Trials Joint Industry Project

Trial Application of API RP 2A — WSD Draft Section 17

Volume II — Participants' Submittals

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1.0 PLATFORM INFORMATION

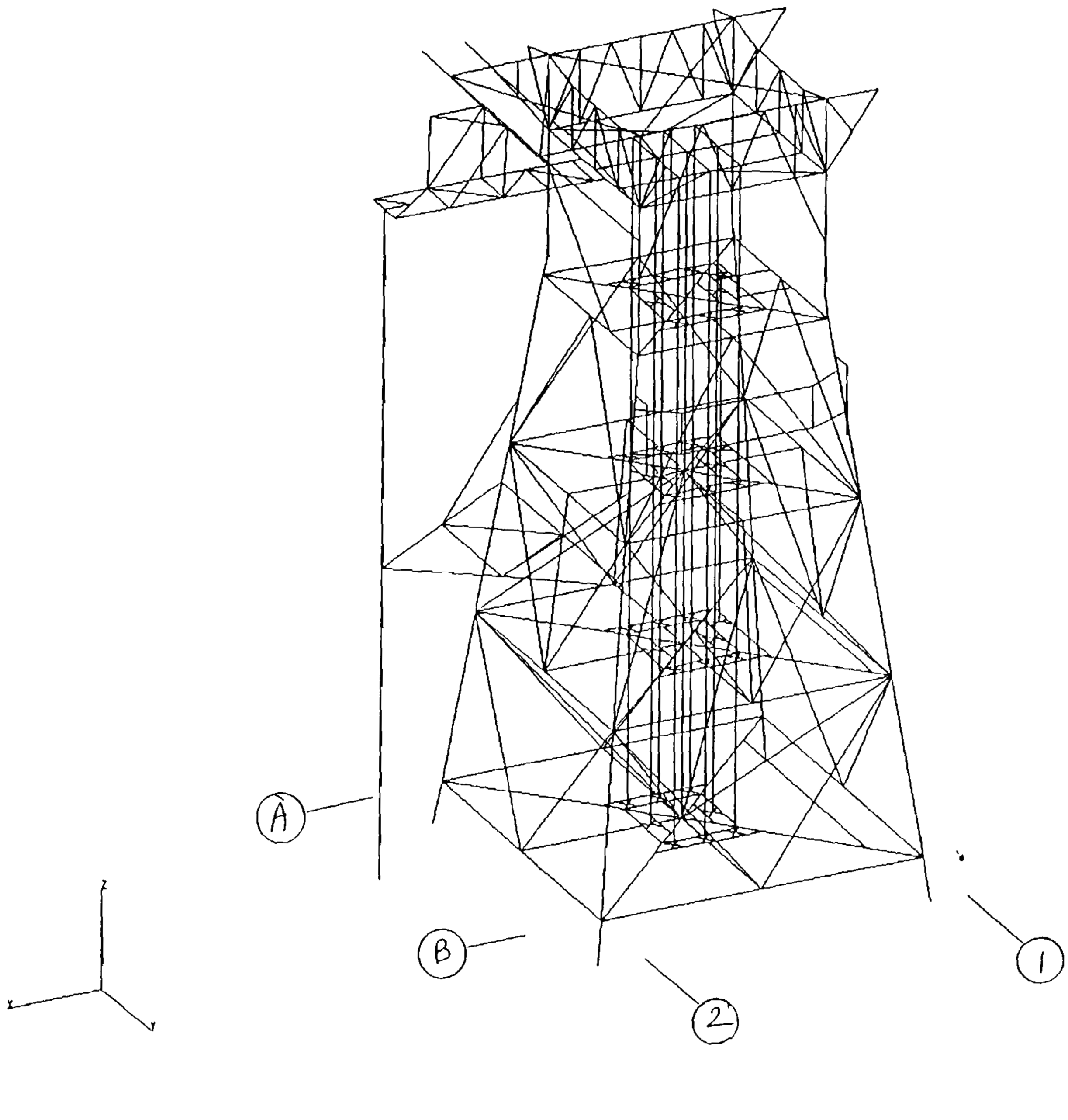
Platform 'A' is located in the East Cameron block of the Gulf of Mexico. The following are the salient features of the platform.

Water Depth	-	103 ft.
No. of Legs & Batter	-	4, 1:7.5 batter on all legs in both directions
No. of Piles	-	4 grouted main piles
Type of Platform	-	Production with quarters, manned and evacuated during storm
Brace Type in Vertical Frames	-	K Braces
Cellar Deck Elevation	-	52.00 ft.
No. of Wells	-	10 (9 original + 1 external well added)
Year Designed	-	1964
Year Installed	-	1964 in 150 ft. water depth, moved to present location in 1969
Miscellaneous	-	One conductor runs through the center of the platform enclosed by a casing. The casing is connected to the jacket leg joints by vertical diagonal members.
Pile Size and Penetrations	-	36"ϕ, 230 ft. penetration
Barge Bumper	-	Two numbers
Boatlanding	-	One full size

The soil boring at the locations was performed and was used for assessment. The last inspection of the platform was done in 1993. Most of the anodes were found to be depleted and steel loss on the members were measured.

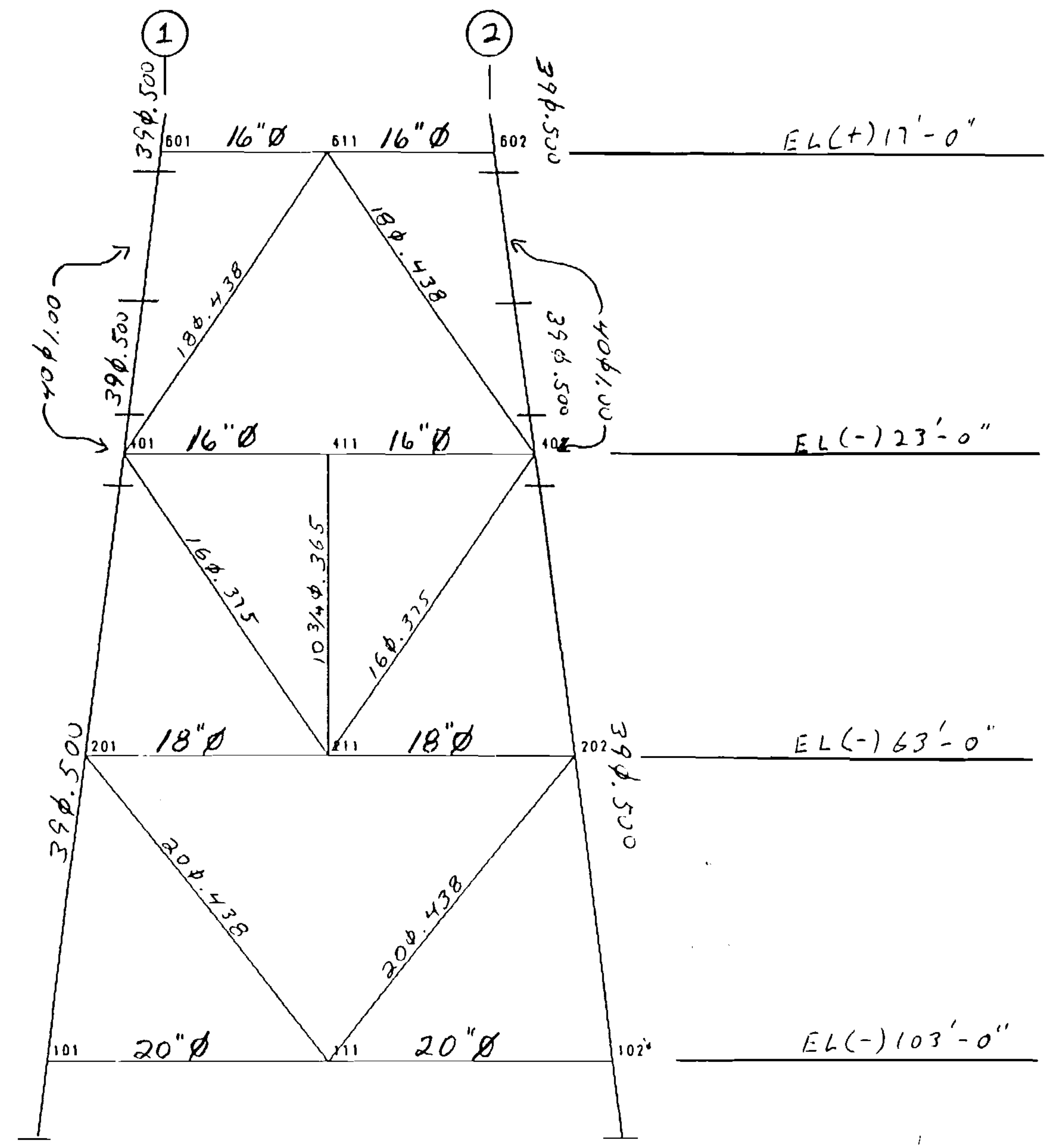
PLATFORM 'A'

3D VIEW



JACKET ROW A

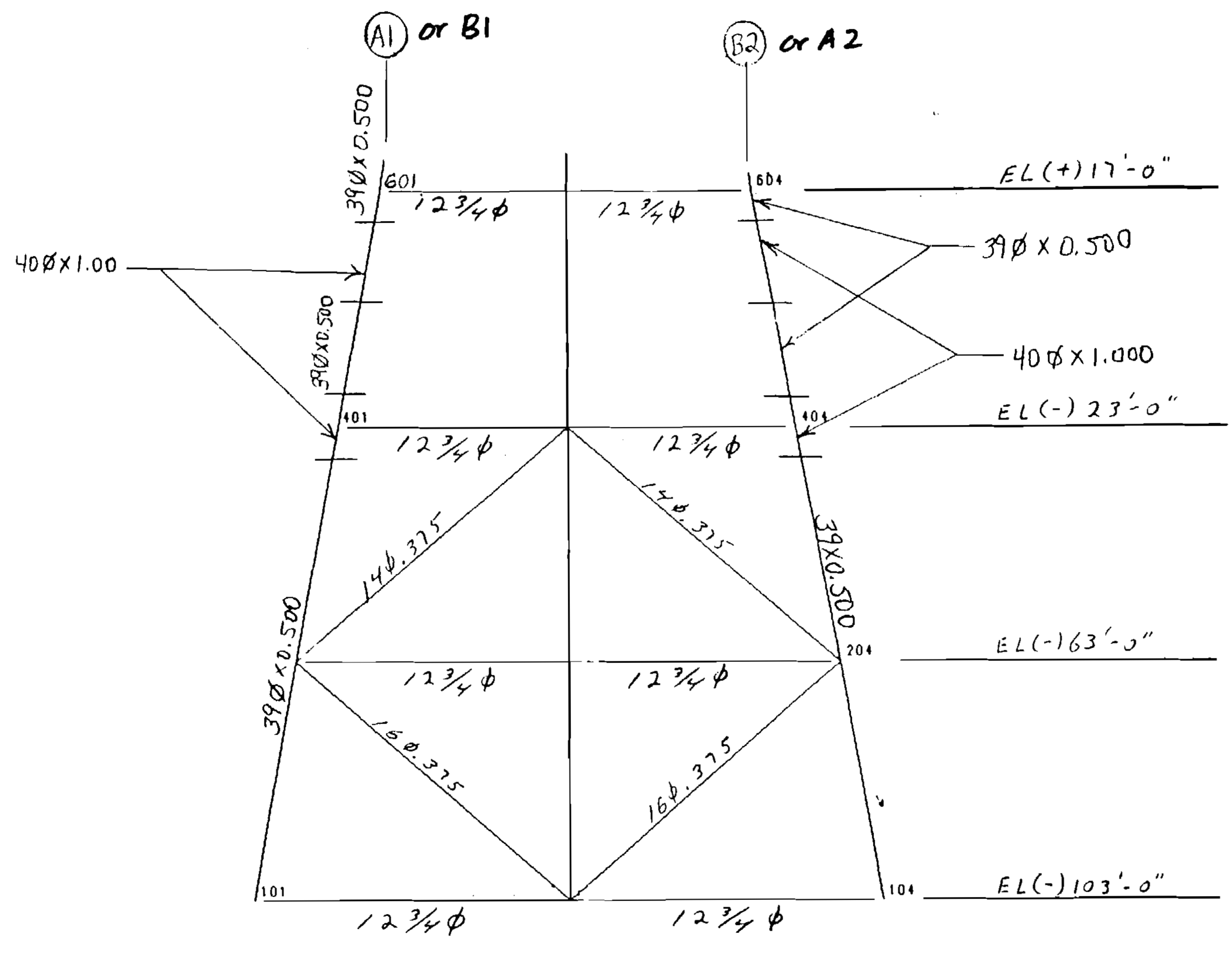
PLATFORM 'A'



(TYPICAL ELEV.)

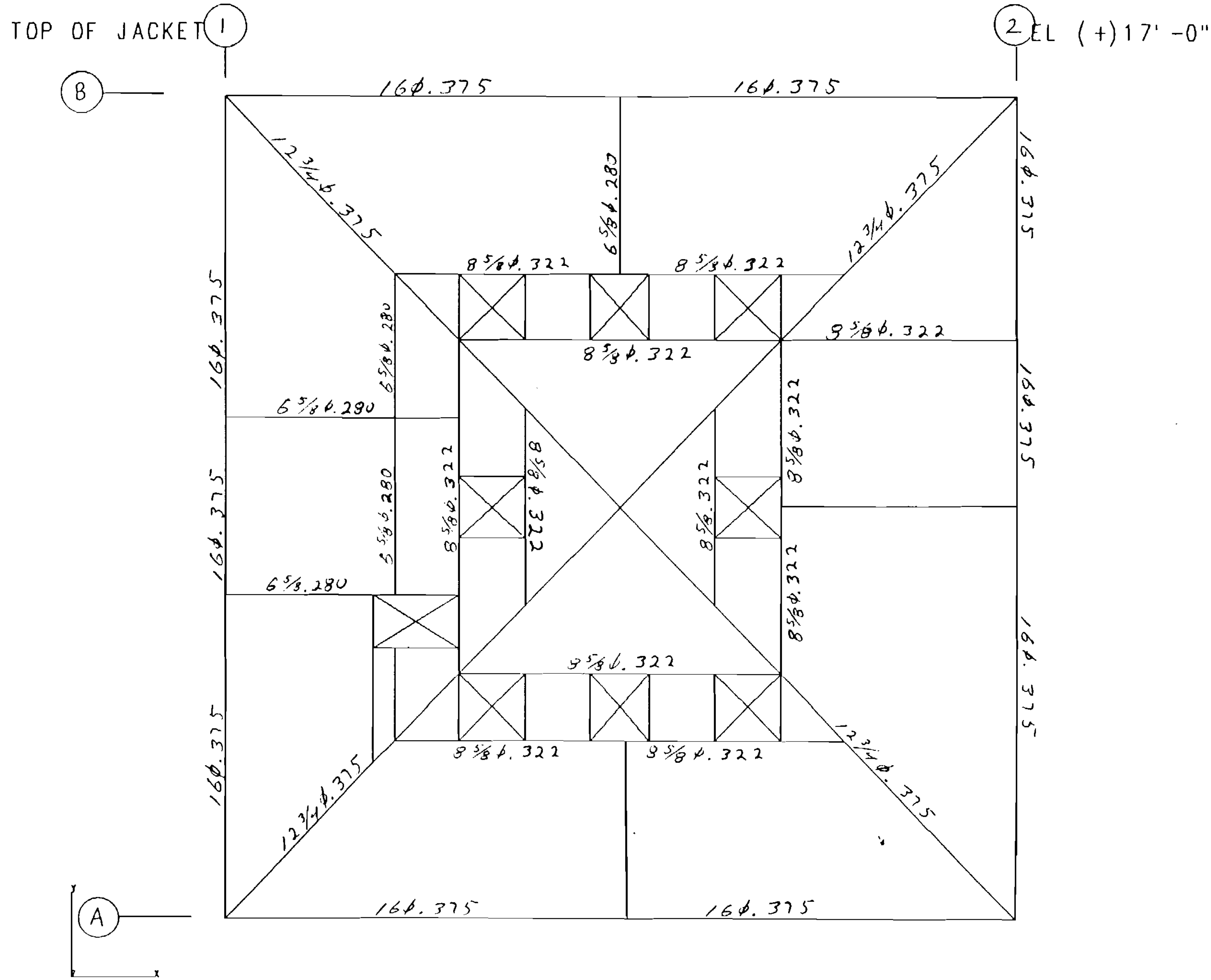
PLATFORM 'A'

JACKET DI AGONAL



TY. 'AL

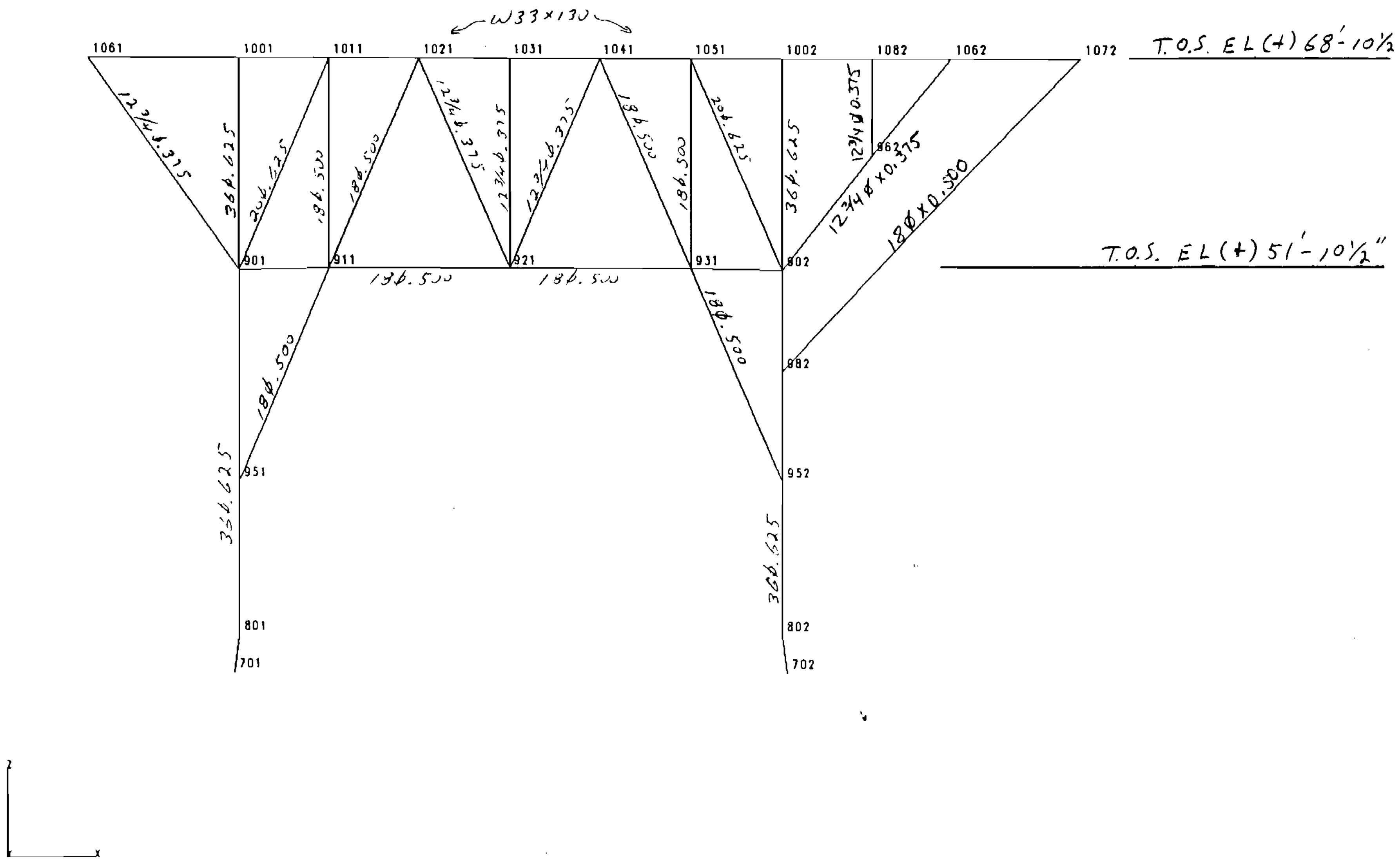
PLATFORM 'A'



TOP OF JACKET
EL (+)17' -0"
1
2
A
B
16ϕ.375
12 3/4ϕ.375
8 5/8ϕ.322
6 5/8ϕ.280
6 5/8.280

PLATFORM 'A'

DECK ROW A



PART A: **PLATFORM ASSESSMENT**

A.1 **PLATFORM SELECTION:** The platform should undergo if one of the conditions noted below exists:

A. **Addition of Personnel:** The manning condition has not changed. Hence, this condition is not satisfied.

B. **Addition of Facilities:** No significant addition of facilities relative to the original operational loads have been made. Hence, this condition is not satisfied.

C. **Increased Loading on Structure:** The addition of one well has not increased the loading by more than 10% beyond the original design loads using the original design criteria. Hence, the increase in load is not significant and this condition is not satisfied.

D. **Inadequate Deck Height:** The minimum deck height required from Fig. 17.6.2-3b for the platform is 38 ft. Since the cellar deck height is 52 ft., inadequate deck height criteria is not satisfied.

E. **Damage Found During Inspection:** The last inspection showed damage in the form of excessive corrosion occurred and a dent was found in one of the horizontal members. The structural integrity of the platform due to the dent was evaluated and found to be not compromised. Since the material loss due to corrosion was considered as significant damage, it is concluded that the platform has to undergo the assessment process.

The damage found on Platform 'A' during inspection was used as a trigger for assessment process.

A.2 **CONDITION ASSESSMENT:** The condition of the platform was assessed by performing annual Level 1 topside survey and conducting a Level IV underwater survey of the platform. The topsides platform drawings have been verified and the facilities arrangement and configurations information collected.

As described earlier, the underwater inspection showed excessive corrosion damage and a dent in one of the members. No fatigue cracks were detected at any joint.

The soil boring log at the platform site performed in 1969 was used for the assessment process.

A.3 **CATEGORIZATION:** For assessment purposes, the platform was categorized as manned, evacuated with insignificant environmental damage.

A.4 **DESIGN BASIS CHECK:** Since the platform has significant corrosion damage and also has been designed prior to 9th Edition of RP2A (1977), a detailed assessment is required.

A.5 **ANALYSIS CHECKS:** For the JIP, both the design level and ultimate strength analysis were performed, although it was not mandatory to perform ultimate strength analysis because of sufficient deck height.

A.5.1 **DESIGN LEVEL ANALYSIS:** The platform model for the design level analysis included all ten conductors and all appurtenances. The reduced thickness of members due to corrosion was not modelled. This was done to check initially whether the platform passes the design level analysis in the intact state. As shown below the platform does not pass the design level analysis requirement in the intact state and hence, the design level analysis was not performed on the platform with reduced member thickness as modelled. SACS program was used for the analysis.

The design level wave, current and wind were applied to the platform in eight different directions 45° apart. Wave kinematic factor and current blockage factor were considered in the analysis. The following sudden hurricane metocean criteria was used for the design level analysis:

Wave Height	-	42 ft. (Fig. 17.6.2-3A) (Omni-directional unless it was found greater than ultimate analysis wave height for certain directions)
Wave Period	-	11.3 secs. (Table 17.6.2.1)
Current Speed	-	1.20 knots (Omni-directional unless greater than ultimate analysis current for certain directions)
Storm Tide	-	3.5 ft. (Fig. 17.62-3A)

Wind Speed (1 hr. @ 10m)	-	55 knots (Table 17.6.2.-1)
Marine Growth	-	1.5"
Minimum Deck Height Required	-	38 ft. (Fig. 17.6.2-3B)

The bottom of the cellar deck beams is at 50'-3". Since minimum deck height required is satisfied, it is not mandatory to perform ultimate strength analysis.

The results from the analysis are summarized below:

Maximum Base Shear (wave direction)	-	1351 kips (90°)
Maximum Overturning Moment (wave direction)	-	98500 ft.-kips (90°)
Maximum Compressive Pile Load (wave direction)	-	1346 kips (45°)
Safety Factor on Pile Compression Capacity	-	2.78
Maximum Tensile Pile Load (wave direction)	-	700 kips (225°)
Safety Factor on Pile Tension Capacity	-	3.57

Various internal vertical diagonal members connecting the jacket leg joints to the central conductor casing were overstressed. This was mainly due to the large KL/r values for these members. A plot of members with unity check greater than 0.85 are shown in Figures 1-3. The platform does not have any joint cans and hence, a lot of joints were found to have punching shear overstress. These joints were mainly the K-brace joints on the jacket horizontal members and the joints on the internal conductor casing member.

Since various primary members and joints overstress were detected in the platform by performing the design level analysis, it was concluded that the platform does not pass the design level analysis. An ultimate strength analysis was then performed as described below.

A.5.2 ULTIMATE STRENGTH ANALYSIS:

The platform model for the non-linear ultimate strength analysis included all ten conductors. The reduced thickness of members due to corrosion were modelled in the analysis. Wave kinematic factor and current blockage was considered for the analysis. USFOS program was used for the analysis.

The ultimate level wave, current and wind were applied to the platform in 90° direction with respect to platform north. Based on preliminary analysis, the 90° waves, current and wind were found to produce the minimum ultimate strength after taking the wave and current directionality factor and the loadings on the platform into account. The following metocean criteria was used for the ultimate strength analysis:

Wave Height	-	49.88 ft. (Fig. 17-6-2-3A and Fig. 17.6.2-4)
Wave Period	-	12.5 sec. (Table 17.6.2-1)
Current Speed	-	1.80 knots (Table 17.6.2-1 and Fig. 17.6.2-4)
Storm Tide	-	3.5 ft. (Fig. 17.6.2-3A)
Wind Speed (1 hr. @ 10m)	-	70 knots (Table 17.6.2-1)
Marine Growth	-	1.5 ~~knots~~ inch

Two sets of ultimate strength analyses were performed. In one analysis, the joint strength was assumed to be infinite and only member failures were allowed. In the other analysis, joint ultimate capacities and member failures were both considered. Equations 4.3.1-4A and 4.3.1-4B from API RP2A with the safety factor removed were used to calculate the ultimate joint capacities. It should be noted that the ultimate strength analysis was performed overseas in June, 1993 before the JIP started and all the results required for JIP are not available. Providing all the results in the format required by JIP would require rerunning the analysis. The cost for re-running the analysis could not be justified. However, all the important results are included in this report.

The load deflection plot from the analysis without considering the joint strength is shown in Figure 4 and for the analysis which also considered the joint capacities in Figure 5. As shown, the RSR value is doubled when the joint strength is not considered. This tendency could be generalized in most of the old platforms with no joint cans.

The failure of the platform when joint strengths were not considered was initiated by the buckling of the vertical face diagonal member in Row 2 between EL(-)23'-0" and EL(-)63'-0". It should be noted that in USFOS program the buckling strength is not based on KL/r value, but the buckling phenomenon is implicit in the energy formulation. This results in less conservative buckling strength than the conventional design using KL/r values.

In the analysis involving joint capacities check the collapse of the platform was due to the K-joints at EL(-)63'-0" and EL(-)103'-0" in Rows 1 and 2. These joints are yielded much before the load peak.

Since the ultimate lateral load capacity of the platform is less than the ultimate strength level load when the joint capacities are included in the analysis, it was concluded that the platform does not pass assessment. This conclusion would be true even if corrosion damage on the platform is neglected. The platform, however, passes assessment when only member strength is considered.

A.6 **MITIGATION ALTERNATIVES:** Because the platform does not pas assessment and the platform is still producing the following mitigation steps would be taken:

1) Strengthen the K-joints either by adding pup pieces or grouting the joint.

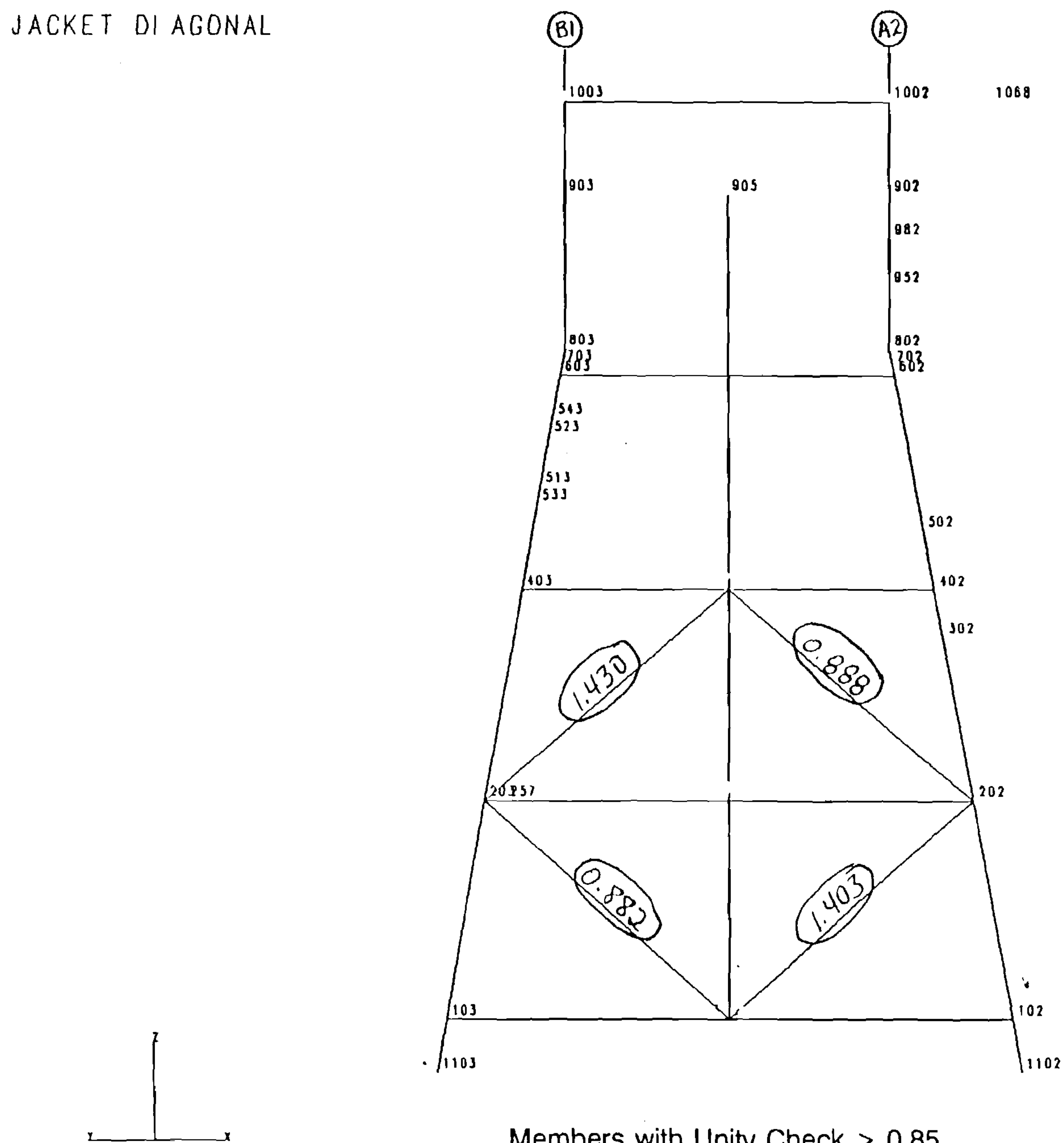
2) Replace depleted anodes.

3) Reduce the load on the platform by removing some of the non-producing wells or cut below the wave zone.

A.7 **SUMMARY NOTE:** The assessment process for Platform 'A' was triggered by corrosion damage found during inspection. The platform was categorized as manned, evacuated with insignificant environmental damage. The platform does not pass the design level analysis. An ultimate strength analysis showed the platform has adequate capacity if joint failures are ignored. However, with joint strength taken into consideration, the platform's ultimate capacity is found to be below the ultimate strength analysis level load and the reference level load (RP-2A, 20th Edition).

Thus, the platform passes the assessment process if joint strengths are not considered and does not pass assessment if joint failures are considered.

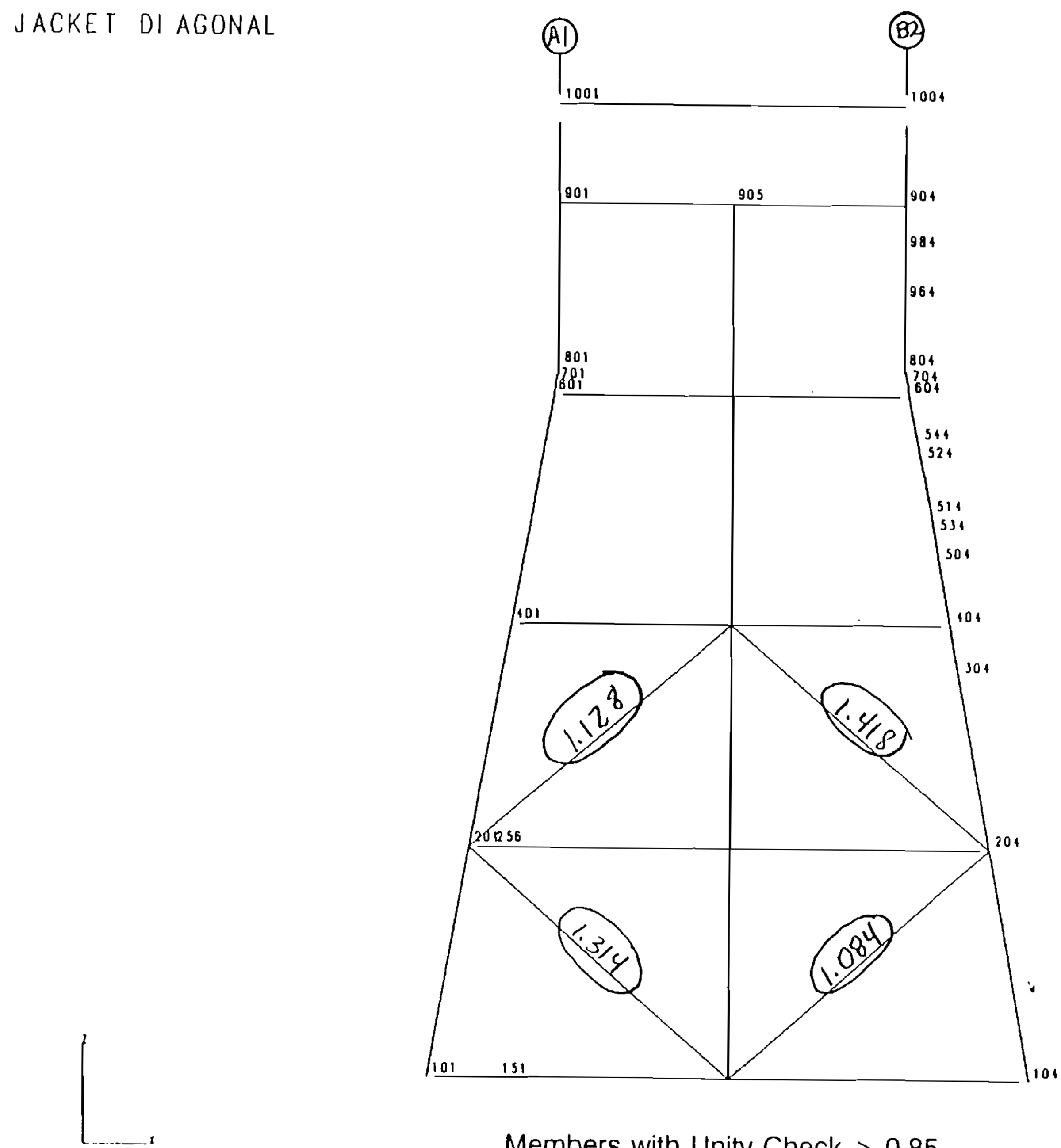
PLATFORM 'A'



Members with Unity Check > 0.85

Figure 1: Desi[illegible] evel Analysis

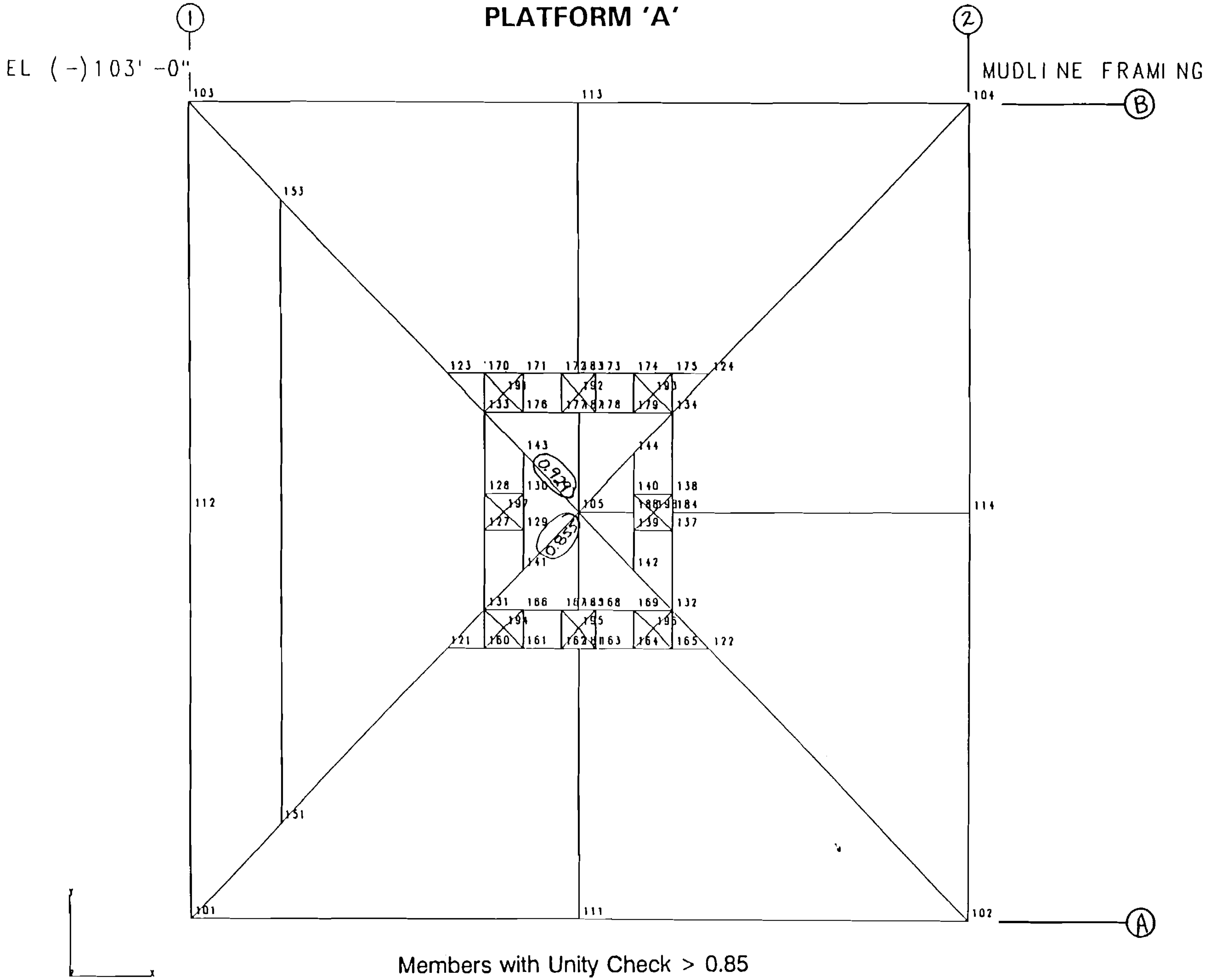
PLATFORM 'A'



Members with Unity Check > 0.85

Figure 2: Design Level Analysis

PLATFORM 'A'



Members with Unity Check > 0.85

Figure 3: Design ' ’el Analysis

PLATFORM 'A' ULTIMATE STRENGTH ANALYSIS

Ultimate Strength Analysis Level Load	-	2197 kips
Load level at which first component reaches yield	-	2212 kips
Reference level load (S_{ref})	-	2390 kips
Ultimate Capacity (R_u)	-	2614 kips
Reserve Strength Ratio (RSR) (R_u/S_{ref})	-	1.094
Platform Failure Mode	-	Jacket face diagonal

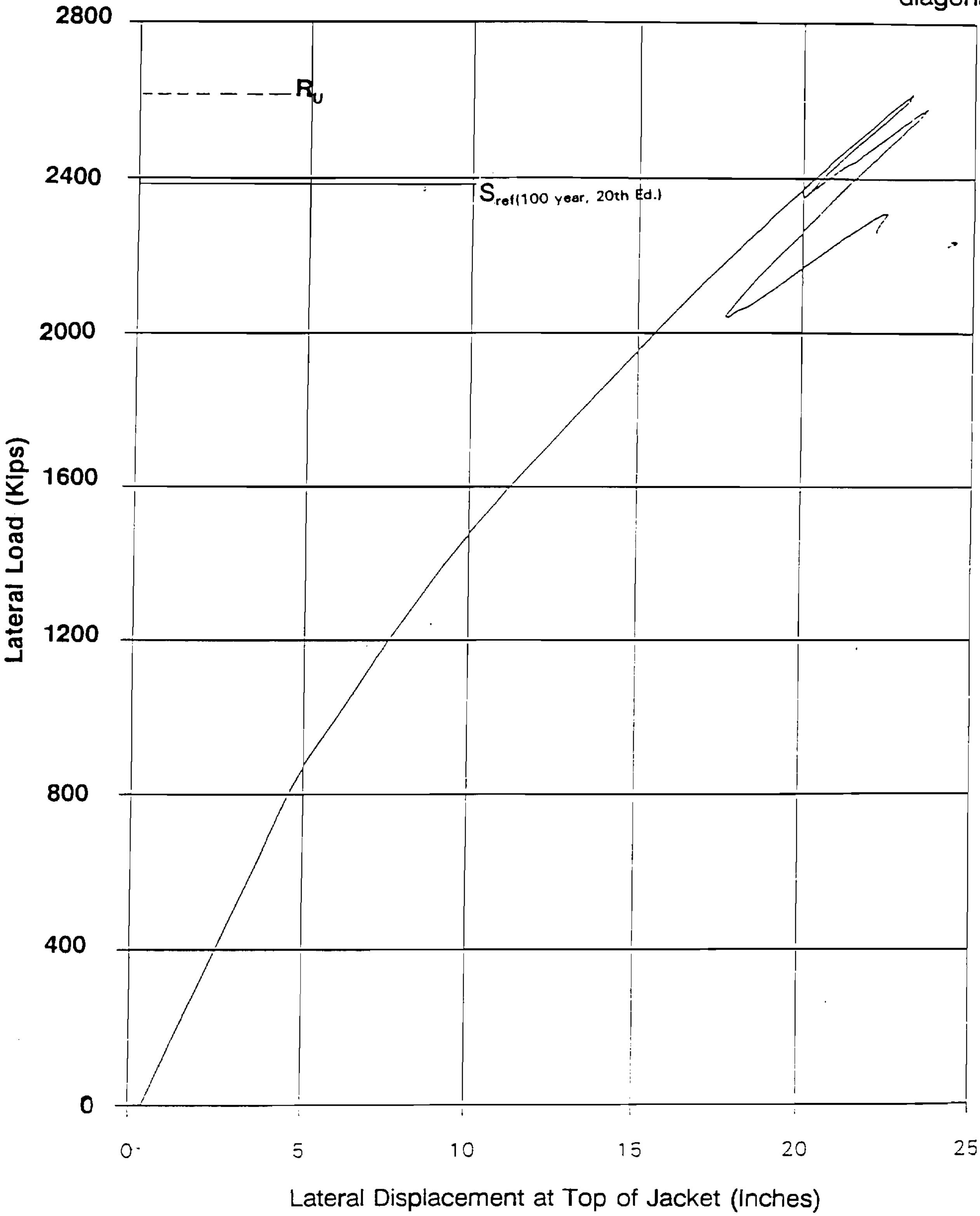


Figure 4: Load Defelction Plot without Considering Joint Capacities

PLATFORM 'A' ULTIMATE STRENGTH ANALYSIS

Ultimate Strength Analysis Level Load	-	2197 kips
Reference level load (S_{ref})	-	2390 kips
Ultimate Capacity (R_u)	-	1307 kips
Reserve Strength Ratio (RSR) (R_u/S_{ref})	-	0.547
Platform Failure Mode	-	Jacket k-brace joints

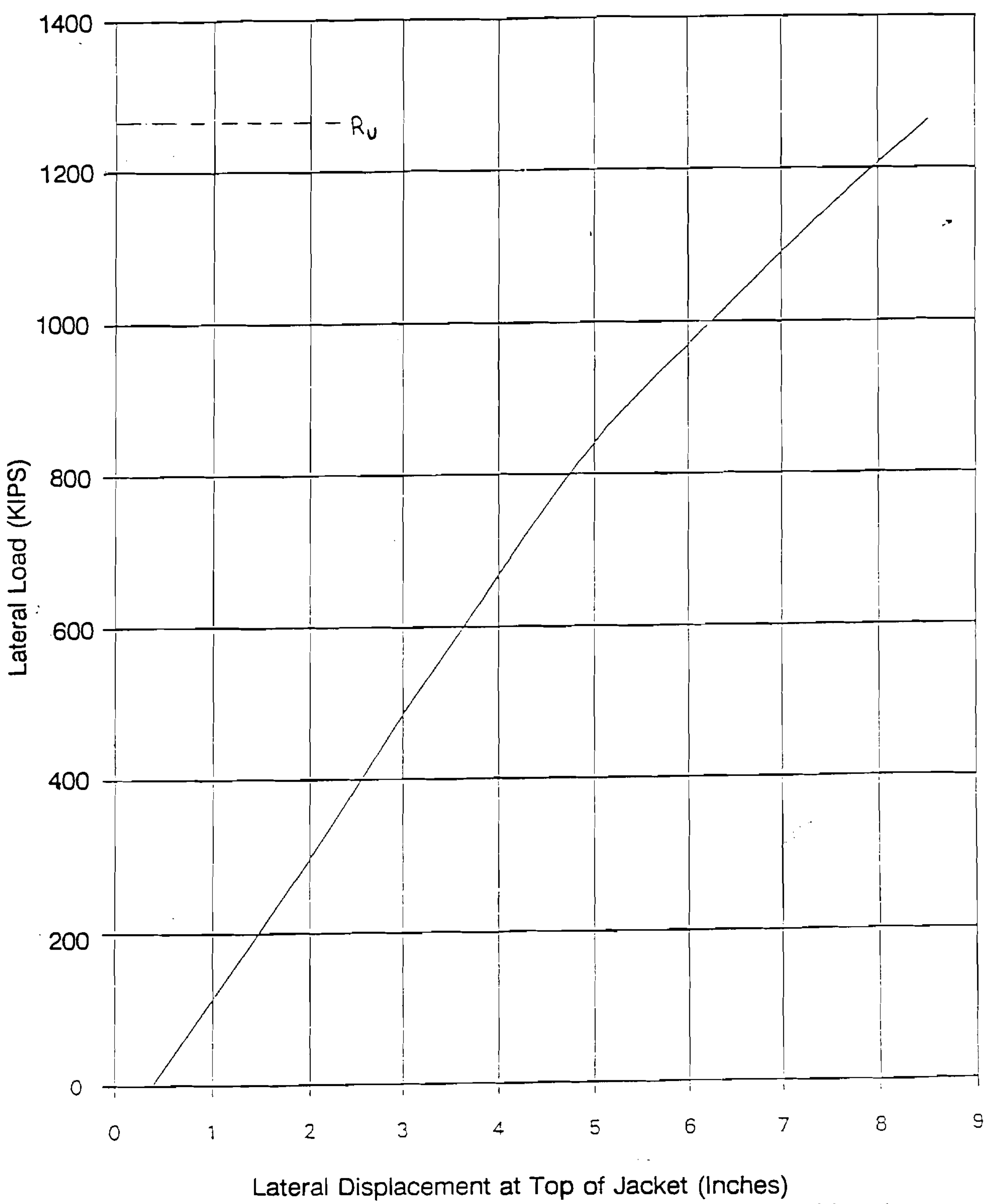


Figure 5: Load Deflection Plot with Joint Capacities Considered

PART B: REVIEW AND FEEDBACK TO THE API TG92-5

The only condition that triggers assessment for Platform 'A' is member degradation due to corrosion. If there was no corrosion damage on the platform, we didn't have to go through the assessment process. But as it turns out the platform does not pass assessment when all the analysis checks are made even when the platform damage is neglected. This probably will be true for many old platforms designed prior to 1970. Most of these old platforms were designed for 25 year storm with no loads due to current used in the design and did not have joint cans. It is our opinion that another trigger to perform assessment should be introduced for platforms designed prior to 1970 (Section 17.2).

All the triggers to perform assessment should be included in the flow chart of Figure 17.5.2 to make it more complete.

PLATFORM "B"

1. Platform Information

Platform B (PLT-B) is currently owned and operated by Trial Participant B. It was originally installed in 1971. PLT-B is located 79 miles off the coast of Louisiana (92° - 38' W and 28° - 25' N), and is oriented N 10° E. PLT-B is a four pile, ungrouted, drilling platform in 182 feet of water. There are two decks (T.O.S. elev. 48', 64'), five wells, and ten conductors. The platform has quarters for a crew of 32, and is bridge connected to a production platform.

An API RP2A level 2 inspection was made on PLT-B in June, '93 by Martech International. One member had a small amount of fretting due to a wire cable (which has since been cut). This minor damage will be ignored in the assessment process. Also found during the inspection was that ten feet of debris has been accumulated on the sea floor beneath the jacket. The debris is most likely drill cutting. This debris neither adds stiffness nor load and will therefore be ignored. No scour was found on the pileheads.

Framing of PLT-B consists mainly of K-braced bays. PLT-B is apparently a modification of an existing jacketed by adding an additional 20 feet to the bottom bay. The Leg diameter is 46 inches. Platform sketches are given in appendix A of this section.

A soil boring was taken in September, '71 in the same lease block to a penetration of 396.5 feet. The soil boring is located at Lambert coordinates of x=1,583,650 and y=-87,669, at a distance of approximately 138 feet from PLT-B. Undrained shear strength of 2 1/4" diameter samples were obtained using miniature vane (MV) tests, and an unconfined compression (UC) tests. In interpreting the shear strength profile the soil boring contractor modified the measured shear strength as follows: $Su_{(design)} \approx$ minimum of $Su_{(mv)}$ and 1.2 X the average of $Su_{(uc)}$ and $Su_{(mv)}$. This calculation is done in order to mimic undrained shear strength using 3" push samples (Section C17.4.3). Soil strength parameters are given in appendix B of this section.

The piles are 42" O.D. with wall thickness varying from 1" to 2". Pile penetration for PLT-B is 255'. The design ultimate axial capacity are 7600 and 7200 kips in compression and tension.

Appendix A - Sketches

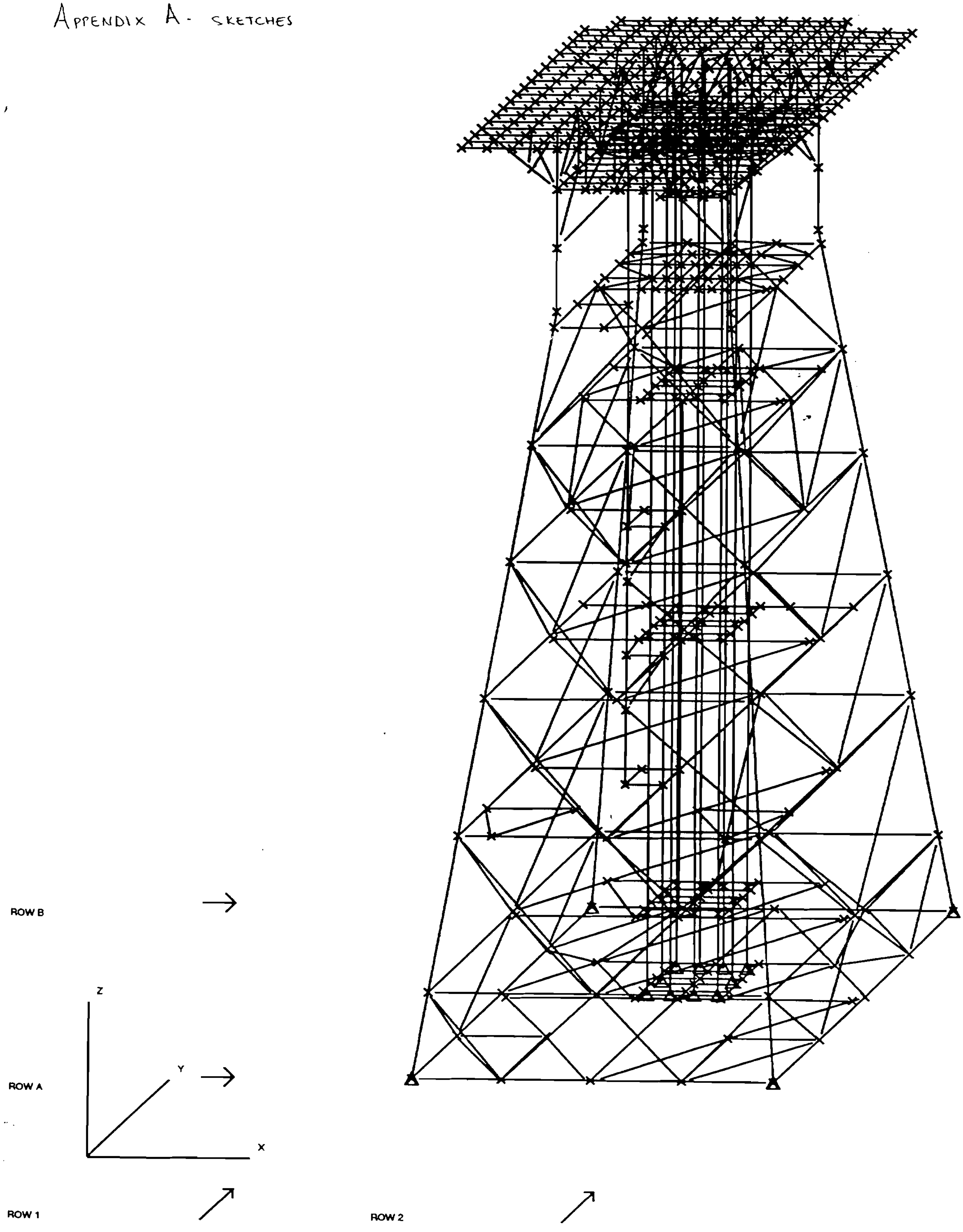
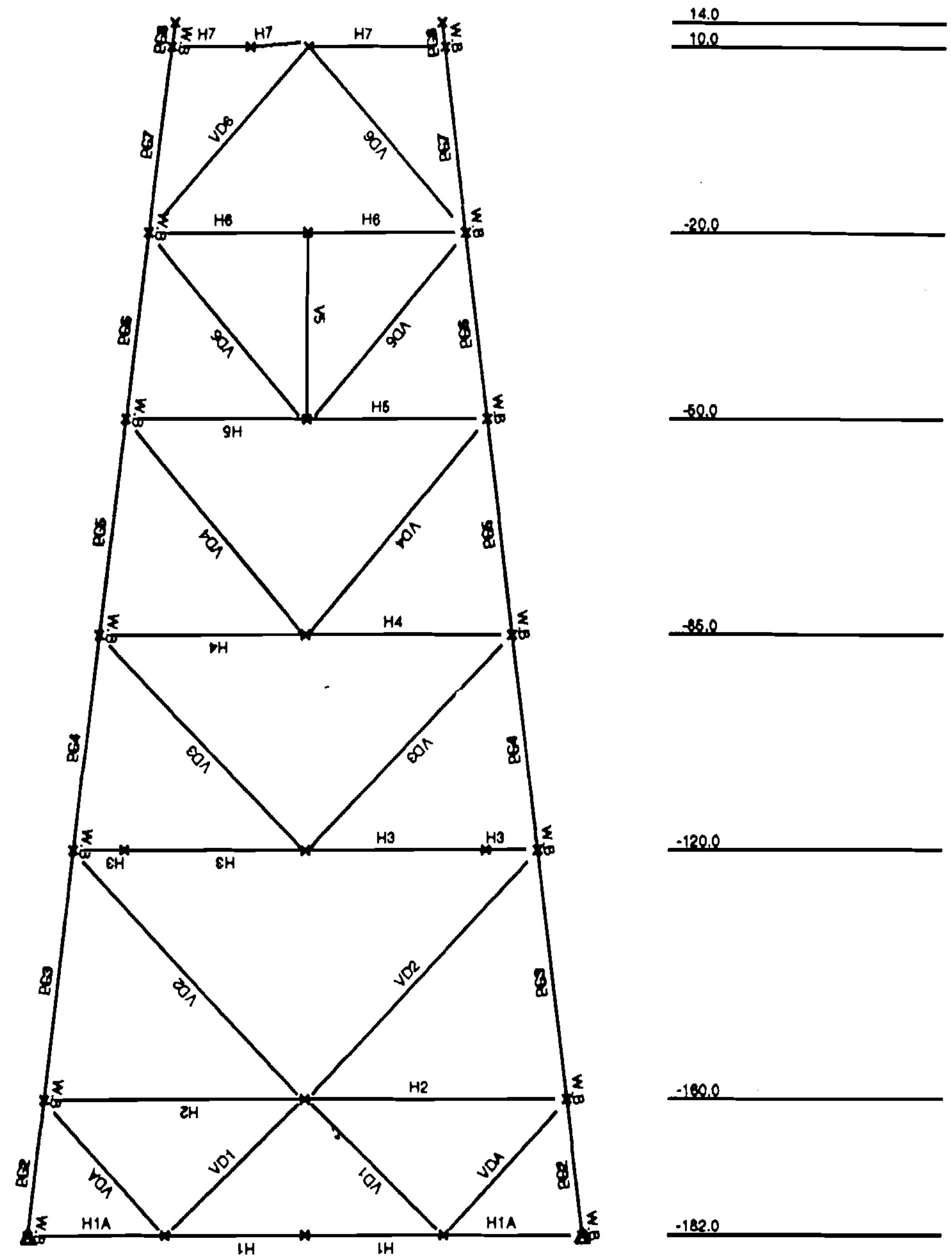


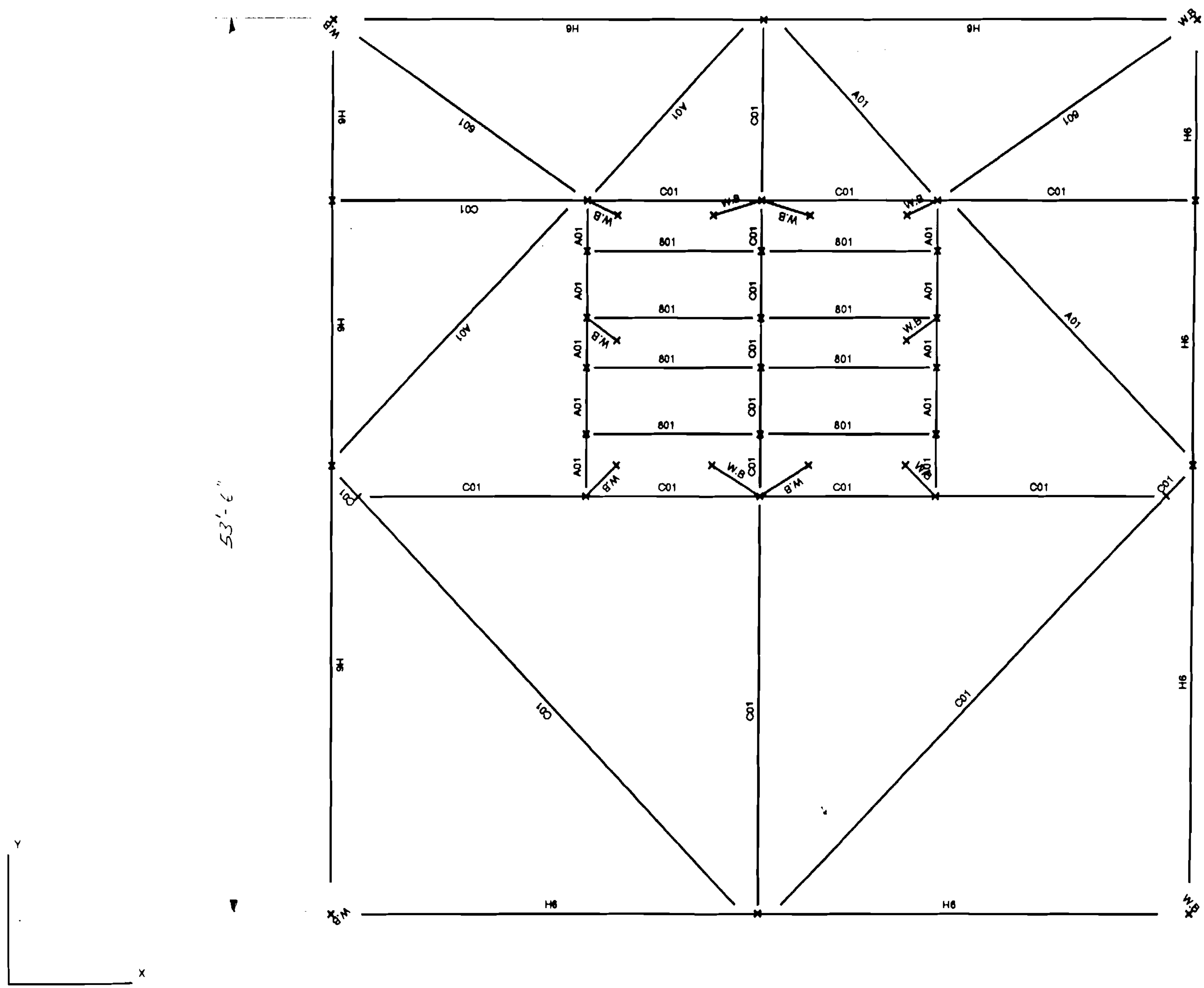
Table of Member Sizes for Platfom B (excluding can sizes)

Legs / Diagonals		Misc. Members		Non Tubulars	
Group	Size	Group	Size	Group	Size
LG2	45 x 0.625	H1	24 x 0.437	8C1	8C11.5
LG3	45 x 0.625	H1A	24 x 0.437	9C1	9C13.4
LG4	44.75 x 0.5	H2	24 x 0.372	W01	W12 x 27
LG5	44.75 x 0.5	H3	16 x 0.375	W02	W 24 x 84
LG6	44.75 x 0.5	H4	16 x 0.375	W03	W 24 x 145
LG7	45 x 0.625	H5	14 x 0.375	W04	W 24 x 110
LG8	46 x 1.125	H6	12.75 x 0.375	W05	W 36 x 170
LG9	46 x 1.125	H7	14 x 0.375	W06	W 16 x 36
LGA	46 x 1.125			W07	W 30 x 172
VD1	18 x 0.375	401	4 1/2 x 0.237		
VD2	20 x 0.438	601	6 5/8 x 0.280		
VD3	16 x 0.375	801	8 5/8 x 0.322		
VD4	16 x 0.375	A01	10 3/4 x 0.365		
VD5	14 x 0.375	A02	10 3/4 x 0.438		
VD6	14 x 0.5	C01	12 3/4 x 0.375		
VDA	18 x 0.375	C02	12 3/4 x 0.438		
		E01	14 x 0.375		
		G01	16 x 0.375		
		G02	16 x 0.656		
		G03	16 x 0.500		
		I01	18 x 0.375		
		I02	18 x 0.625		
		K01	20 x 0.375		
		K02	20 x 0.500		
		K03	20 x 0.593		
		O01	24 x 0.562		
		O02	24 x 0.593		
		Z01	26 x 0.5		

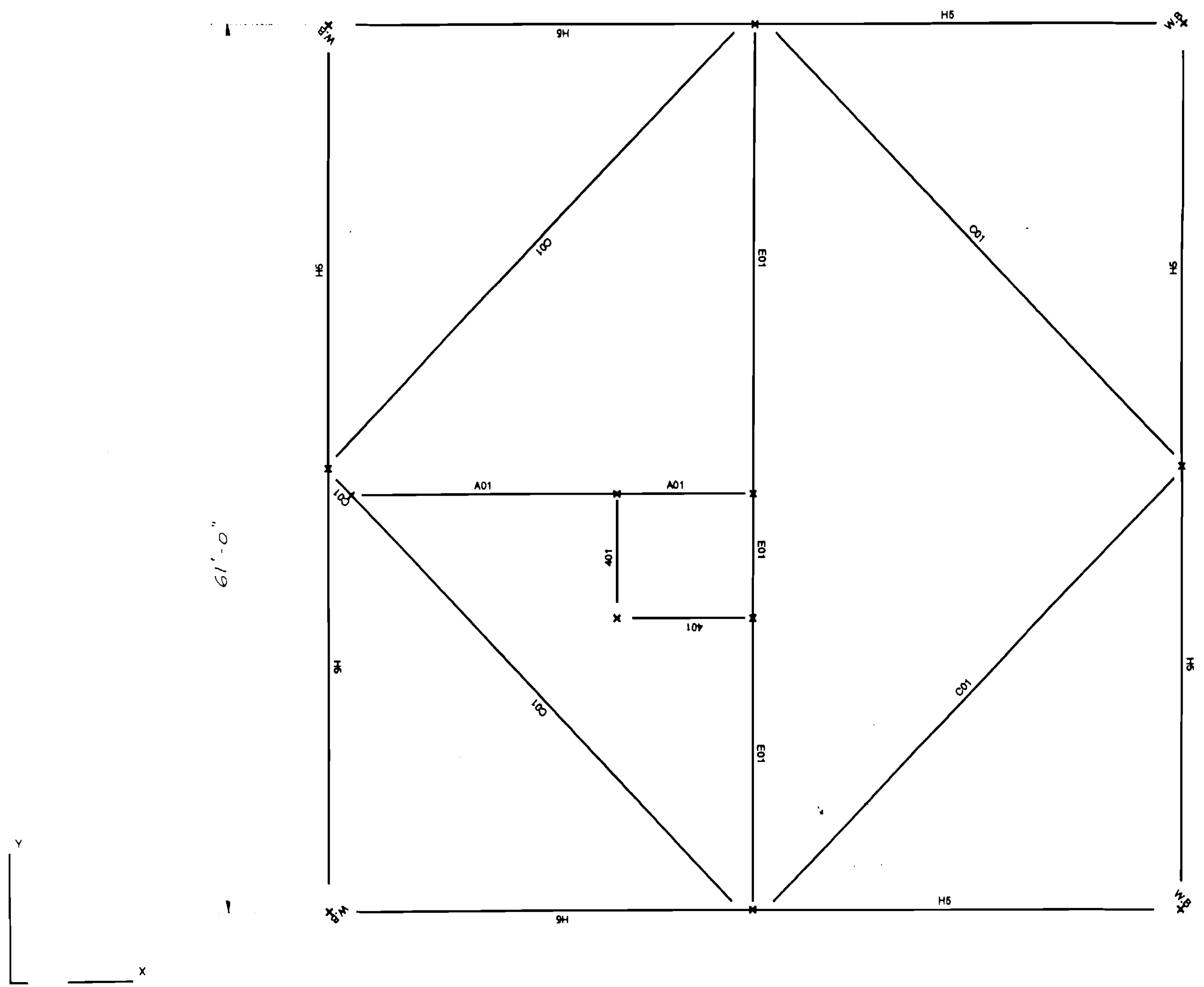
ROW A

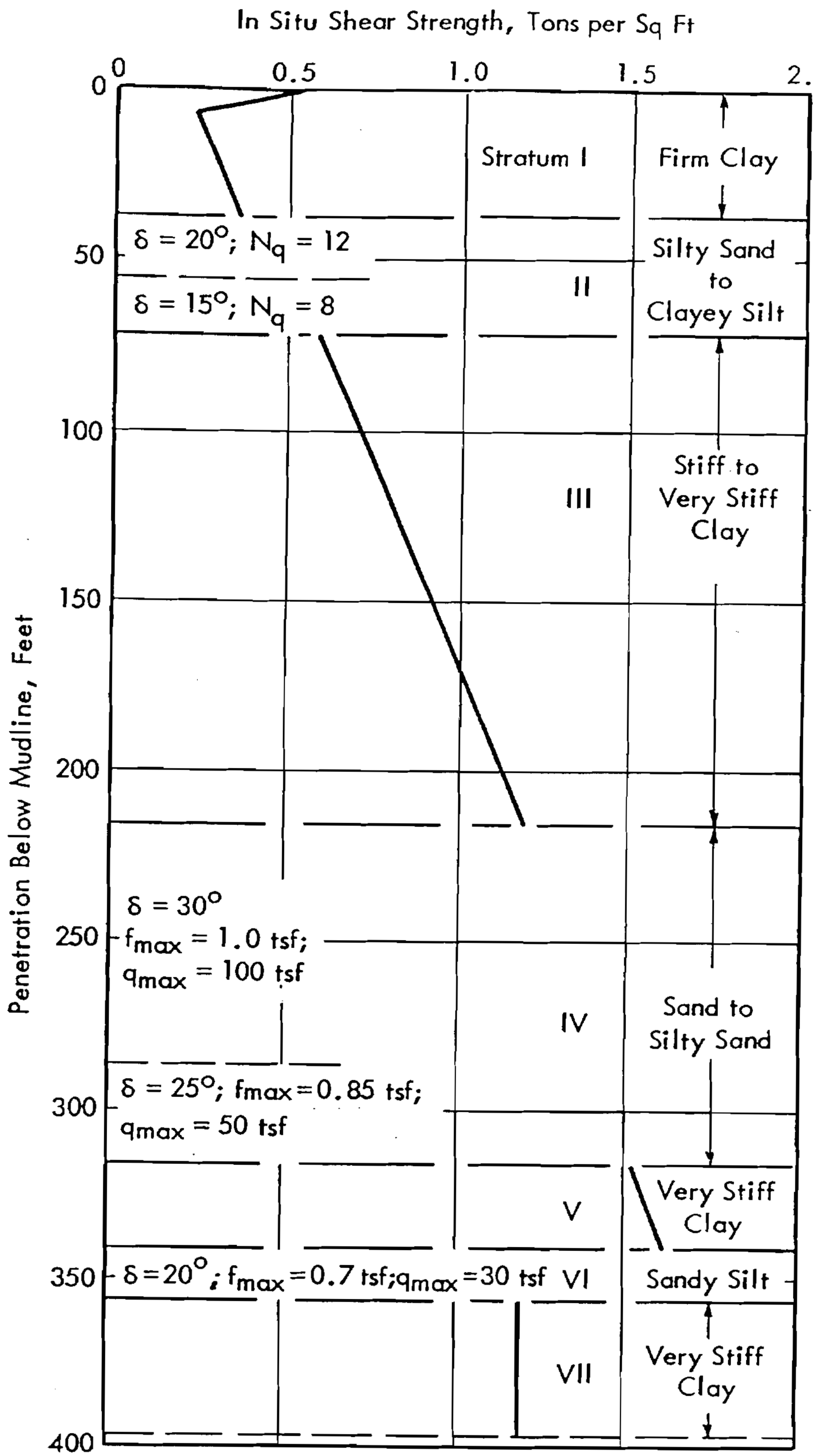

14.0
10.0
-20.0
-50.0
-85.0
-120.0
-160.0
-182.0
W.B
H7
H6
H5
H4
H3
H2
H1
H1A
V5
VD6
VD4
VD3
VD2
VD1
VDA
BG8
BG7
BG6
BG5
BG4
BG3
BG2
Z
X

PL IEW AT -20.00



PL IEW AT -50.00





STRENGTH PARAMETERS

SOIL PROPERTIES FOR PLT-B

PART A: Platform Assessment

A.1 Platform Selection

PLT-B does NOT need assessment according to Draft Section 17.2 (April 4, 1994 version). There has been no 'Addition of Personnel', 'Addition of Facilities', 'Increased Loading on the Structure', or 'Significant Damage found on the Structure'.

A.2 Condition Assessment

There has been no change in the usage of the platform since its installation. There has been no significant physical changes to the structure. The most recent inspection was an API RP2A Level 2 survey made in June, '93. No indication of fatigue or corrosion was discovered during the survey.

A.3 Categorization

According to Draft Section 17, PLT-B is classified as a Manned-Evacuated structure with Insignificant Environmental Impact. There is no oil storage tanks on board. There are two 142 barrel storage tanks with drilling fluid for the workover rig. This capacity represents a low environmental impact for a structure 79 miles off the coast.

A.4 Design Basis Checks

The underside of the cellar deck is 46 feet above MWL, which is above the minimum deck height required for a design basis check. However, the structure was designed prior to the 9th edition and for this reason PLT-B does not pass assessment by design basis check.

A.5 Analysis Checks

A.5.1 Metocean, seismic, and ice criteria/loads

PLT-B is almost symmetrical and therefore loading will be considered from the three most critical directions: 1) End-On (10° w/respect to true north), 2) Diagonal (325° w/respect to true north), 3) Broadside (280° w/respect to true north). These directions have the greatest load factors for the sudden hurricane wave criteria.

API RP2A 20th ed. environmental loading criteria was used unless otherwise defined in Draft Section 17. In this case metocean loads for an insignificant environmental impact/manned structure are defined by the sudden hurricane criteria in Draft Section 17. For PLT-B **design level analysis** this corresponds to:

Wave Height (H)	= 46 feet	(Sec 17)
Wave Period (T)	= 11.3 seconds	(Sec 17)
Inline Current (U)	= 1.2 knots	(Sec 17)
Wind Speed (V)	= 55 knots (1 hour @ 10 meters)	(Sec 17)
Storm Tide	= 3 feet	(Sec 17)
CD (smooth/rough)	= 0.65 / 1.05	(API 20th)
CM (smooth/rough)	= 1.6 / 1.2	(API 20th)

The metocean criteria for the **ultimate strength analysis** are:

Wave Height (H)	= 57.5 feet
Wave Period (T)	= 12.5 seconds
Max Current (U)	= 1.8 knots= 3.04 ft/sec (0 - 150') ; Dir=275° w/r true north
Wind Speed (V)	= 70 knots (1 hour @ 10 meters)
Storm Tide	= 3 feet
CD (smooth/rough)	= 0.65 / 1.05
CM (smooth/rough)	= 1.6 / 1.2

Table 1. Sudden Hurricane Rosette Factors for Ultimate Strength Analysis

Wave Direction	True Direction	Factor	Wave Height	Current > 150'
End-On (0°)	10°	0.95	54.6'	1.46 @ -49°; x =-1.7, y=1.0
Diagonal (45°)	325°	1.00	57.5'	1.64 @ 287°; x = -1
Broadside (90°)	280°	0.90	51.8'	1.72 @ 278°; x = -1

The API RP2A 20th ed. criteria was used to compute the denominator of the Reserve Strength Ratio (RSR). For PLT-B, the API 20th ed. criteria are as follows:

Wave Height (H)	= 64.2 feet
Wave Period (T)	= 13.0 seconds
Max Current (U)	= 2.1 knots
Wind Speed (V)	= 80 knots (1 hour @ 33 feet)
Storm Tide	= 3.5 feet
CD (smooth/rough)	= 0.65 / 1.05
CM (smooth/rough)	= 1.6 / 1.2

Table 2. 20th ed. Rosette Factors

Wave Direction	True Direction	Factor	Wave Height	Current
End-On (0°)	10°	0.85	54.6'	1.68 @ -51°
Diagonal (45°)	325°	0.95	61.0'	1.90 @ 286°
Broadside (90°)	280°	1.0	64.2'	2.09 @ 289°

For all Metocean criteria the following was used:

7th Order stream function.
Marine growth was specified at 1.5" thick from 150' depth to 1' above MWL.
Wave Kinematics factor of 0.88
Current blockage factors of 0.8, 0.85, and 0.8 for the End-On, Diagonal, and Broadside cases respectively.
No seismic or ice loading criteria.

A.5.2 Screening

Draft Section 17.7.3 ***(old Section 17)*** states screening is not required. For PLT-B this step was skipped.

A.5.3 Design Level

A linear model of PLT-B was analyzed using sudden hurricane wave criteria, and force/stress equations from API RP2A 20th ed. Several joints failed the Design Level Analysis (DLA) with unity checks greater than 1. These joints are near the mudline, and are not considered critical. Considering the conservatism associated with the joint design formula a case can be made for PLT-B to pass assessment in the DLA step. For the purposes of the project, the assessment process will continue to the Ultimate Strength Analysis step. The following table summarizes the DLA results:

Table 3. Design Level Wave: Direction vs. Base Shear and Unity Check*

Wave Direction	Base Shear (kips)	Pile Strength		Joint Strength		Member Strength		
		Joint	U.C.	Joint	U.C.	1st Jnt	2nd Jnt	U.C.
End-On	1092	107	0.21	117	1.15	512	601	0.83
Diagonal	1113	105	0.28	117	0.95	105	205	0.72
Broadside	1077	101	0.23	116	1.08	407	311	0.75

* - Components with UC greater than 0.85 are shown in figures A.5.3-1 and A.5.3-2.

The soil capacities for each pile were 7600 kips in compression and 7200 in tension. The maximum compression and tension in a pile was 1102 and 1025 kips respectively, in the DLA (diagonal wave), yielding a factor of safety of 6.9 and 7.0.

The trial application required a Working Stress Design (WSD) analysis be performed in order to find the base shear causing each component a unity check of 1.0. Trial applicant B used the Load Resistance Factored Design (LRFD) method due to an automation feature already in place. The results of this analysis follow:

Table 4. Summary of Component Unity Checks of 1.0 using LRFD

Wave Dir	Pile Capacity			Pile Strength			Joint Strength			Member Strength			
	Jnt	BS (kps)	Wave Ht (feet)	Jnt	BS (kps)	Wave Ht (feet)	Jnt	BS (kps)	Wave Ht (feet)	1st Jnt	2nd Jnt	BS (kps)	Wave Ht (feet)
EndOn	101	3303	69.1	107	3560	71.0	212	1537	52.3	512	601	1179	47.3
Diag	103	2433	61.2	105	2768	64.3	212	1804	54.9	105	205	1269	48.3
Broad	107	3281	69.3	101	3372	70.0	211	1504	52.2	407	311	1276	49.0

A Simplified Ultimate Strength (SUS) analysis (*"An Automated Procedure for Platform Strength Assessment,"* OTC 7474 (1994) w/ F_y=42 ksi, K_{diag}=0.65, L_{eff} = center to center, first component failure) was also performed. The wave height and current associated with the first component failure exceed that of Draft Section 17 ultimate strength analysis, and therefore PLT-B would pass assessment using an SUS. The results of the SUS analysis follow:

Table 5. Summary of First Component Failure using SUS

Wave	Pile Capacity			Pile Strength			Joint Strength			Member Strength			
Dir	Jnt	BS (kps)	Wave Ht (feet)	Jnt	BS (kps)	Wave Ht (feet)	Jnt	BS (kps)	Wave Ht (feet)	1st Jnt	2nd Jnt	BS (kps)	W I (f
EndOn	101	5114	82.8	101	5345	84.3	212	2983	66.0	107	207	2544	6
Diag	103	4014	73.2	103	4804	80.0	211	3345	68.0	105	205	2151	5
Broad	107	5057	82.9	107	5684	87.1	211	2857	65.6	101	201	2368	6

A.5.4 Ultimate Strength

A nonlinear model of PLT-B was analyzed to assess PLT-B on the basis of an ultimate strength analysis and to determine its ultimate capacity. PLT-B clearly passed assessment. In fact, Draft Section 17 wave loads were not sufficient to cause any inelastic events. The wave load profile was then increased by a factor of two in order to determine PLT-B's ultimate capacity. Past experience showed that the ultimate capacity of a platform is not very sensitive to the choice of load profile. Figures A.5.4-1-3 plot lateral load versus deck displacement for the ultimate case. Table 6 gives a summary of the ultimate capacity results. Tables 7-9 list the initial components to yield for each loading condition.

Table 6. Summary of Ultimate Strength Analysis

Dir	Sudden Hurricane 100 yr (Ult criteria)	First Yield	Ult Cap	RSR	F_{robust}	Robustness
End-On	1800	2660	2660	1.66	2280	1.67
Diagonal	2100	2750	3500	1.52	2960	1.18
Broadside	1700	2650	2650	1.02	2130	1.24

Where:
RSR = Ult Capacity / (100 yr - 20th ed.)
ULR = Ult Capacity / (Case with DLA = 1.0)
LRF = Load Reduction Factor = (UC = 1.0 for 20th ed. / 100 yr - 20th ed.)
F_{robust} = Lateral load supported by portal frame action of jacket
Robustness = Ult Cap / F_{robust}

Table 7. Initial Failure Elements for End-On Wave Loading

Load Step	Lateral Deck Disp. (feet)	Lateral Load (kips)	Element Name	Element Type	Failure Description
13	1.4	2659	Diags1-97	Strut	Buckling
35	1.3	2313	Diags1-93	Strut	Buckling
56	1.3	2141	Diags1-91	Strut	Buckling
80	1.3	2148	Diags2-113	Strut	Buckling
113	1.2	1904	Diags2-111	Strut	Buckling
154	1.2	1755	Diags2-115	Strut	Buckling

The End-On case is governed by strut buckling. Several struts at various levels all fail at about the same base shear. Therefore, when one strut fails, the others fail at a lower base shear. Failing at the same base shear results in the ultimate capacity to be the same as the first component failure. Once the vertical diagonals fail, the platform is supported by a portal frame action of the legs.

Table 8. Initial Failure Elements for Diagonal Wave Loading

Load Step	Lateral Deck Disp. (feet)	Lateral Load (kips)	Element Name	Element Type	Failure Description
10	1.5	2750	FN_pil-2362	Beam Column	Initial Yield
11	1.6	2800	FN_pil-2334	Beam Column	Initial Yield
			FN_pil-2339	Beam Column	Initial Yield
13	1.7	2916	FN_pil-2308	Beam Column	Initial Yield
			FN_pil-2365	Beam Column	Initial Yield
			FN_pil-2366	Beam Column	Initial Yield
			LG_pil-329	Beam Column	Initial Yield
14	1.7	3003	FN_pil-2312	Beam Column	Initial Yield
			FN_pil-2337	Beam Column	Initial Yield
			FN_pil-2338	Beam Column	Initial Yield
			FN_pil-2342	Beam Column	Initial Yield
			FN_pil-2343	Beam Column	Initial Yield
			LG_pil-315	Beam Column	Initial Yield
15	1.8	3110	FN_pil-3411	Beam Column	Initial Yield
205	2.4	3512	Diags2-115	Strut	Buckling

The diagonal case has the greatest ultimate capacity. Foundation piles initially yield in tension and compression (not buckling) but still resist load. This resistance results in the ultimate capacity being greater than the initial yield value.

Table 9. Initial Failure Elements for Broadside Wave Loading

Load Step	Lateral Deck Disp.	Lateral Load	Element Name	Element Type	Failure Description
	(feet)	(kips)			
15	1.388	2654	DiagsB-80	Strut	Buckling
33	1.289	2360	DiagsB-78	Strut	Buckling
65	1.260	2180	DiagsB-76	Strut	Buckling
105	1.370	2255	DiagsA-66	Strut	Buckling
107	1.370	2255	DiagsA-68	Strut	Buckling
149	1.295	1870	DiagsA-70	Strut	Buckling

The broadside ultimate capacity loading is very unique in comparison to the 20th ed. loading. Draft Section 17 uses a rosette in which the broadside factors for PLT-B are 0.95. The 20th ed. rosette is rotated by 45 (using a factor of 1.0) and resulted in a much larger wave. This difference causes the 20th ed. wave load forces to be substantially higher than the Draft Section 17 waves.

A.6 Consideration of Mitigations

PLT-B passes assessment, therefore mitigation alternatives were not considered.

A.7 Summary Note - Part A

Platform Categorization

PLT-B was installed in 1971. It is a Manned-Evacuated structure with Insignificant Environmental Impact and will therefore use sudden hurricane criteria. There has been no change in the structure since its installation and therefore would pass assessment according to Draft Section 17.2.

DLA

The maximum UC for a member was 0.83 Several joints had UC greater than 1.0 (max=1.15). The platform therefore failed the DLA portion of platform assessment. Because joint criteria appears to be conservative it is believed that PLT-B would have passed assessment with a DLA.

An SUS analysis was performed. PLT-B passed assessment using SUS.

Ult Capacity

The critical components for the End-On case were the vertical diagonals. These diagonals all fail at about the same base shear. Therefore, the Ult Capacity was identical to the first component failure. PLT-B has little redundancy in the End-On direction.

Initial yielding occurred in the piles for the Diagonal case. This yielding did not cause failure, but the piles continued to resist some load. The ultimate capacity was reached when the vertical diagonals began to buckle. PLT-B has a lot of redundancy in the Diagonal direction.

The Broadside resistance and capacity is very much like the End-On case. The loads for the sudden hurricane are approximately the same as well. The loads from the 20th edition Broadside case (Hmax = 64') are much greater than the End-On case (Hmax = 55') due to the rosette factors. This difference results in a vastly different RSR.

The following table summarizes the results for all cases:

Table 10. Summary of Results - Base Shear (kips)

Wave Dir	Wave Forces			Structure Resistance			Ratios / Factors		
	9th ed.	20th ed.	Sudden Hurricane	DLA = 1.0	SUS	Ult Cap.	RSR	ULR	LRF
End-On	1460	1600	1800	1180	2540	2660	1.66	2.25	0.74
Diag	1387	2300	2100	1270	2150	3500	1.52	2.76	0.55
Broadsd	1460	2600	1700	1280	2370	2650	1.02	2.07	0.49

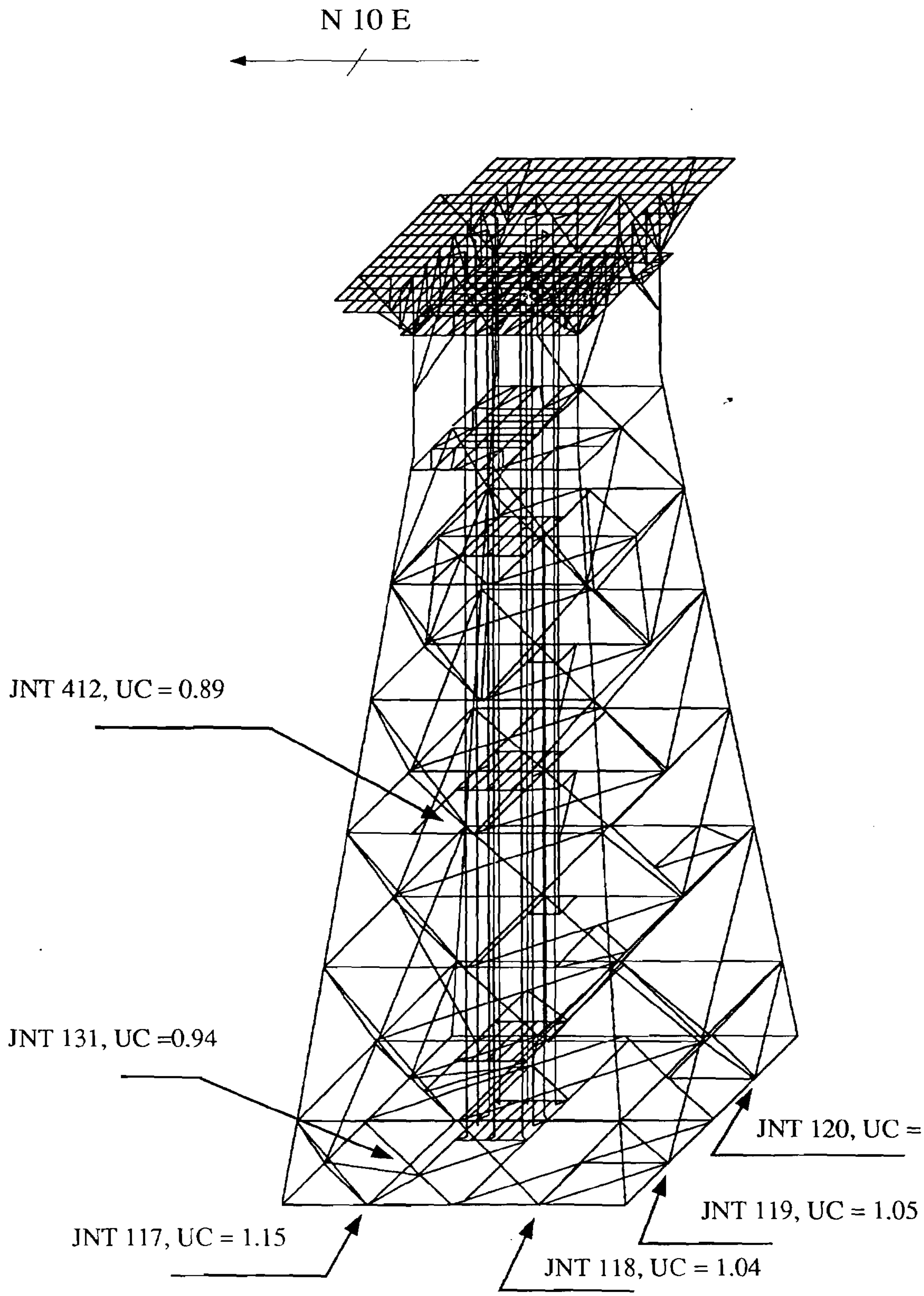


Figure A.5.3-1. PLT-B components with Unity Checks > 0.85. *View from the West*

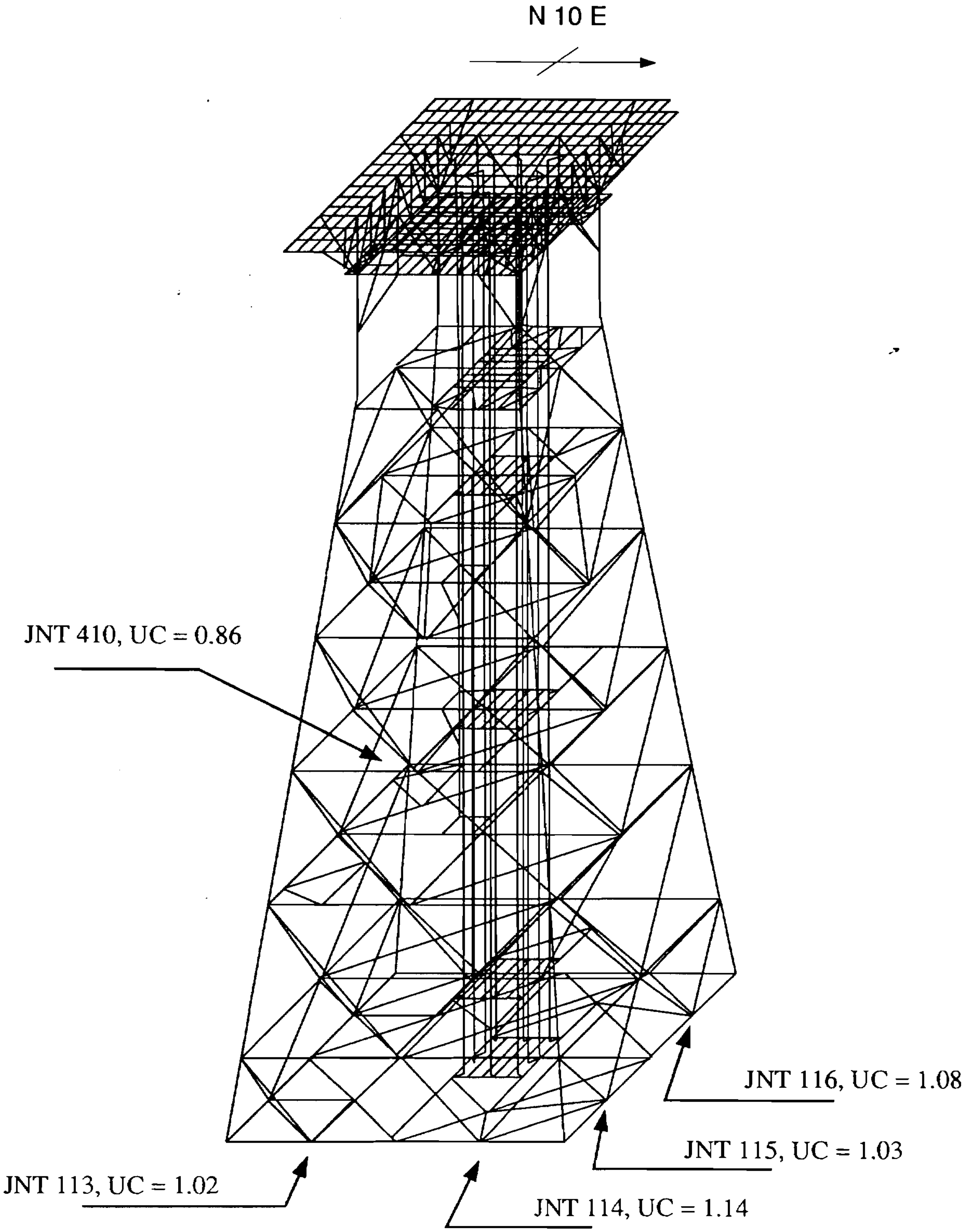


Figure A.5.3-2. PLT-B Components with Unity Checks > 0.85. *View from the East.*

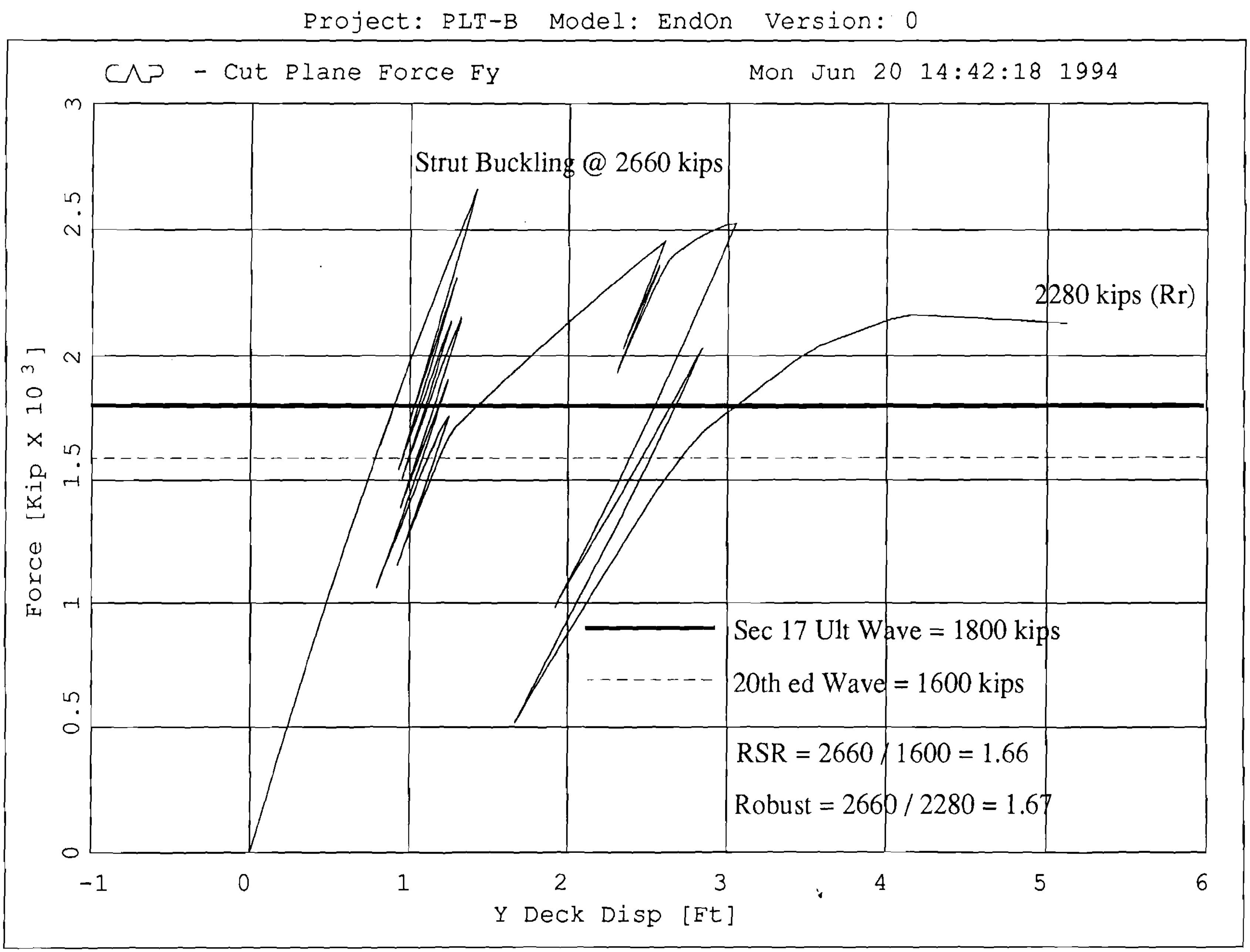


Figure A.5.4-1 PLT-B Forces vs. Deck Displacement - End-On Wave. *Wave forces greater than section 17 to determine ultimate capacity*

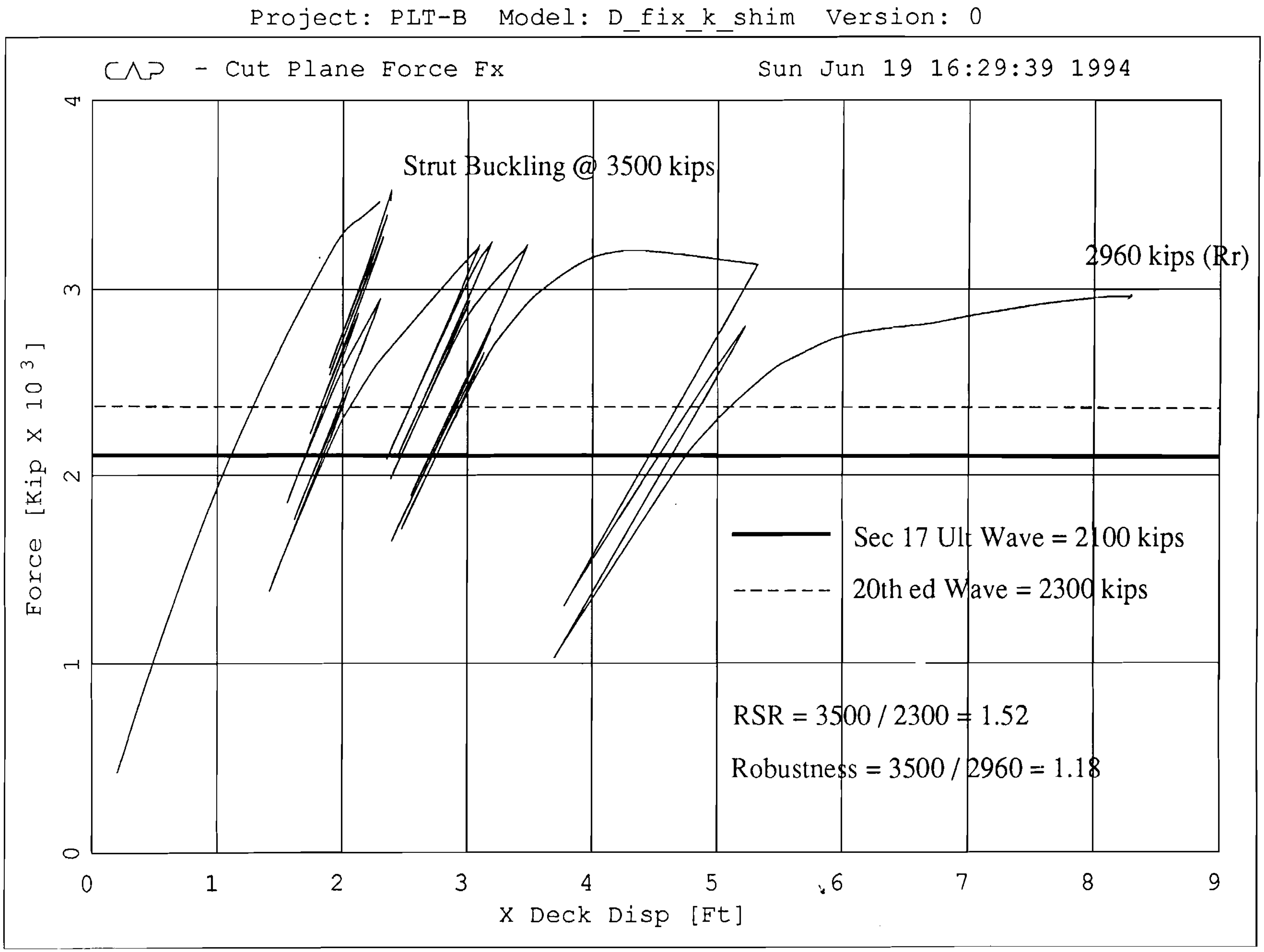


Figure A.5.4-2 PLT-B Forces vs. Deck Displacement - Diagonal Wave. *Wave forces greater than section 17 to determine ultimate capacity*

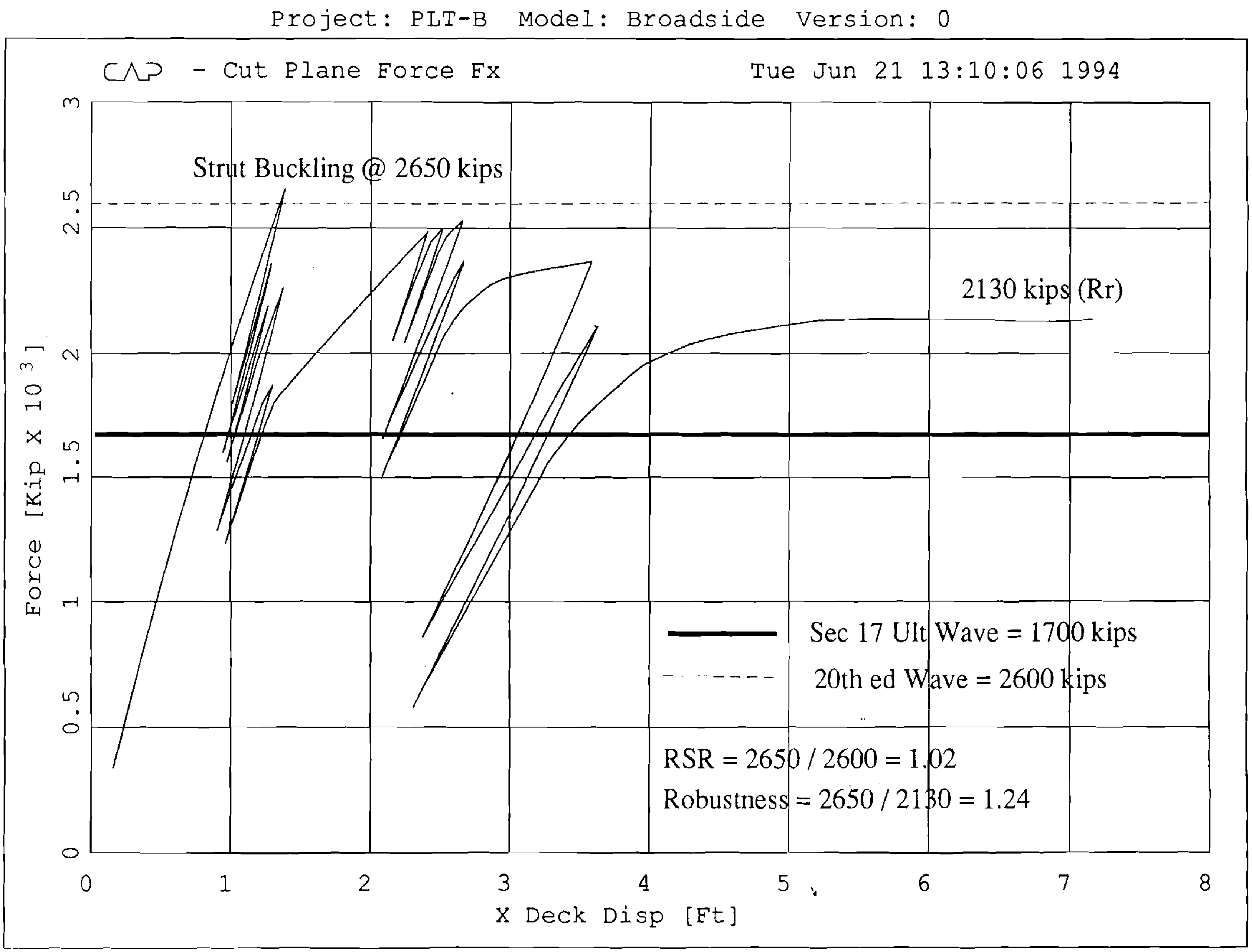


Figure A.5.4-3 PLT-B Forces vs. Deck Displacement - Broadside Wave. *Wave forces greater than section 17 wave to determine ultimate capacity*

PART B: Review and Feedback to the API TG 92-5 (April/94 version of Draft Section 17)

- Section 17.2 states, "An existing platform should undergo the assessment process if one or more of the conditions noted in Sections 17.2.1 through 17.2.4 exits. Sections 17.2.1 through 17.2.4 consider 'Addition of Personnel', 'Addition of Facilities', 'Increased Loading on the Structure', and 'Significant Damage.' Please consider adding that platform assessment may also be required from an MMS initiated assessment.

- Figure 17.5.2 Page 6, Note 1: *'Design Level Check'*. It is not clear if what is meant is a 'Design Level Analysis' or 'Design Basis Check'.

- Sections 17.7.3a and 17.7.3b (P. 26) are not clear. Is a Linear Global Analysis the same as a Simplified Ultimate Strength Analysis? Is a Local Overload Analysis simply considering removing over stressed members and rerunning the Linear Global Analysis? Could these sections please be rewritten?

- Fig 17.6.2-4 (P 20) contains a rosette entitled: "Sudden Hurricane Wave Directions and Factors to Apply to the Omnidirectional Wave Heights in Fig. 17.6.2-3a for Ultimate Strength Analysis." Is this for currents also? Does it apply only for deep water?

- Are there any comments on dynamic analysis for deep water platforms (other than fatigue)?

PART C: Miscellaneous Information

During the trial application it became apparent that not all of information required would be easily attainable. The lack of reliable data could affect the results on the assessment of other platforms. A few areas of concern are:

- **Obtaining complete and readable drawings.** For many older platforms, the quality of drawings is not very good.

- **Determining if a structure is grouted or not.** Whether a structure is grouted or not cannot always be determined from the structural drawings. Other evidence such as grout lines (or lack there of) may be used to determine if a structure is grouted.

- **Determining the pile penetration.** Without adequate pile driving reports, the pile penetration be determined accurately.

- **Determining the soil profile close to the structure.** Many soil boring information logs are not available.

PLATFORM "C"

EUGENE ISLAND BLOCK 330C
DRILLING AND PRODUCTION PLATFORM
255 FT. WATER DEPTH

EXISTING PLATFORM ASSESSMENT
TRIAL APPLICATION DOCUMENT

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EXECUTIVE SUMMARY

API Task Group 92-5 has developed a draft version of API RP 2A Section 17.0 – Assessment of Existing Platforms. This is a product of collective oil industry expertise with contributions mainly from members of the API Task Group 92-5 as well as other API members. A joint industry project (JIP) was proposed by PMB Engineering of San Francisco, California for the purpose of testing the methodology presented in the draft recommendations. This program is sponsored by the Minerals Management Services (MMS) with the participants mainly from oil companies and/or operators and consulting engineering companies.

As a participant,
perform the trial applications of API RP 2A Section 17.0 (draft) using one of its existing platforms. The selected platform is the "C" drilling and production platform located at Eugene Island Block 330 in the Gulf of Mexico in 255 feet of water.

The platform information was provided by which included as-is condition details such as physical features, operational information, pertinent inspection information, structural assessment data, soil boring logs and shear strength profile. The platform sketches were generated from the as-built drawings.

As the first phase in the platform assessment, the platform selection, condition assessment, categorization and design basis checks were evaluated. Secondly, the analysis checks were performed for the design level and ultimate strength (both linear and inelastic) conditions. Finally, from the results of the analyses (code checks and static push-over RSR's, etc.), the consideration of mitigation was addressed.

For the purpose of this study, the platform is categorized as "insignificant environmental impact/manned - evacuated", and the metocean criteria/loads corresponding to that category are used. A design level analysis and two ultimate strength analyses are performed.

Design Level Analysis

For the design level analysis, both "tubular member checks" and "joint checks" are performed. The results of the tubular member check showed that most of members with a stress ratio exceeding 0.85 were governed by hydrostatic conditions. Only five members (Member Nos. 272, 273, 422, 560 and 566) with stress ratios exceeding 0.85 were governed by strength check conditions. The tubular member check results were satisfactory, i. e., stress ratios did not exceed unity (other than the mudline framing level members mentioned later). See Section A.5.3.5 for further explanations.

The joint check results showed several joints with stress ratio exceeding 1.0. After reviewing the results, it was concluded that a refined model would reduce those stress ratios, especially those joints which are part of the conductor framing at the mudline. See Section A.5.3.6 for further details.

Ultimate Strength Analyses

For the ultimate strength analysis, Section 17 allows for either a linear (elastic) or inelastic (nonlinear) analysis to be performed. The analysis to be performed for a particular case is determined mutually by the Owner and the Design Consultant.

For this study, prior to performing the inelastic static push-over analysis, an alternative or pseudo ultimate strength analysis and code check was performed using the linear model (design level analysis). The loads used for the linear analysis were determined from the ultimate strength metocean criteria as called for in Section 17. The tubular member checks and joint checks were performed using modified checking equations to remove the factors of safety. The tubular member check results (using Fy = 36 ksi) indicate that the stress ratios of some horizontal members are slightly higher than that of the design level analysis. However, the stress ratios of vertical braces are much less than that of the design level analysis. Overall, the results of tubular member check were satisfactory except the mudline framing level members mentioned later. The joint check results (using Fy = 42 ksi) show that the stress ratios are much less than that of design level analysis. See Section A.5.4.6 for the results of both tubular member and joint checks.

For the inelastic, static push-over analysis, four wave directions were considered (195, 240, 180 and 270 degrees) using the appropriate 100-year lateral load for each direction. The calculated platform reserve strength ratio (RSR) varied between 1.18 and 1.39 depending on the wave direction considered. The RSR is defined as the ratio of a platform's ultimate lateral load-carrying capacity (as determined by the static pushover analysis) to its 100-year environmental lateral loading condition determined by the present API RP 2A procedures (20th edition). The lowest RSR is **1.18** which occurred in the 195 degree wave direction. See Sections A.5.4.7 through A.5.4.11 for further detail.

The ultimate metocean lateral loads as described above were also computed for these same four wave directions and compared to the platform's ultimate lateral load-carrying capacity in each direction. These ratios of factors (ultimate lateral load-carrying capacity/ultimate metocean lateral load) varied between 1.4 and 2.4 (+) depending on the wave direction.

Ultimate strength (inelastic analysis) joint checks were not performed as it was assumed the joint strength was sufficient to develop the failure capacity of the adjoining member.

Fatigue Analysis

The draft Section 17 does not require a fatigue analysis in those cases where sufficient inspection has been performed (Level II or greater). For the "C" platform it was assumed that sufficient inspection had been performed and, therefore, a fatigue analysis was not required.

Consideration of Mitigation

For the design level analysis, results of the tubular member check for the horizontal framing at the mudline (EL. (-) 254'-0) showed several members with stress ratio exceeding 1.00. It appears that this was due to modeling inaccuracies. It is recommended that further investigation on those horizontal members (at the mudline) be carried out. However, at this point, no consideration of mitigation is immediately required. The same argument is also applied to the two ultimate strength analyses.

The joint check results (both from the design level analysis and the pseudo ultimate strength analysis) showed that the joints with a stress ratio exceeding either 0.85 or even 1.00 are mainly K-joints. Again, the use of an improper modeling approach appears to be the main cause for the resulting high stress ratios for the overlapping joints and for the mudline level joints mentioned previously. A refined model should be developed for a further investigation, especially on those joints at the mudline horizontal framing. Furthermore, a finite element analysis of some selected joints is recommended before any physical mitigation is initiated.

In the inelastic static push-over analysis, it was assumed that the joint capacity could reach or exceed its full member-end strength requirements either by refining the model for further analysis or by any other measure that is feasible and practical.

1.0 PLATFORM INFORMATION

1.1 AS-IS CONDITION DETAILS

1.1.1 Physical Features

The "C" structure is an 8-leg, 8-pile, drilling and production platform with 21 wells (18 original plus three recently added wells) in 255 feet of water located at Eugene Island Block 330, Gulf of Mexico.

The platform is composed of deck, jacket and piles. The deck consists of drilling and production levels and both are 136 feet long by 72 feet wide. The top of steel for the production and drilling deck levels is at El. (+) 45'-6" and El. (+) 63'-11" respectively.

The jacket which serves as a template for the eight 42-inch diameter piles is 277'-7/16" tall, 40'-7 1/2" x 110'-7 1/2" at top of jacket and 175'-6" x 105'-6" at the base. The bracing type used in the vertical frames is either a K-brace or a single diagonal brace. No grout was provided between the jacket-leg and pile.

Each 42-inch diameter pile has a wall thickness of 2 1/2" at the mudline. The pile wall thickness is incrementally decreased from 2 1/2" at the mudline to 7/8" in the lower portion of the pile. The pile penetration of the four corner piles and four interior piles is 360' and 312', respectively.

1.1.2 Operational Information

The platform was installed in 1972 and has been operating as a "manned-evacuated" platform since that time. It is expected, in the event of hurricane, that the personnel on the platform will be evacuated. Since there is no H_2S or sulfur production nor any significant oil storage on the platform, the platform is classified within the "*insignificant* environmental impact" category.

1.1.3 Pertinent Inspection Information

The platform has been maintained and inspected regularly following normal industry practice and guidelines required by the MMS. Most of the inspection data on this platform can be obtained through the office

1.1.4 Structural Assessment Data

A complete set of as-built structural drawings has been provided by in this study. In addition, the wave report prepared by the metocean consultant, A. H. Glenn specifically for this platform was made available Previously, in 1990, , to reassess the strength of the platform due to the proposed installation of three additional wells in the northeast side of the platform. Therefore, the structural model could be and was retrieved from the archive files in

1.1.5 Soil Boring Logs and Shear Strength Profile

The information on the soil boring logs, shear strength profile, P-Y data and pile driving records for this platform are available. However, the soil information, such as T-Z and Q-Z curves was retrieved from the existing structural model which was developed previously by using their MicroSAS program. MicroSAS is a McDermott computer program specifically developed for offshore platforms design and analysis.

1.2 PLATFORM SKETCHES

1.2.1 Platform Computer Model Orientation

The global coordinates system of the "C" platform model is defined by Cartesian coordinates (X, Y, Z). The X-axis is oriented parallel to the longitudinal direction with (+) X to the platform South. The Z-axis is oriented parallel to the transverse direction with (+) Z to the platform West. The Y-axis is oriented in the vertical direction with the (+) Y upward. The origin of the global structural coordinates system is located at the geometric center of the horizontal framing with Y = 0 at the mudline.

The platform orientation is such that the platform north is 10 degrees counter clockwise from the true north. The positive X-axis is along the platform south direction, while the positive Z-axis is along the platform west direction. The positive Y-axis is upward from the mudline (using right-hand rules).

See Figure 1 - Platform Orientation and Wave Directions.

NOTE: The current direction shown in Figure 1 corresponds to the shallow water (depth < 150') as provided in API RP 2A 20th edition. For water depth between 150' and 300', the actual current direction must be calculated per the provision of API RP 2A 20th edition.

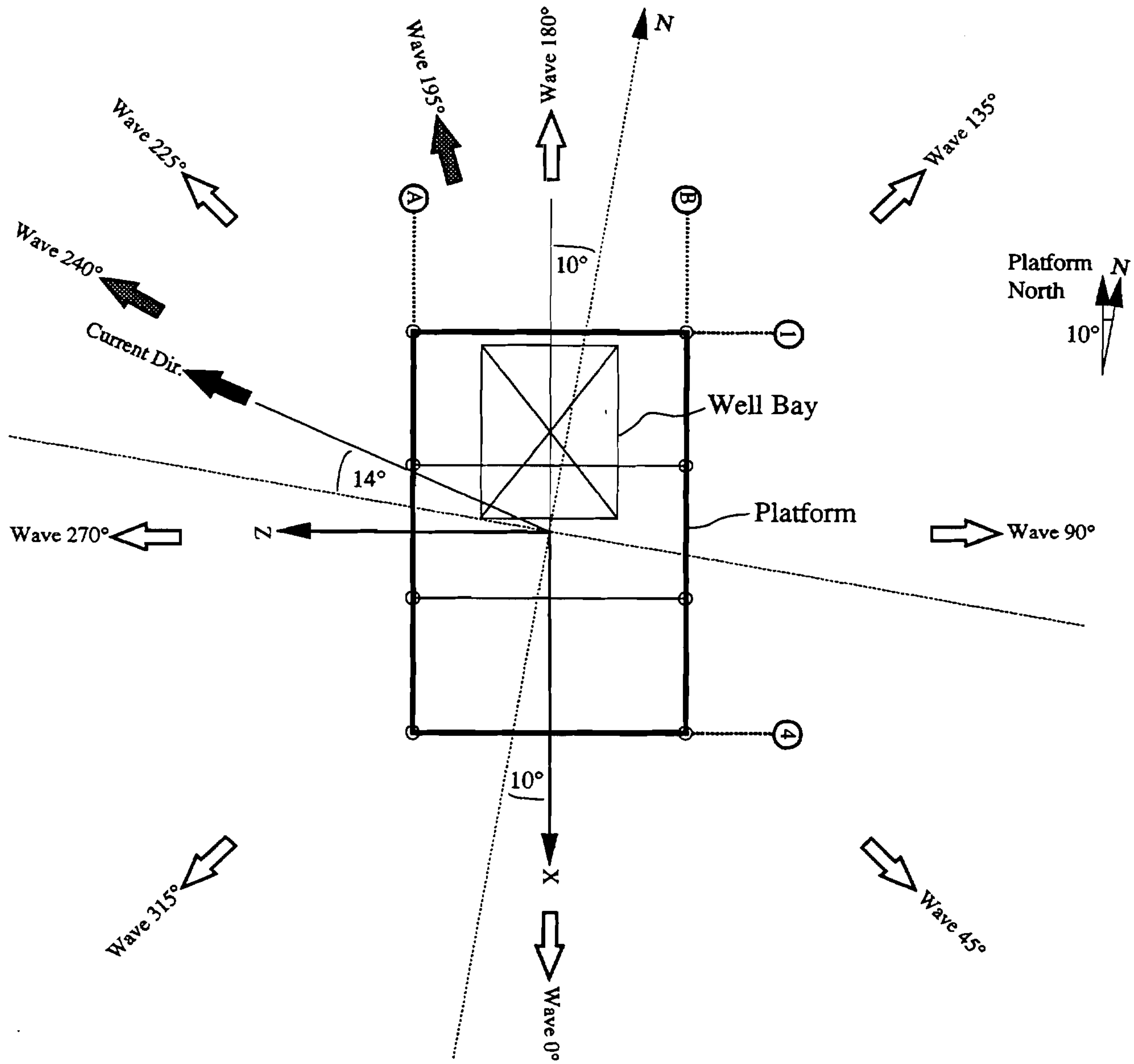


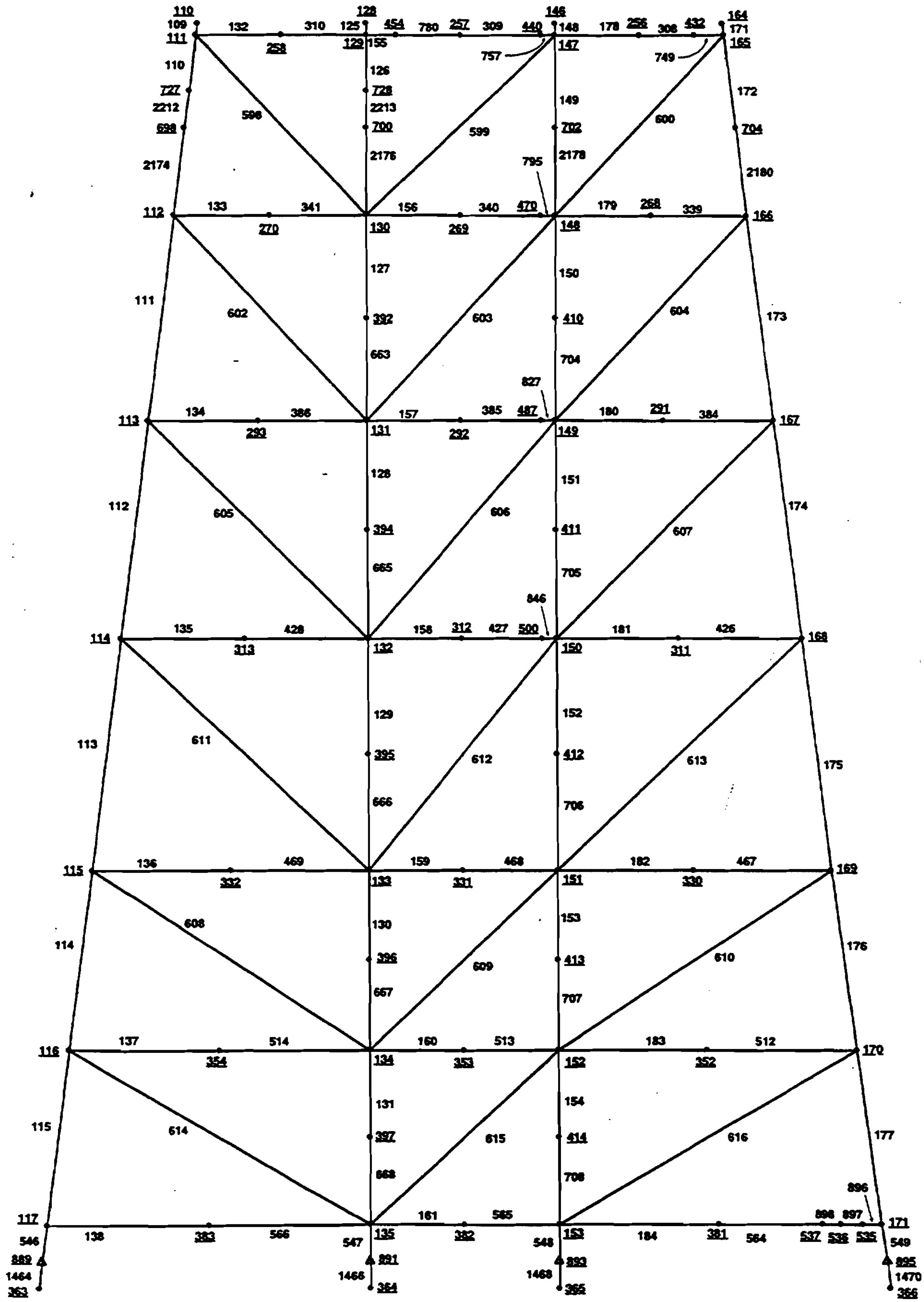
Figure 1. Platform Orientation and Wave Directions

1.2.2 Vertical Framing Elevations

The structural framing system is identified by Row A and Row B in the longitudinal direction and Rows 1 through 4 in the transverse direction. See Figure 1 - Platform Orientation and Wave Directions for further detail.

The structural model plots of vertical framing elevations of jacket and deck (Row A, Row B, Row 1, Row 2, Row 3 and Row 4) are shown in the following attachments.

The structural model was generated from the as-built structural drawings. The actual structural member properties, size and lengths can be found in the Member Data and Joint Data in the Appendix.



JACKET
ROW A

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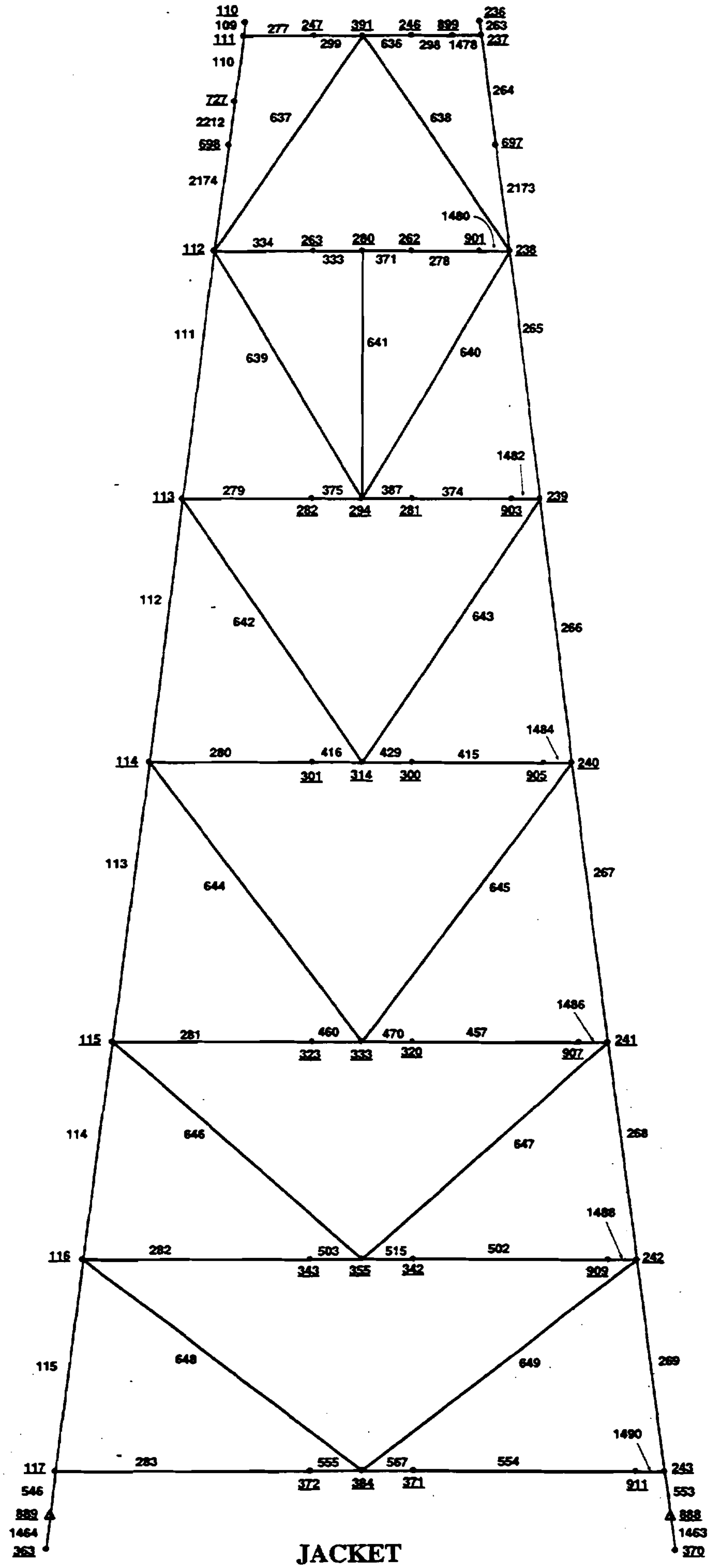

JACKET
ROW 1

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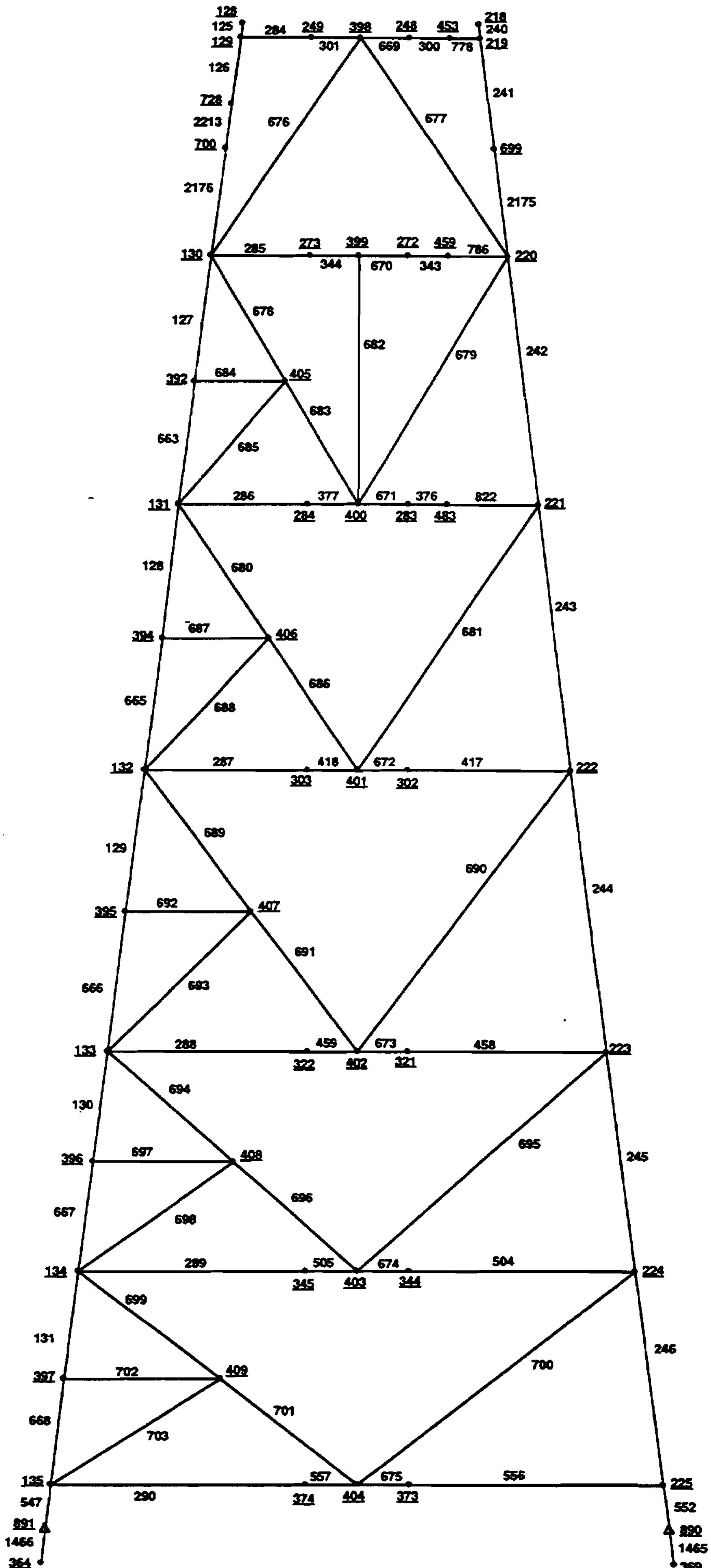

JACKET
ROW 2

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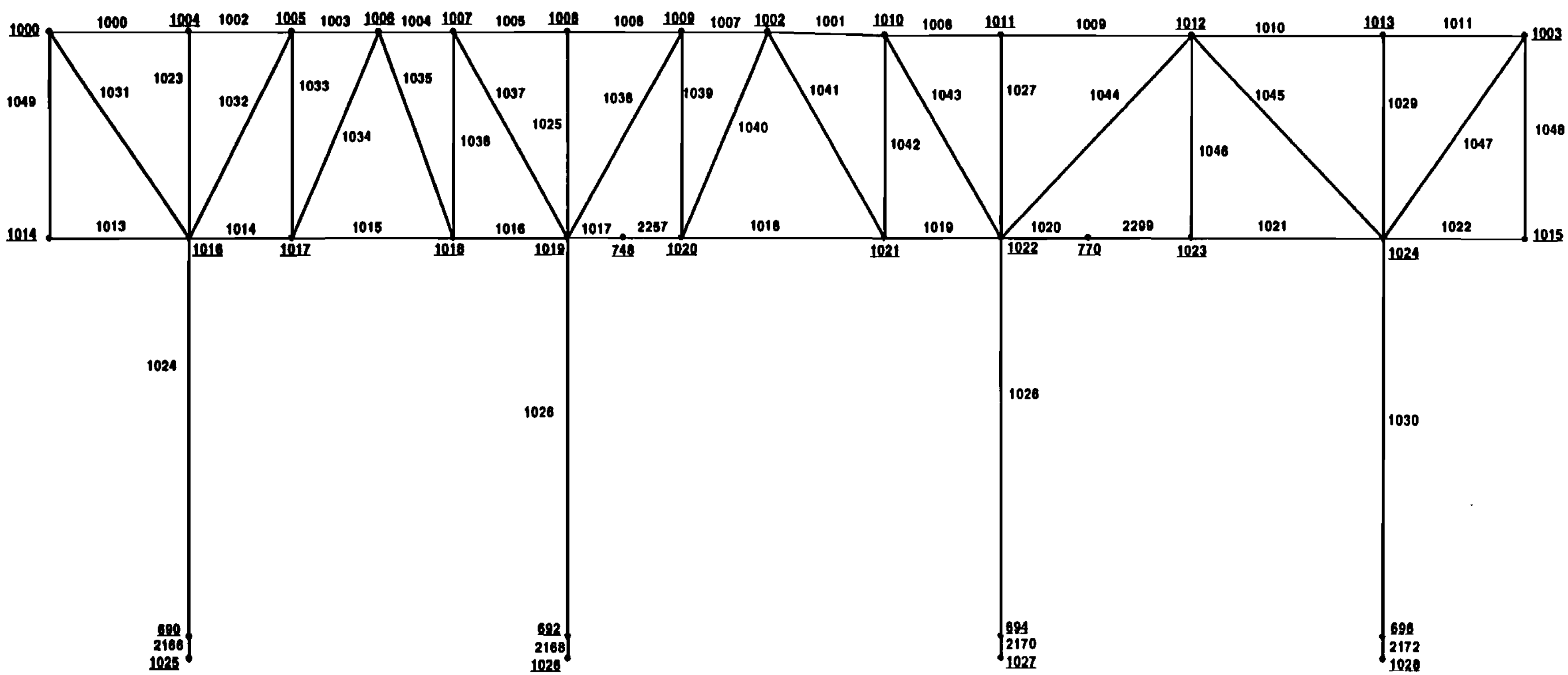
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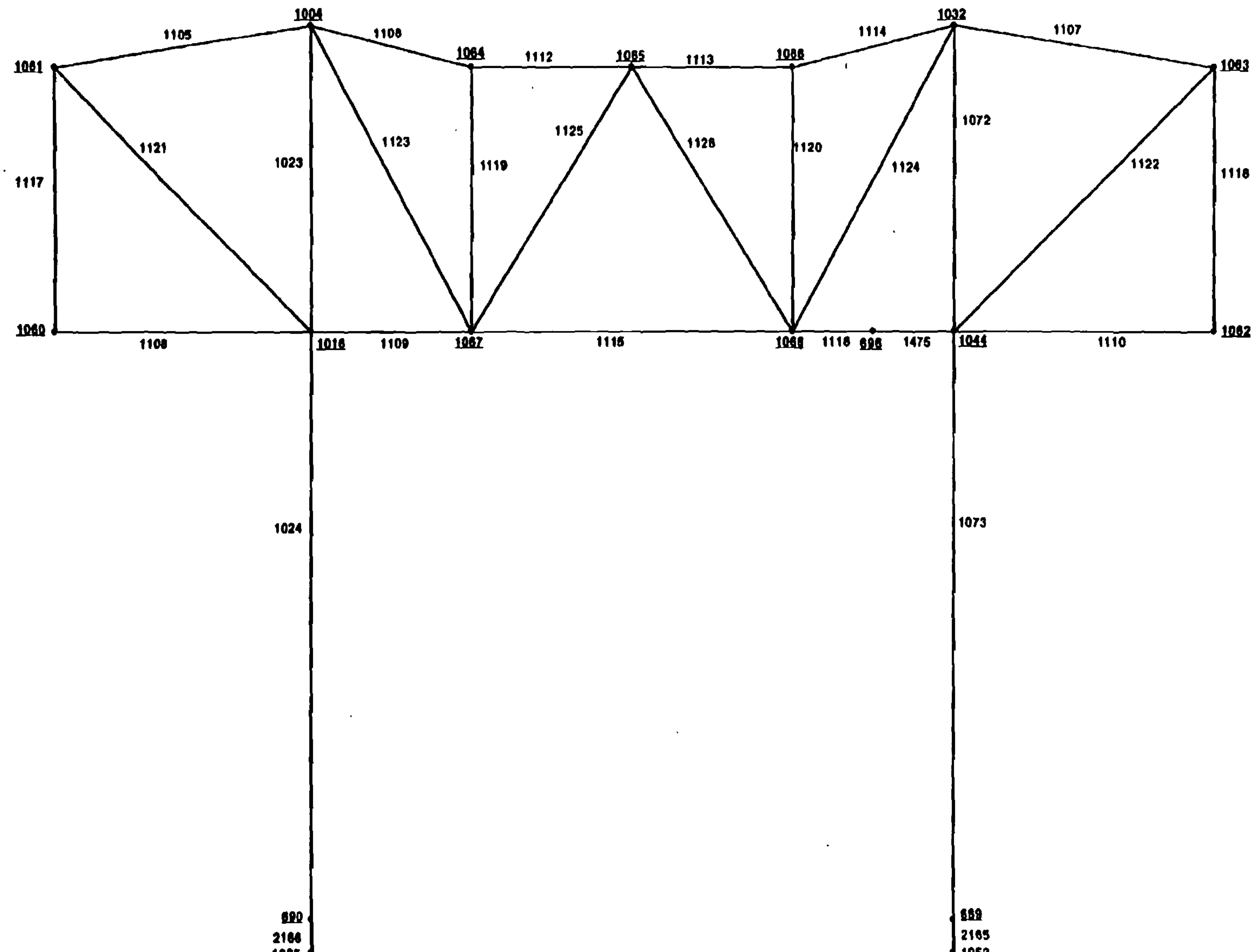
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DECK

1.2.3 Horizontal Framing Plans

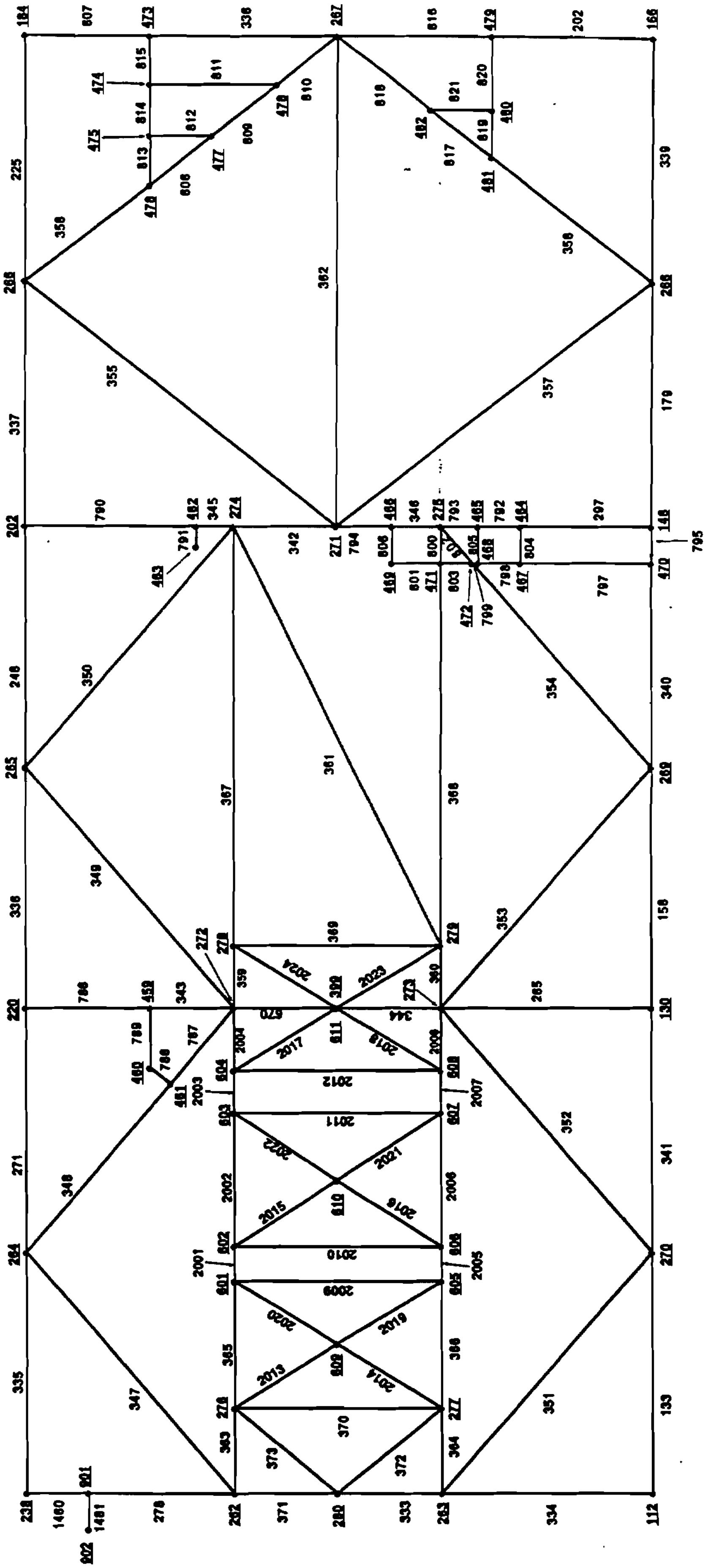
The horizontal framing plans are identified as "Elevations" in reference to Mean Low Water (MLW) level. The following is the list of horizontal framing plans used in the as-built drawings as well as the structural model:

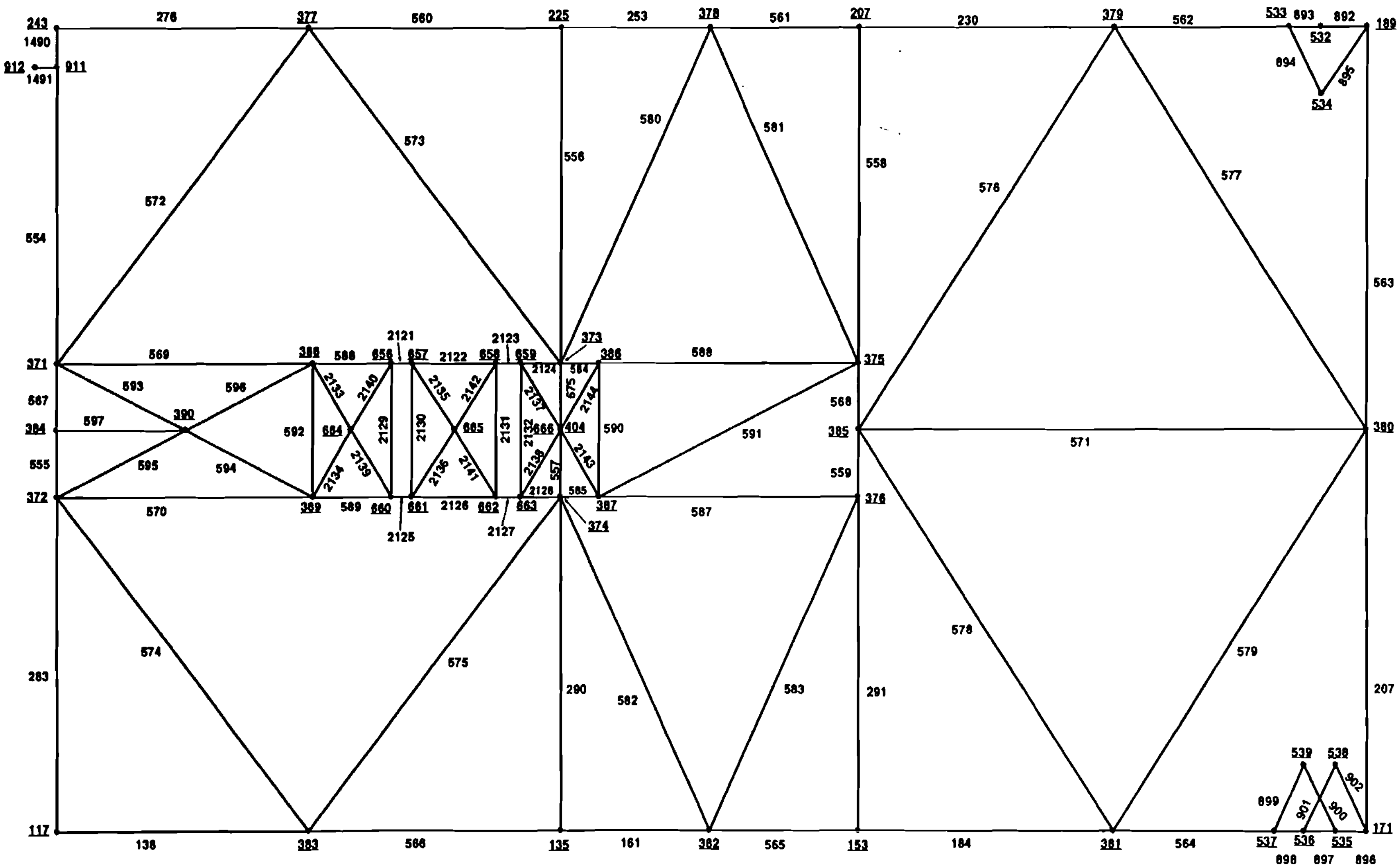
- Jacket
  - EL. (+) 3' - 0"
  - EL. (-) 36' - 0"
  - EL. (-) 80' - 0"
  - EL. (-) 127' - 0"
  - EL. (-) 177' - 0"
  - EL. (-) 216' - 0"
  - EL. (-) 254' - 0"

- Decks

Drilling Deck -	EL. (+) 71' - 10 7/8"	(original drawing)
Production Deck -	EL. (+) 53' - 6"	(original drawing)

The structural model plots of the jacket horizontal framing elevations and drilling and production decks are shown in the following attachments.

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897
896

EL (-) 254' - 0''

1.2.4 Pile Make-up and Details

There are two sets of pile make-ups. One set for external piles (total four) and another set for inner piles (total four) due to different pile penetrations.

1) External Piles: (A-1), (A-4), (B-1) and (B-4)
   Pile Penetration: 360'
   Pile Make-up: (Pile Material Fy = 36 ksi)

Pile Diameter (inch)	Wall Thickness (inch)	Pile Segment Length (foot)
42	2.500	12.91 (from mudline)
42	2.250	50.00
42	2.000	10.00
42	1.750	10.00
42	1.500	10.00
42	1.250	60.00
42	1.000	30.00
42	0.875	165.00
42	1.250	12.09 (pile tip)

2) External Piles: (A-2), (A-3), (B-2) and (B-3)
   Pile Penetration: 312'
   Pile Make-up: (Pile Material Fy = 36 ksi)

Pile Diameter (inch)	Wall Thickness (inch)	Pile Segment Length (foot)
42	2.500	5.04 (from mudline)
42	2.250	50.00
42	2.000	10.00
42	1.750	10.00
42	1.500	10.00
42	1.250	60.00
42	1.000	30.00
42	0.875	125.00
42	1.250	11.96 (pile tip)

1.2.5 Other Information

- Number of conductors: 21
- Conductor size: 26" diameter x 1/2" wall thickness
- Material Yield Strength: Fy = 36 ksi

PART A: PLATFORM ASSESSMENT

A.1 PLATFORM SELECTION

API RP 2A, Section 17.0 (Draft), provides guidelines for determining when an assessment is required for existing platforms. Presently it states that a platform assessment is required if one or more of the following conditions exists:

1) Addition of Personnel
2) Increase Loading on Structure (Significantly)
3) Inadequate Deck Height
4) Significant Damage Found During Inspections

Significant Damage is defined as the point when the total of the cumulative damage or cumulative changes from the original design premise (increases in load) decreases the capacity or increases the loading in excess of 10%.

The "C" platform was originally designed to accommodate 18 wells. However, in 1990, three additional wells were installed on the platform. This resulting increase in loading on the structure did not exceed 10% (or was not significant), therefore, according to the new Section 17.0 (draft) guidelines mentioned above, a platform assessment might not be required.

A.2 CONDITION ASSESSMENT

The platform assessment information is very critical to the success of subsequent engineering analysis and assessment to the platform. In general, the original platform design file, as-built structural drawings, fabrication, welding and construction specifications, platform history, present conditions and soil data should be made available for review.

For the "C" platform, the as-built drawings and platform design file are available for review. The design wave data was provided by A. H. Glenn. A soil report was available, however, some of the soil data (T-Z, Q-Z curves) was obtained from previous design information prepared by in 1990. The platform assessment information is adequate for carrying out the subsequent engineering analysis.

A.3 CATEGORIZATION

Based on the guideline provided in API RP 2A Section 17 (Draft) - Assessment of Existing Platforms, the "C" platform can be categorized as "insignificant environmental impact" and "manned - evacuated." Since there is no H_2S or sulfur production nor any significant oil storage on the platform, the failure of the platform (structural or operational) is determined to have an *insignificant* environmental impact on conditions in the Gulf of Mexico. It is expected that in the event of hurricane, the personnel on the platform will be evacuated. Therefore, the platform can be considered as "manned/evacuated".

A.4 DESIGN BASIS CHECK

The "C" platform was installed in 1972 and API RP 2A 9th Edition was published in November, 1977. Since the "C" platform was designed and installed prior to the issue of the 9th Edition and since there has been an increase in load (addition of three conductors), it was concluded that the option of design basis check is not applicable to this platform. However, because of the increase in load, the more thorough Analysis Checks must be performed. The detailed analysis checks of the platform have been carried out in this study and documented in this report.

A.5 ANALYSIS CHECKS

A.5.1 Metocean Criteria / Loads

The metocean criteria are based on Table 17.6.2-1 of API RP 2A Section 17 (Draft) - Assessment of Existing Platforms. The design waves are based on sudden hurricane. The wave, current, wind and storm tide data required for the Design Level Analysis and both Ultimate Strength Analyses (linear and inelastic) are summarized as follows:

Water Depth: 255 feet

1) Design Level Analysis

Wave Height and Storm Tide:	47' (Fig. 17.6.2-3a), 2.5'(Storm Tide)
Deck Height:	36.25' (Fig. 17.6.2-3b)
Wave & Current Direction:	Omni-Dir.**
Current Speed:	1.2 knots (2.03 ft / sec)
Wave Period:	11.3 sec.
Wind Speed (1-hr @ 10m):	55 knots (63.29 mph)
Wave Crest:	26.26' (Omni Direction)

2) Ultimate Strength Analysis

Wave Height and Storm Tide:	59' (Fig. 17.6.2-3a), 2.5'(Storm Tide)
Deck Height:	36.25' (Fig. 17.6.2-3b)
Wave & Current Direction:	Fig.17.6.2-4 (see attachment)
Current Speed:	1.8 knots (3.04 ft / sec)
Wave Period:	12.5 sec.
Wind Speed (1-hr @ 10m):	70 knots (80.55 mph)
Wave Crest:	33.45' (Wave Dir. 195 deg.)

** If the wave height or current versus direction exceeds that required for ultimate strength analysis, then the ultimate strength criteria will govern.

For comparison purpose, the wave, current, wind and storm tide data required for new platform design (API RP 2A 20th edition) are presented as follows:

3) New Platform Design (Working Stress Design)

Wave Height and Storm Tide:	67' (Fig. 2.3.4-3, RP 2A 20th Ed.), Wave Dir. 240 deg. 3.2' (Storm Tide)
Deck Height:	48.8' (Fig. 2.3.4-8, RP 2A 20th Ed.)
Wave & Current Direction:	Figs. 2.3.4-4, 2.3.4-5 (RP 2A 20th Ed.)
Current Speed:	2.1 knots (3.54 ft / sec)
Wave Period:	13.0 sec.
Wind Speed (1-hr @ 10m):	80 knots (92 mph)
Wave Crest:	38.48' (Wave Dir. 240 deg.)

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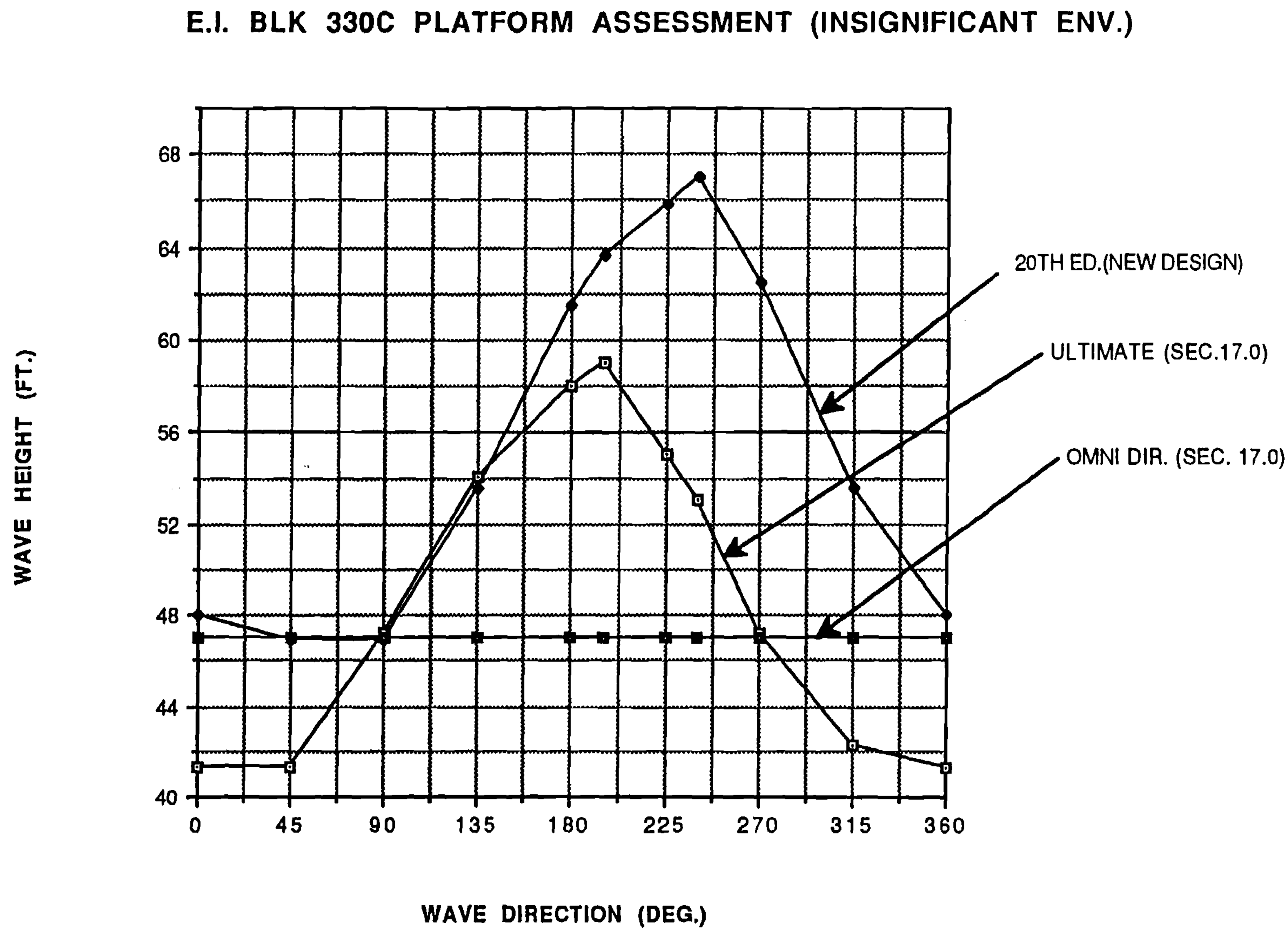
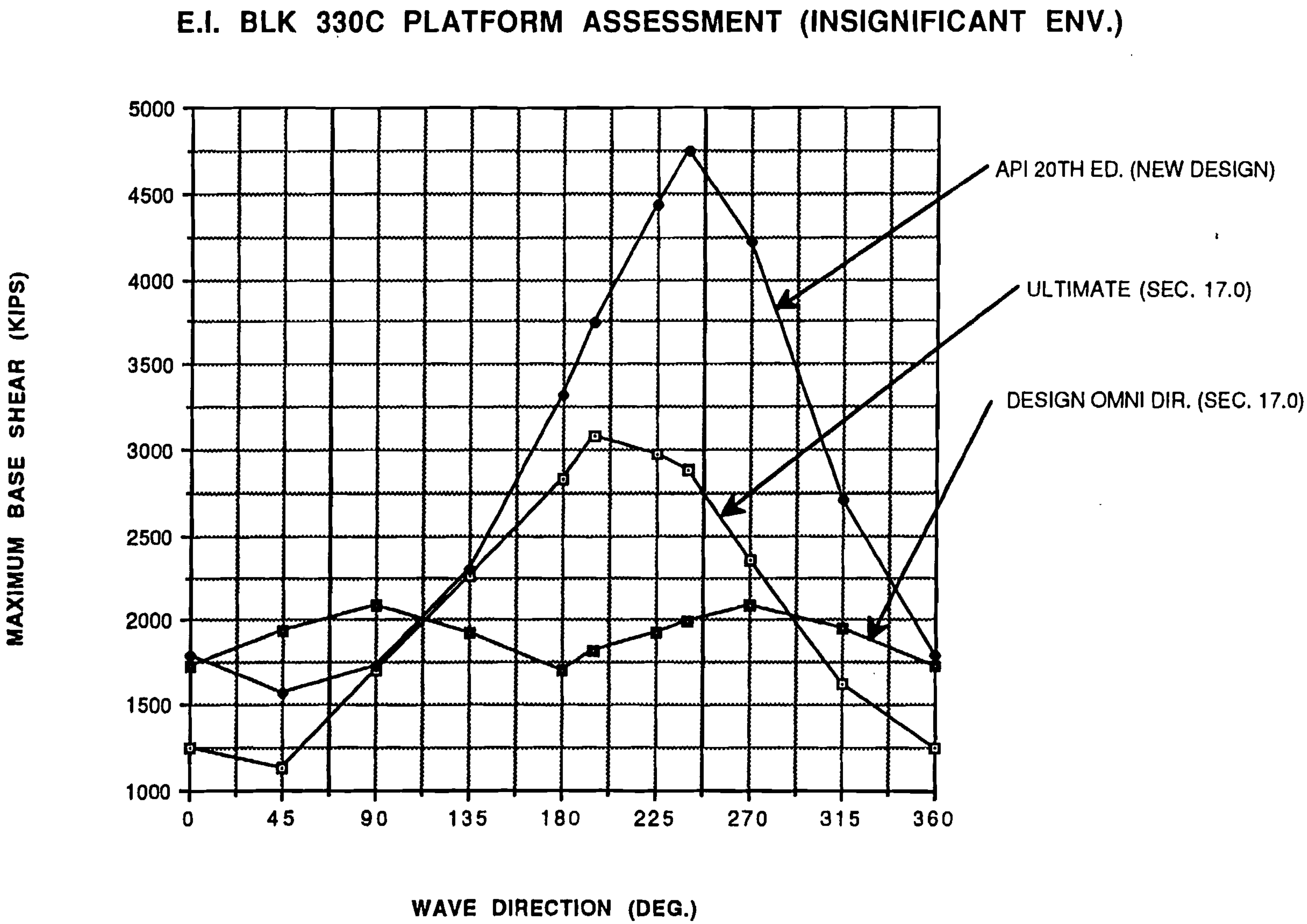

E.I. BLK 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)
WAVE HEIGHT (FT.)
68
64
60
56
52
48
44
40
0
45
90
135
180
225
270
315
360
WAVE DIRECTION (DEG.)
20TH ED.(NEW DESIGN)
ULTIMATE (SEC.17.0)
OMNI DIR. (SEC. 17.0)

FIG. 2



E.I. BLK 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)
5000
4500
4000
3500
3000
2500
2000
1500
1000
0
45
90
135
180
225
270
315
360
MAXIMUM BASE SHEAR (KIPS)
WAVE DIRECTION (DEG.)
API 20TH ED. (NEW DESIGN)
ULTIMATE (SEC. 17.0)
DESIGN OMNI DIR. (SEC. 17.0)

FIG. 3

E.I. BLK 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)

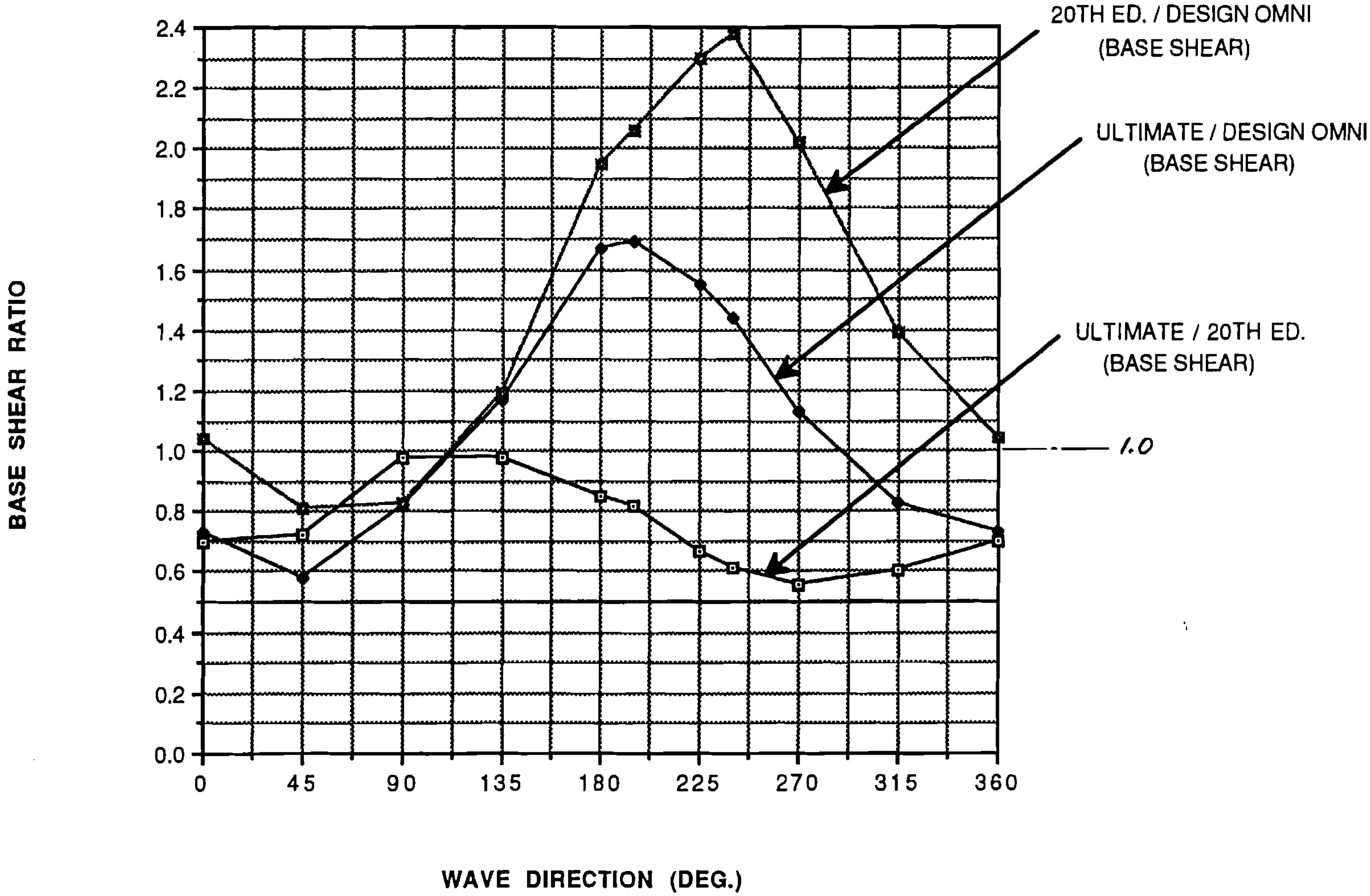
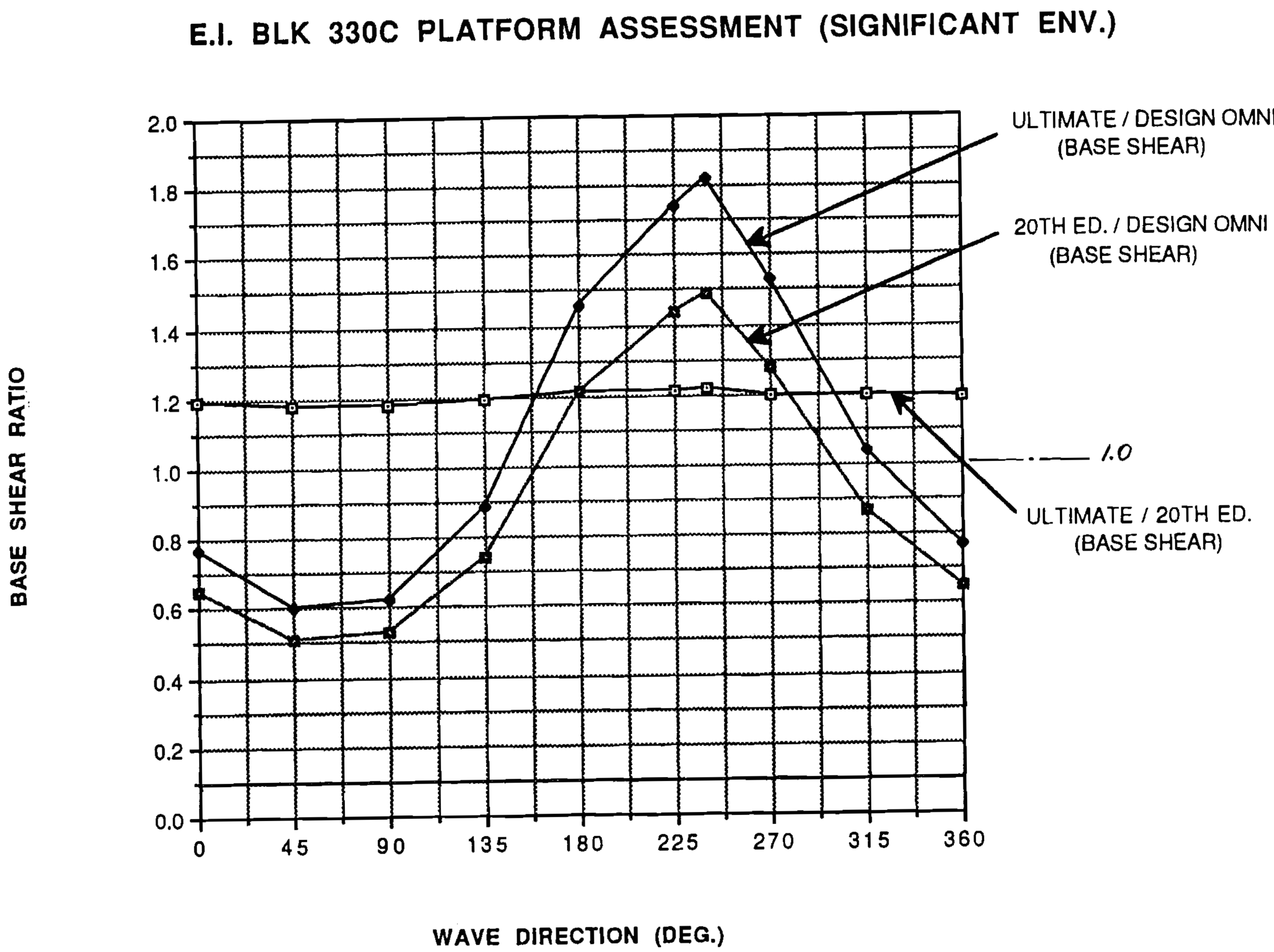


FIG. 4



E.I. BLK 330C PLATFORM ASSESSMENT (SIGNIFICANT ENV.)
BASE SHEAR RATIO
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
45
90
135
180
225
270
315
360
ULTIMATE / DESIGN OMNI (BASE SHEAR)
20TH ED. / DESIGN OMNI (BASE SHEAR)
1.0
ULTIMATE / 20TH ED. (BASE SHEAR)
WAVE DIRECTION (DEG.)

FIG. 4A

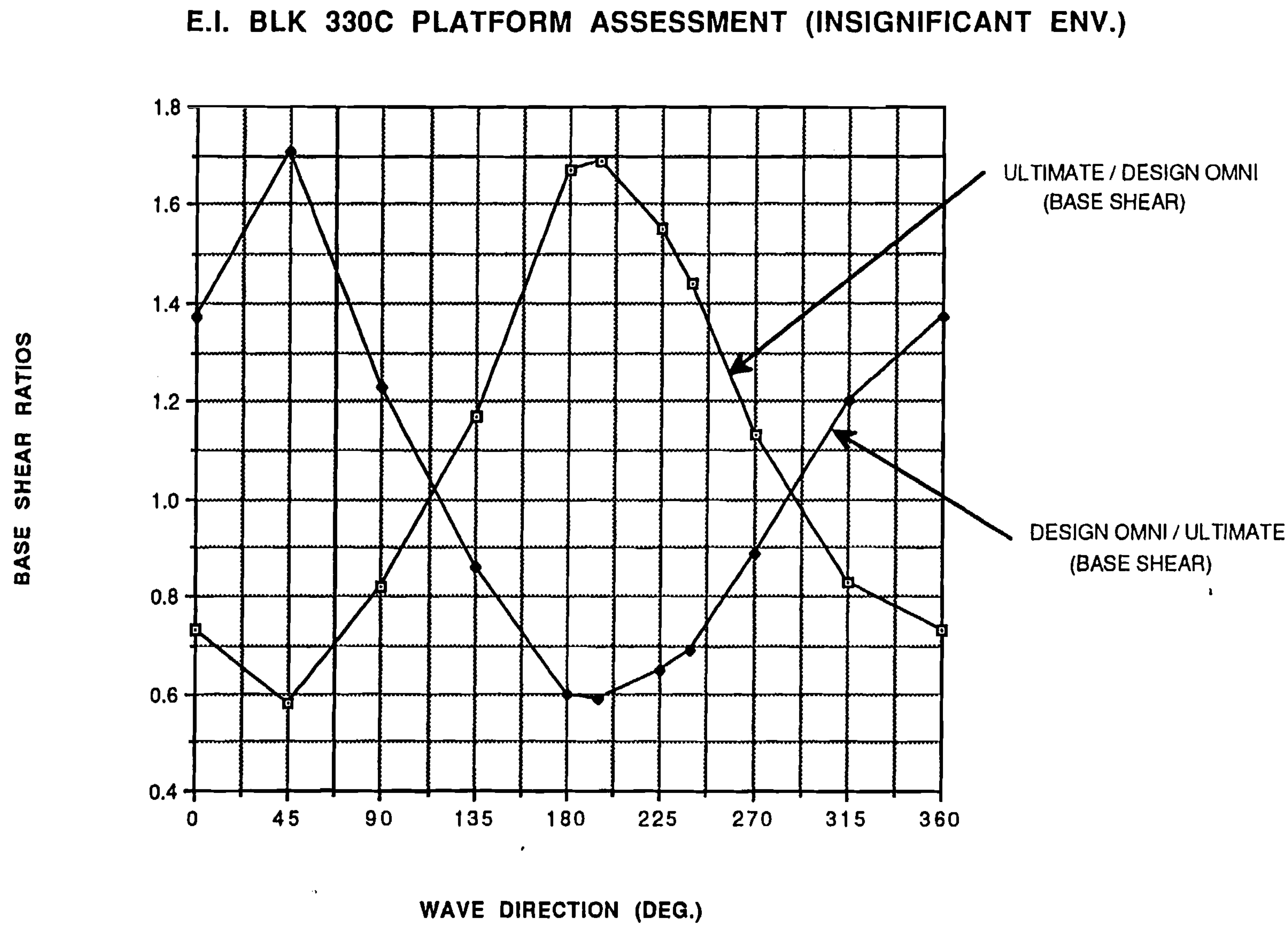

E.I. BLK 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)
BASE SHEAR RATIOS
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0
45
90
135
180
225
270
315
360
ULTIMATE / DESIGN OMNI
(BASE SHEAR)
DESIGN OMNI / ULTIMATE
(BASE SHEAR)
WAVE DIRECTION (DEG.)

FIG. 5

E.I. BLK 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)

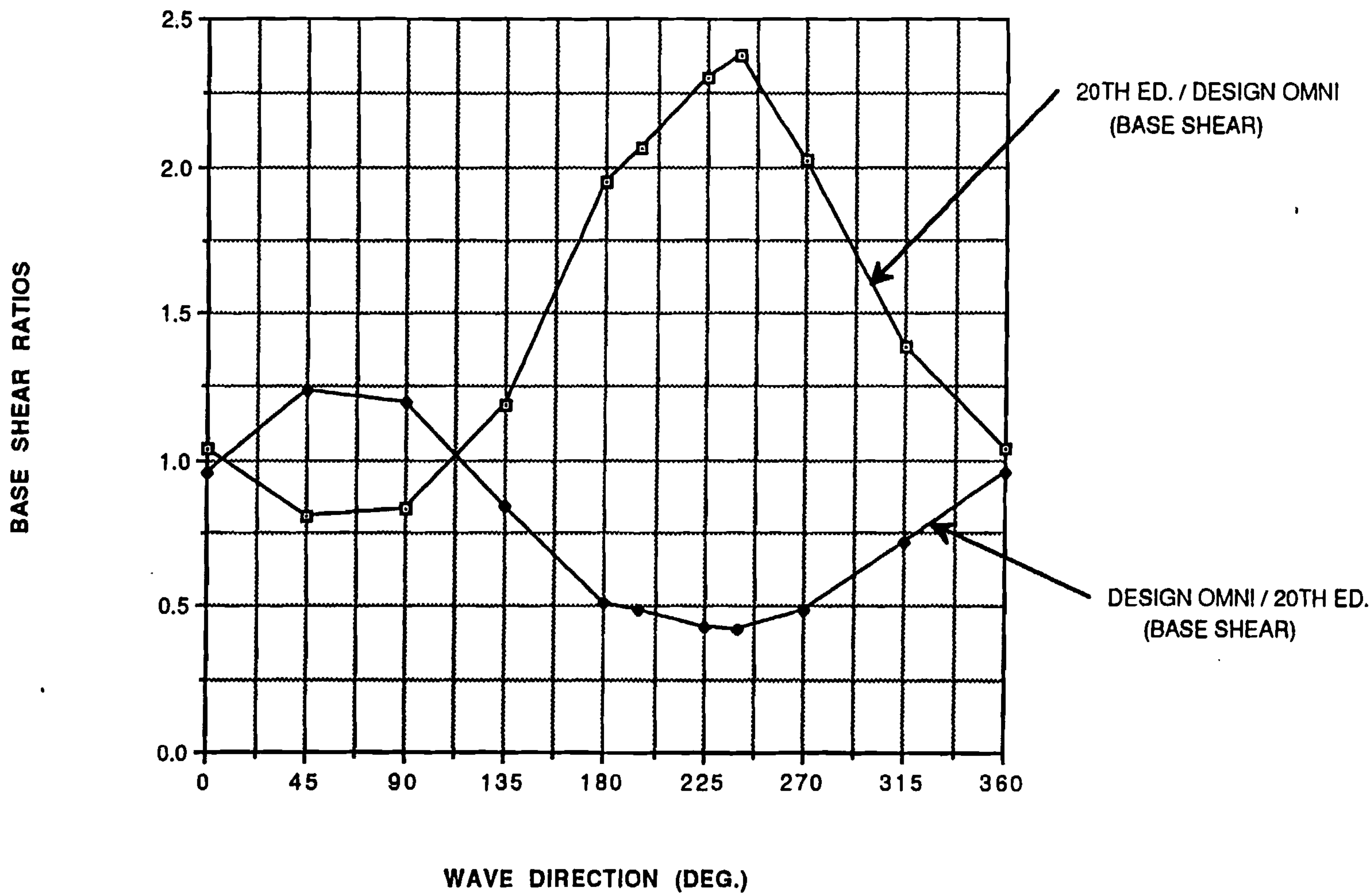


FIG. 6

E.I. BLK. 330C PLATFORM ASSESSMENT (INSIGNIFICANT ENV.)

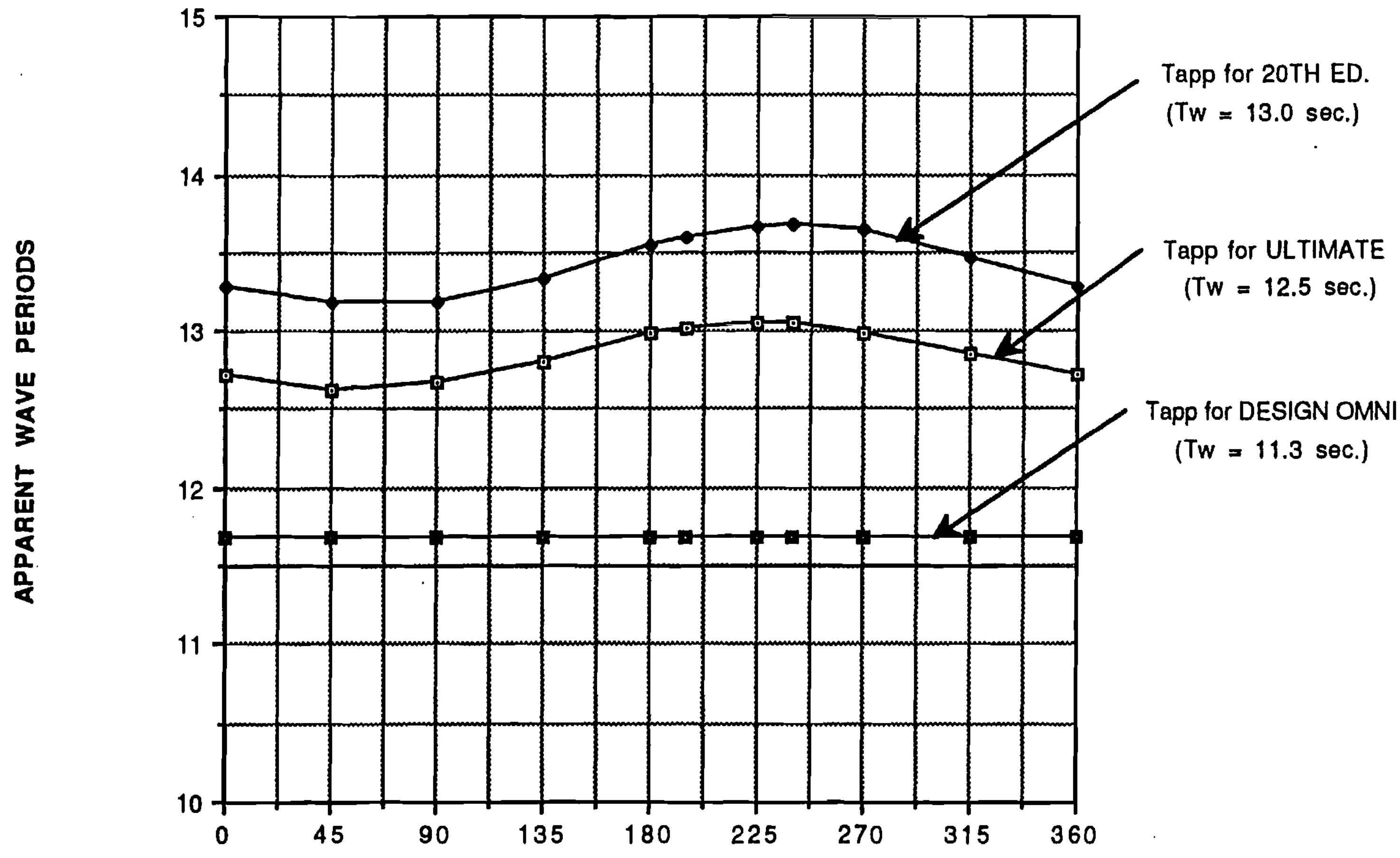


FIG. 7

A.5.2 Screening

The screening process used to assess the strength of existing platform can be varied. It depends on the data base of existing platforms which are comparable to the platform being evaluated. Experience and engineering judgment must be exercised with caution. It is intended that the result of screening process will always be on the conservative side. For structures with marginal safety of factors, the screening process might not be able to provide satisfactory results.

The screening process option was not exercised in this study.

A.5.3 Design Level Analysis

A.5.3.1 Metocean Criteria

The platform is considered as insignificant environmental impact/manned-evacuated. The metocean criteria for design level analysis are based on 100-year force due to the combined sudden hurricane and winter storm population. The wave height is assumed to be constant in all directions. The associated in-line current is also assumed to be constant in all directions. The required wave height, wave period and current magnitude are described above (see Section A.5.1). In some non-critical wave directions, the omni-directional criteria might exceed the ultimate strength analysis values, in which case the ultimate strength analysis values will govern. See Figure 3 in Section A 5.1 Metocean Criteria/Loads for further detail on the base shear plots versus wave direction for the comparison of different wave criteria.

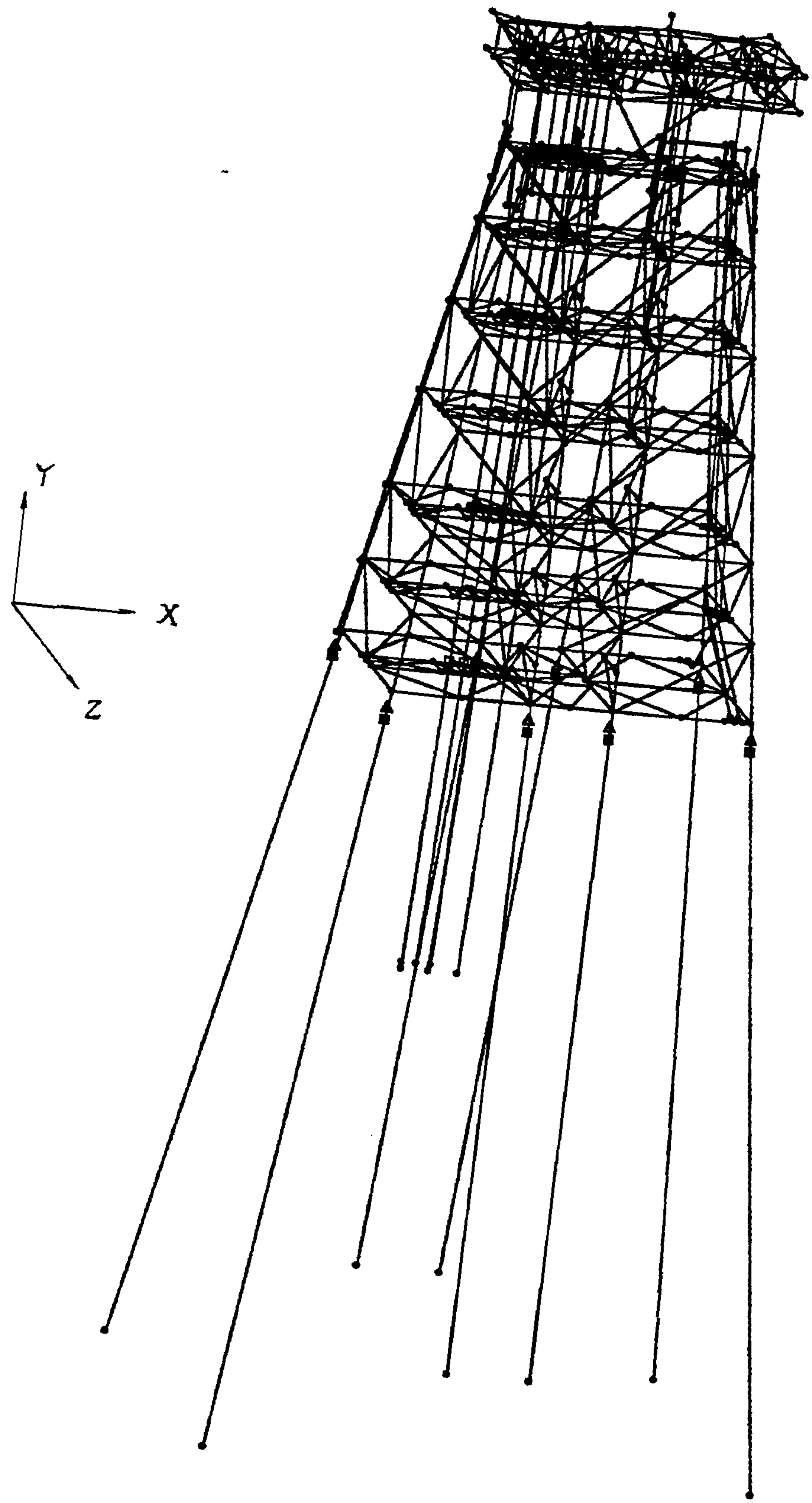
A.5.3.2 Structural Model and Boundary Conditions

The structural model is a three-dimensional model. The members of the platform are modeled as 3-D Beam Finite Elements. The beam element is a two-node element with six degrees of freedom at each node (three translational and three rotational degree of freedoms). Since this is a nongrouted structure, the jacket-leg node and pile node which have the same coordinates (common node) are connected by two orthogonal springs (zero length) so that the lateral load can be transferred from the jacket-leg to the pile at the common node. The information of member section properties and the material yield strength are obtained from as-built drawings.

The structural model consists of 816 joints (nodes) and 1,636 members (beam elements). The deck members consist of wide flange beams and tubular members. The deck loads are supported by eight main piles, which are extended below the mudline to their full design pile penetration. See Section 1.2.4 - Pile Make-up and Details for the pile penetrations.

In the conductor modeling, the original 18 conductors were lumped into three equivalent conductors (with each conductor model representing six actual conductors). The three additional conductors were modeled individually. The conductors were explicitly modeled to 150' below the mudline.

The boundary conditions of the superstructure are the pile foundation with soil reactions modeled as soil springs (P-Y, T-Z and Q-Z curves), which are attached to the nodes of pile elements. The soil properties at different elevations were properly modeled.


Y
X
Z

A.5.3.3 Design Loads

The platform was designed to support the drilling and production operations (live and dead loads). In addition, the platform must be designed to withstand lateral loads such as storm waves/operation waves, current and wind load, etc. In this study, the design load information was retrieved from the previous design model, which was available ir lesign file.

The design gravity loads used in this study can be summarized as follows:

Buoyancy:	1,762.96 kips
Gravity Load:	4,491.57 kips (including 5% contingency)
Deck Live Load:	7,504.00 kips
Deck Dead Load:	754.00 kips

The lateral loads (wave, current and wind) are generated by MicroSAS computer program using the metocean criteria as described in Section A.5.1 - Metocean Criteria/Loads. See Section A.5.3.4 - Lateral Load Level for further information.

A.5.3.4 Lateral Load Level

For the Design Level Analysis, its wave height (H_W) is constant in all directions. In this study, H_W = 47' is used. The current direction is in-line with the wave direction. Current blockage factors are directionally dependent with respect to platform's orientation (longitudinal, transverse or diagonal) per API RP 2A 20th edition guideline. Wave kinematics factor of 0.88 is used. Drag and inertia coefficients take into account the surface effects of tubular members (smooth or rough). The conductor shielding factor is also considered. The marine growth is also modeled per API RP 2A 20th edition.

Typically, eight wave directions are used in the calculations of static base shears. In this study 10 wave directions are applied (0 , 45 , 90 , 135 , 180 , 225 , 270 , 315 , 195 and 240 degs.). The wave directions are defined with respect to platform coordinates system (see sketch of platform orientation). The wave direction of 240 deg. corresponds to the worst wave direction (wave height factor = 1.0) in API RP 2A 20th edition, which is equivalent to the exposure category of significant environmental impact/manned-evacuated. The wave direction of 195 deg. corresponds to the worst wave condition for platforms classified as insignificant environmental impact/manned-evacuated case. The reason to include the wave direction of 240 deg. is that in the subsequent ultimate strength analysis, the reserve strength ratio is defined with reference to API RP 2A 20th edition guideline (which corresponds to the significant environmental impact/manned - evacuated case).

The lateral load level due to wave and current effects is summarized as follows:

Design Wave Height: 47 ft.
Design Wave Period: 11.3 sec (Apparent Wave Period: 11.69 sec.)

Wave Direction (deg.)	Lateral Load (Wave & Current) (kips)
0	1,724 (1,258)*
45	1,939 (1,133)*
90	2,083 (1,700)*
135	1,930
180	1,696
225	1,924
270	2,090
315	1,952 (1,622)*
240	1,996
195	1,818

* Each value shown in parentheses is the lateral load corresponding to ultimate strength analysis, which is less than that of design level analysis, and will be used in the subsequent stiffness analysis, SPIA (soil-pile-structure interaction analysis), tubular member check, joint check etc.

The maximum lateral load is 2,090 kips in the wave direction of 270 deg. See Figure 3 in Section A.5.1 for the comparison of lateral loads (wave and current) for different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition).

A.5.3.5 Members with Utilization Ratio > 0.85

Tubular member check was performed by MicroSAS program, an in-house developed computer program specifically for offshore platforms design and analysis. The material yield strength is 36 ksi (nominal) typical. The effective length factors of tubular members are calculated based on API RP 2A guideline (jacket legs, diagonal braces, horizontal members, truss members etc.). An allowable stress increase factor of 1.33 is used for design level analysis.

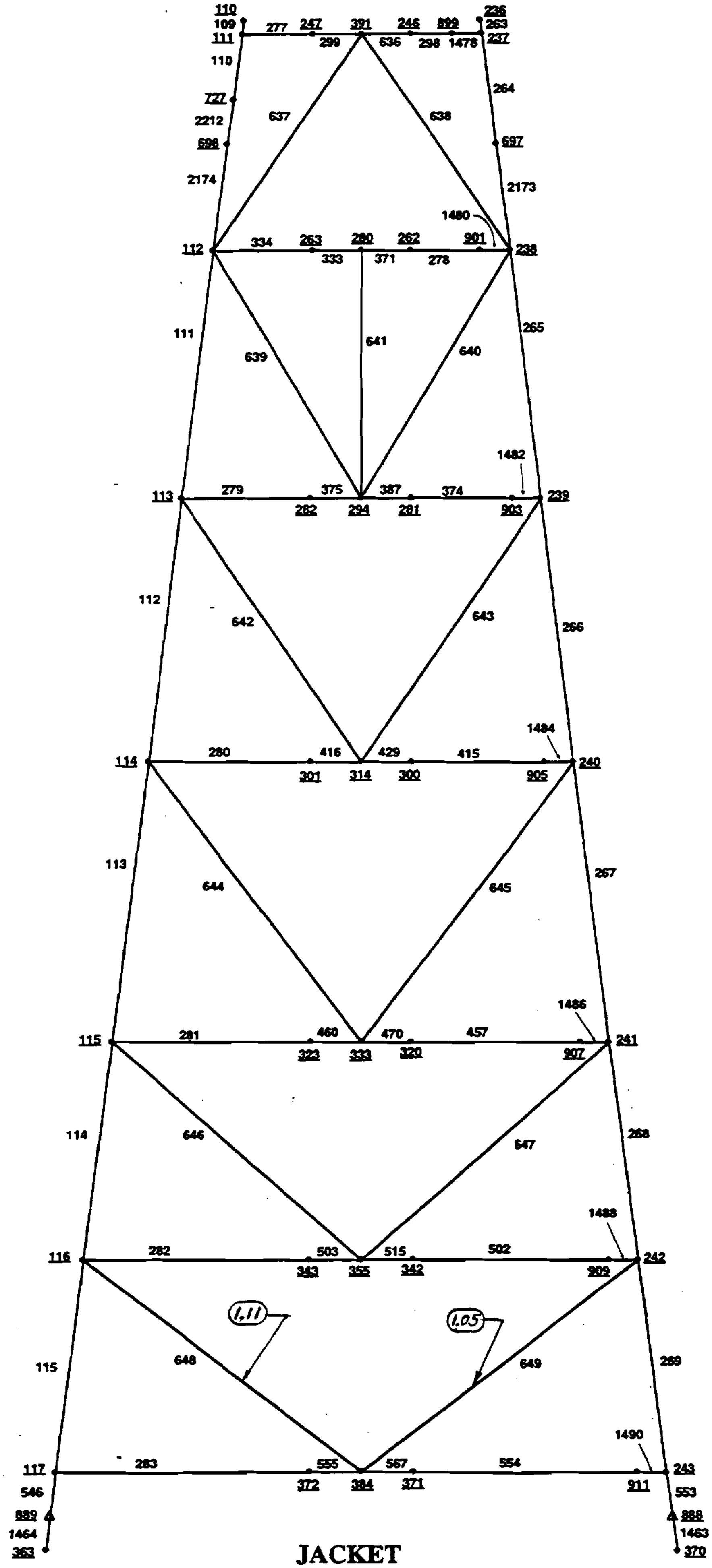
The tubular member check results are shown in the attachment (Excel spreadsheets). Although, the attached sketches show members with the stress ratio > 0.85, it should be pointed out that most of members with stress ratio exceeding 0.85 were attributed to the hydrostatic check conditions. Only five members (#272, #273, #422, #560 and #566) with stress ratios exceeding 0.85 are attributable to the strength check conditions. The reason so many members are governed by the hydrostatic check (in comparison with strength check) is due to the fact that the drag coefficients have been increased significantly in comparison to previous editions of API RP 2A (prior to 9th edition). Also, the safety factors used in the hydrostatic check are higher than that used in the pure strength check. In addition, the marine growth thickness and the marine growth region (below waterline) have significantly increased in API RP 2A 20th edition. Thus, structural members with higher marine growth would result in higher lateral loads and member stresses.

Some secondary members with stress ratio greater than 1.0 are due to structural modeling difficulties. Their stress ratios are included in this report just to be consistent with the reported computer output. They can be ignored as far as the stress ratio is concerned.

The details of the stress ratio and governing loading case for the tubular members checked can be found in the Appendix of the final report - computer output.

E.I. BLOCK 330 C PLATFORM ASSESSMENT					8/7/94
Insignificant Environmental Impact / Manned - Evacuated					
TUBECHECK (MEMBER CHECK) RESULTS					
DESIGN LEVEL ANALYSIS		**(Fy = 36 ksi)**			
MEMBER NO.	LOADING CASE	STRESS RATIO	Controlling Case	REMARKS	
EL. (+) 3' - 0					
#744 (18"x0.375")	KTORM240	0.93	HYDROSTATIC		L = 3.15'
#772 (6.625"x0.375")	KTORM270	1.45	HYDROSTATIC	Secondary Member	L = 1.97'
#776 (6.625"x0.280")	KTORM270	1.07	HYDROSTATIC	Secondary Member	L = 1.37'
#783 (6.625"x0.280")	KTORM090	1.08	HYDROSTATIC	Secondary Member	L = 2.55'
#785 (6.625"x0.280")	KTORM240	1.47	HYDROSTATIC	Secondary Member	L = 4.14'
#1478 (18"x0.375")	KTORM135	**1.13**	HYDROSTATIC		L = 3.03'
#1479 (8.625"x0.322")	KTORM135	**3.60**	HYDROSTATIC		L = 1.58'
EL. (-) 80' - 0					
#272 (16"x0.375")	KTORM195	0.91	STRENGTH		
EL. (-) 127' - 0					
#273 (18"x0.375")	KTORM195	0.89	STRENGTH		
#422 (18"x0.375")	KTORM195	0.91	STRENGTH		
EL. (-) 216' - 0					
#160 (24"x0.375")	KTORM195	0.92	HYDROSTATIC		
#252 (24"x0.375")	KTORM195	0.91	HYDROSTATIC		
#509 (24"x0.375")	KTORM195	1.00	HYDROSTATIC		
#513 (24"x0.375")	KTORM180	**1.01**	HYDROSTATIC		
EL. (-) 254' - 0					
#138 (24"x0.375")	HTORM315	**1.13**	HYDROSTATIC		
#184 (24"x0.375")	KTORM195	**1.04**	HYDROSTATIC		
#207 (24"x0.375")	HTORM090	0.98	HYDROSTATIC		
#230 (24"x0.375")	KTORM195	**1.06**	HYDROSTATIC		
#276 (24"x0.375")	HTORM000	**1.10**	HYDROSTATIC		
#283 (24"x0.375")	HTORM090	**1.10**	HYDROSTATIC		
#290 (24"x0.375")	HTORM090	**1.04**	HYDROSTATIC		
#291 (24"x0.375")	HTORM090	**1.01**	HYDROSTATIC		
#554 (24"x0.375")	KTORM270	**1.09**	HYDROSTATIC		
#555 (24"x0.375")	HTORM090	**1.01**	HYDROSTATIC		
#556 (24"x0.375")	KTORM270	**1.12**	HYDROSTATIC		
#557 (24"x0.375")	HTORM090	**1.13**	HYDROSTATIC		
#558 (24"x0.375")	KTORM270	**1.07**	HYDROSTATIC		
#559 (24"x0.375")	KTORM135	1.00	HYDROSTATIC		
#560 (24"x0.375")	HTORM000	**1.11**	STRENGTH		

MEMBER NO.	LOADING CASE	STRESS RATIO	Controlling Case		
#564 (24"x0.375")	KTORM195	**1.14**	HYDROSTATIC		
#566 (24"x0.375")	KTORM195	**1.01**	STRENGTH		
#567 (24"x0.375")	KTORM270	**1.06**	HYDROSTATIC		
#568 (24"x0.375")	KTORM225	**1.05**	HYDROSTATIC		
#675 (24"x0.375")	KTORM270	**1.25**	HYDROSTATIC		
#892 (24"x0.375")	KTORM195	**1.32**	HYDROSTATIC		
#893 (24"x0.375")	KTORM195	**1.24**	HYDROSTATIC		
#896 (24"x0.375")	KTORM135	**1.26**	HYDROSTATIC		
#897 (24"x0.375")	KTORM195	**1.23**	HYDROSTATIC		
#898 (24"x0.375")	KTORM195	**1.21**	HYDROSTATIC		
#1490 (24"x0.375")	KTORM225	**1.30**	HYDROSTATIC		
#1491 (8.625"x0.322")	KTORM195	1.25	HYDROSTATIC	Secondary Member	L=1.33'
ROW 1					
#648 (24"x0.375")	KTORM270	**1.11**	HYDROSTATIC		
#649 (24"x0.375")	HTORM090	**1.05**	HYDROSTATIC		
ROW 2					
#699 (24"x0.375")	KTORM270	0.88	HYDROSTATIC		
#700 (24"x0.375")	HTORM090	**1.04**	HYDROSTATIC		
#701 (24"x0.375")	KTORM270	**1.21**	HYDROSTATIC		
ROW 3					
#731 (24"x0.375")	KTORM270	0.86	HYDROSTATIC		
#732 (24"x0.375")	HTORM090	1.00	HYDROSTATIC		
#733 (24"x0.375")	KTORM270	**1.10**	HYDROSTATIC		
ROW 4					
#661 (24"x0.375")	KTORM270	0.95	HYDROSTATIC		
#662 (24"x0.375")	HTORM090	0.92	HYDROSTATIC		
ROW A					
#608 (24"x0.375")	KTORM180	0.89	HYDROSTATIC		
#614 (30"x0.500")	KTORM195	0.86	HYDROSTATIC		
ROW B					
#633 (30"x0.500")	KTORM195	0.94	HYDROSTATIC		



JACKET ROW 1

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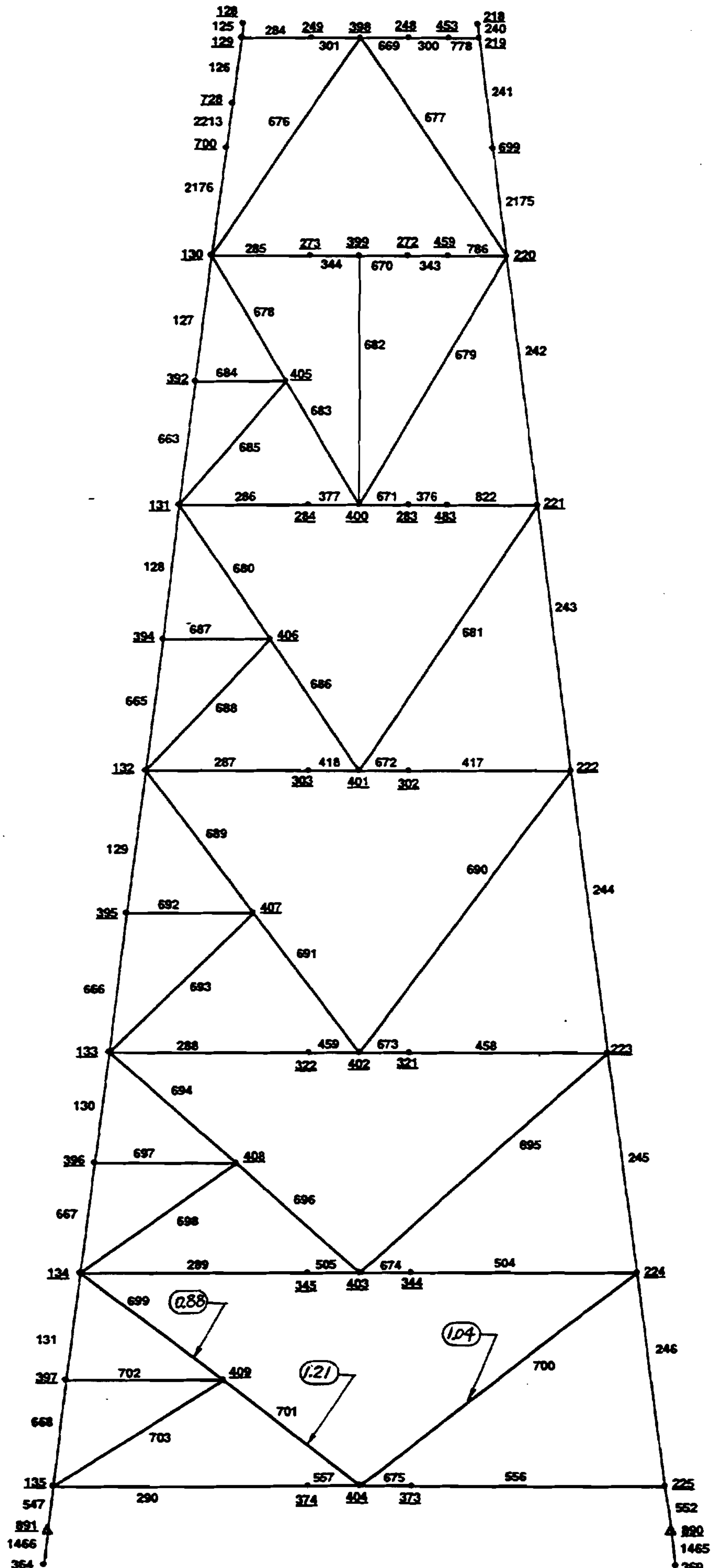

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ROW 2

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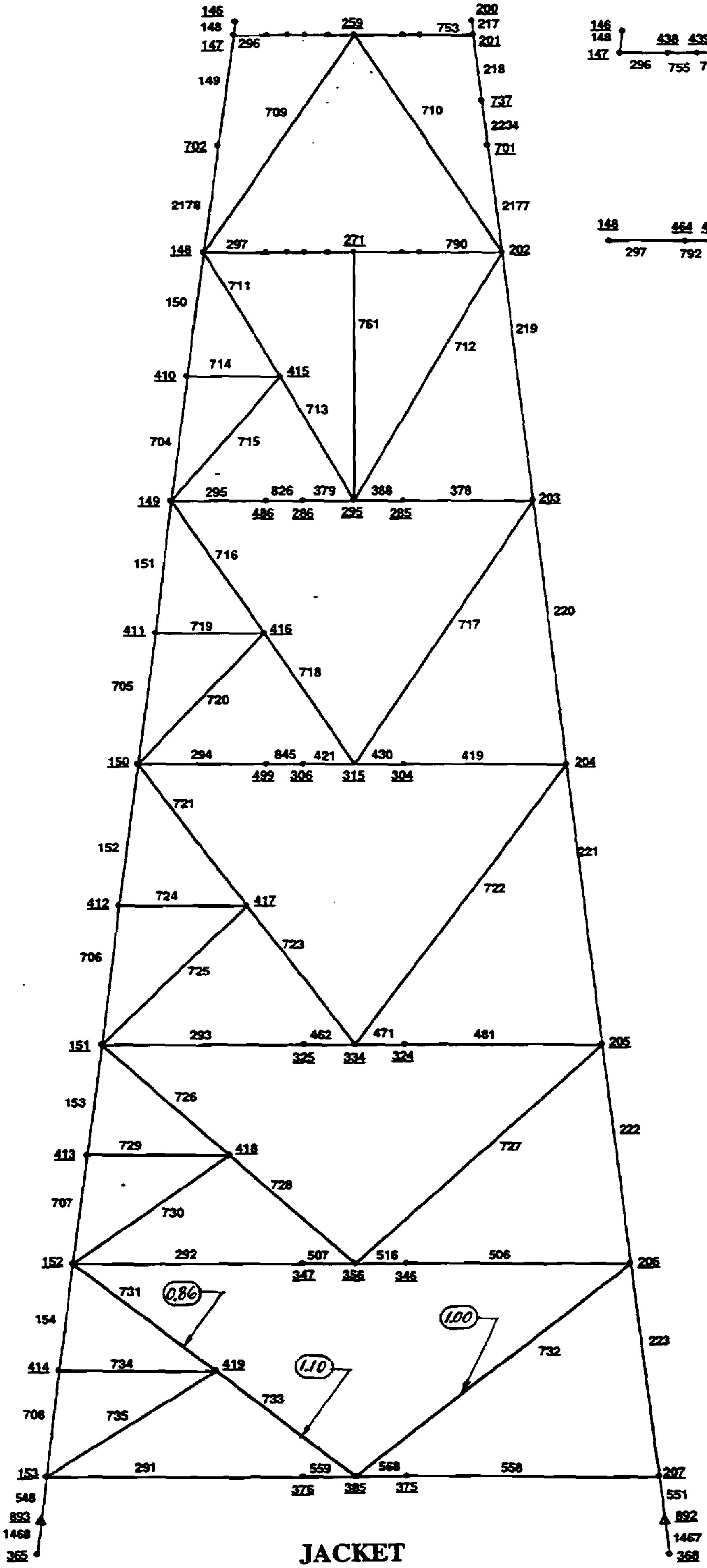

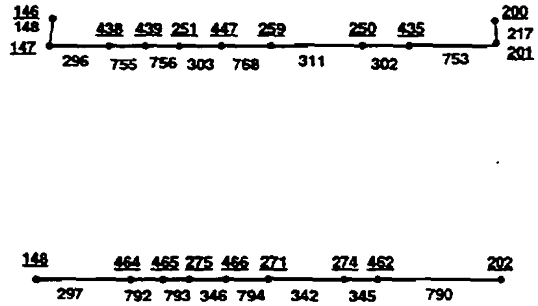

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ROW 3

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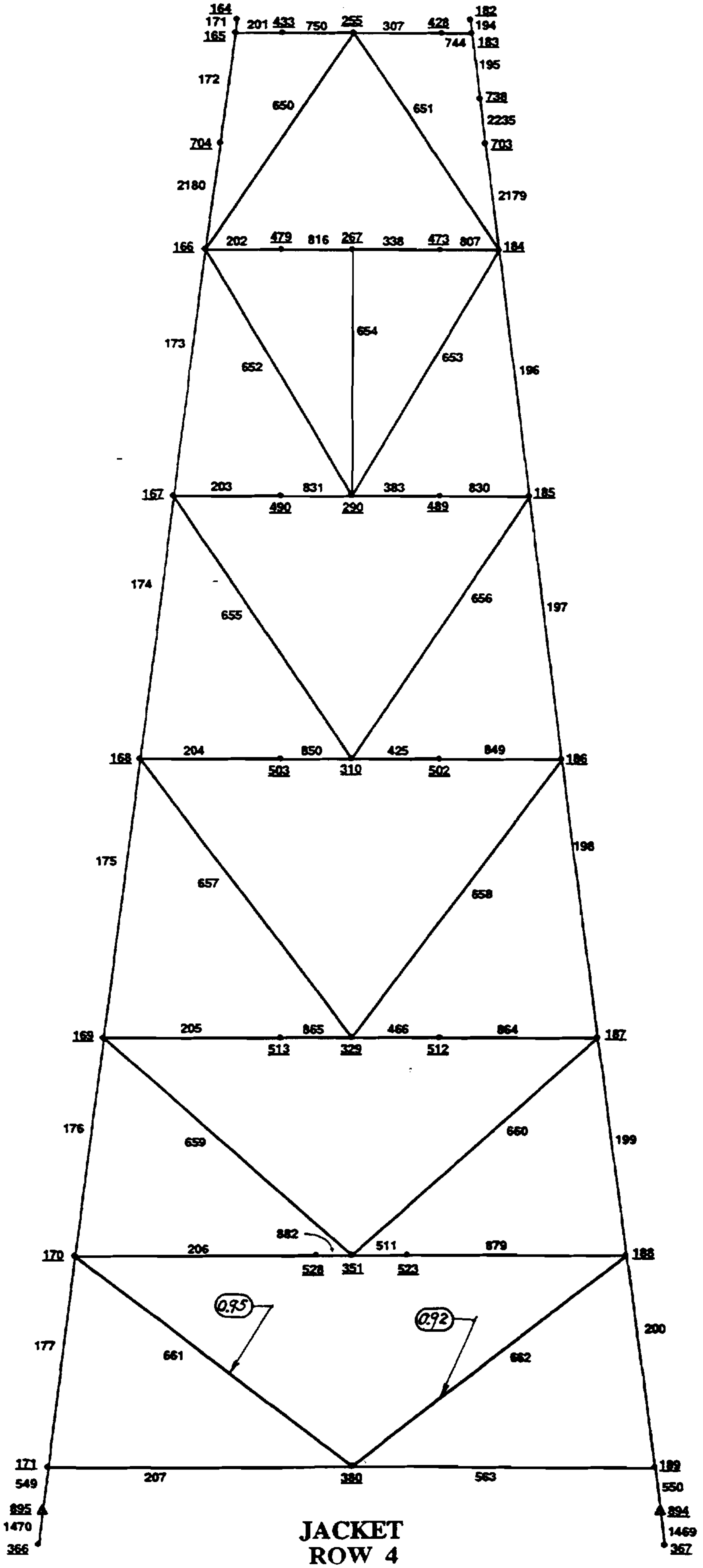

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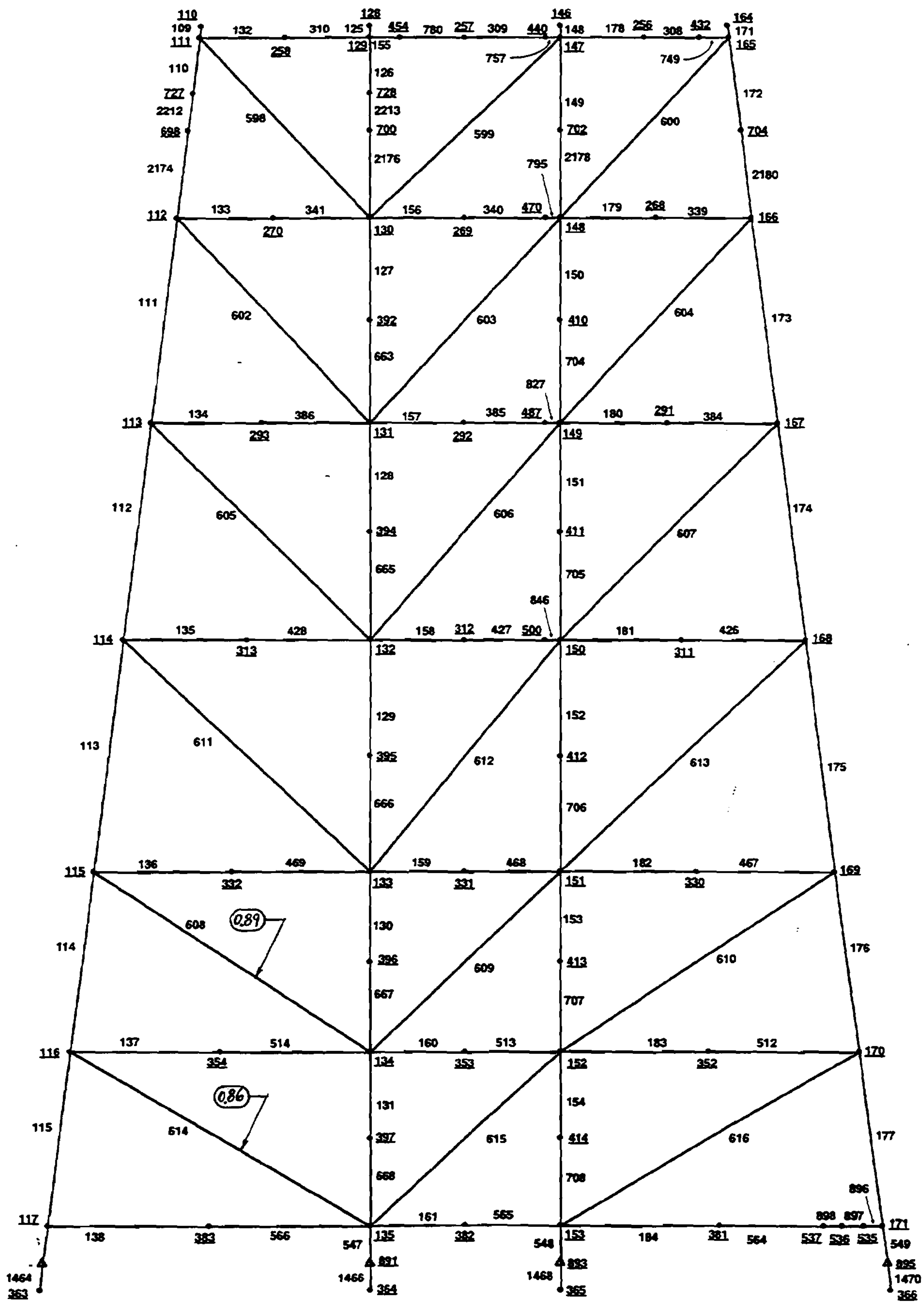
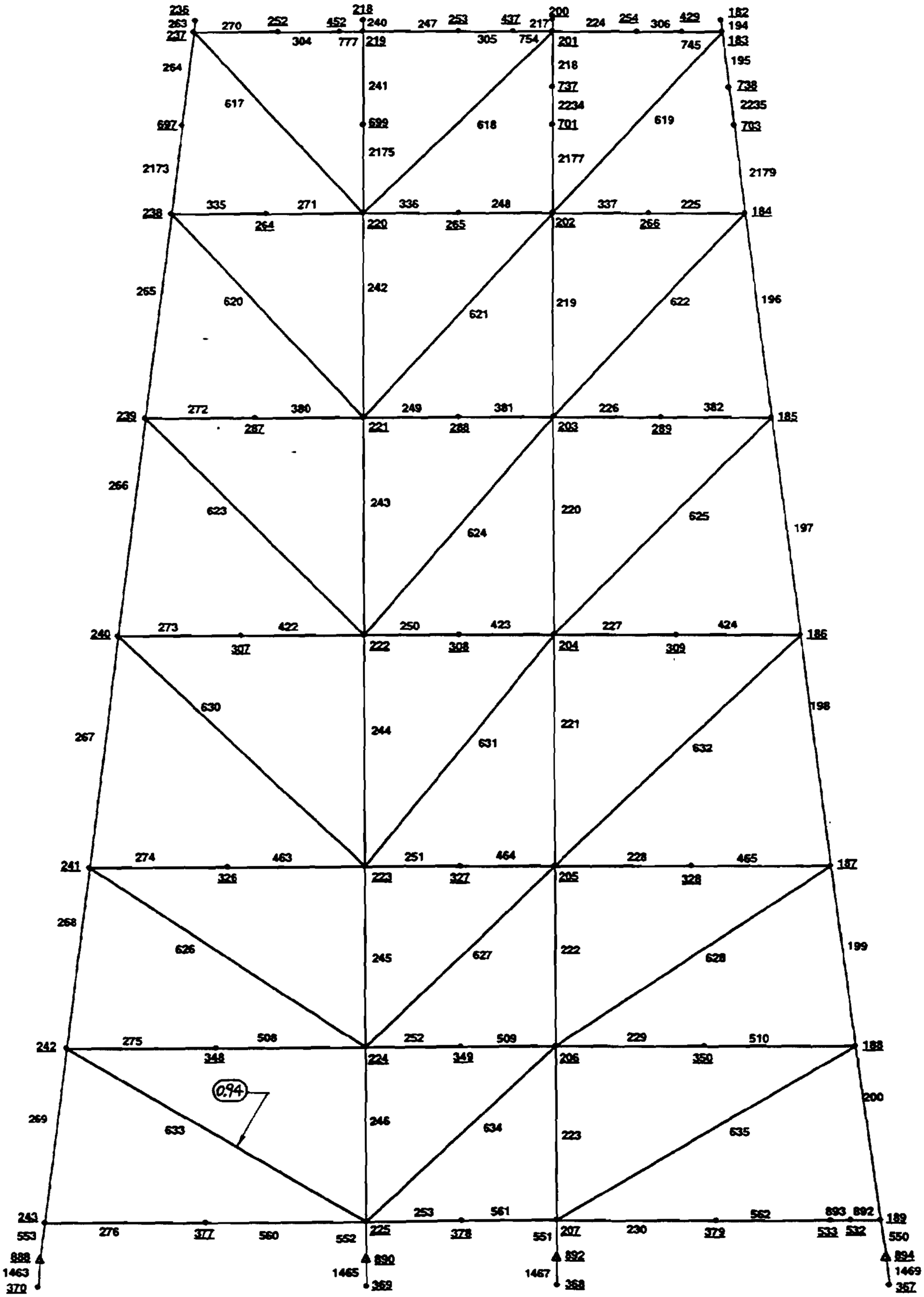


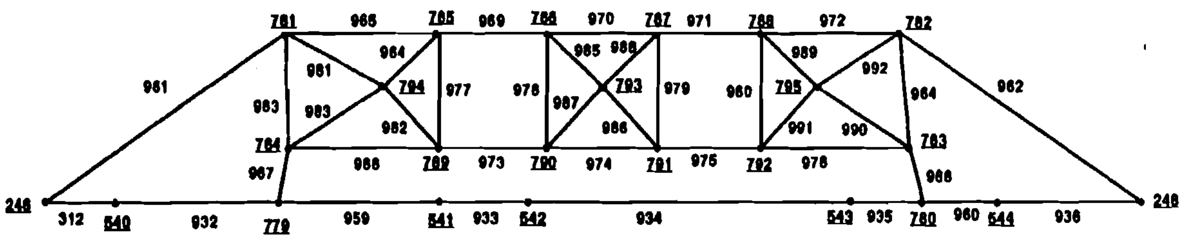
JACKET
ROW A

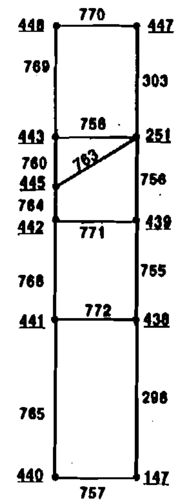
4/23/94

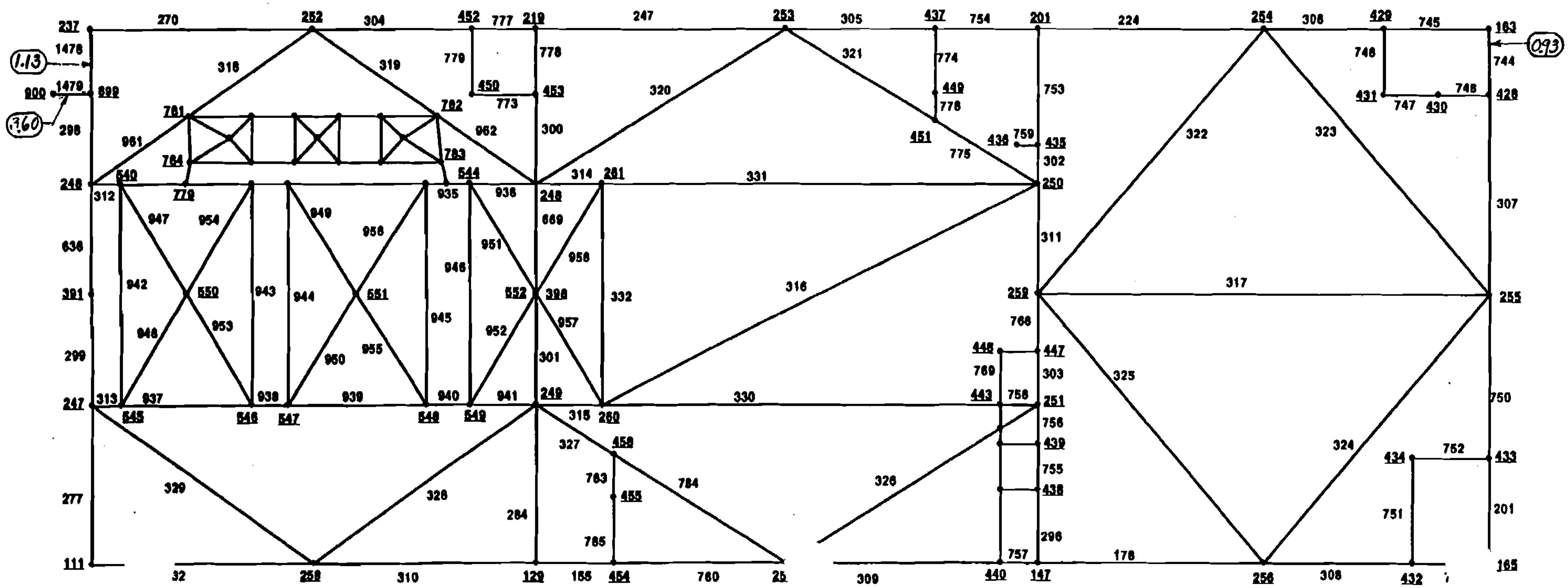


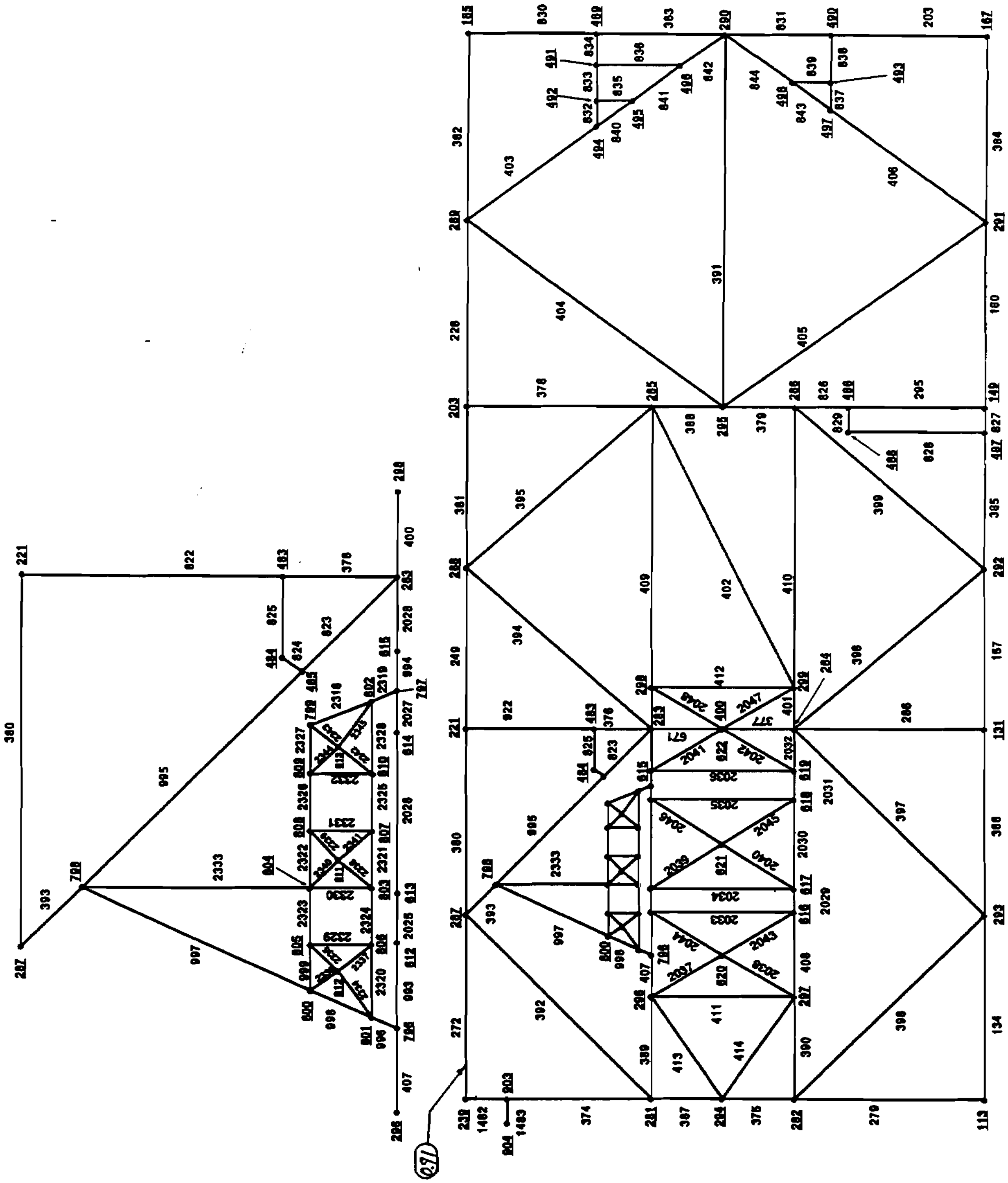
236 263 237 270 252 304 452 777 218 240 219 247 253 305 437 754 217 200 201 224 254 306 429 745 182 194 183
264 617 241 618 218 737 2234 619 195 738 2235
697 699 701 703
2175 2177 2173 2179
238 335 264 271 220 336 265 248 202 337 266 225 184
265 620 242 621 219 622 196
239 272 287 380 221 249 288 381 203 226 289 382 185
266 623 243 624 220 625 197
240 273 307 422 222 250 308 423 204 227 309 424 186
267 630 244 631 221 632 198
241 274 326 463 223 251 327 464 205 228 328 465 187
268 626 245 627 222 628 199
242 275 348 508 224 252 349 509 206 229 350 510 188
0.94
269 633 246 634 223 635 200
243 553 276 377 560 552 225 253 378 561 551 207 230 379 562 893 533 892 532 189 550
888 1463 370 890 1465 369 892 1467 368 894 1469 367

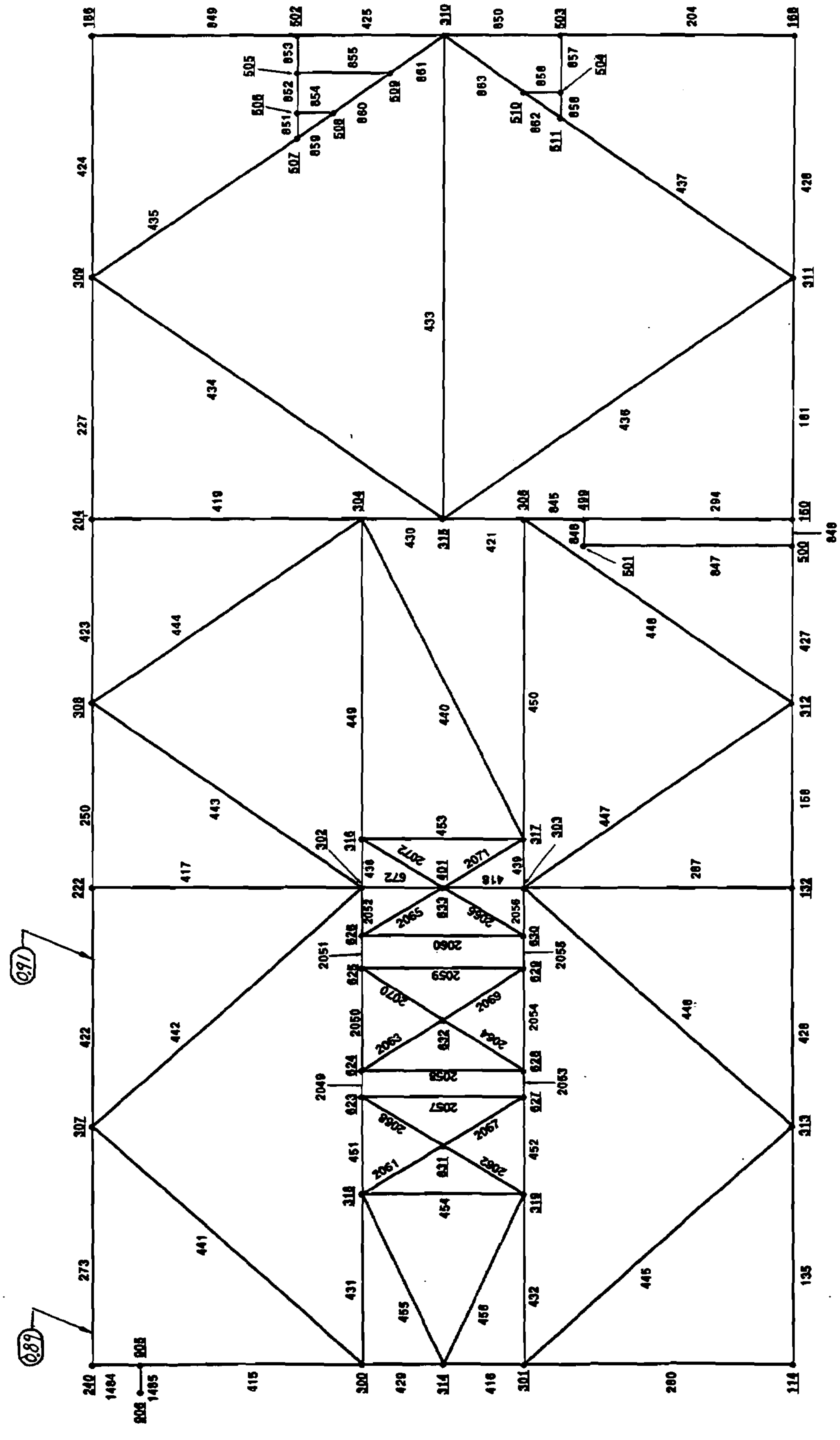
JACKET
ROW B

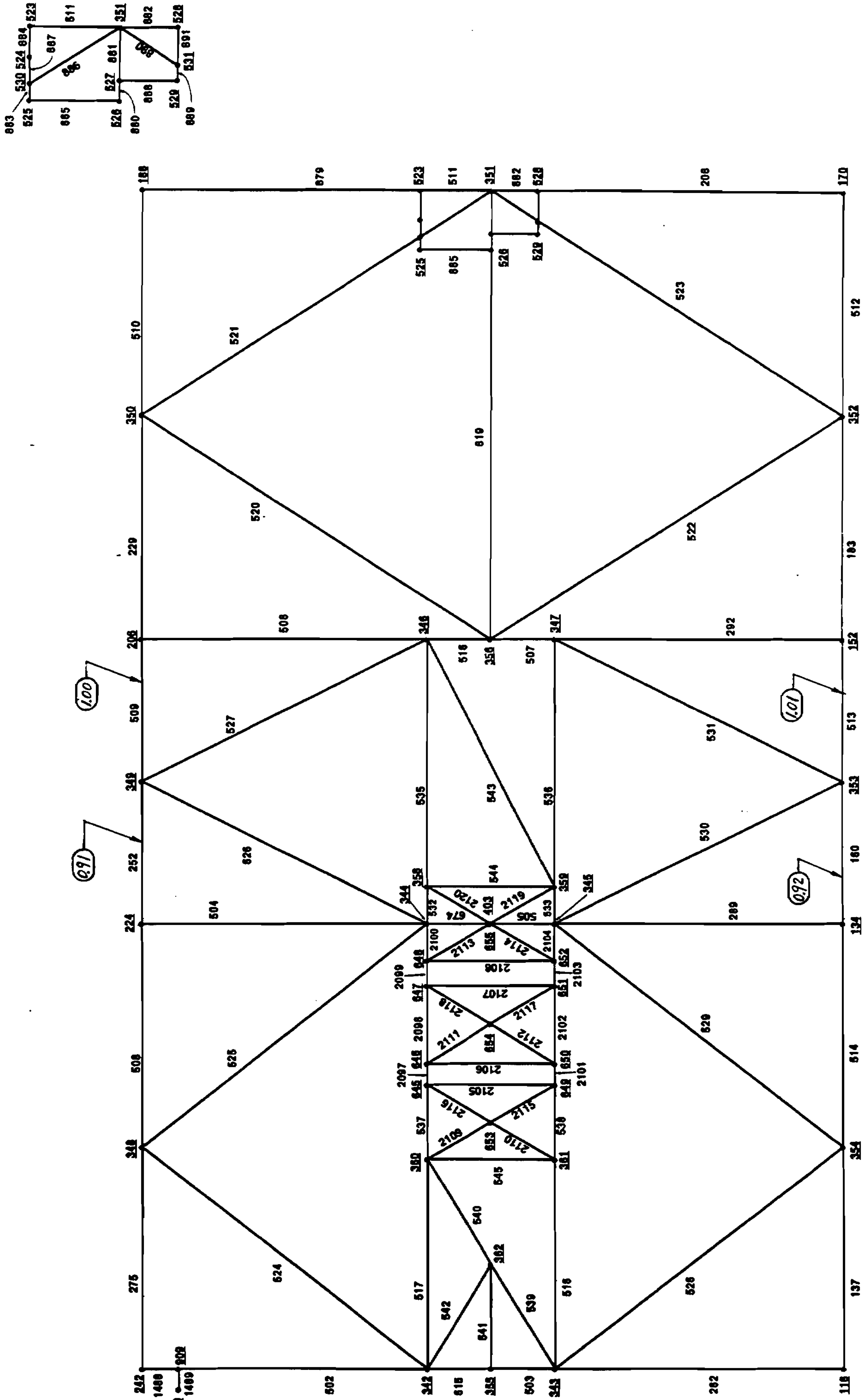

781 965 785 969 786 970 787 971 788 972 782
961 981 984 794 977 978 985 988 793 979 980 795 989 992 962
983 982 987 986 991 990 964
784 966 789 973 790 974 791 975 792 976 783
967 968
248 312 540 932 779 959 541 933 542 934 543 935 780 960 544 936 248

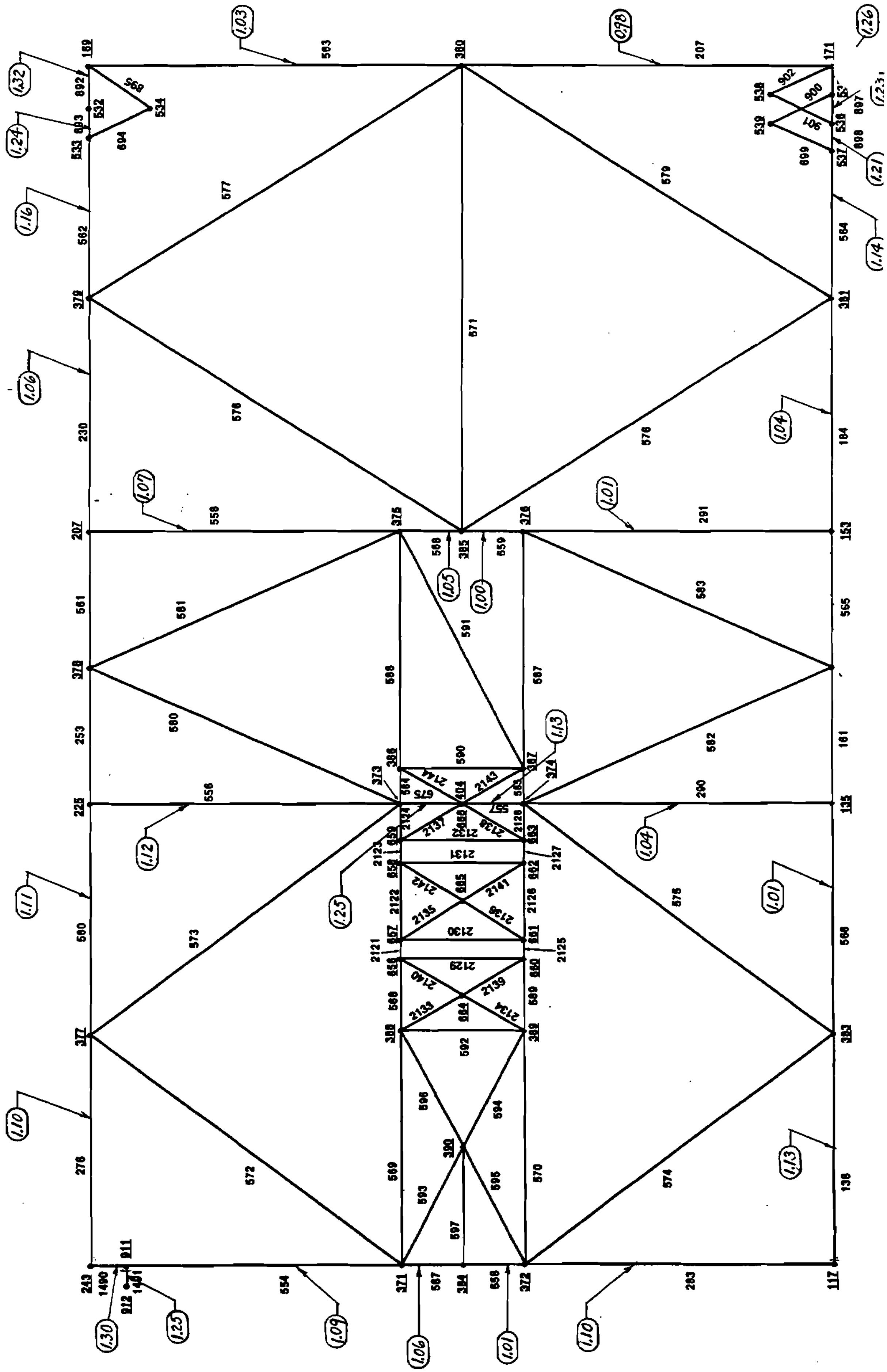

446 770 447
769 303
443 758 251
760 763 756
445
764
442 771 439
768 755
441 772 438
765 298
440 757 147


237 270 252 304 452 777 219 247 253 305 437 754 201 224 254 308 429 745 163
1478 1.13 318 319 776 778 774 746 744 0.93
900 1479 899 450 773 453 449 431 747 430 428
3.60 320 321 753 322 323
298 781 782 962 300 776 451 775 436 759 435
961 784 783 302
248 540 779 935 544 936 248 314 261 331 250
312 947 954 949 956 669 307
636 942 951 958 311
391 550 943 944 551 945 946 552 398 332 316 259 317 255
948 953 957 766
299 950 955 952 301 448 447 325
769 303
247 313 937 938 939 940 941 249 330 443 758 251 750
545 546 547 548 549 315 260 756
327 456 439 324
329 328 783 784 326 755 434 752 433
277 455 438
284 751 201
785 296
111 32 259 310 129 155 454 780 25 309 440 757 147 176 256 308 432 165









A.5.3.6 Joint / Brace with Utilization Ratio > 0.85

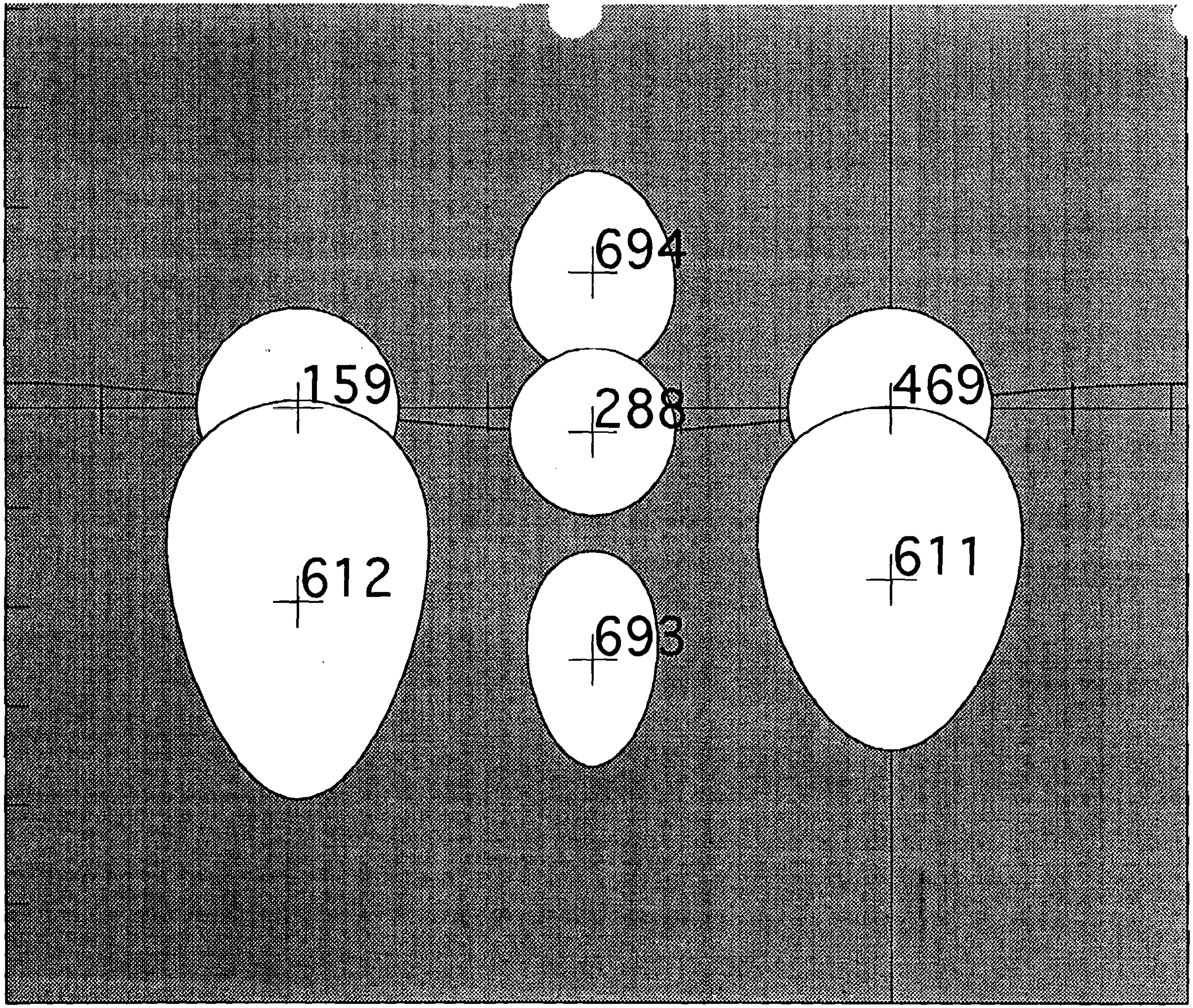
Tubular joint check was performed by MicroSAS program, an in-house developed computer program specifically for offshore platforms design and analysis. A material yield strength of 36 ksi was specified. An allowable stress increase factor of 1.33 was used. The nominal loads approach as provided in API RP 2A was used in the Joint Check. The joint check results are shown in the attachment (Excel spreadsheets). Although the attached sketches show the joints/braces with stress ratio > 0.85, it should be noted that most of the joints/braces with interaction stress ratio exceeding 0.85 are mainly K-joints. The factor that attributed to these high stress ratios is that the brace and chord wall thickness is identical. Consequently, the ratio of brace wall thickness to the chord wall thickness, tau, is equal to one, which will induce high stress in the chord (in the sense of chord punching stress).

Most of the K-joints in Rows #1 through #4 are overlapping joints. Those overlapping joints are specifically considered in the MicroSAS joint check program, in order to take advantage of the strength of overlapping joints. Three overlapping joint plots (joints #133, #294 and #355) are shown in the following attachment.

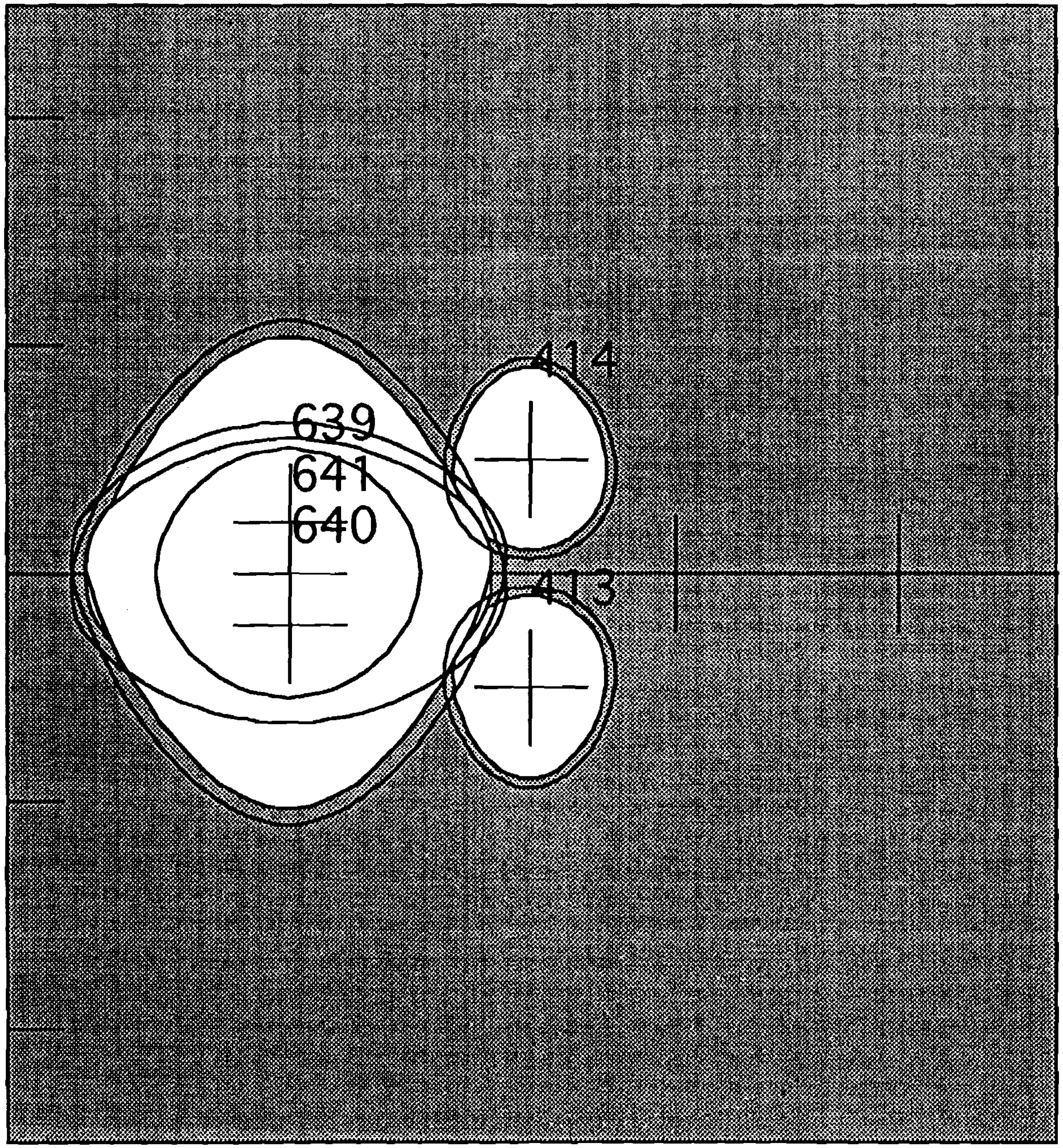
The summary of joint check results with the interaction stress ratios greater than 0.85 is shown in the list (Excel spreadsheet). The plots of joint/brace pairs with interaction stress ratio greater than 0.85 are also shown in the following attachment.

It should be noted that joint/brace #133/#130 and #151/#153 are overlapping T-K joints , in which three coplanar braces are connected at a common joint. When two out of three braces are overlapped with a gap to the third brace, some extra manual effort is required to calculate the actual stress ratio for joint check.

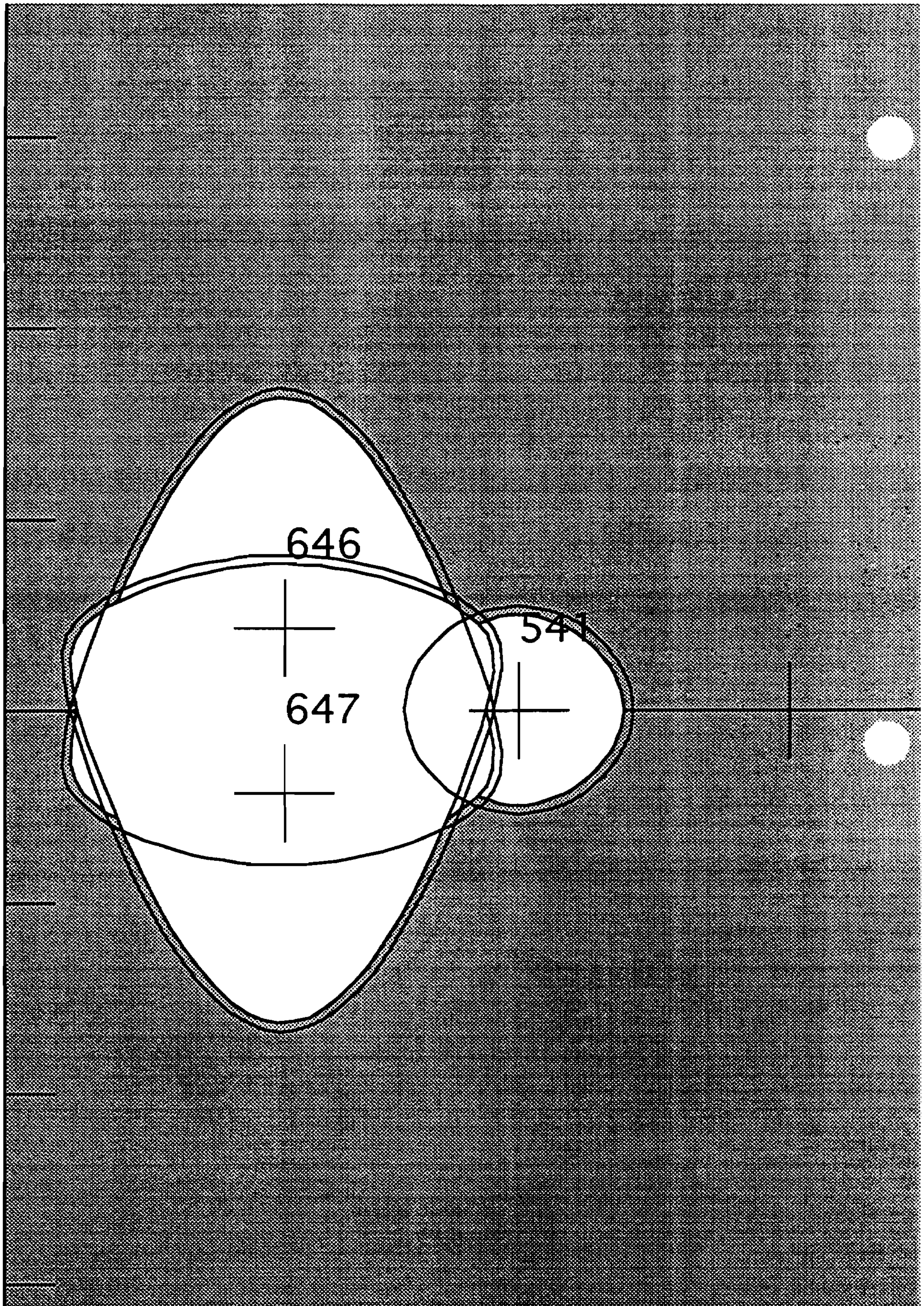
The joint check results show several joints with stress ratio exceeding 1.0. After reviewing the results, it was concluded that a refined model would reduce most of these stress ratios, especially those joints which are part of the conductor framing at the mudline.



JOINT #133



JOINT #294

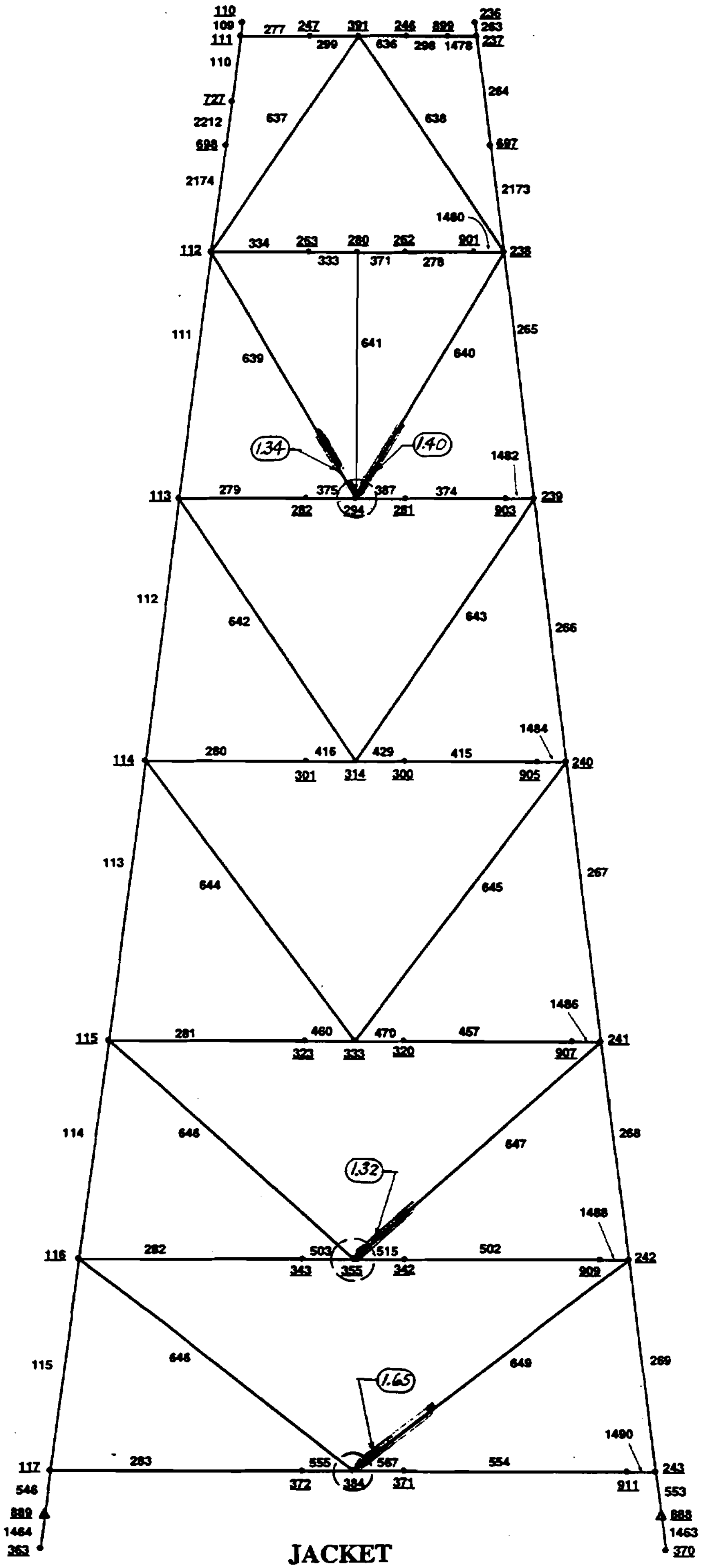


JOINT # 355

E.I. BLOCK 330C PLATFORM ASSESSMENT				8/3/94
Insignificant Environmental Impact / Manned - Evacuated				
JOINT CHECK RESULTS OF OVERLAPPING JOINTS				
			(Fy = 36 ksi)	
		DESIGN LEVEL	DESIGN LEVEL	
			W/ Overlapping	
JOINT NO./CHORD NO.	BRACE NO.	LOADING CASE	Stress Ratio	
ROW 1				
#294 / #375	#639 (18"x0.563")	KTORM270	**1.34**	
(18"x0.563")	#640 (18"x0.563")	KTORM270	**1.40**	
#355 / #503	#647 (20"x0.375")	KTORM270	**1.32**	
(20"x0.375")				
#384 / #555	#649 (24"x0.375")	KTORM270	**1.65**	
(24"x0.375")				
ROW 2				
#133 / #130	#693 (16"x0.50")	HTORM000	N / A	
(46.5"x1.25")				
#400 / 377	#683 (18"x0.563")	KTORM270	**1.40**	
(18"x0.563")	#679 (18"x0.563")	KTORM270	**1.41**	
#401 / 418	#681 (18"x0.563")	KTORM270	0.86	
(18"x0.563")				
#402 / 459	#690 (20"x0.562")	KTORM270	0.87	
(20"x0.562")				
#403 / 505	#695 (20"x0.375")	KTORM270	**1.45**	
(20"x0.375")				
#404 / #557	#700 (24"x0.375")	KTORM270	**1.74**	
(24"x 0.375")				
ROW 3				
#151 / #153	#725 (16"x0.50")	HTORM000	N / A	
(46.5"x1.25")				
#295 / #379	#713 (18"x0.563")	KTORM270	**1.06**	
(18"x0.563")	#712 (18"x0.563")	KTORM270	**1.08**	
#356 / #507	#727 (20"x0.375")	KTORM270	**1.11**	
(20"x 0.375")				

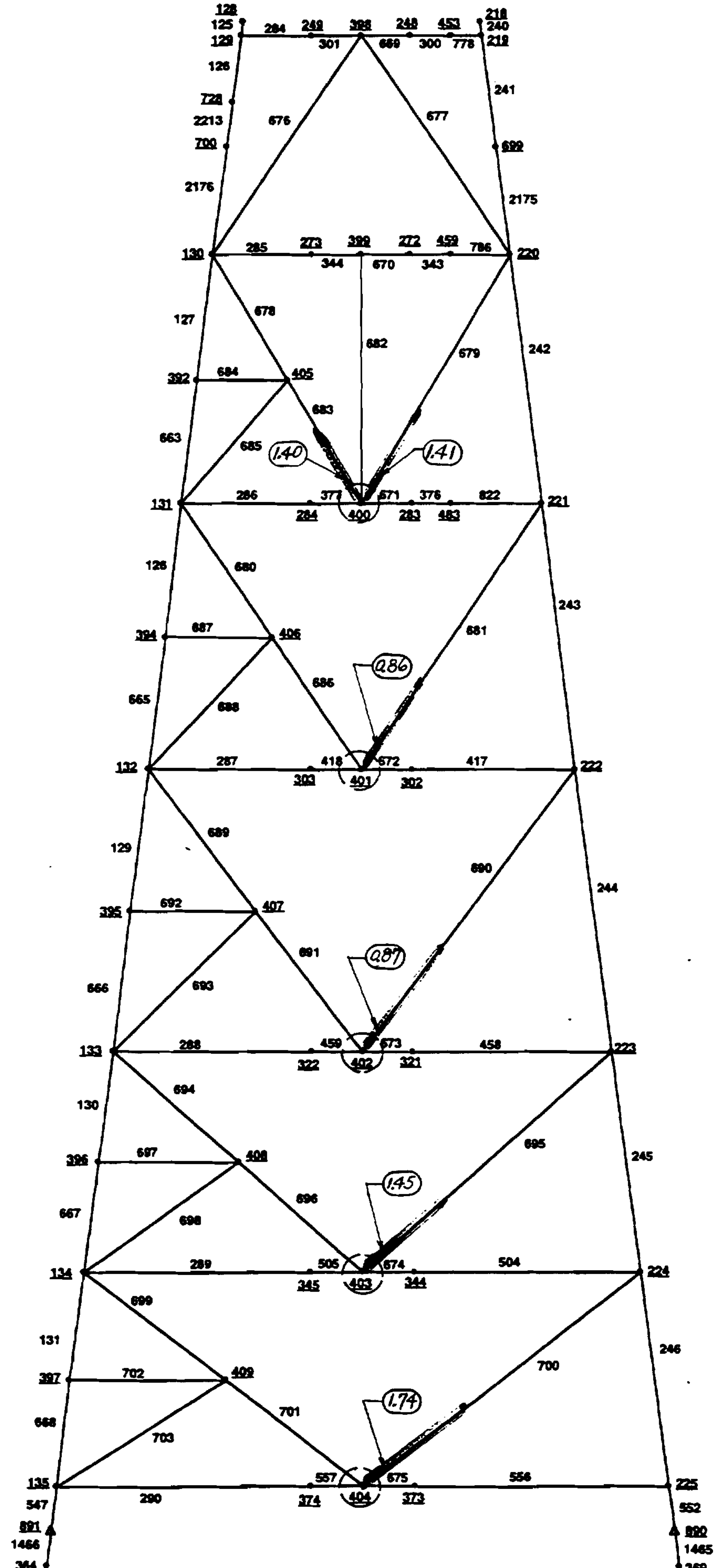
		DESIGN LEVEL	DESIGN LEVEL	
			W/ Overlapping	
JOINT NO./CHORD NO.	BRACE NO.	LOADING CASE	Stress Ratio	
#385 / #559	#732 (24"x0.375")	KTORM270	**1.29**	
(24"x 0.375")				
ROW 4				
#290 / #383	#653 (18"x0.563")	KTORM270	0.96	
(18"x0.563")	#652 (18"x0.563")	KTORM270	0.92	
#380 / #207	#571 (20"X0.375")	KTORM195	**1.07**	
(24"x 0.375")	#662 (24"X0.375")	KTORM270	0.97	
EL. (+) 3' - 0				
#246 / #298	#312 (14"X0.375")	HTORM090	**1.24**	
(18"x0.375")				
#247 / #277	#313 (14"X0.375")	KTORM240	**2.03**	
(18"x0.375")				
EL. (-) 216' - 0				
#348 / #275	#525 (16"x0.375")	KTORM195	0.90	
(24"x 0.375")				
EL. (-) 254' - 0				
#371 / #554	#569 (22"x0.375")	HTORM045	**1.03**	
(24"x 0.375")				
#372 / #283	#570 (22"x0.375")	HTORM315	**1.12**	
(24"x 0.375")				
#373 / #556	#2124 (22"x0.375")	KTORM270	**6.64**	
(24"x 0.375")	#580 (16"x0.375")	KTORM195	**1.06**	
	#584 (12.75"x0.375")	KTORM270	**11.98**	
#374 / #290	#2128 (22"x0.375")	KTORM270	**7.22**	
(24"x 0.375")	#585 (12.75"x0.375")	KTORM270	**12.57**	
	#582 (16"x0.375")	KTORM135	**1.07**	
#375 / #558	#591 (12.75"x0.375")	KTORM270	**1.02**	
(24"x 0.375")				
#376 / #291	#587 (12.75"x0.375")	KTORM180	**1.32**	
(24"x 0.375")				
#377 / #276	#572 (18"x0.375")	KTORM195	0.91	
(24"x 0.375")	#573 (18"x0.375")	HTORM315	**1.03**	

		DESIGN LEVEL	DESIGN LEVEL	
			W/ Overlapping	
JOINT NO./CHORD NO.	BRACE NO.	LOADING CASE	Stress Ratio	
#383 / #138	#574 (18"x0.375")	KTORM180	0.85	
(24"x 0.375")	#575 (18"x0.375")	HTORM000	0.97	



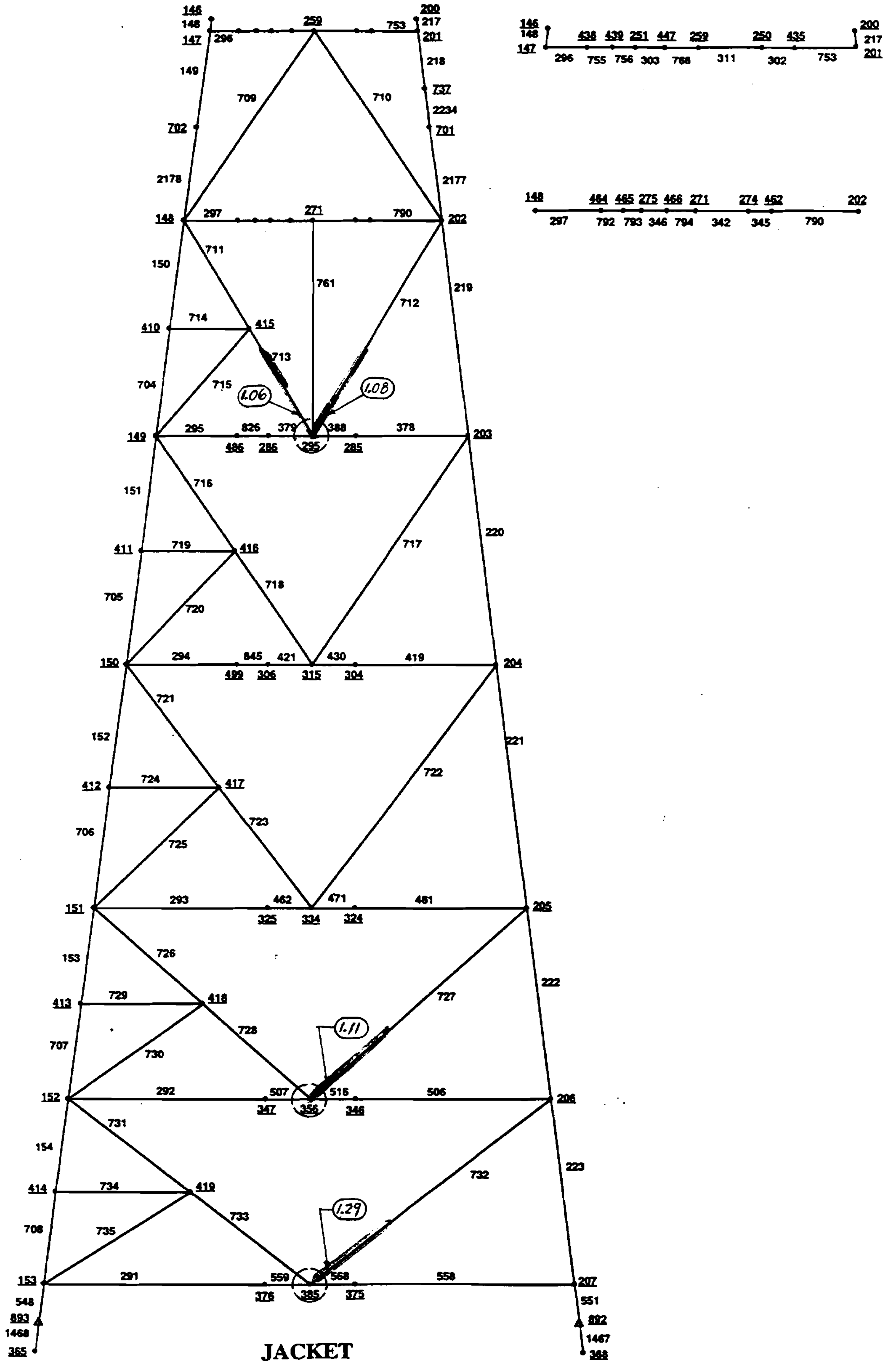
JACKET
ROW 1

4/23/94



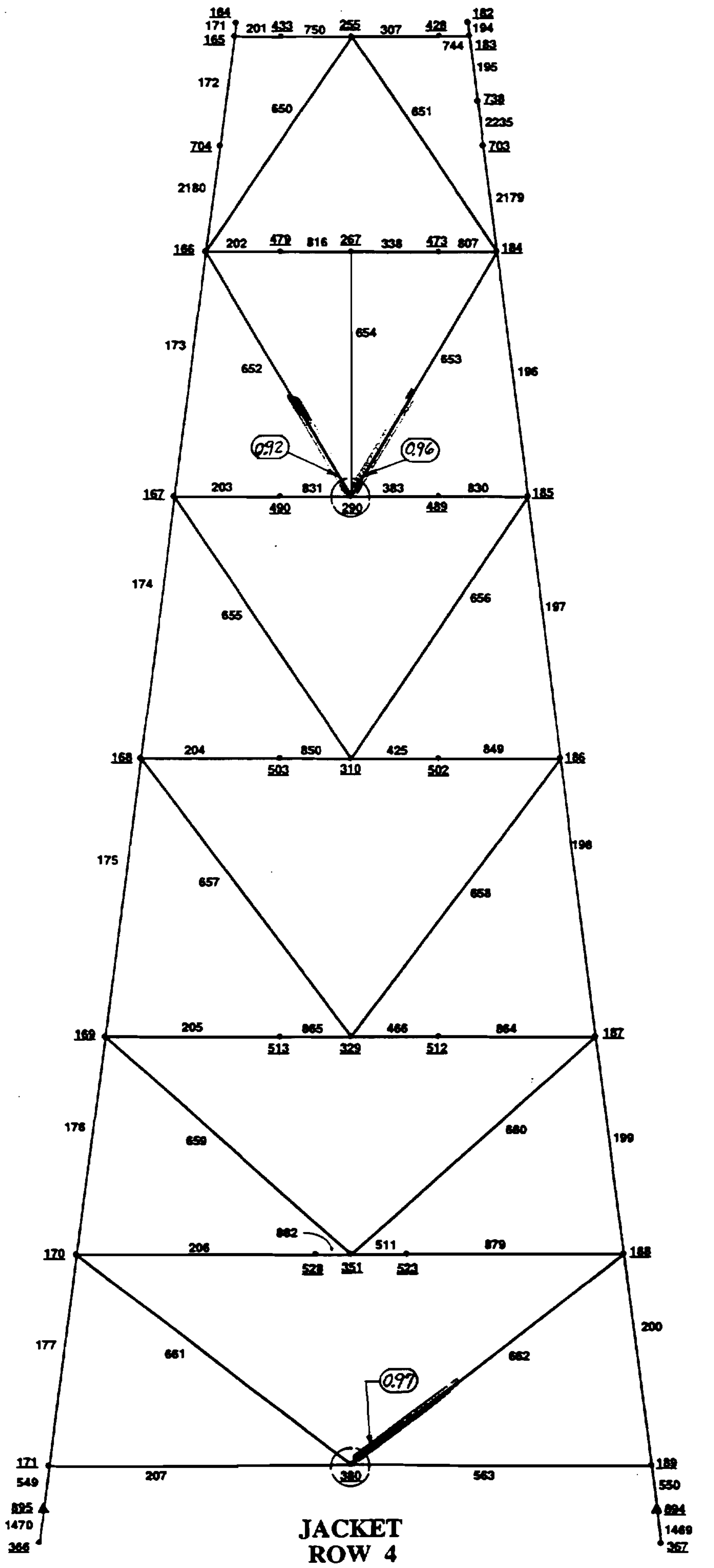
JACKET
ROW 2

4/23/94



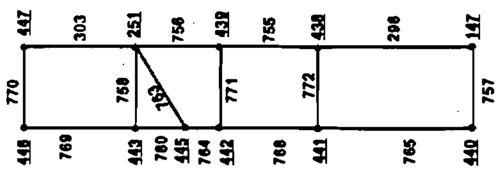
JACKET
ROW 3

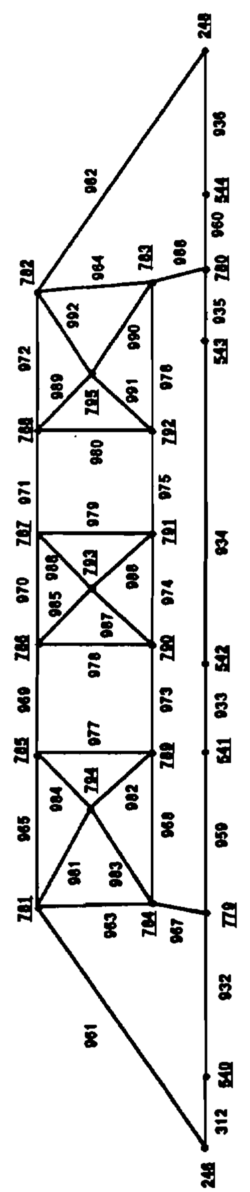
4/23/94

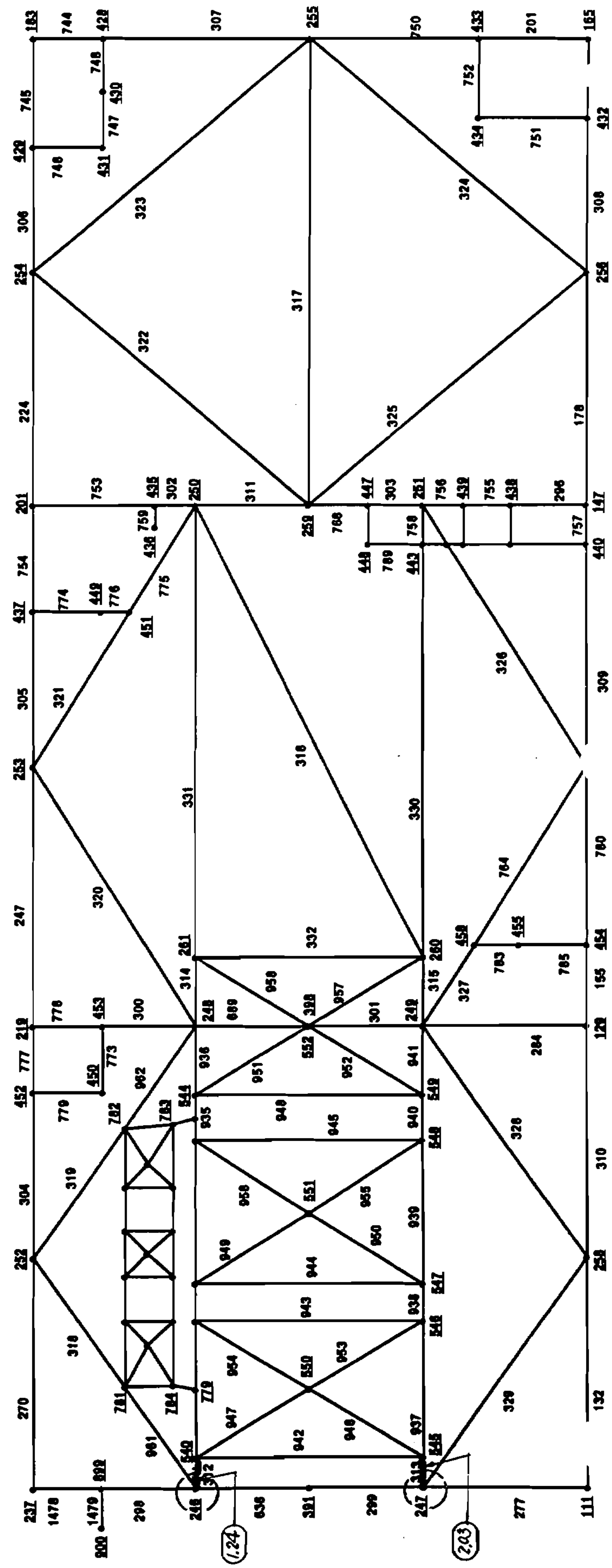


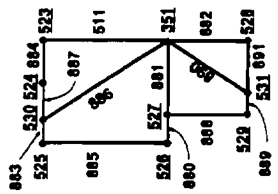
JACKET
ROW 4

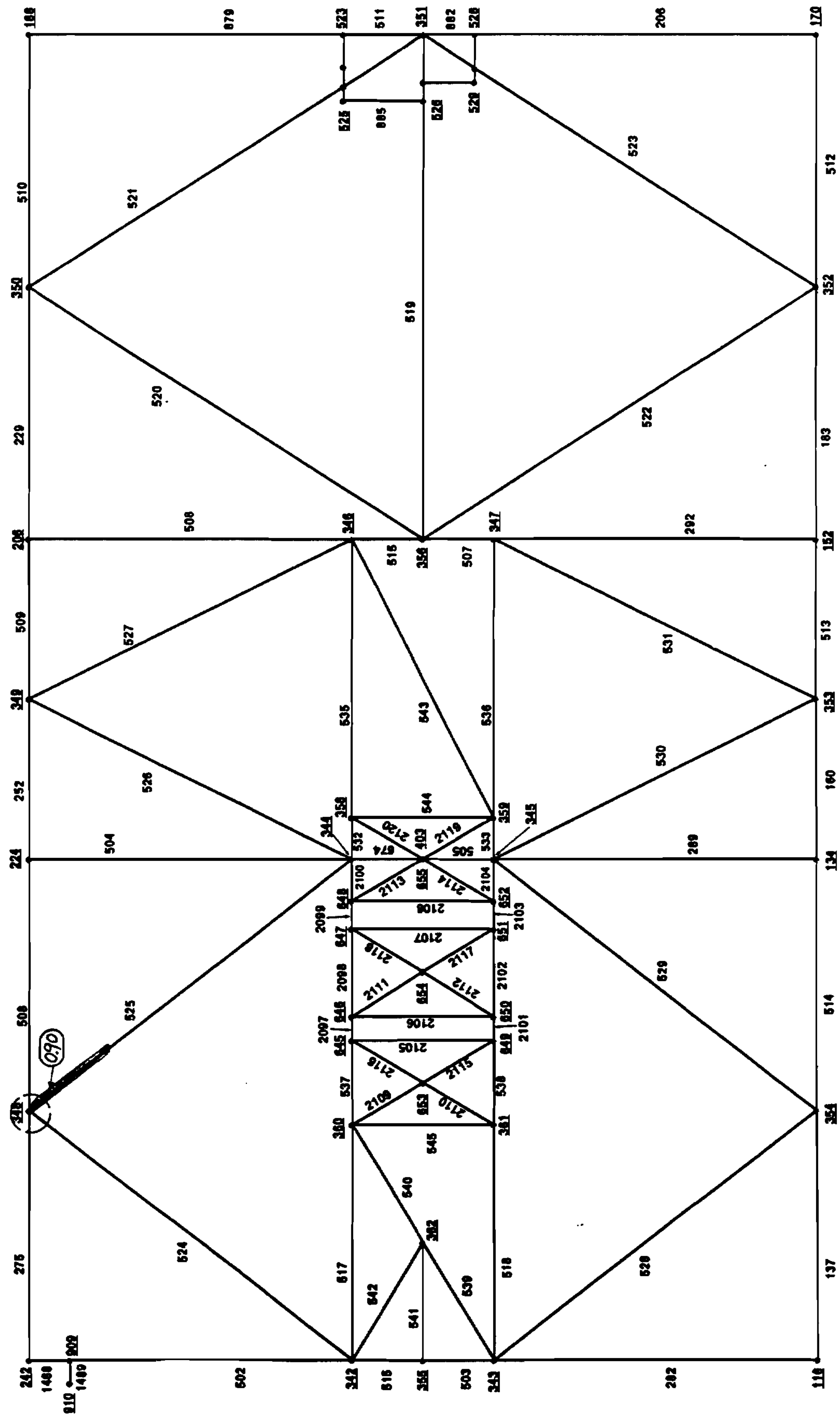
4/23/94

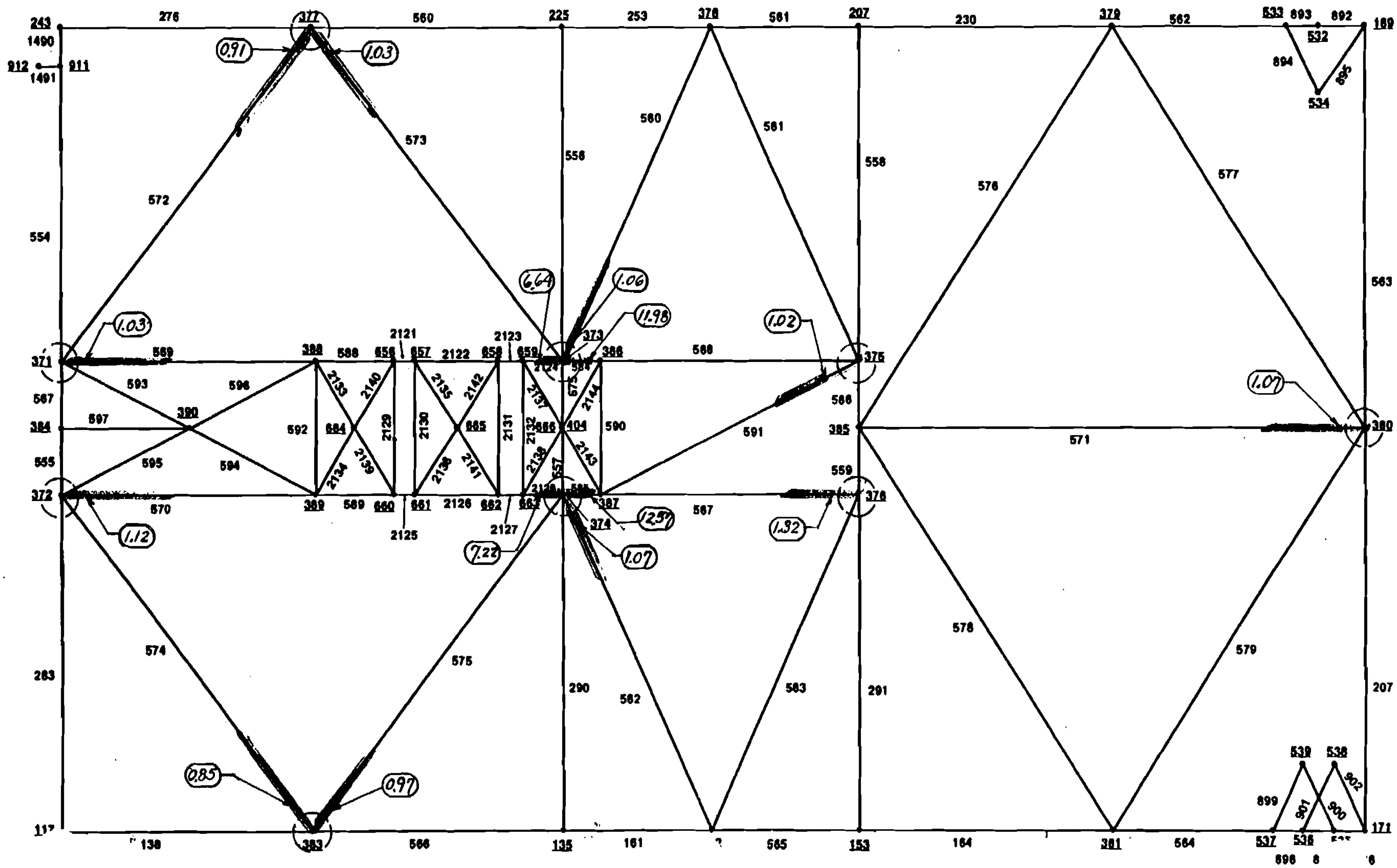











243 276 377 560 225 253 376 561 207 230 379 562 533 893 892 189
1490
912 911
1491
532
894 895
534
0.91 1.03
573
560 561
556 558
572
576 577
554
563
6.64 1.06
11.98 1.02
1.03
371 569 388 588 656 2121 657 2122 658 2123 659 373 386 568 375
593 596
567
390
384 597 592 684 2129 2130 665 2131 666 404 590 591 385 571 380
1.07
555 595 594 586
559
372 570 389 589 660 2125 661 2126 662 2127 663 387 567 376
1.12 7.22 374 12.57 1.32
1.07
574 575 578 579
283 290 582 583 291 207
0.85 0.97
539 538
899 901 900 902
117 138 383 566 135 161 565 153 164 381 564 537 536 171
898

A.5.3.7 Strength Check of Piles

The results of SPIA analysis were used to perform the pile strength check. The material yield strength of the piles is 36 ksi (nominal) typically. An allowable stress increase factor of 1.33 is used in the pile strength check.

The pile strength check results showed that the highest stress ratio was 0.499 which occurred in the pile (B-1); it was due to the combined loading in the wave direction of 135 deg. Apparently, the piles have appreciable reserve strength and the pile's response was still in the linear stress range at this design load level.

The results of the strength check of piles for design level analysis, ultimate strength analysis and API RP 2A 20th edition are shown in the following attachment.

E.I. BLK 330C PLATFORM ASSESSMENT 8/3/94

INSIGNIFICANT ENVIRONMENTAL IMPACT (MANNED - EVACUATED)

SPIA ANALYSIS RESULTS

PILE NO.	SUPER NODE	ULTIMATE Fy (ksi)	ULTIMATE LOADING Case	ULTIMATE Stress Ratio	Fy (ksi)	DESIGN LEVEL LOADING Case	DESIGN LEVEL Stress Ratio	20TH EDITION LOADING Case	20TH EDITION Stress Ratio
#1419 (A-4)	#423	36.00	HTORM225	0.531	36.00	KTORM270	0.426	DTORM270	0.760
#1420 (A-3)	#422	36.00	HTORM195	0.722	36.00	KTORM270	0.457	DTORM240	0.893
#1421 (A-2)	#421	36.00	HTORM195	0.764	36.00	KTORM270	0.462	DTORM240	0.945
#1422 (A-1)	#420	36.00	HTORM195	**0.793**	36.00	KTORM225	0.495	DTORM240	**1.066**
#1423 (B-1)	#424	36.00	HTORM195	0.791	36.00	KTORM135	**0.499**	DTORM195	0.822
#1424 (B-2)	#425	36.00	HTORM180	0.660	36.00	KTORM135	0.446	DTORM195	0.653
#1425 (B-3)	#426	36.00	HTORM180	0.624	36.00	KTORM135	0.424	DTORM195	0.610
#1426 (B-4)	#427	36.00	HTORM195	0.601	36.00	HTORM090	0.380	DTORM225	0.739

A.5.4 Ultimate Strength Analysis

A.5.4.1 Metocean Criteria

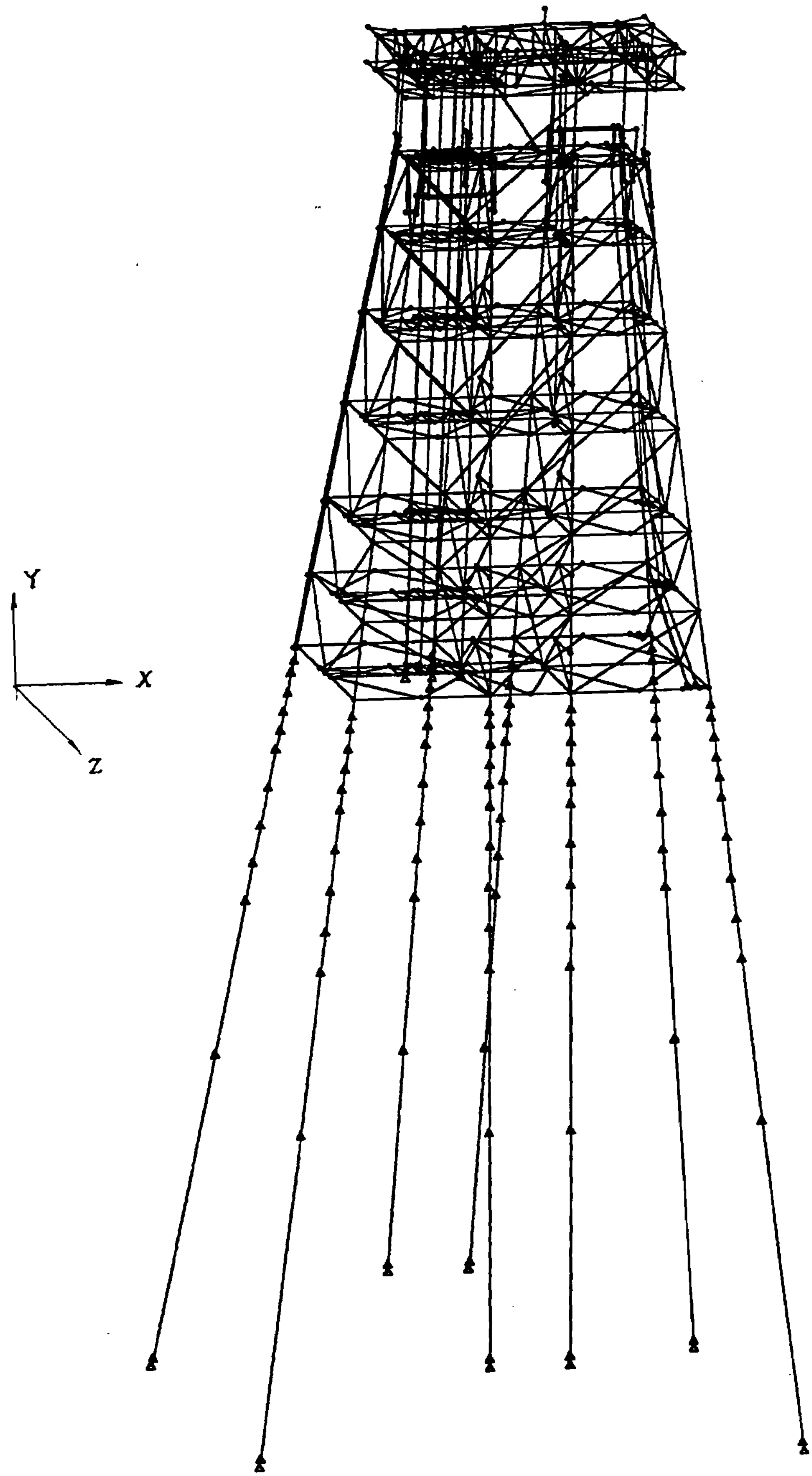
The platform is considered as insignificant environmental impact/manned-evacuated. The metocean criteria for both ultimate strength analyses are based on 100-year force due to the combined sudden hurricane and winter storm population. The directionality of the wave and current must be taken into account. The wave height, associated current and profile as a function of direction, are calculated based on API PR 2A 20th edition guidelines, except that the directional factors are based on Figure 17.6.2-4 of Section 17.0 (draft) of API RP 2A. The required wave height, wave period and current magnitude are described above (see Section A.5.1). See Figure 3 in Section A 5.1- Metocean Criteria/Loads for further detail on the base shear plots versus wave direction for the comparison of different wave criteria.

A.5.4.2 Nonlinear Analysis Model and Boundary Conditions

The structural model is a three-dimensional model. The structural model consists of 832 joints (nodes) and 1,676 members (elements). The nonlinear model is comprised of linear and nonlinear elements. In the jacket part, the STRUT-BEAM element is used to model vertical diagonal bracings, horizontal diagonal bracings and the primary horizontal members around the perimeter at each level. The jacket-legs and piles below the mudline are modelcd as BEAM-COLUMNs. The rest of the structural members are modeled as BEAM elements. Typically, six degrees of freedom are considered at each node (joint).

In the nonlinear analysis model, the main piles below the mudline are discretized into 10 BEAM-COLUMN elements in each pile. The soil P-Y, T-Z and Q-Z curves are properly modeled in the pile foundation. The conductor model is slightly different from that of design level analysis in that the conductors were extended only four feet below the mudline and terminated by hinge supports (three translational restraints at each support).

A sketch of the nonlinear analysis model is shown in the following attachment.


Y
X
Z

A.5.4.3 Selection of Wave Directions for Static Push-Over Analysis

In the design level analysis, 10 wave directions are considered (0, 45, 90, 135, 180, 225, 270, 315 ,195 and 240 degrees). The first eight wave directions are starting from the platform X-axis (longitudinal direction) as 0 degree and increased the angles incrementally in 45 deg. counter-clockwise. The wave direction of 195 deg. corresponds to the worst case (wave height factor = 1.0) for insignificant environmental impact/manned - evacuated metocean criteria. Whereas, the wave direction of 240 deg. corresponds to the maximum case for API RP 2A 20th edition. For nonlinear analyses, it would be very expensive to perform the static push-over analysis for all 10 wave directions as described.

In the following static push-over analyses, four wave directions (195, 240, 180 and 270 degrees) were selected. The reason for the selection of those four wave directions was based on the results of wave base shear calculations (see Figure 3 in Section 5.1) and that of code checks for ultimate strength analysis.

A.5.4.4 Static Push-Over Analysis (No Joint Capacity Determined)

In the static push-over analysis, first one applies the vertical loads to the structural system to evaluate the structural response, such as joint displacements, member end forces etc. The vertical loads include structural weight, deck dead and live loads, conductor weight, buoyancy etc. Secondly, lateral load is applied (wave, current and wind) to the structure in increments with each load step corresponding to a fraction (or multiple) of the ultimate lateral load. Each wave direction is considered separately. Four wave directions were performed in this study. The incremental load factors applied to wave direction 195 degrees, for example, are: 0.20, 0.40, 0.60, 1.00, 1.20, 1.21, 1.22, 1.23, 1.24, 1.25, 1.26, 1.27, 1.28, 1.30 and 1.40. For other wave directions, refer to the plots of ultimate lateral load factors versus deck displacements for further details (see Figures 13, 15, 17 in Section A.5.4.8, which will be presented in the latter part of this report.).

It should be pointed out that no fatigue checks were performed for the design level or ultimate strength analyses in this study. Furthermore, in the subsequent static push-over analysis, it is assumed that the major joints in the structure have sufficient joint capacity to withstand whatever resultant (member-end) load that occurred for each incremental step of the static push-over analysis up to failure.

A.5.4.5 Load Level at Ultimate Capacity of Platform

For Ultimate Strength Analyses, the wave height (H_W), current value and current profile are function of wave directions. Current blockage factors are directionally dependent with respect to platform's orientation (longitudinal, transverse or diagonal) per API PR 2A 20th edition guideline. A wave kinematics factor of 0.88 is used. Drag and inertia coefficients take into account the surface effects of tubular members (smooth or rough). The conductor shielding factor is also considered. The marine growth is also modeled per API RP 2A 20th edition.

Typically eight wave directions are used in the calculations of static base shears. In this study, 10 wave directions are applied (0 , 45 , 90 , 135 , 180 , 225 , 270 , 315 , 195 and 240 degs.). The wave directions are defined with respect to the platform coordinates system (see sketch of platform orientation). The wave direction of 240 deg. corresponds to the worst wave direction (wave height factor = 1.0) in API RP 2A 20th edition, which is equivalent to the exposure category of significant environmental impact/manned-evacuated. The wave direction of 195 deg. corresponds to the worst wave condition for platforms classified as insignificant environmental impact/manned-evacuated. The reason the wave direction of 240 deg. is included is that in the subsequent inelastic ultimate strength analysis, the reserve strength ratio must be determined and is defined with reference to API RP 2A 20th edition guideline (which corresponds to significant environmental impact/manned - evacuated).

The lateral load level due to wave and current effects is summarized as follows:

Design wave height: 59 ft.
Design wave period: 12.5 sec (Apparent wave periods: varied)

Wave Direction (deg.)	Wave Height (ft.)	Lateral Load (Wave & Current) (kips)
0	41.30	1,258
45	41.30	1,133
90	47.20	1,700
135	54.10	2,254
180	58.02	2,831
225	55.05	2,975
270	47.20	2,356

continued

Wave Direction (deg.)	Wave Height (ft.)	Lateral Load (Wave & Current) (kips)
315	42.30	1,622
240	53.10	2,881
195	59.00	**3,077**

The maximum lateral load is 3,077 kips in the wave direction of 195 deg. See Figure 3 in Section A.5.1 for the comparison of lateral loads (wave and current) for different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition).

A.5.4.6 Alternative Linear Approach to Ultimate Strength Analysis

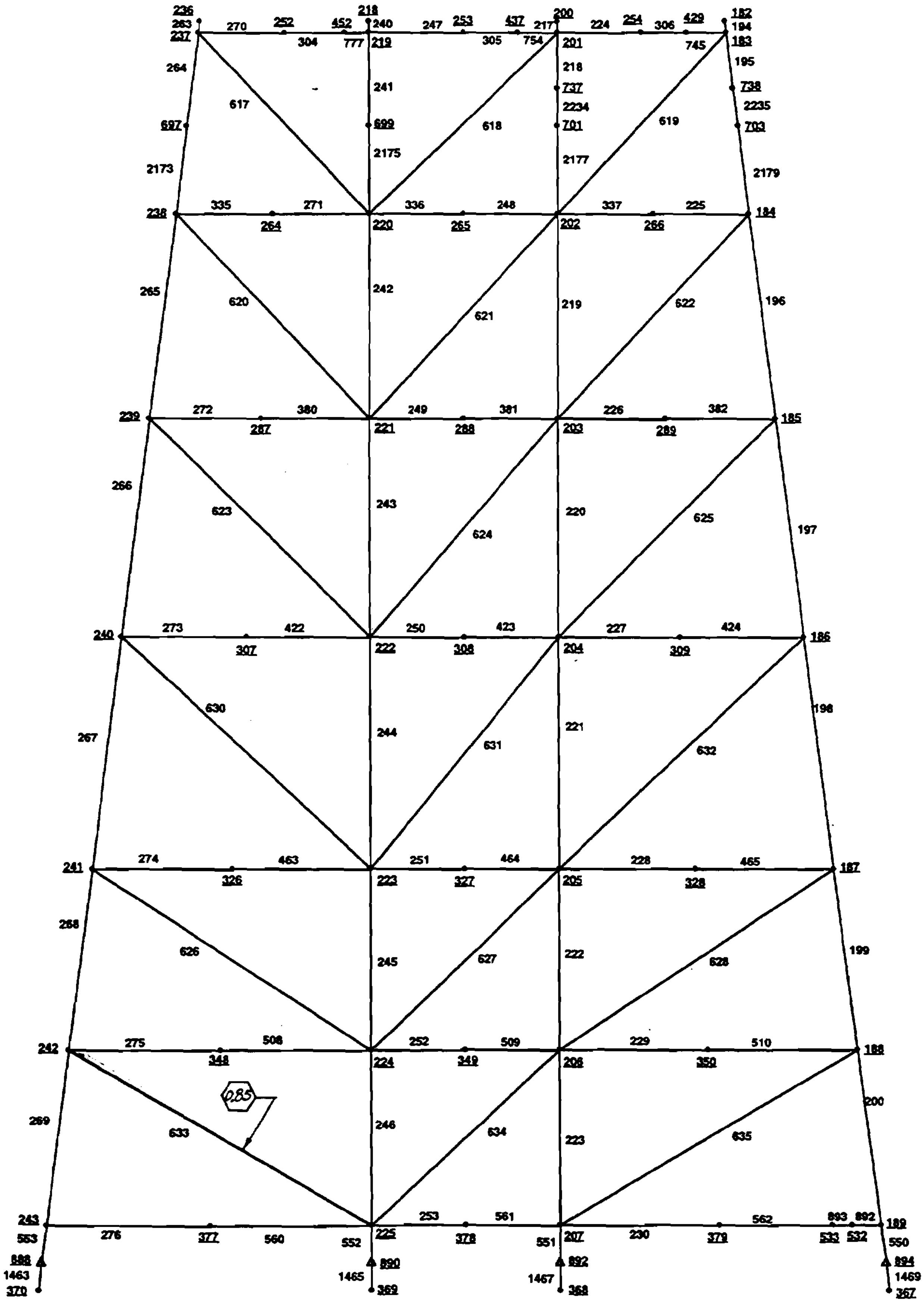
Per API RP 2A Section 17.0 (draft) – Assessment of Existing Platforms, the ultimate strength of undamaged members, joint and piles may be established using the formulas of Sections 3, 4, 6 and 7 of API RP 2A 20th edition with all safety factors removed (i.e, a safety factor of 1.0). The ultimate strength of joints may also determined using a mean "formula or equation" versus the lower bound formulas for joints in Section 4.

Tubular member check, joint check and pile strength check were performed for this ultimate strength analysis using the same structural model and linear stiffness analysis used in the design level analysis. The material yield strength (Fy) was 36 ksi for both tubular member check and pile strength check. However, for joint check, the material yield strength (Fy) of 42 ksi was used. The allowable stress increase factor of 1.0 was specified since the removal of safety factors in the checking equations made the allowables, in effect, ultimate capacities.

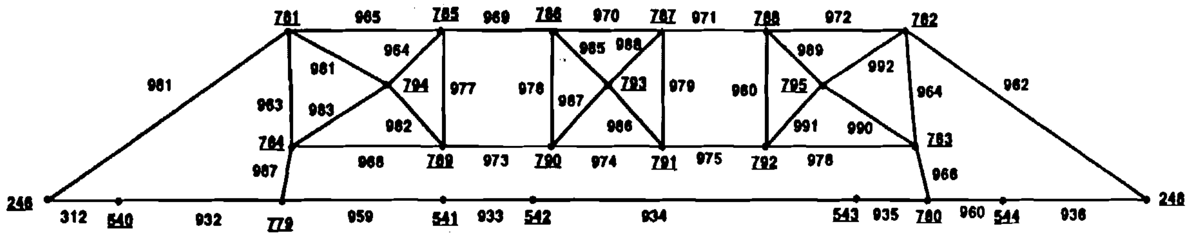
The results of the tubular member check, joint check and pile strength check with stress ratio greater than 0.85 are summarized in the Excel spreadsheet. The members and joints/braces with interaction stress ratio greater than 0.85 are also shown in the attached sketches.

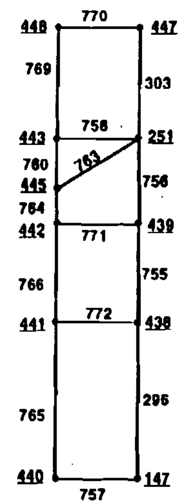
E.I. BLOCK 330 C PLATFORM ASSESSMENT					8/7/94
Insignificant Environmental Impact / Manned - Evacuated					
TUBECHECK (MEMBER CHECK) RESULTS					
ULTIMATE STRENGTH ANALYSIS		(Fy = 36 ksi)			
MEMBER NO.	LOADING CASE	STRESS RATIO	Controlling Case	REMARKS	
EL. (+) 3' - 0					
#744 (18"x0.375")	HTORM240	**1.07**	HYDROSTATIC		L = 3.15'
#748 (10.75"x0.365")	HTORM225	1.17	HYDROSTATIC	Secondary Member	L = 2.58'
#772 (6.625"x0.375")	HTORM240	1.20	HYDROSTATIC	Secondary Member	L = 1.97'
#774 (6.625"x0.280")	HTORM225	0.86	HYDROSTATIC	Secondary Member	L = 4.14'
#776 (6.625"x0.280")	HTORM270	0.88	HYDROSTATIC	Secondary Member	L = 1.37'
#783 (6.625"x0.280")	HTORM135	1.02	HYDROSTATIC	Secondary Member	L = 2.55'
#785 (6.625"x0.280")	HTORM225	1.63	HYDROSTATIC	Secondary Member	L = 4.14'
#1478 (18"x0.375")	HTORM135	**1.20**	HYDROSTATIC		L = 3.03'
#1479 (8.625"x0.322")	HTORM135	**3.13**	HYDROSTATIC		L = 1.58'
EL. (-) 36' - 0					
#1481 (8.625"x0.322")	HTORM180	0.92	HYDROSTATIC	Secondary Member	L = 1.74'
EL. (-) 80' - 0					
#134 (16"x0.375")	HTORM180	0.85	STRENGTH		
#272 (16"x0.375")	HTORM195	1.00	STRENGTH		
EL. (-) 127' - 0					
#273 (18"x0.375")	HTORM195	**1.02**	STRENGTH		
#422 (18"x0.375")	HTORM195	1.00	STRENGTH		
EL. (-) 254' - 0					
#230 (24"x0.375")	HTORM195	0.85	HYDROSTATIC		
#253 (24"x0.500")	HTORM195	0.86	HYDROSTATIC		
#560 (24"x0.375")	HTORM195	**1.43**	STRENGTH		
#562 (24"x0.375")	HTORM195	0.93	HYDROSTATIC		
#564 (24"x0.375")	HTORM195	0.87	HYDROSTATIC		
#566 (24"x0.375")	HTORM195	**1.21**	STRENGTH		
#892 (24"x0.375")	HTORM195	**1.21**	HYDROSTATIC		
#893 (24"x0.375")	HTORM195	**1.04**	HYDROSTATIC		
#896 (24"x0.375")	HTORM195	0.96	HYDROSTATIC		
#897 (24"x0.375")	HTORM195	0.97	HYDROSTATIC		
#898 (24"x0.375")	HTORM195	0.94	HYDROSTATIC		
#902 (10.75"x0.365")	HTORM240	0.95	HYDROSTATIC	Secondary Member	
#1490 (24"x0.375")	HTORM225	**1.10**	HYDROSTATIC		
#1491 (8.625"x0.322")	HTORM195	**2.87**	HYDROSTATIC	Secondary Member	L=1.33'

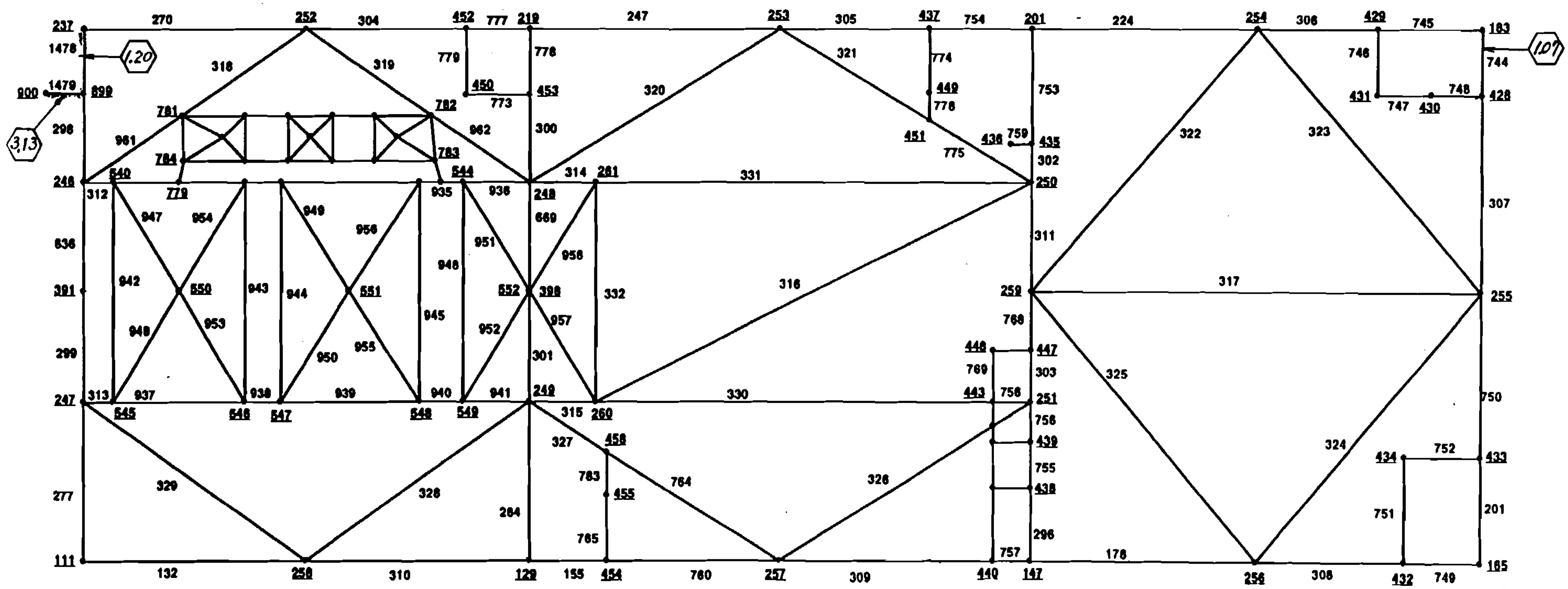
MEMBER NO.	LOADING CASE	STRESS RATIO	Controlling Case		
ROW B					
#633 (30"x0.500")	HTORM195	0.85	HYDROSTATIC		

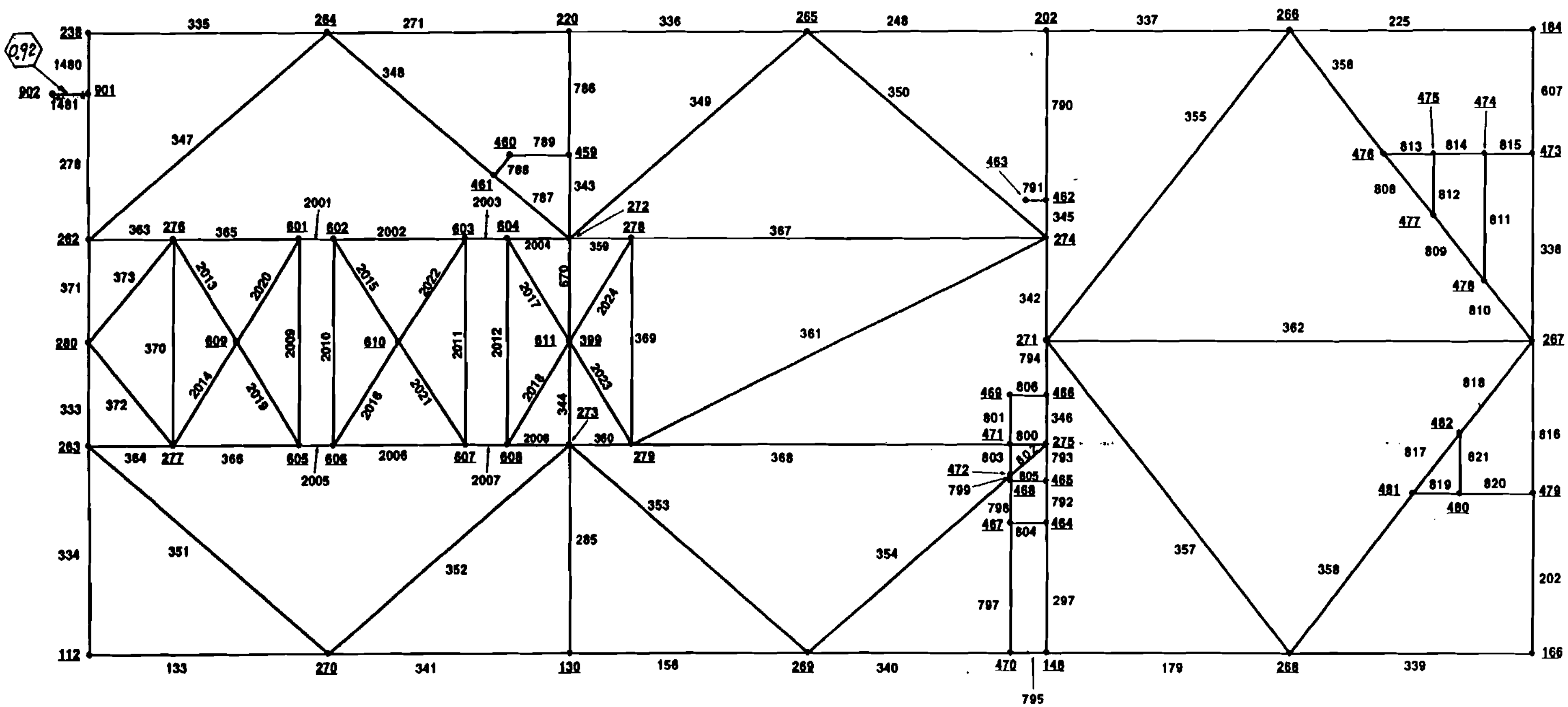


JACKET
ROW B

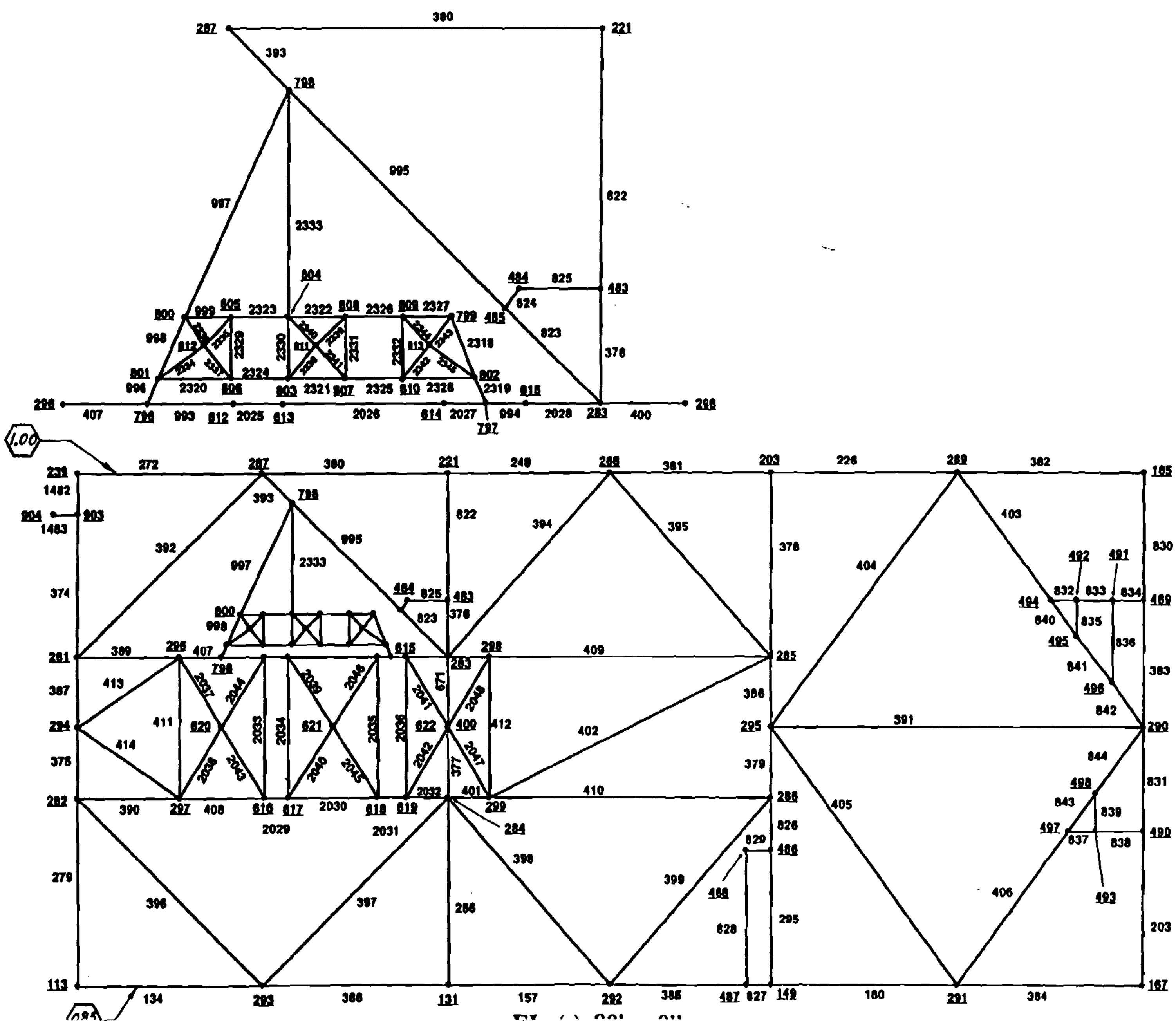


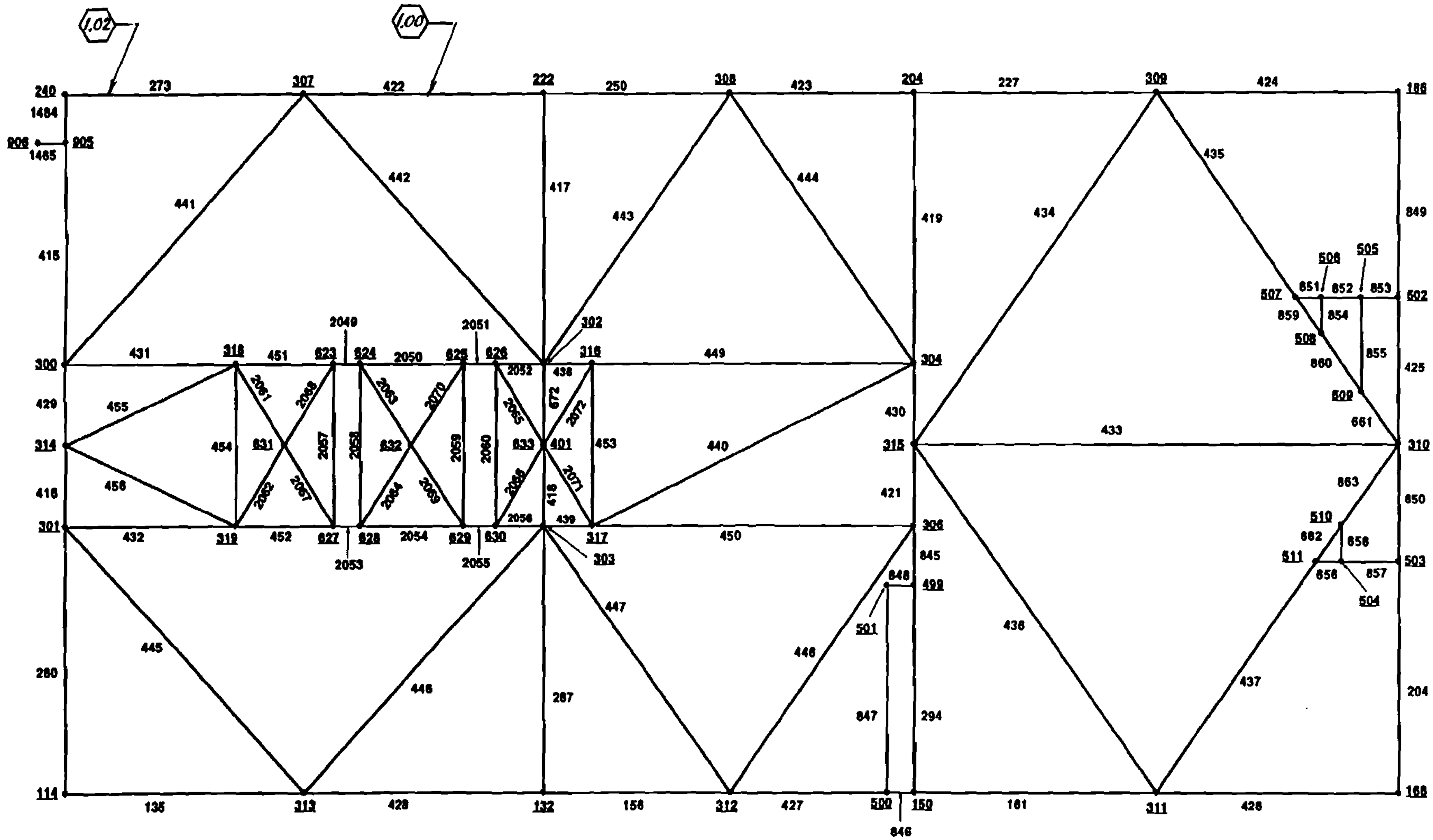




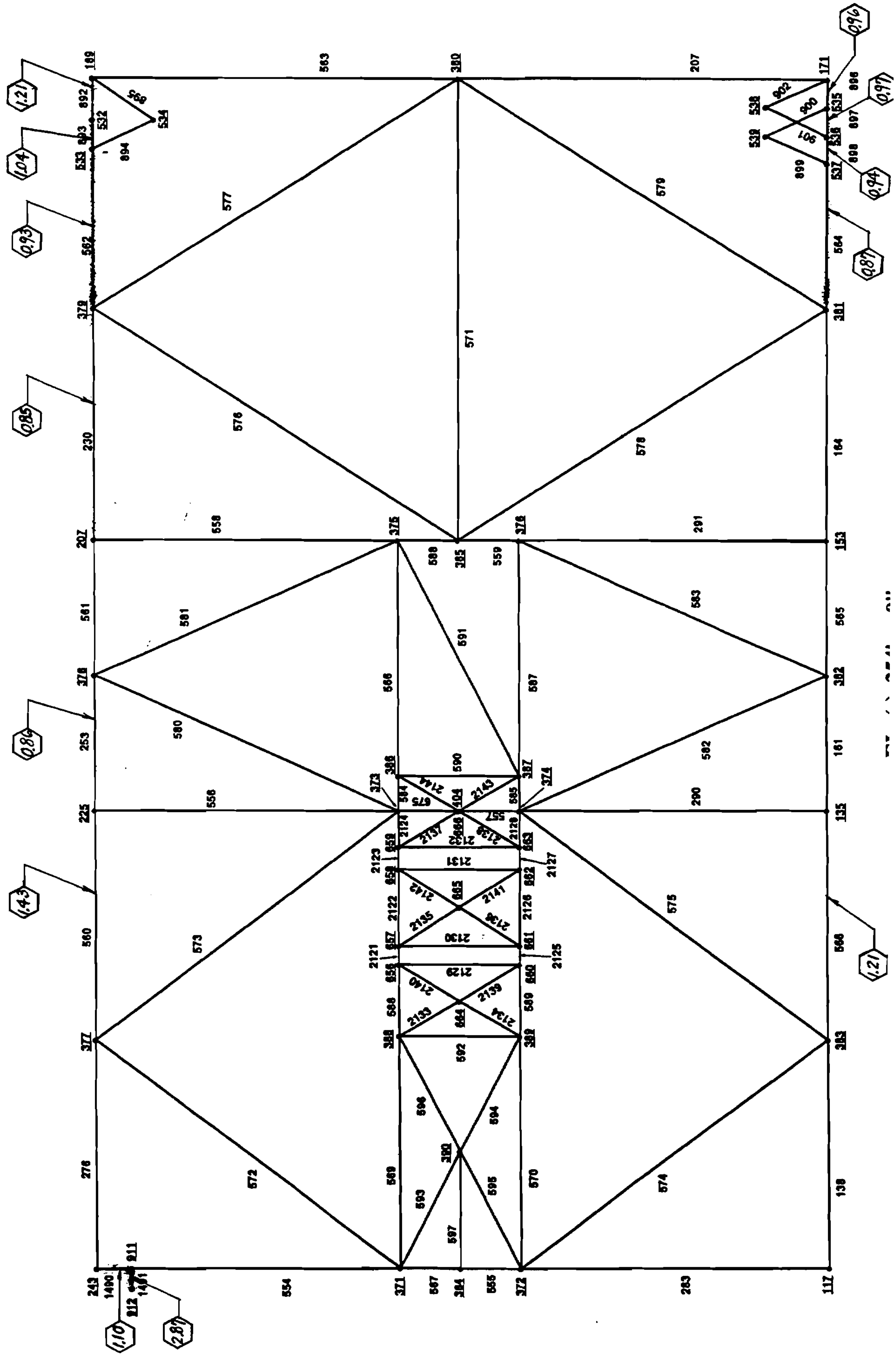


0.92
238 335 284 271 220 336 265 248 202 337 266 225 164
1480
902 901
1481
348 786 349 350 790 355 356 607
347 460 789 459 463 475 474
278 788 343 791 462 476 813 814 815 473
461 787 345 808 812
2001 2003 272 477 811
262 363 276 365 601 602 2002 603 604 2004 278 367 274
359 809 338
371 373 2013 2020 2015 2022 2017 570 2024 342 476
370 609 2009 2010 610 2011 2012 611 399 369 361 810
280 271 362 267
794
333 372 2014 2019 2016 2021 2018 2023 344 273 469 806 466 818
801 346
263 364 277 366 605 606 2006 607 608 2008 360 279 368 471 800 275 482 817 816
2005 2007 803 807 793 821
472 805 465 481 819 820 479
799 468 792 480
353 798
334 351 352 285 467 804 464
354 357 358
797 297 202
112 133 270 341 130 156 269 340 470 148 179 268 339 166
795




1.02
1.00
240
273
307
422
222
250
308
423
204
227
309
424
186
1484
906
905
1465
441
442
417
443
444
419
434
435
849
415
506
505
507
851
852
853
502
859
854
508
855
860
425
509
661
2049
2051
302
300
431
318
451
623
624
2050
625
626
2052
316
438
449
304
429
455
2061
2068
2063
2070
2065
672
2072
430
314
454
631
2057
2058
632
2059
2060
633
401
453
440
315
433
310
416
456
2062
2067
2064
2069
2066
418
2071
421
863
850
301
432
319
452
627
628
2054
629
630
2056
439
317
450
306
510
862
858
2053
2055
303
845
511
856
857
503
848
499
504
447
501
436
445
446
280
287
847
294
437
204
114
135
313
428
132
156
312
427
500
150
161
311
426
168
846

EL (-) 1 - 0''



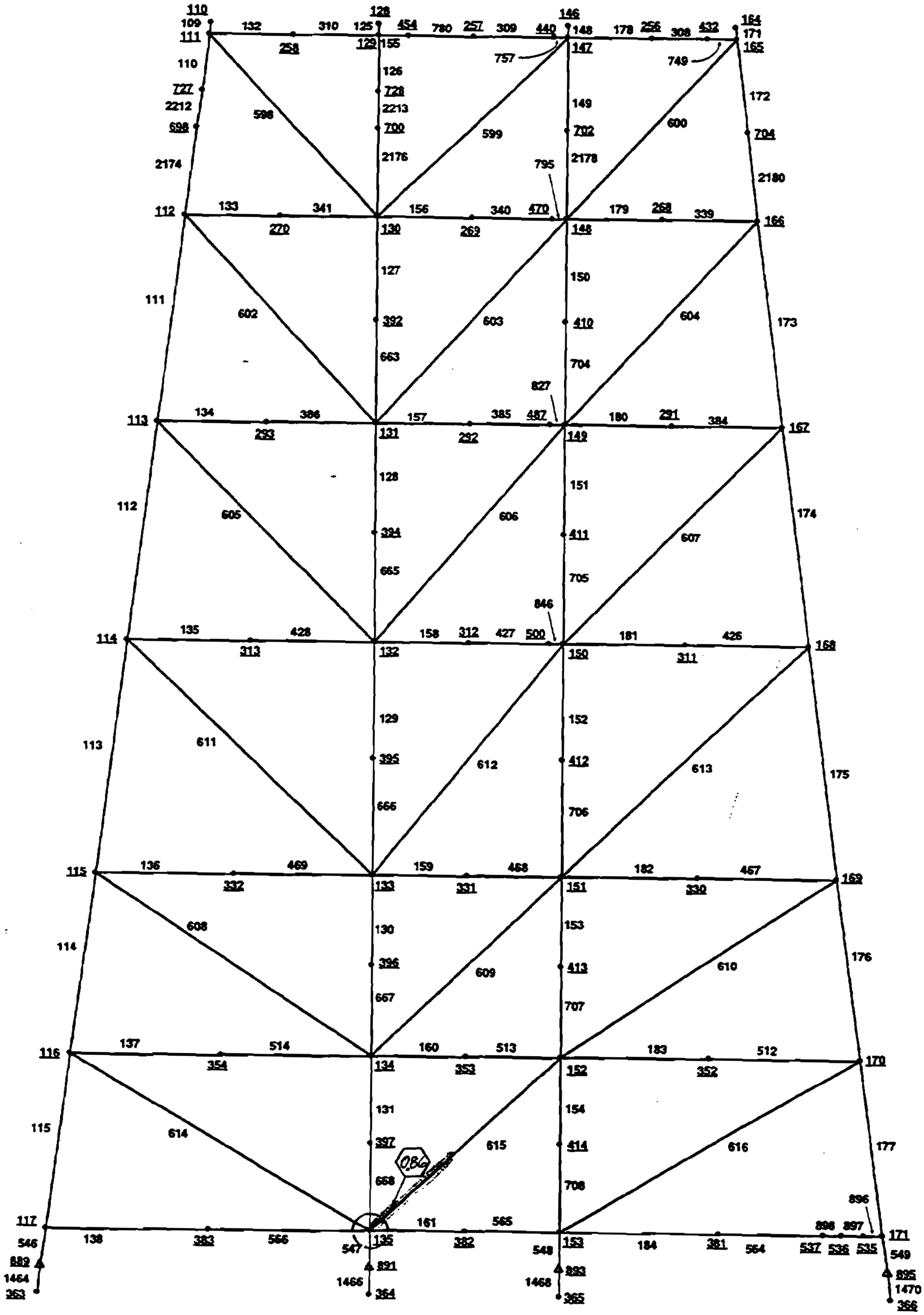
ULTIMATE STRENGTH ANALYSIS

JOINT CHECK RESULTS

LIST OF JOINT / BRACE WITH STRESS RATIO > 0.85

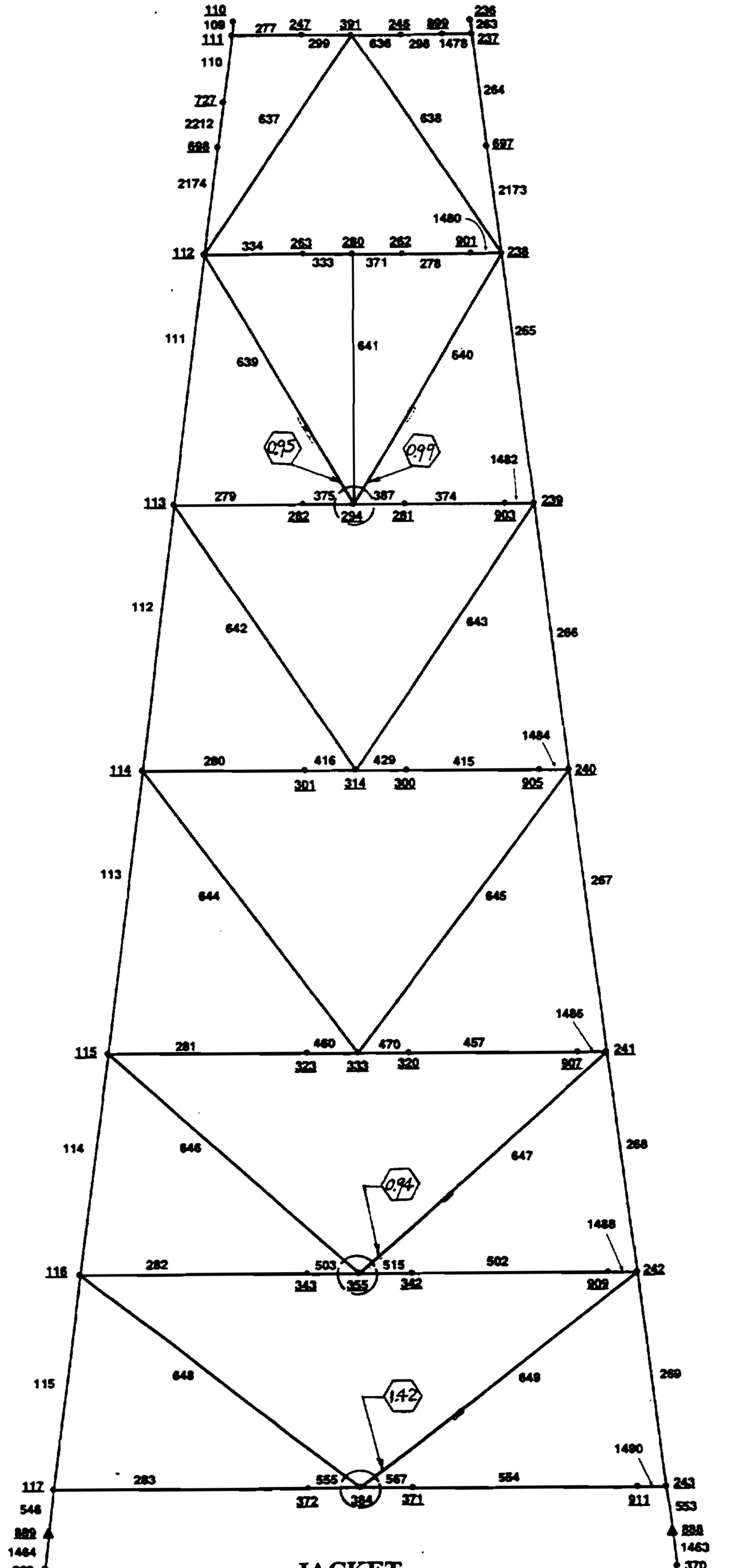
E.I. BLOCK 330C PLATFORM ASSESSMENT				8/3/94
Insignificant Environmental Impact / Manned - Evacuated				
JOINT CHECK RESULTS OF OVERLAPPING JOINTS				
			(Fy = 42 ksi)	
		ULTIMATE	**ULTIMATE**	
			W/ Overlapping	
JOINT NO./CHORD NO.	BRACE NO.	LOADING CASE	Stress Ratio	
ROW A				
#135 / #547	#615 (30"x0.75")	HTORM195	0.86	
(46.5"x1.25")				
ROW 1				
#294 / #375	#639 (18"x0.563")	HTORM240	0.95	
(18"x0.563")	#640 (18"x0.563")	HTORM240	0.99	
#355 / #503	#647 (20"x0.375")	HTORM240	0.94	
(20"x0.375")				
#384 / #555	#649 (24"x0.375")	HTORM240	**1.42**	
(24"x0.375")				
ROW 2				
#133 / #130	#693 (16"x0.50")	HTORM000	N / A	
(46.5"x1.25")				
#400 / 377	#683 (18"x0.563")	HTORM240	1.00	
(18"x0.563")	#679 (18"x0.563")	HTORM240	**1.01**	
#403 / 505	#695 (20"x0.375")	HTORM240	**1.09**	
(20"x0.375")				
#404 / #557	#700 (24"x0.375")	HTORM240	**1.52**	
(24"x 0.375")				
ROW 3				
#151 / #153	#725 (16"x0.50")	HTORM000	N / A	
(46.5"x1.25")				
#356 / #507	#727 (20"x0.375")	HTORM240	0.86	
(20"x 0.375")				
#385 / #559	#732 (24"x0.375")	HTORM240	**1.10**	
(24"x 0.375")				

		ULTIMATE	ULTIMATE	
			W/ Overlapping	
JOINT NO./CHORD NO.	BRACE NO.	LOADING CASE	Stress Ratio	
EL. (+) 3' - 0				
#246 / #298	#312 (14"X0.375")	HTORM135	0.85	
(18"x0.375")				
#247 / #277	#313 (14"X0.375")	HTORM240	**1.34**	
(18"x0.375")				
EL. (-) 254' - 0				
#380 / #207	#571 (20"X0.375")	HTORM195	0.87	
(24"x 0.375")				
#373 / #556	#2124 (22"x0.375")	HTORM240	**6.06**	
(24"x 0.375")	#580 (16"x0.375")	HTORM195	**1.09**	
	#584 (12.75"x0.375")	HTORM240	**9.83**	
#374 / #290	#2128 (22"x0.375")	HTORM240	**5.99**	
(24"x 0.375")	#585 (12.75"x0.375")	HTORM240	**10.59**	
	#582 (16"x0.375")	HTORM180	0.96	
#375 / #558	#591 (12.75"x0.375")	HTORM225	0.90	
(24"x 0.375")				
#376 / #291	#587 (12.75"x0.375")	HTORM180	**1.27**	
(24"x 0.375")				



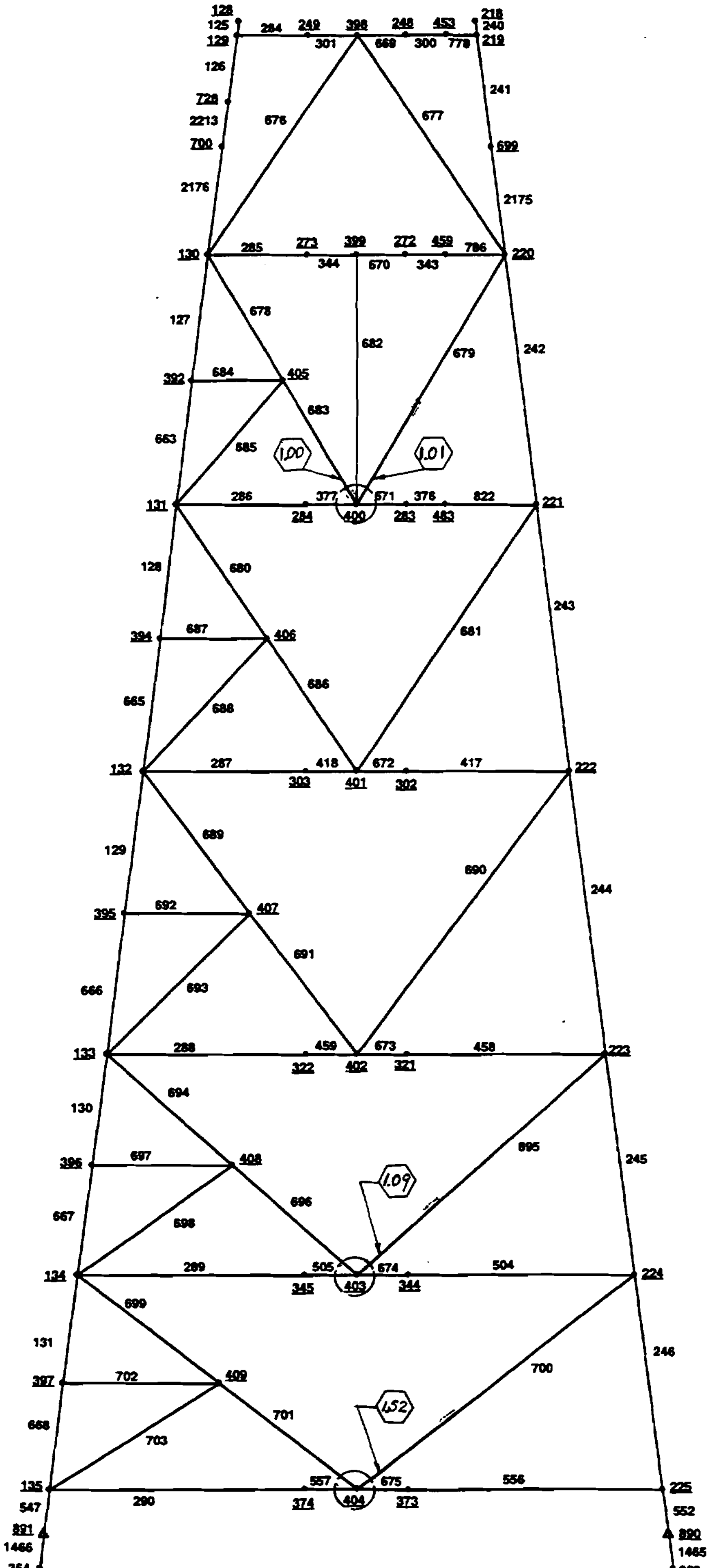
110 109 111 132 310 125 128 454 780 257 309 440 146 148 178 256 308 432 164 171 165
258 129 155 757 147 749
110 126 172
727 728
2212 2213 149
698 700 702 704
598 599 600
2174 2176 795 2178 2180
112 133 341 156 340 470 179 268 339 166
270 130 269 148
111 127 150 173
602 392 603 410 604
663 704
827
113 134 386 157 385 487 180 291 384 167
293 131 292 149
112 128 151 174
605 606 607
394 411
665 705
846
114 135 428 158 312 427 500 181 426 168
313 132 150 311
113 129 152 175
611 612 613
395 412
666 706
115 136 469 159 468 182 467 169
332 133 331 151 330
114 608 130 153 176
396 413
609 610
667 707
116 137 514 160 513 183 512 170
354 134 353 152 352
115 131 154 177
614 397 615 414 616
OBG
668 708
896
117 161 565 896 897 171
546 138 383 566 547 135 382 548 153 184 381 564 537 536 535 549
889 891 893 895
1464 1466 1468 1470
363 364 365 366

JACKET
ROW A



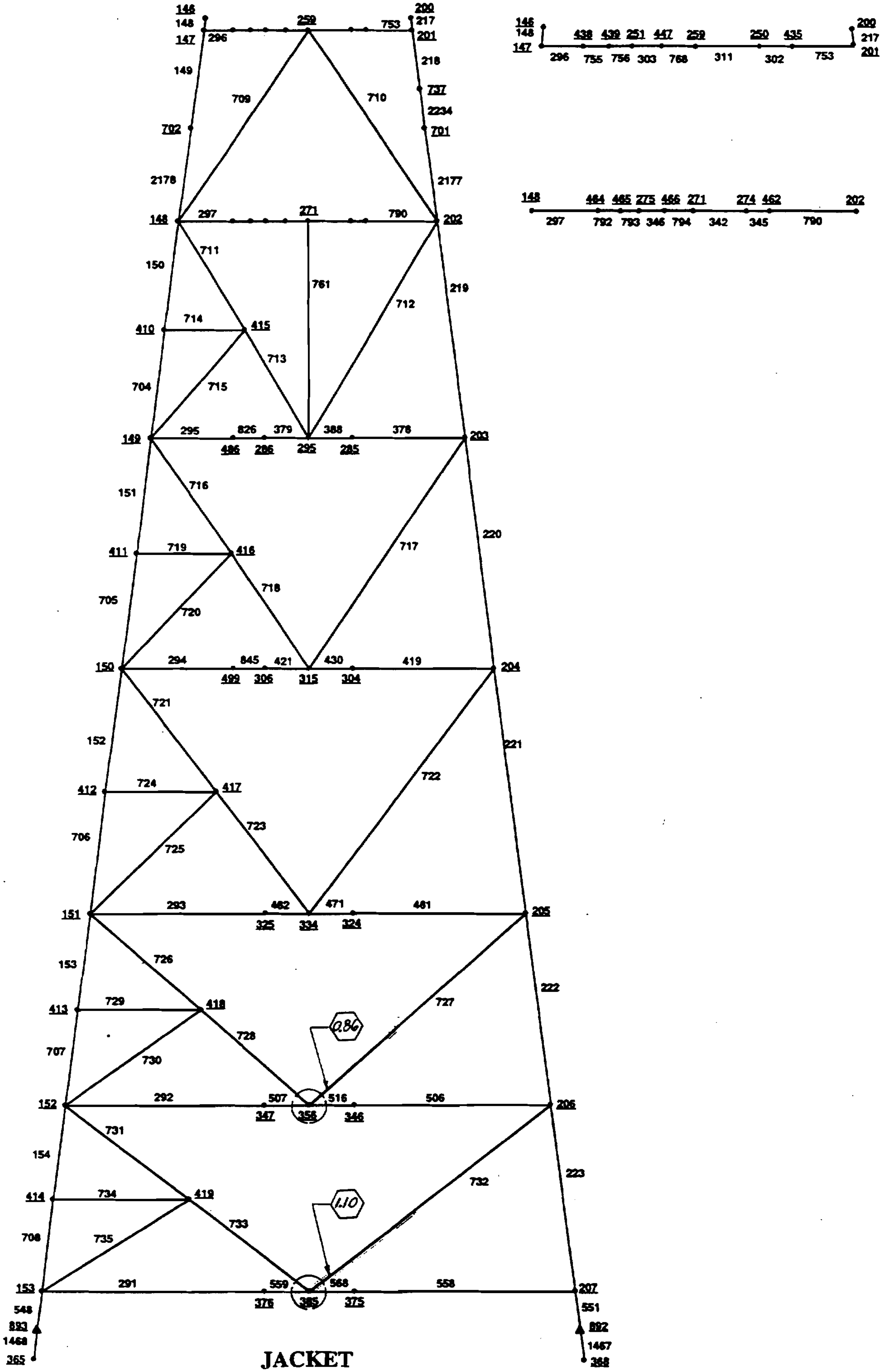
JACKET
ROW 1

4/23/94



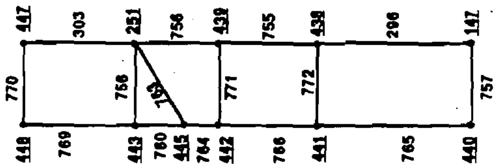
JACKET
ROW 2

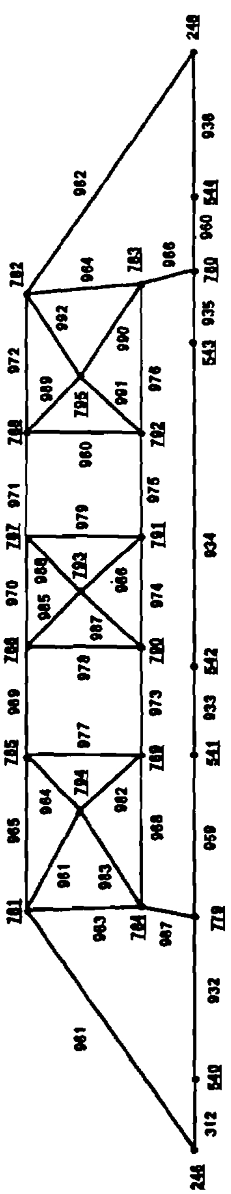
4/23/94

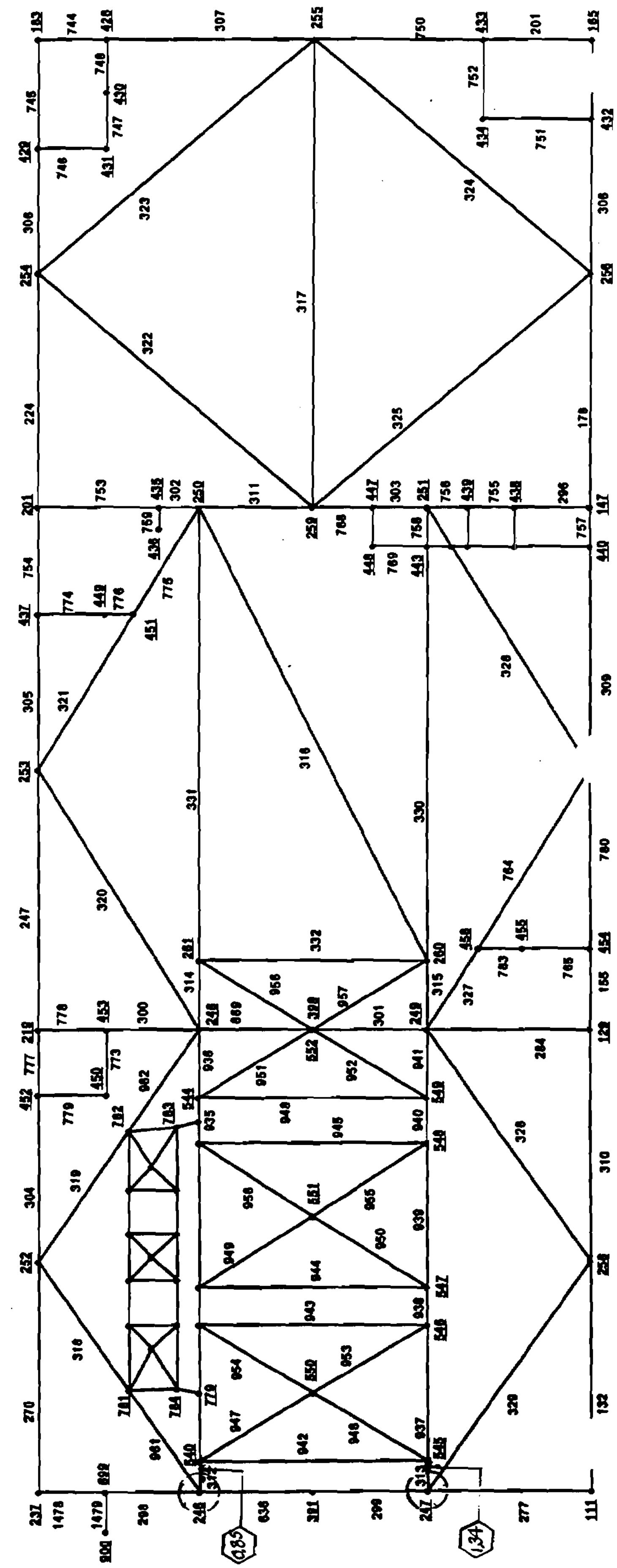


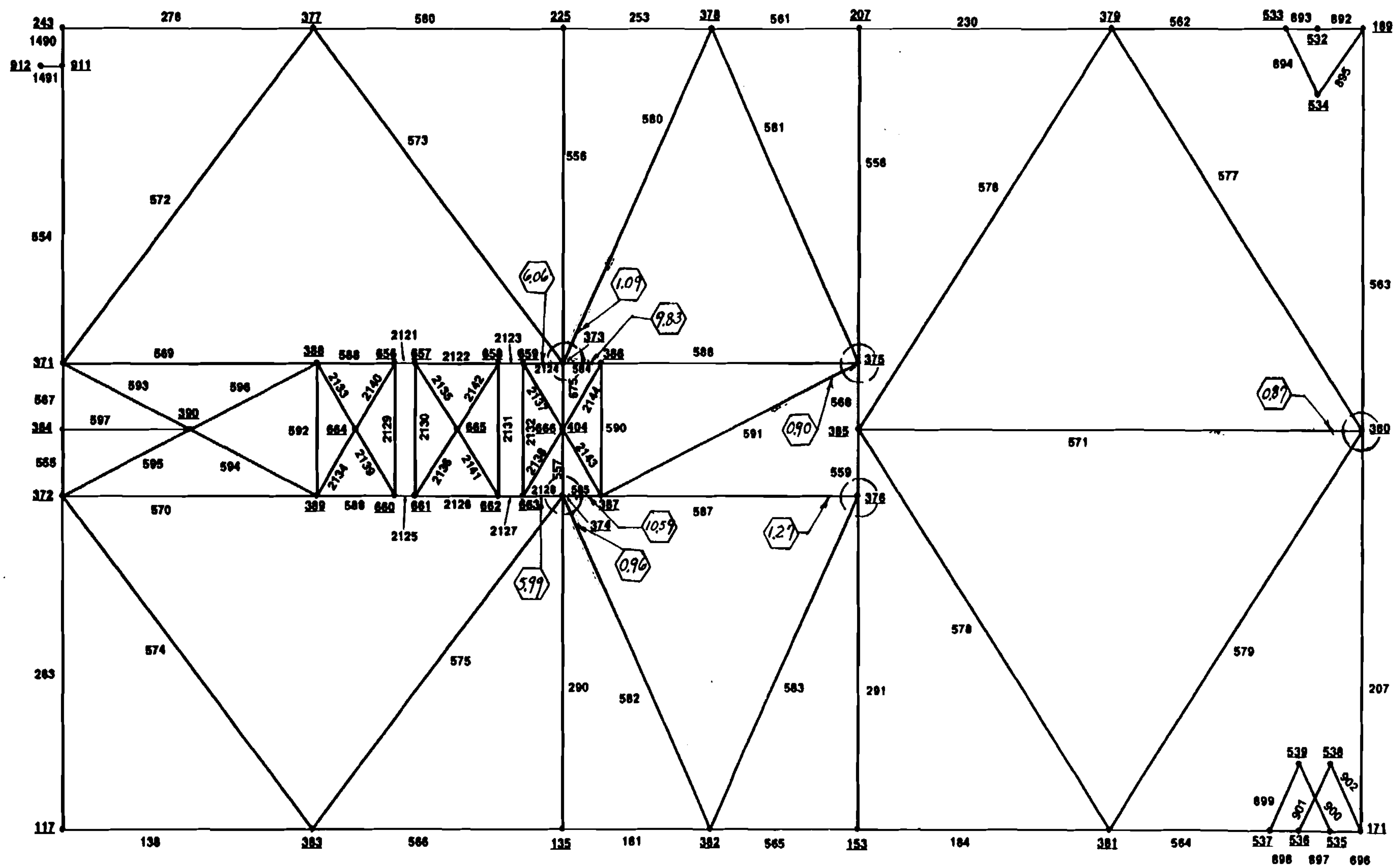
JACKET
ROW 3

4/23/94









EL (-) 254' - 0''

A.5.4.7 Pushover Load Level

From the results of the SPIA (Soil-Pile-Structure Interaction Analysis) analysis, Tubular Member Check and Joint Check for the linear Ultimate Strength Analysis, and from the lateral external load due to wave and current combinations as shown above, it was concluded that wave directions of 195 and 240 degs. would be the dominant directions to the inelastic push-over response of the platform. In the static pushover analysis, four wave directions are considered (195, 240, 180 and 270 degs.). The 180 deg. is in the longitudinal axis of the platform, while 270 deg. is in its orthogonal direction. The pushover load in each wave direction is incrementally increased by multiplying a load factor times the lateral load corresponding to the ultimate strength analysis metocean criteria in that particular wave direction. The pushover load level reached for the wave directions considered is summarized as follows:

Wave Direction (deg.)	Base Ultimate Lateral Load (Wind, Wave & Current) (kips)	Pushover Lateral Load (kips)
195	3,161	4,425
240	2,996	5,991 (+)
180	2,892	4,338
270	2,453	5,886 (+)

A.5.4.8 Load - Deflection Plots

The load-deflection plots are made for four wave directions (195, 240, 180, and 270 degs.). The deck joint #1032 located at the drilling deck leg (B-1) is selected as a representative joint in reference to the deck displacements. In fact, eight deck joints (Joints #1004, #1008, #1011, #1013, #1032, #1036, #1039 and #1041) at the drilling deck were investigated first, and it was concluded that variation of global response is not significant as to which of the eight deck joints should be used. See Figures 8 through 17 in Section A.5.4.8 for load-deflection plots for further detail.

Two types of load-deflection plot were generated. One is the lateral load versus deck displacement. The other one is the ultimate lateral load factor versus deck displacement. The base ultimate lateral load as shown above corresponds to the ultimate lateral load factor = 1.0. See attached load-deflection plots for further detail.

It can be seen that the ultimate lateral load factor of the wave direction of 195 deg. is approximately 1.40. In the wave direction of 180 deg., the ultimate lateral load factor is approximately 1.50. Whereas, in the wave direction of 240 deg., the ultimate lateral factor is approximately 2.0 (+) in this analysis. From the trend of load-deflection plots of wave direction 240, the ultimate lateral load factor can be increased over 2.0, if another load increment was made. For the wave direction of 270 deg., the ultimate lateral load factor is beyond 2.40. The results of this plot (wave direction 270 deg.) showed that the response of the structural system was still in the linear load-displacement mode. Further analysis might be helpful, but not necessary in this case since the lower RSR for the 195 deg. wave direction was already exceeded.

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 195)

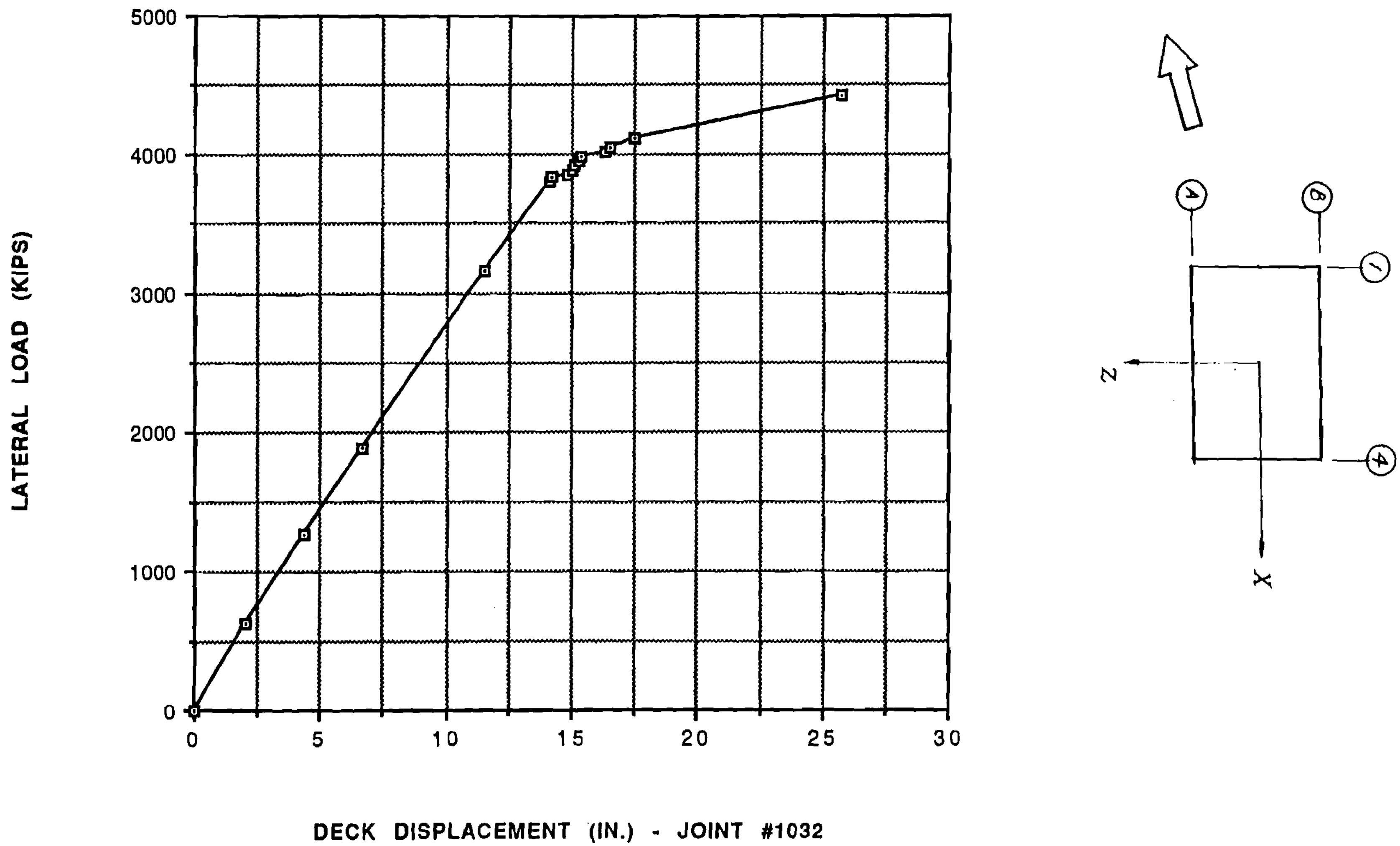


FIG. 8

E.I.BLK 330C PLATFORM - STATIC PUSH-OVER ANALYSIS(WAVE DIR 195)

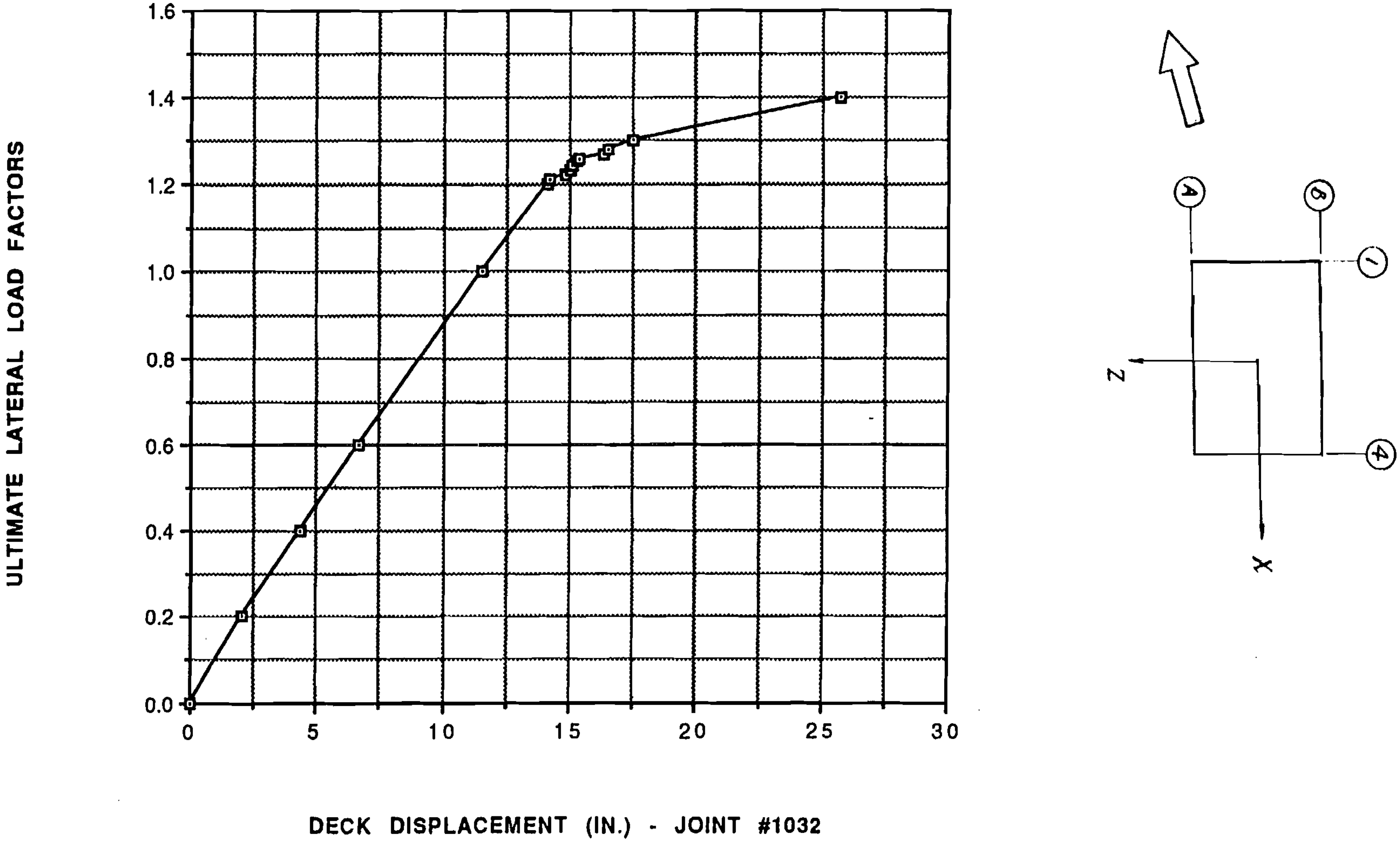
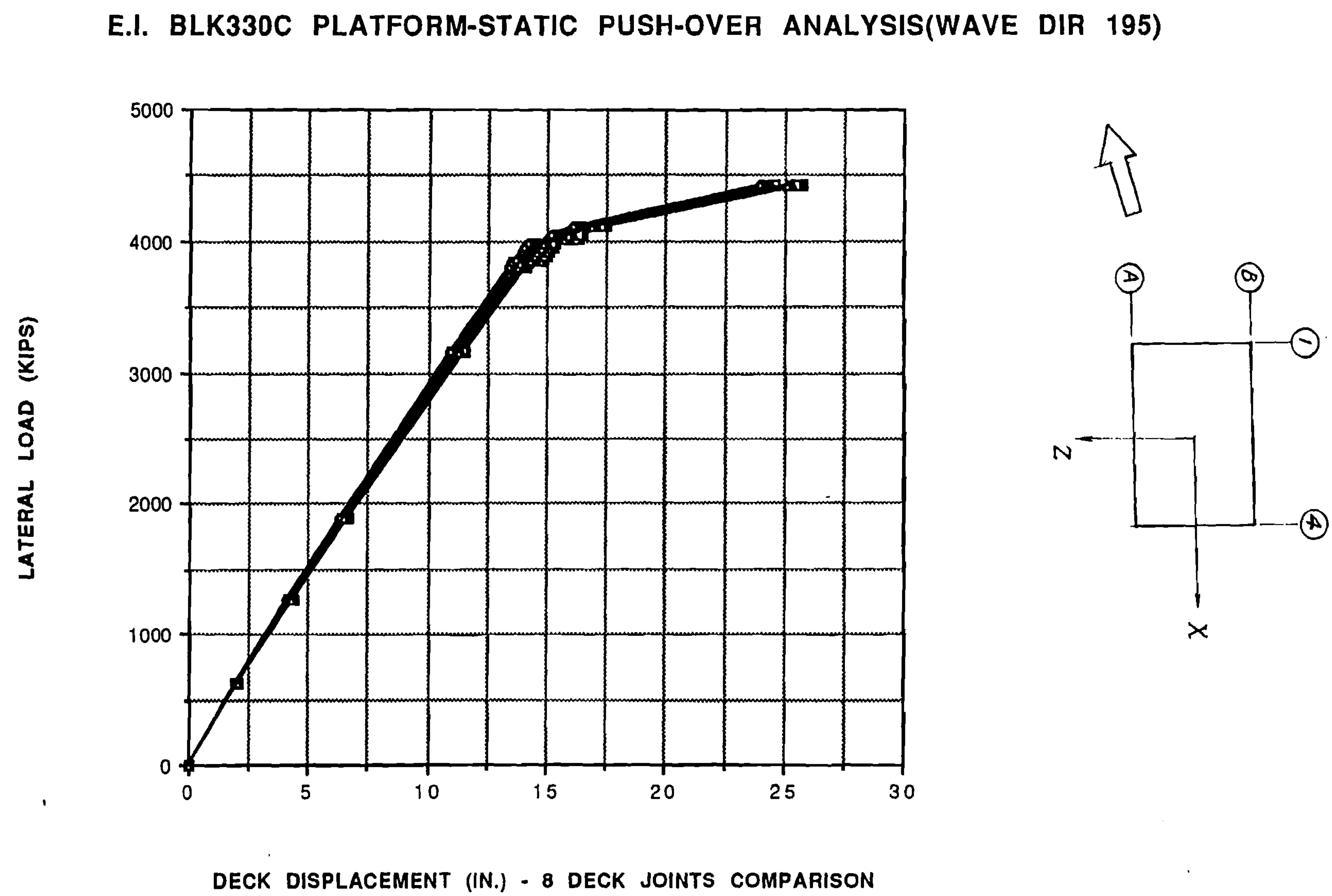


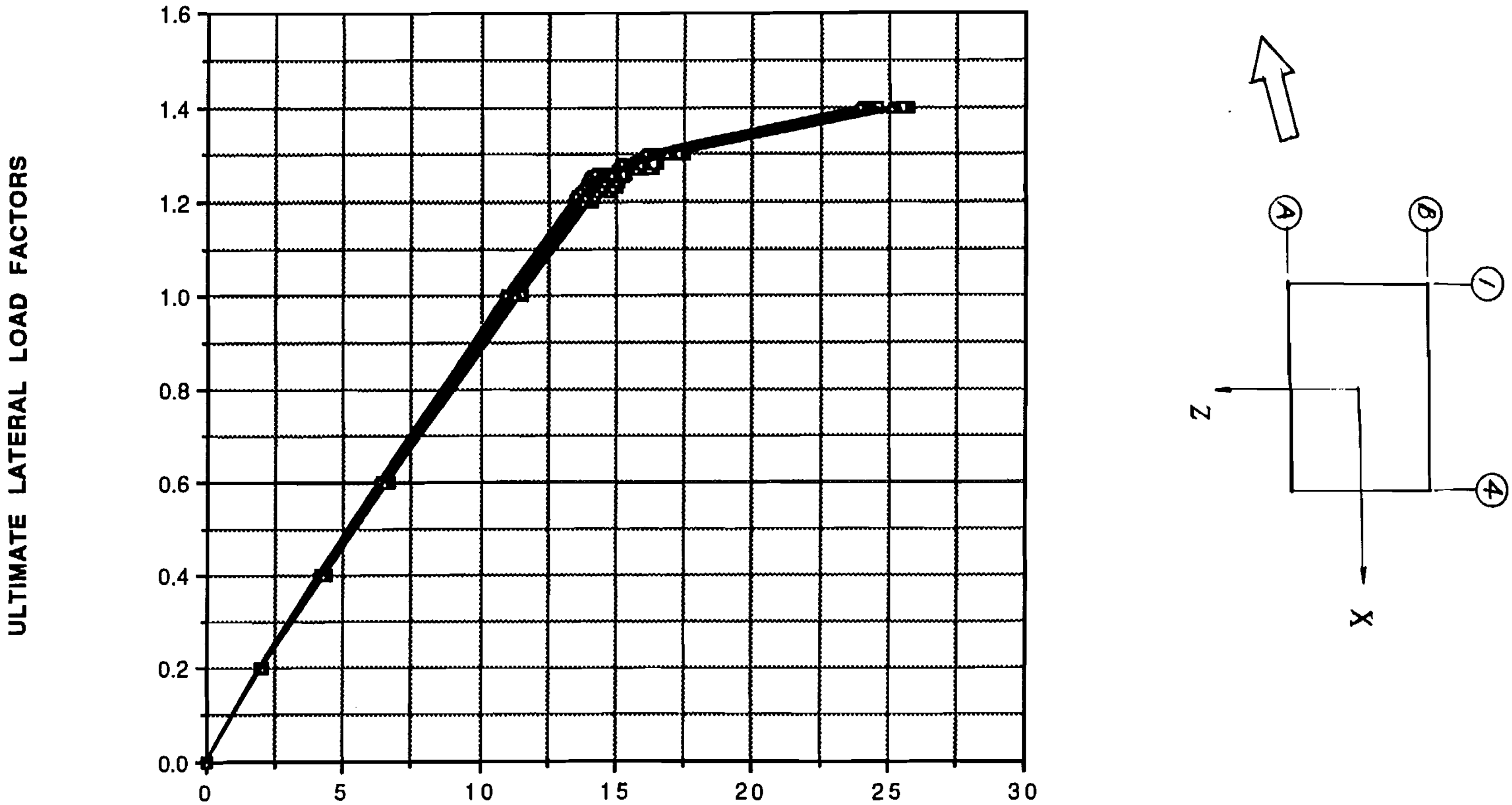
FIG. 9



E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 195)
LATERAL LOAD (KIPS)
5000
4000
3000
2000
1000
0
0
5
10
15
20
25
30
DECK DISPLACEMENT (IN.) - 8 DECK JOINTS COMPARISON
A
B
1
4
Z
X

FIG. 10

E.I.BLK 330C PLATFORM - STATIC PUSH-OVER ANALYSIS(WAV DIR 195)



DECK DISPLACEMENT (IN.) - 8 DECK JOINTS COMPARISON

FIG. 11

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 240)

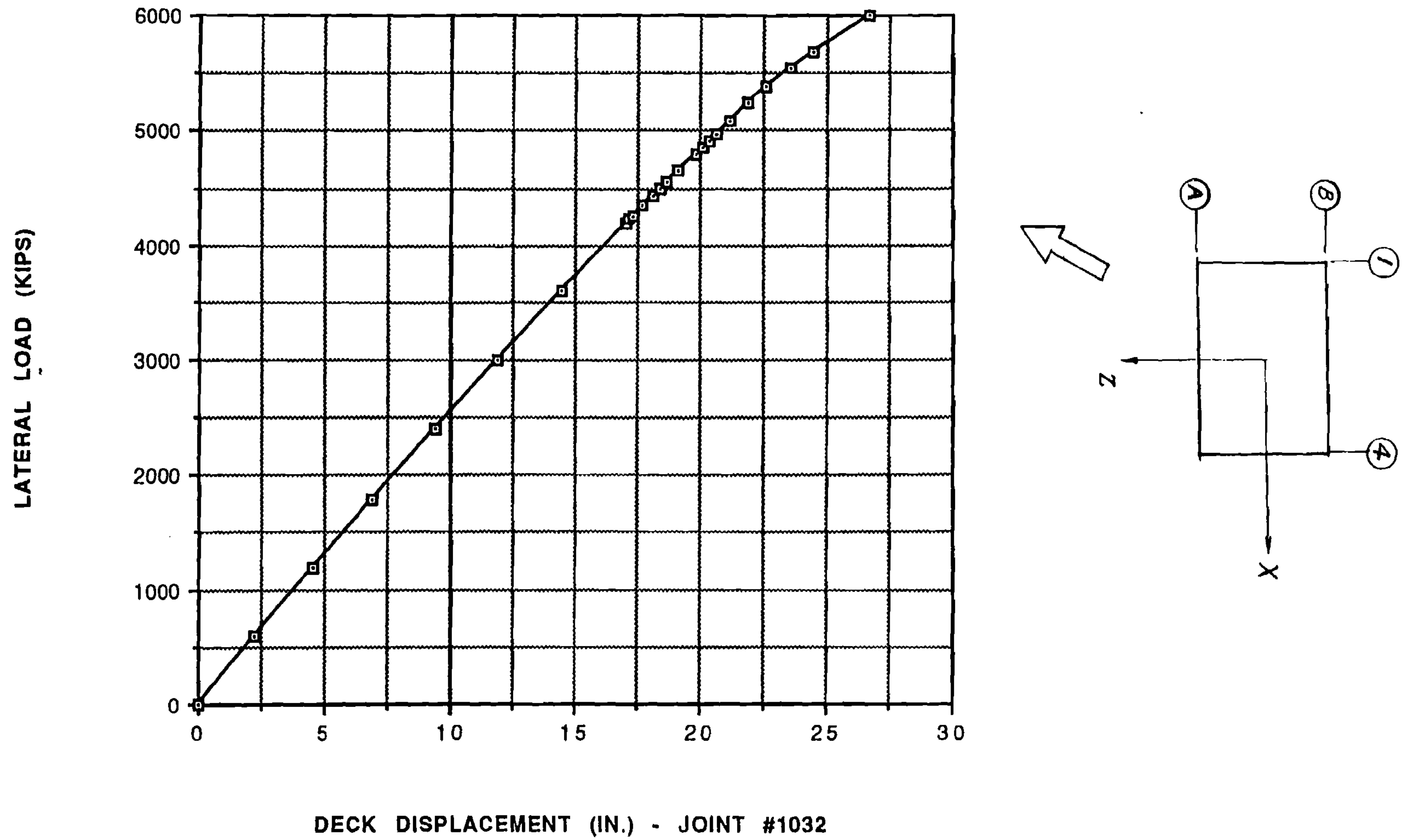


FIG. 12

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 240)

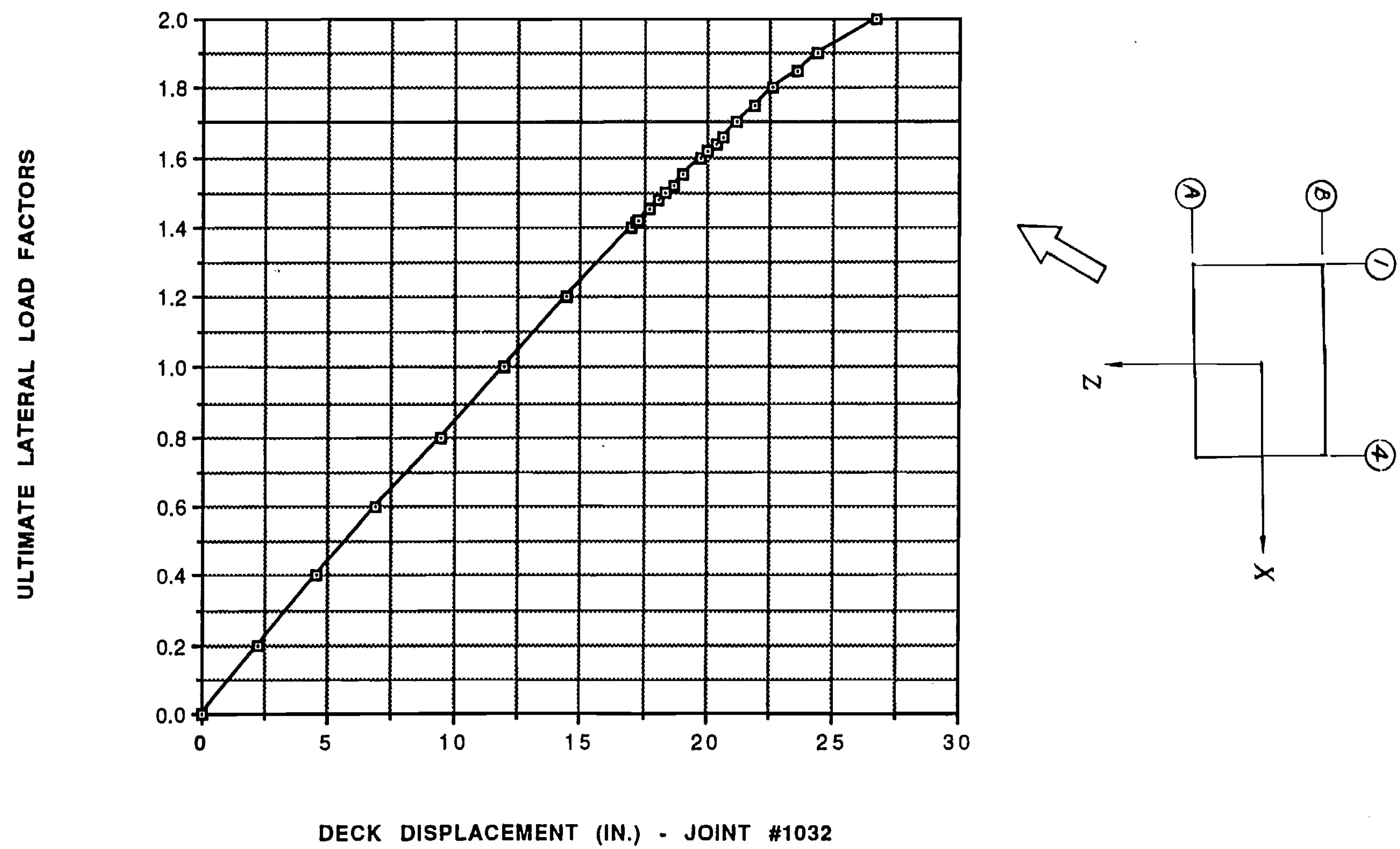


FIG. 13

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 180)

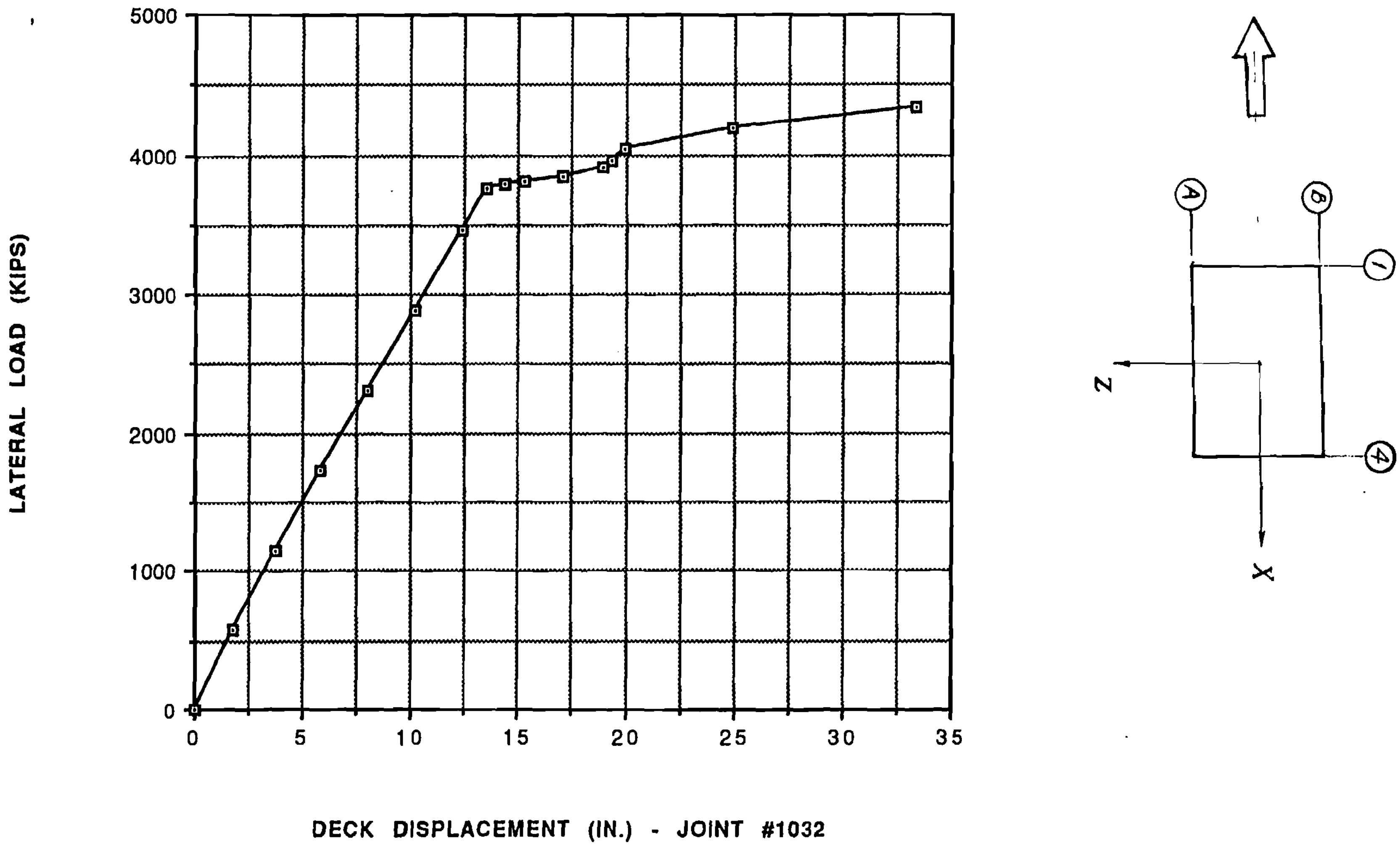


FIG. 14

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 180)

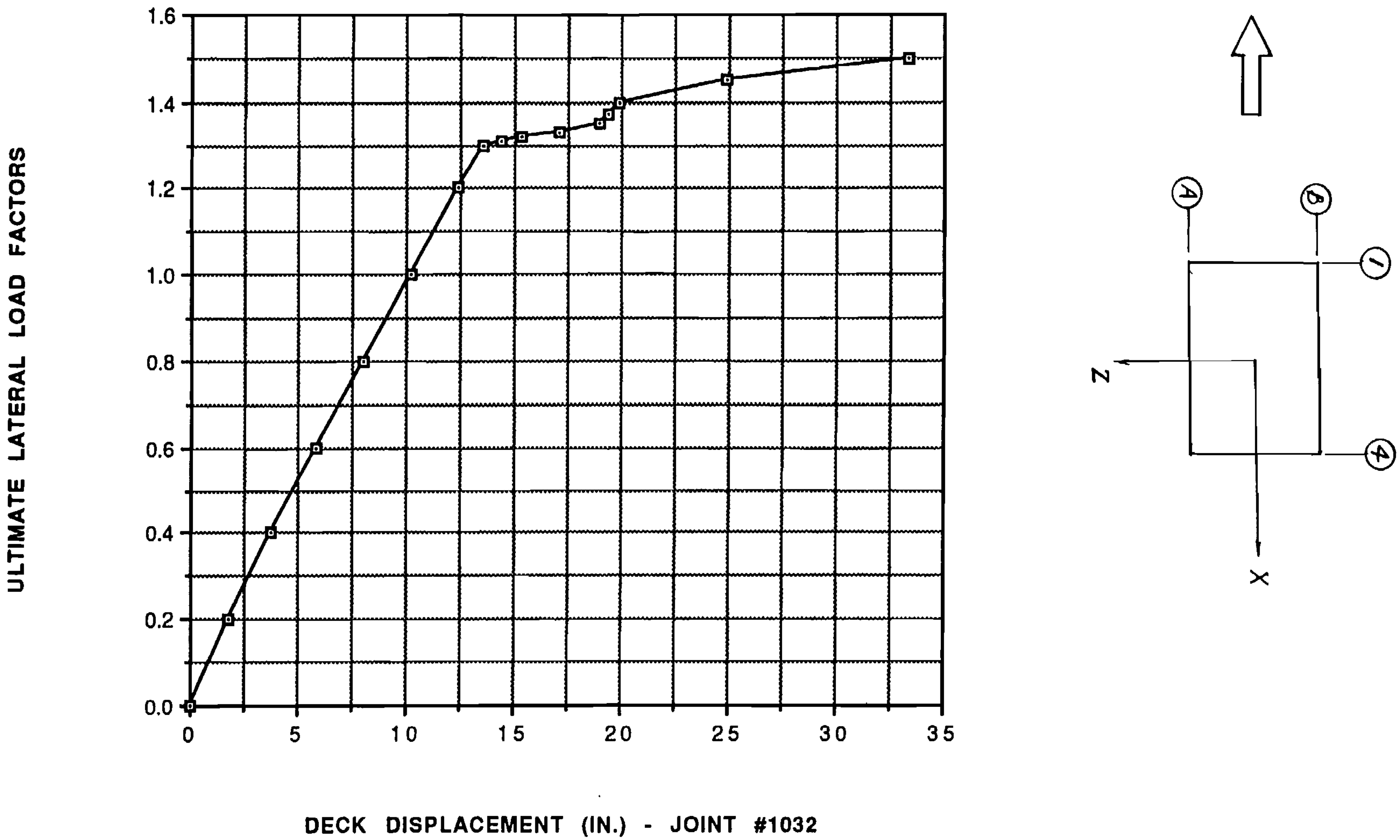


FIG. 15

E.I. BLK330C PLATFORM-STATIC PUASH-OVER ANALYSIS(WAVE DIR 270)

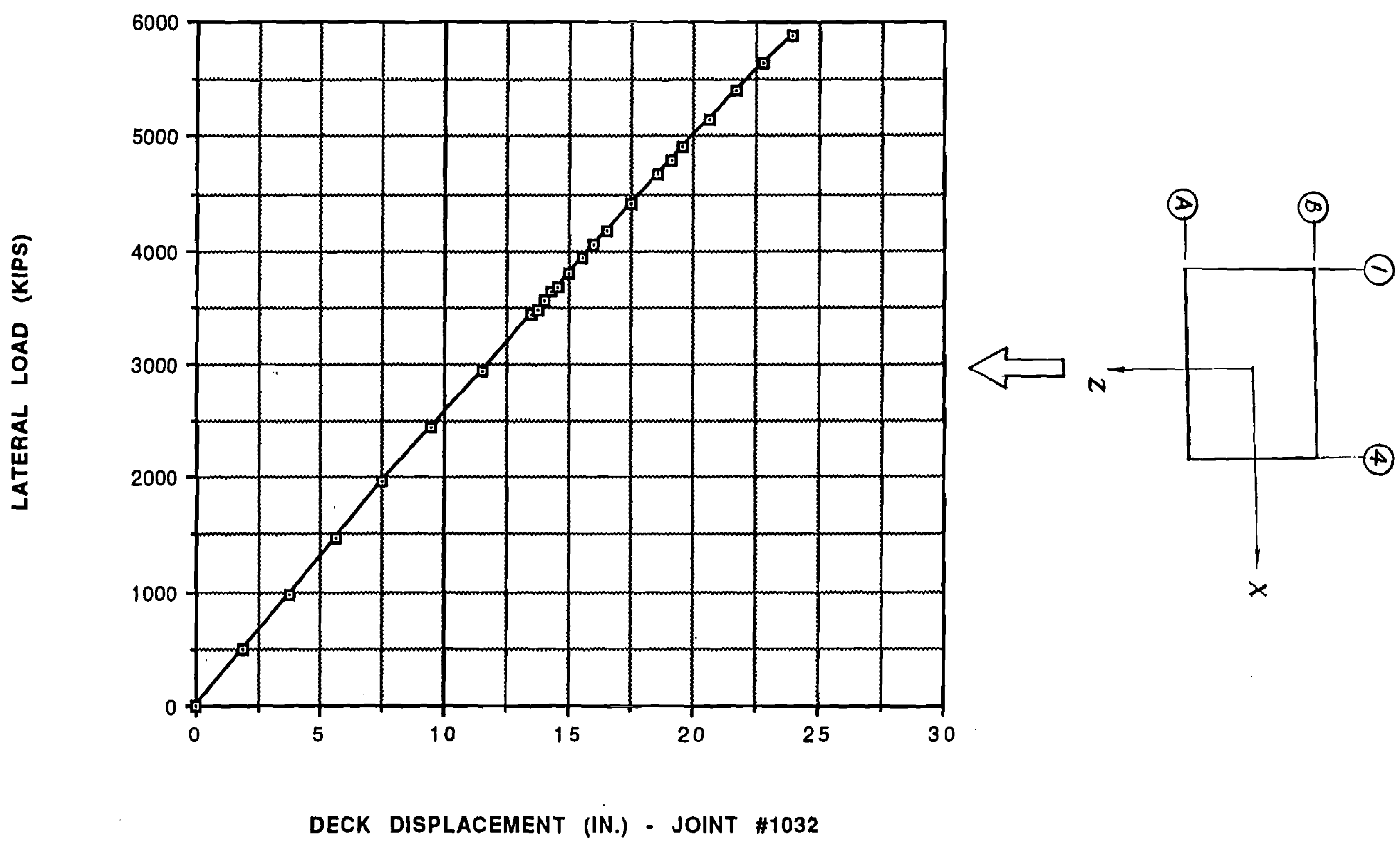


FIG. 16

E.I. BLK330C PLATFORM-STATIC PUSH-OVER ANALYSIS(WAVE DIR 270)

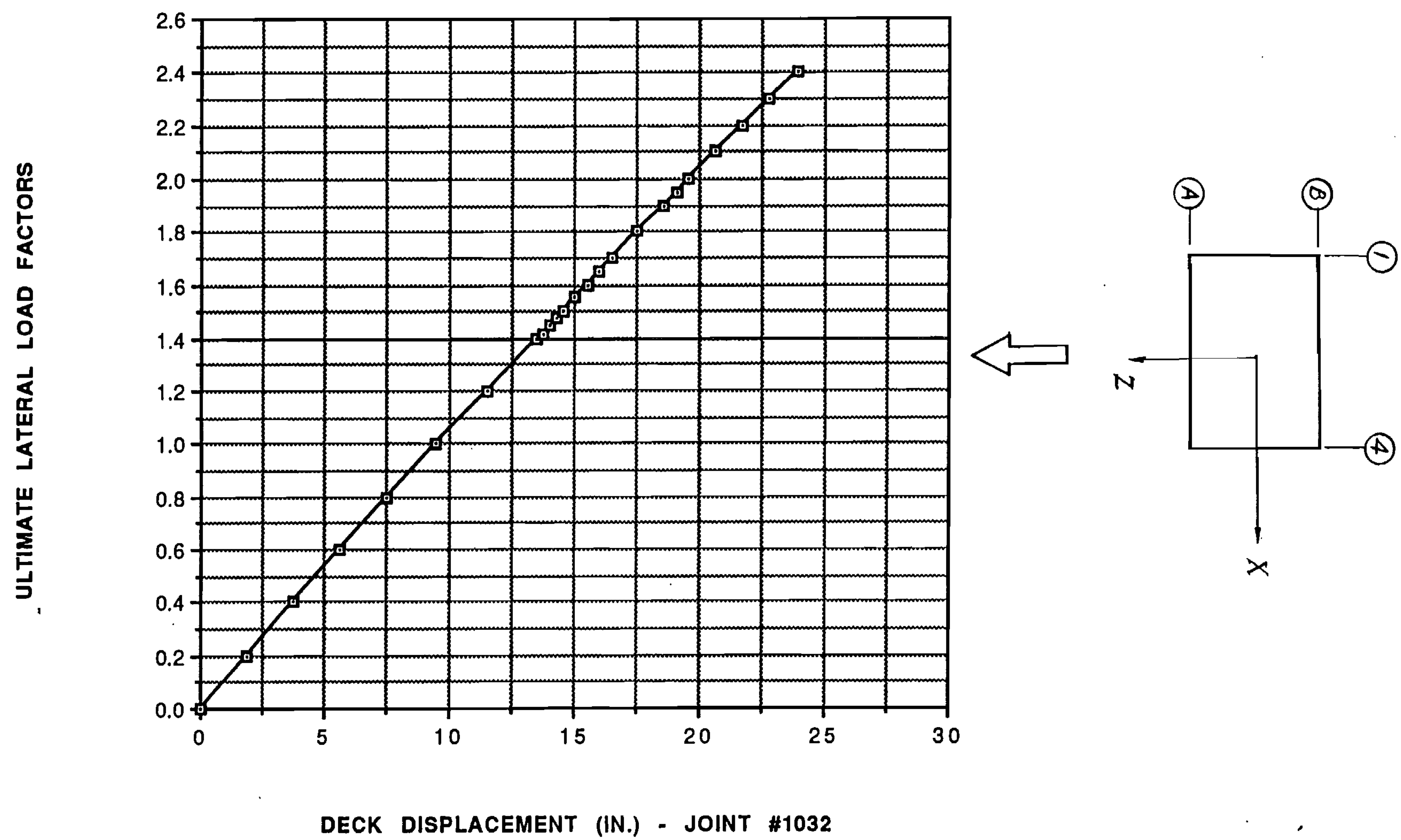


FIG. 17

E.I. BLK	330C PLATFORM	ASSESSMENT			8/6/94
ULTIMATE STRENGTH	ANALYSIS RESULTS	FOR EACH WAVE	DIRECTION		
ANALYSIS CASE	3D MODEL				
Wave Direction	**195 deg.**				
Lateral load level	for first member	with unity check	1.00	(Mem. #272	,horizontal member)
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck Level				
	(in.)	(kips)			
1	2.05	632.15			
2	4.31	1264.29			
3	6.66	1896.44			
4	11.57	3160.73			
5	14.07	3792.88			
6	14.20	3824.48			
7	**14.82**	**3856.09**	Mem #272	Buckled	Horizontal member
8	14.97	3887.70	See Note 1		
9	15.10	3919.31	See Note 1		
10	15.24	3950.91	See Note 1		
11	15.37	3982.52	See Note 1		
12	16.33	4014.13	See Note 1		
13	16.55	4045.73	See Note 1		
14	17.46	4108.95	See Note 1		
15	25.72	4425.02	See Note 1		
NOTE 1 :					
	Member Buckled or Yield				
Step 7	#272	(Buckled)			
Step 8	#272, #273	(Buckled)			
Srep 9	#272, #273	(Buckled)			
Step 10	#272, #273	(Buckled)			
Step 11	#272, #273	(Buckled)			
Step 12	#272, #273, #560, #566, #580		(Buckled)		
Step 13	#272, #273, #560, #566, #580		(Buckled)		
Step 14	#335, #134, #272, #135,#273, #274, #275, #560, #566				
Step 15	#132, #270, #133, #335, #134, #272, #135, #273, #136, #274				
	#275, #137, #560, #566, #571, #604, #606, #607, #619, #621				
	#622, #624, #625, #580, #582		(Buckled)		
	#598 (Yield)				

Wave Direction	**240 deg.**				
Lateral load level	for first member	with unity check	1.00	(Mem. #2179	jacekt leg)
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck Level				
	(in.)	(kips)			
1	2.22	599.12			
2	4.52	1198.25			
3	6.89	1797.37			
4	9.41	2396.50			
5	11.90	2995.62			
6	14.44	3594.74			
7	17.03	4193.87			
8	17.16	4223.82			
9	17.29	4253.78			
10	17.68	4343.65			
11	18.09	4433.52			
12	18.37	4493.43			
13	18.65	4553.34			
14	19.07	4643.21			
15	19.78	4792.99			
16	20.06	4852.90			
17	20.35	4912.82			
18	20.63	4972.73			
19	21.20	5092.55			
20	21.87	5242.34			
21	**22.61**	**5392.12**	Mem #2179	Yield	Jacket leg
22	23.60	5541.90	See Note 2		
23	24.43	5691.68	See Note 2		
24	26.68	5991.24	See Note 2		
NOTE 2 :					
	Member Buckled or Yield				
Step 21	#2179	(Yield)			
Step 22	#2179	(Yield)			
Step 23	#2179	(Yield)			
	#580	(Buckled)			
Step #24	#2174, #2179	(Yield)			
	#289, #574, #580	(Buckled)			

Wave Direction	**180 deg.**				
Lateral load level	for first member	with unity check	1.00	(Mem. #272	,horizontal member)
Load Step	Lateral Displacement at Deck Level (in.)	Lateral Load (kips)	Element Failures	Component Failure Mode	Remark
1	1.79	578.41			
2	3.79	1156.82			
3	5.82	1735.22			
4	7.98	2313.63			
5	10.20	2892.04			
6	12.39	3470.45			
7	13.50	3759.65			
8	**14.37**	**3788.57**	Mem #272	Buckled	Horizontal member
9	15.28	3817.49	See Note 3		
10	17.07	3846.41	See Note 3		
11	18.92	3904.25	See Note 3		
12	19.32	3962.09	See Note 3		
13	19.92	4048.86	See Note 3		
14	24.91	4193.46	See Note 3		
15	33.36	4338.06	See Note 3		
NOTE 3 :					
	Member Buckled or Yield				
Step 8	#272, #273, #560, #580		(Buckled)		
Step 9	#134, #272, #135, #273, #275, #560, #566, #580, #582				(Buckled)
Step 10	#270, #133, #335, #134, #272, #135, #273, #274, #275				
	#137, #560, #566, #580, #582		(Buckled)		
Step 11	#270, #133, #335, #134, #272, #135, #273, #136				
	#274, #275, #137, #560, #566, #580, 582			(Buckled)	
Step 12	#270, #133, #335, #134, #272, #135, #273, #136				
	#274, #275, #137, #560, #566, #580, #582			(Buckled)	
Step 13	#270, #133, #335, #134, #272, #135, #273, #136				
	#274, #275, #137, #560, #566, #580, #582			(Buckled)	
Step 14	#270, #133, #335, #134, #272, #135, #273, #136, #274				
	#275, #137, #560, #566, #604, #606, #607, #621, 622				
	#624, #625, #580, #582		(Buckled)		
	#598 (Yield)				
Step 15	#270, #133, #335, #134, #272, #135, #273, #136, #274				
	#275, #137, #560, #566, #571, #603, #600, #604, #606				
	#607, #619, #621, #622, #624, 625, #580, #582			(Buckled)	
	#598 (Yield)				

Wave Direction	**270 deg.**				
Lateral load level	for first member	with unity check	1.00	(N/A)	
Load Step	Lateral	Lateral	Element	Component	Remark
	Displacement	Load	Failures	Failure Mode	
	at Deck Level				
	(in.)	(kips)			
1	1.87	490.52			
2	3.72	981.04			
3	5.58	1471.55			
4	7.49	1962.07			
5	9.48	2452.59			
6	11.49	2943.11			
7	13.52	3433.63			
8	13.72	3482.68			
9	14.02	3556.26			
10	14.33	3629.83			
11	14.53	3678.89			
12	15.03	3801.51			
13	15.52	3924.14			
14	16.02	4046.77			
15	16.50	4169.40			
16	17.54	4414.66			
17	18.56	4659.92			
18	19.07	4782.55			
19	19.58	4905.18			
20	20.65	5150.44			
21	21.74	5395.70			
22	22.79	5640.96			
23	23.94	5886.22	See Note 4		
Note 4					
Step 23	The system reponse was still in linear mode, no member was yieled or buckled.				

A.5.4.9 Load Level at Which First Component Reaches IR = 1.0

The load level at which the first component reaches an IR = 1.0 is defined as the first member that buckles or yields in the static pushover analysis in the following discussion:

i) Wave Direction 195 deg.

As the lateral load was increased to a load factor of **1.22** (load step #7 in the static pushover analysis), at that particular snap shot, the member #272 (16" diameter x 0.375" wall thickness), a horizontal member in Row B at Elevation (-) 80'- 0", buckled first which corresponded to the lateral load level of **3,856 kips.**

ii) Wave Direction 240 deg.

As the lateral load was increased to a load factor of **1.80** (load step #21 in the static pushover analysis), at that particular snap shot, the member #2179 (45" diameter x 0.625" wall thickness), a segmented jacket-leg (B-4) just below the waterline, reached its maximum strength (Yield) which corresponded to the lateral load level of **5,392 kips**. Member #2179 consists of two segments with different wall thicknesses (0.625" and 1.250"). The segment with the wall thickness of 0.625" yielded first.

iii) Wave Direction 180 deg.

As the lateral load was increased to a load factor of **1.31** (load step #8 in the static pushover analysis), at that particular snap shot, the member #272 (16" diameter x 0.375" wall thickness), a horizontal member in Row B at Elevation (-) 80'- 0", buckled first which corresponded to the lateral load level of **3,789 kips.**

iv) Wave Direction 270 deg.

The lateral load was increased to a load factor of **2.40** (load step #15 in the static pushover analysis) which corresponded to the lateral load level of **5,886 kips**. It should be pointed out that at this particular snap shot, the structural system response was still in the linear mode, i. e., no member as yet had buckled. The ultimate lateral load factor could be increased beyond the last factor of 2.40 tried, but due to time constraints, we did not continue to perform the static push-over analysis.

A.5.4.10 Reference Level Load (API RP 2A 20th Edition, 100 Year Return Period)

The reference level load is defined in API RP 2A 20th edition, and is the design wave with a 100-year return period. The reference level load due to wave and current effects is summarized as follows:

Design wave height: 67 ft. (Maximum)
Design wave period: 13.0 sec (Apparent wave periods: varied)

Wave Direction (deg.)	Wave Height (ft.)	Lateral Load (Wave & Current) (kips)
0	48.04	1,789
45	46.90	1,566
90	46.90	1,732
135	53.60	2,300
180	61.44	3,314
225	65.86	4,426
270	62.51	4,223
315	53.60	2,704
240	67.00	**4,746**
195	63.65	3,736

The maximum lateral load is 4,746 kips in the wave direction of 240 deg. See Figure 3 in Section A.5.1 for the comparison of lateral loads (wave and current) for different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition).

A.5.4.11 Reserve Strength Ratio

The reserve strength ratio (RSR) is defined as the ratio of a platform's ultimate lateral load-carrying capacity to its 100-year environmental condition lateral loading for a particular wave direction as defined in the present API RP 2A procedures.

In this study, a static push-over analysis is used to calculate the ultimate lateral load-carrying capacity of the platform for each wave direction considered. Results or output depend on the structural model (different types of finite elements, such as Strut, Beam-Column, Beam, etc.) used in the nonlinear analysis (static push-over analysis), numerical solutions scheme implemented in the computer program, and the specified tolerances of unbalanced forces and displacements in the nonlinear analysis. Also, the ultimate strength of the platform might be varied, especially in the local response of the platform. But overall, the trend of global response of the platform should be consistent, if the same guidelines, such as Section 17.0 (draft) of API RP 2A are used.

The following is the summary of the reserve strength ratio of the platform being studied:

Wave Direction (deg.)	Ultimate Lateral Load (kips)	20th Ed. Reference Lateral Load (kips)	Reserve Strength Ratio (RSR)
195	4,425	3,736	**1.18**
240	5,991 (+)	4,746	1.26
180	4,338	3,314	1.31
270	5,886 (+)	4,223	1.39

From the results shown above, the lowest "reserve strength ratio" of this platform is 1.18, and its associated wave direction is 195 deg. This is simply a comparison of the ultimate lateral load with the reference lateral load of API RP 2A 20th edition instead of with the section 17.0 metocean criteria..

It should be pointed out that the "reserve strength ratio" is directionally dependent. It raises the question about how many wave directions should be considered in the platform assessment to ensure that the platform's reserve strength is properly evaluated.

Ultimate Strength Analysis Results Summary

i) Wave Direction: 195 deg.
Load Level at which First Component reaches I.R. of 1.0 (S_I): 3,856 kips
Reference Level Load (Sref): 3,736 kips
Ultimate Capacity (Ru): 4,425 kips
Reserve Strength Ratio (RSR): 1.18
Platform Failure Mode: Jacket

ii) Wave Direction: 240 deg.
Load Level at which First Component reaches I.R. of 1.0 (S_I): 5,392 kips
Reference Level Load (Sref): 4,746 kips
Ultimate Capacity (Ru): 5,991 kips (+)
Reserve Strength Ratio (RSR): 1.26 (+)

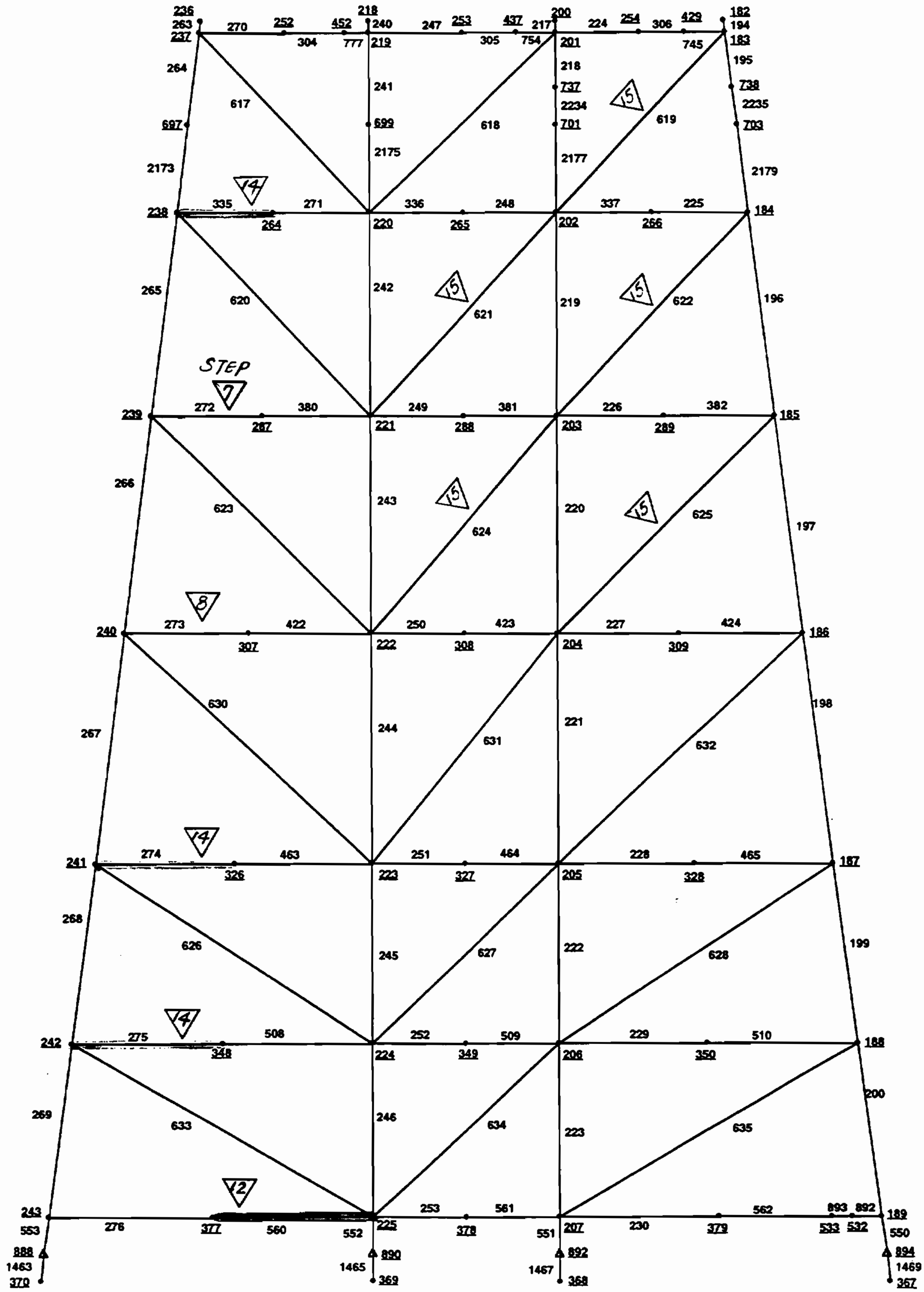
	Platform Failure Mode:	Jacket
iii)	Wave Direction:	180 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	3,789 kips
	Reference Level Load (Sref):	3,314 kips
	Ultimate Capacity (Ru):	4,338 kips
	Reserve Strength Ratio (RSR):	1.31
	Platform Failure Mode:	Jacket
iv)	Wave Direction:	270 deg.
	Load Level at which First Component reaches I.R. of 1.0 (S_I):	5,886 kips (+) *
	Reference Level Load (Sref):	4,223 kips
	Ultimate Capacity (Ru):	5,886 kips (+) *
	Reserve Strength Ratio (RSR):	1.39 (+)
	Platform Failure Mode:	*

* Platform response was still in linear mode. Further nonlinear analysis is required.

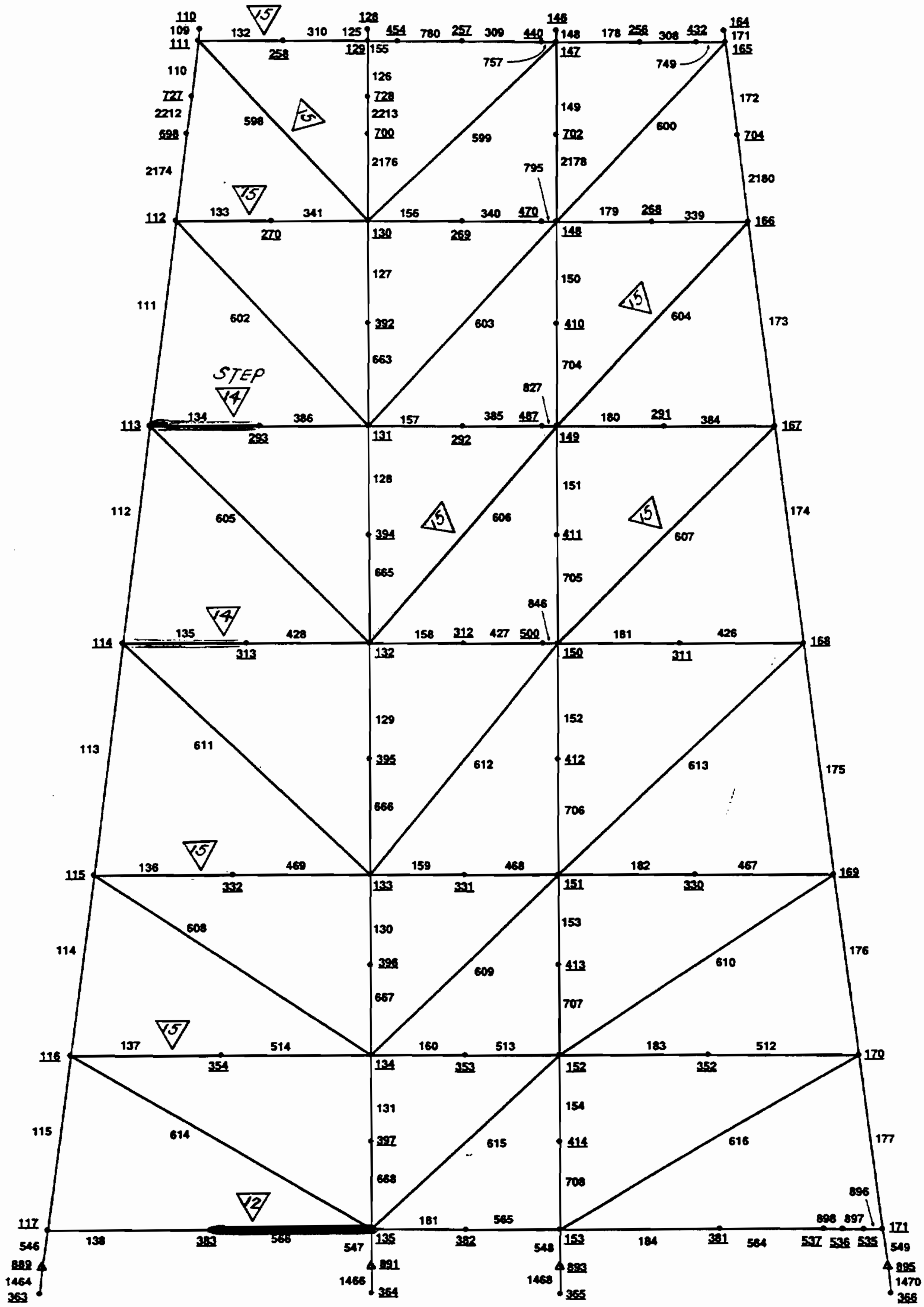
STATIC PUSH-OVER ANALYSIS RESULTS

PLOTS OF MEMBERS BUCKLED OR YIELDED

WAVE DIRECTION - 195 DEGREES

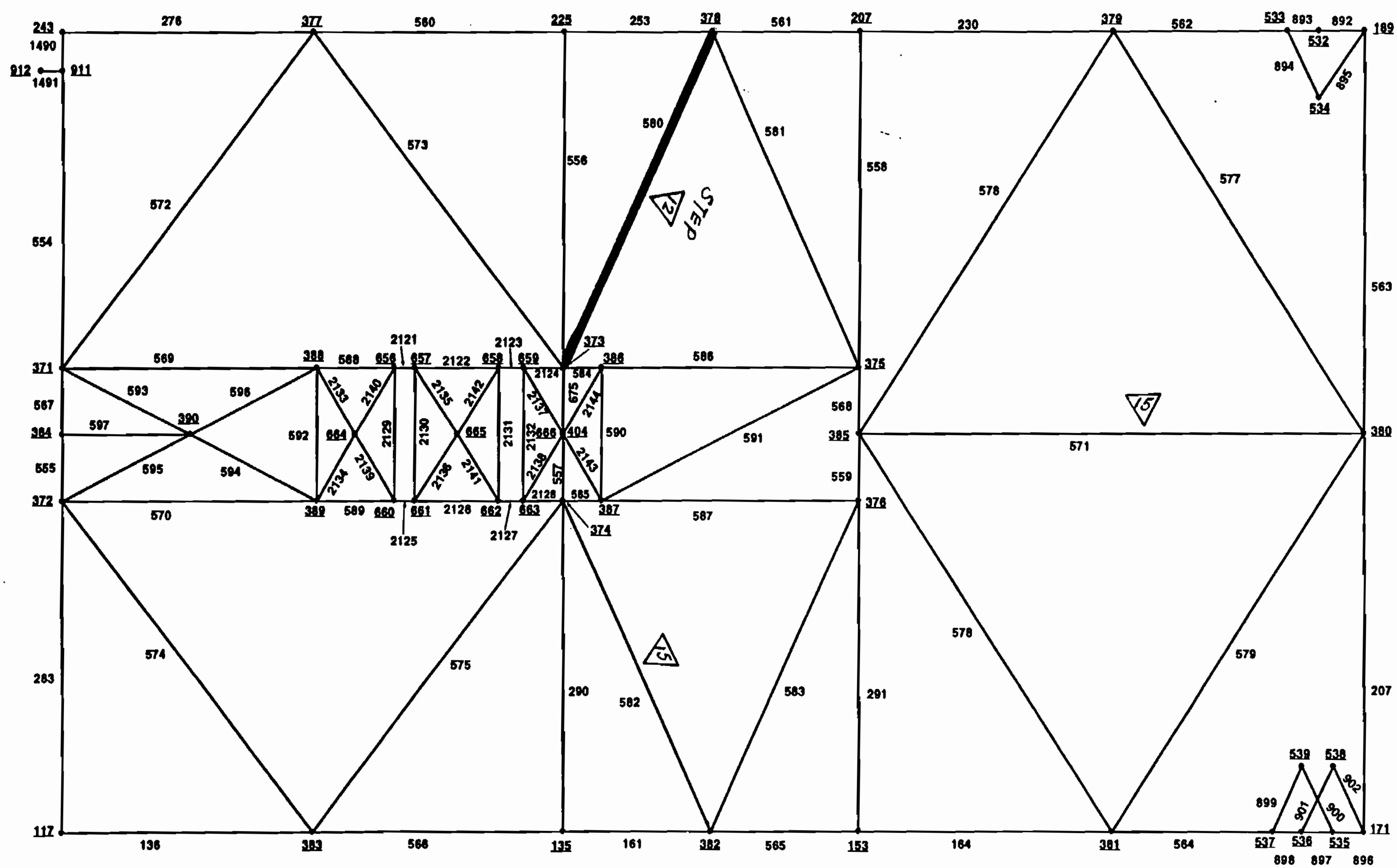


JACKET
ROW B



JACKET
ROW A

4/23/94



EL (-) 254' - 0"

4/23/74

A.6 CONSIDERATION OF MITIGATION

According to API RP 2A Section 17.0 (draft), the mitigation actions are defined as modifications or operational procedures that reduce loads, increase capacities, or reduce consequences.

In the design level analysis, the results of the tubular member check showed that a majority of members with stress ratio exceeding 0.85 were governed by the hydrostatic stress check. Only five members were governed by the strength check. There were some secondary members with stress ratios greater than 0.85 which were included in the report (see Section A.5.3.5) just for the sake of completeness. The effects of secondary members overstress or collapse should not be a major concern. It should be pointed out that at the horizontal framing of EL. (-) 254'-0, which is at the mudline, most of the members with a stress ratio exceeding 1.00 were due to the modeling difficulty. There are eight mudmats at the mudline framing, one at each jacket-leg location. If the mudmat effect were properly taken into account, they would provide lateral support to those horizontal members. Thus, the effective length factor (K) could be reduced. It is recommended that a further investigation of these horizontal members (at the mudline) be carried out. At this point, however, no consideration of mitigation is immediately required. The same argument is also applied to the (linear) ultimate strength analysis. It should be noted that the material yield strength (Fy) of 36 ksi was used in the design level analysis and the linear ultimate strength analysis.

On the joint check results (both design level analysis and "linear" ultimate strength analysis), the joints with stress ratio exceeding 0.85 or even greater than 1.00 are mainly K-joints. Due to the fact that a larger material mean yield strength (Fy = 42 ksi) was used in the (linear) ultimate strength analysis, the results of joint check for the linear ultimate strength analysis were much less severe than that of the design level analysis (see Sections A.5.3.6 and A.5.4.6 for further detail).

Again, the modeling difficulty mentioned earlier was the main cause of the resulting high stress ratios. A refined model should be developed for a further investigation, especially on those joints at the mudline horizontal framing level. Furthermore, a finite element analysis of some selected joints would be feasible especially before any physical mitigation is initiated.

In the static push-over analysis, it was assumed that the joint capacity could reach or exceed its full member-end strength requirements either by refining the model through further analysis or by any other measure that is feasible and practical.

A.7 SUMMARY NOTE - PART A

Part A - Platform Assessment involved the tasks of platform selection, condition assessment, categorization, design basis checks, analysis checks, and the consideration of mitigation. Analysis checks are the major tasks in this study which include the topics of metocean criteria/loads, screening, design level analysis, and ultimate strength analyses. Particularly in the ultimate strength analyses, an advanced nonlinear analytical tool and the engineering expertise are a necessity.

This section summarizes the highlights of the platform assessment.

i) Metocean Criteria/Loads

Based on different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition), the wave heights versus wave directions and the platform base shear versus wave directions were calculated and presented in Figures 2 and 3 in Section A.5.1. This information turned out to be the most valuable guideline in the subsequent analyses and code checks. In addition, the base shear ratios of ultimate to design level and ultimate to API RP 2A 20th edition were plotted for comparison (see Figure 4 in Section A.5.1).

ii) Design Level Analysis

A three-dimensional model was generated from the as-built drawings. The structural model consists of 816 joints (nodes) and 1,636 members (beam elements). Six degrees of freedom were considered at each joint (node). There were 21 conductors (18 original plus three recently added wells) in the platform. The pile foundation consists of eight main piles. The soil resistance was properly modeled using P-Y, T-Z and Q-Z curves.

The design gravity load included structural weights, deck live and dead loads, and buoyancy.

The lateral loads included wave, current and wind loads. The lateral loads were calculated for each wave directions. Ten wave directions (0, 45, 90, 135, 180, 225, 270, 315, 195 and 240 degrees) were considered in the design level analysis. The maximum lateral load due to wave and current is 2,090 kips which occurred in the wave direction of 270 degrees. See Figure 3 in Section A.5.1 for further detail.

The tubular member check was performed using the MicroSAS computer program, a McDermott in-house developed computer software specifically for offshore structural design and analysis. A material yield strength (Fy) of 36 ksi and an allowable increase factor of 1.33 were used in the design level analysis. The results of the tubular member check showed that most of members with stress ratio exceeding 0.85 were governed by the hydrostatic stress check. Only five members (#272, #273, #422, #560 and #566) with stress ratio exceeding 0.85 were attributable to the strength check. The tubular member check results were satisfactory, i.e., stress ratios did not exceed unity (other than the mudline framing level mentioned earlier). See Section A.5.3.5 for further explanations.

The tubular joint check was performed using the MicroSAS computer program. A material yield strength of 36 ksi (nominal) was specified. An allowable stress increase factor of 1.33 was used. The nominal loads approach as provided in API RP 2A was used in the Joint Check. It should be noted that most of the joints/braces with interaction stress ratio exceeding 0.85 are mainly K- joints. The factor that attributed to these high stress ratios is that the brace and chord wall thickness is identical. Consequently, the ratio of brace wall thickness to the chord wall thickness, tau, is equal to one, which will induce a high stress in the chord (in the sense of chord punching stress). Most of the K-joints in Rows #1 through #4 are overlapping joints which are specifically considered in the MicroSAS joint check program in order to take advantage of the strength of overlapping joints. The joint check results showed several joints with stress ratio exceeding 1.0. After reviewing the results, it was concluded that a refined model would reduce these stress ratios, especially those joints which are part of the conductor framing at the mudline. See Section A.5.3.6 for further details.

Pile strength check was performed using the results of SPIA analysis. A material yield strength of 36 ksi for the piles and an allowable stress increase factor of 1.33 were used in the pile strength check. The pile strength check results showed that the highest stress ratio was 0.499 which occurred in the pile (B-1) due to the combined loading in the wave direction of 135 deg. Apparently, the piles have appreciable reserve strength since the pile response was still in the linear stress range.

iii) Ultimate Strength Analysis

The metocean criteria used for the ultimate strength analysis is quite different from that used in the design level analysis. The design criteria are based on 100-year waves due to the combined sudden hurricane and winter storm population. The ultimate strength method's metocean criteria of wave height, associated current and profile as a function of wave direction, are calculated based on API PR 2A 20th edition guidelines, except that the directional factors are based on Figure 17.6.2-4 of Section 17.0 (draft) of API RP 2A. See Figure 3 in Section A 5.1- Metocean Criteria/Loads for further detail on the base shear plots.

The inelastic analysis model is a three-dimensional model, consisting of 832 joints (nodes) and 1,676 members (elements). This model is comprised of linear and nonlinear elements. Typically, six degrees of freedom are considered at each node (joint). STRUT-BEAM elements are used to model vertical diagonal bracings, horizontal diagonal bracings and the primary horizontal members around the perimeter at each level. BEAM-COLUMN elements are used to model jacket-legs and piles which were below the mudline. Linear BEAM elements are used to model the rest of the structural members. The soil P-Y, T-Z and Q-Z curves are properly modeled in the pile foundation. The conductor model is slightly different from that of design level analysis in that the conductors were extended only four feet below the mudline and terminated by hinge supports (three translational restraints at each support).

After reviewing the results of the platform wave base shear calculations (see Figure 3 in Section 5.1), and the code check results in both design level analysis and linear ultimate strength analysis, it was concluded that four wave directions (195, 240, 180 and 270 degrees) would be sufficient to carry out the subsequent static push-over analysis. See Section A.5.4.3 for further detail.

In the static push-over analysis, first one applies the vertical loads to the structural system to evaluate the structural response, such as joint displacements, member end forces, etc. Secondly, lateral load is applied (wave, current and wind) to the structure in increments with each load step corresponding to a fraction (or multiple) of the ultimate lateral load. Each wave direction is considered separately. Four wave directions were performed in this study. See Section A.5.4.4 for further detail.

The external load level (base shear) of the platform was calculated for 10 wave directions (0, 45, 90, 135, 180, 225, 270, 315, 240 and 195 degrees) based on the metocean criteria for the ultimate strength analysis requirements. The maximum lateral load is 3,077 kips in the wave direction of 195 deg. See Figure 3 in Section A.5.1 for the comparison of lateral loads (wave and current) for different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition).

Prior to performing static push-over analysis, an alternative (linear) approach was taken to perform the code checks using the linear model (design level analysis model) and the load level corresponding to the ultimate strength analysis requirement. A material yield strength (Fy) of 36 ksi was used for both the tubular member check and the pile strength check. However, for the joint check, a material yield strength (Fy) of 42 ksi was used. An allowable stress increase factor of 1.0 was specified. See Section A.5.4.6 for the results of both tubular member and joint checks.

Four wave directions were considered (195, 240, 180 and 270 degrees) in the static push-over analysis. See Section A.5.4.7 for further detail.

The pushover load level for the wave directions considered is summarized as follows:

Wave Direction (deg.)	Base Ultimate Lateral Load (Wind, Wave & Current) (kips)	Pushover Lateral Load (kips)
195	3,161	4,425
240	2,996	5,991 (+)
180	2,892	4,338
270	2,453	5,886 (+)

The load-deflection plots were made for four wave directions (195, 240, 180, and 270). The deck joint #1032 located at the drilling deck leg (B-1) was selected as a representative joint with reference to the deck displacements. Two types of load-deflection plots were generated. One is the lateral load versus deck displacement. Another is the ultimate lateral load factor versus deck displacement. The base ultimate lateral load is defined as the magnitude of load that corresponds to the appropriate or required metocean criteria. See Figures 8 through 17 in Section A.5.4.8 for load-deflection plots for further detail.

The static push-over analysis results showed that the ultimate lateral load factor is approximately 1.40 in the wave direction of 195 deg. The ultimate lateral load factor is approximately 1.50 in the wave direction of 180 deg. Whereas, in the wave direction of 240 deg., the ultimate lateral load factor is greater than 2.0. In the wave direction of 270 deg., the ultimate lateral load factor is beyond 2.40. The lowest ultimate load factor is **1.40** (in this inelastic push-over analysis), which occurred in the wave direction of **195** degrees.

In this study, the lateral load level at which the first component reaches an IR = 1.0 is defined as the first member that buckled or yielded in the static push-over analysis.

- In the wave direction of 195 degrees, the lateral load level at which the first component buckled (member #272) was **3,856 kips.**
- In the wave direction of 240 degrees, the lateral load level was **5,392 kips** (member #2179 reached its maximum strength).
- In the wave direction of 180 degrees, the lateral load level was **3,789 kips** (member #272 buckled).
- In the wave direction of 270 degrees, the lateral load level was **5,886 (+) kips** (the structural system response was still in a "linear" mode, no member had yet buckled). See Section A.5.4.9 for further detail.

The Reference level load is based on the API RP 2A 20th edition; it is the design wave with a 100-year return period. A design wave height of 67' and wave period of 13 sec. was used. Ten wave directions (0, 45, 90, 135, 180, 225, 270, 315, 240 and 195 degrees) were specified.

The maximum lateral load was **4,746 kips** which occurred in the wave direction of 240 deg. See Figure 3 in Section A.5.1 for the comparison of lateral loads (wave and current) for different metocean criteria (design level analysis, ultimate strength analysis and API RP 2A 20th edition).

The reserve strength ratio (RSR) is defined as the ratio of a platform's ultimate lateral load-carrying capacity to its 100-year environmental condition lateral loading computed according to present API RP 2A procedures. In this study, the analysis results showed that the platform has a reserve strength ratio (RSR) between 1.18 and 1.39 depending on which wave direction is considered. The lowest RSR is **1.18** which occurred in the wave direction of 195 degrees.

It should be pointed out that the reserve strength ratio is directionally dependent. It raises the question about how many wave directions one should consider in the platform assessment to ensure that the platform's reserve strength is properly evaluated.

In this study, no fatigue analyses were performed. Furthermore, it was assumed that the major tubular joints in the structure had sufficient joint capacity to withstand whatever member-end loads which resulted in each load increment of the static push-over analysis.

iv) Consideration of Mitigation

In the design level analysis, the tubular check showed that at the mudline horizontal framing, EL. (-) 254'-0, the several members with stress ratio exceeding 1.00 were the result of modeling difficulties. There are eight mudmats at the mudline framing level, one at each jacket-leg location. If the mudmat effect was properly taken into account, they would provide lateral support to these horizontal members. Thus, the effective length factor (K) could be reduced. It is recommended that a further investigation of these horizontal members (at the mudline) be carried out. At this point, however, no consideration of mitigation is immediately required. The same argument is also applicable to the linear ultimate strength analysis results.

In the joint check results (both design level and linear ultimate strength analyses), the joints with stress ratio exceeding 0.85 or even greater than 1.00 are mainly K-joints. Again, a modeling problem was the main cause of the resulting high stress ratios. Further investigation with a refined model should be conducted, especially for those joints at the mudline horizontal framing level. Furthermore, a finite element analysis of some selected joints would be feasible especially before any physical mitigation is initiated.

In the static push-over analysis, it was assumed that the joint capacity could reach or exceed its full member-end strength requirements either by refining the model through further analysis or by any other measure that is feasible and practical.

PART B: REVIEW AND FEEDBACK TO API TG 92-5

B.1. OVERALL OPINION OF GUIDELINE

The purpose of API RP 2A Section 17.0 - Assessment of Existing Platforms (draft) is to provide some practical guideline to the designers in the assessment of existing platforms. The contents of API RP 2A Section 17.0 (draft) have been improved significantly since its early version of November 3, 1993.

As the results of both JIP trial application and Benchmark platform exercise, we would like to offer some comments regarding the provision of Section 17.0 (draft) as follows :

B.2 CONSTRUCTIVE CRITICISM / SUGGESTIONS

1) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS
   17.6.2a Gulf of Mexico Criteria
   4. Design Level and Ultimate Strength Analyses :

Comment : (p.13) Table 17.6.2-1 " Gulf of Mexico Metocean Criteria" has provided all information required for both design level and ultimate strength analyses for different exposure categories. For the evaluation of ultimate strength analysis results, it seems to be more meaningful to the engineer to know what RSR value has been achieved than just plain pass or fail the ultimate analysis. For example, if a platform passes the insignificant environmental impact / manned evacuated metocean criteria (Gulf of Mexico) for ultimate strength analysis, what is the equivalent RSR value? The current text of Section 17.0 (draft) has not mentioned it except implicitly by referring to OTC paper #7482 by Krieger et al. It is suggested that the RSR values (for Gulf of Mexico) should be provided in the text or in the commentary. Alternatively, the RSR values can be inserted in Table 17.5.2a of Figure 17.5.2 (p.6).

2) 17.5 ASSESSMENT PROCESS

(p.5) First paragraph

" ------. However, it is permissible to bypass the design level analysis and to proceed directly with an ultimate strength analysis. ---- "

Comment : This option should be reflected in Figure 17.5.2 (continued) (p.7). See attachment.

3) 17.5 ASSESSMENT PROCESS

(p.5) First paragraph

The design level analysis is a simpler and more conservative check, while the ultimate strength analysis is more complex and less conservative. It is generally more efficient to begin with a design level analysis, only proceeding with ultimate strength analysis as needed. ----

Also see Figure 17.5.2 (continued) on "Design Level Analysis" check box -----> passes -----> Platform passes assessment

Comment : This implies that if the design level analysis is performed and passed then no ultimate strength analysis is required. In the trial application of the "C" platform for insignificant environmental impact / manned - evacuated metocean criteria, one interesting but not surprising result has been found that unity check ratio of certain members (mainly horizontal members) for design level analysis is less than that of the ultimate strength analysis. This means that the statement mentioned in the text that "the design level analysis is simpler and more conservative check" might be not always the case. This finding is confirmed from the results of wave load base shear calculations. The base shear ratio (Ultimate / Design) is ranging from 0.58 to 1.69 in 10 wave directions considered. It is recommended that this finding should be incorporated, at least , in the commentary.

4) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

17.6.2b West Coast Criteria

4. Design Level and Ultimate Strength Analyses :

(p.23) First paragraph

------. An ultimate strength check will be needed if the platform does not pass the design level or if the deck height is not adequate.

Comment : Is this statement always true? (see the discussion in the item 3 above).

5) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

17.6.2a Gulf of Mexico Criteria

4. Design Level and Ultimate Strength Analyses :

(p.13) Table 17.6.2-1 " Gulf of Mexico Metocean Criteria"

** If the wave height or current vs direction exceeds that required for ultimate strength analysis, then the ultimate strength criteria will govern.

Comment : The background of using omni-direction wave is not clearly explained in the text or commentary. The mixing of omni - direction and ultimate strength criteria makes sense only if the design level analysis is solely required. Two different metocean criteria must be used to derive the required design wave load in each wave direction. The benefit of using mixed mode (criteria) is not clear.

6) 17.7 STRUCTURAL ANALYSIS FOR ASSESSMENT

(p. 25) 17.7.2d Fatigue

As part of the assessment process for future service life, consideration should be given to accumulated fatigue degradation effects. Where Levels III and IV surveys are made (see Section 14.3) and any known damage is assessed and / or repaired, no additional analytical demonstration of fatigue life is required. Alternatively, adequate fatigue life can be demonstrated by means of an analytical procedure compatible with Section 5.

(p. 38) C17.7.2d Fatigue (Commentary)

Last paragraph

" -------. Should cracks be indicated, no further analysis is required if these are repaired. The use of analytical procedures for evaluation of fatigue may be adequate if only Level II survey is done."

Comments : The results of fatigue analysis can provide valuable information to the platform owner / operator to identify any critical joints in the structure which might be known or unknown having potential fatigue problems. This information might be available from the platform's design file or a fatigue analysis compatible with Section 5 should be performed.

The last sentence of C17.7.2d Fatigue (Commentary) read " The use of analytical procedures for evaluation of fatigue may be adequate if only Level II survey is done." This implies that if you have Level II survey information, it is sufficient to carry out the fatigue analysis. Is there a better word to replace the "only" word in that sentence. You can have Level III or Level IV surveys if you want to (even though that is impractical) before proceeding any fatigue analysis.

7) Static Push-Over Analysis - How many wave directions should be performed? Are three wave directions sufficient?

Comment: The static push-over analysis results showed that the reserve strength ratio (RSR) is directionally dependent, as expected. It raises the question about how many wave directions should be considered in the platform assessment to ensure that the platform's reserve strength is properly evaluated? Of course, this is an engineering judgment call. The experiences learned in this JIP - trial applications by all participants might have sufficient data to incorporate the answer to that question in the commentary. In this study (for , 4 wave directions were selected for the static push-over analysis. The results showed that the range of reserve strength ratio (RSR) is between 1.18 and 1.39.

8) Reduce Joint Check Conservatism

Comment: In the ultimate strength analysis, the mean value of material yield strength (instead of the lower bound value) can be used in the joint check. This is a reasonable approach taken to reduce the conservatism built in the joint check formulas. There are other joint check parameters which should be brought to the task group's attention, such as the chord stress reduction factor, Q_f (see Figure C4.3-3 in API RP 2A 20th edition. also in the attachment). Especially for the in-plane load case, the factor Qf decreased drastically as the factor A approaches 1.0. There were only two test data shown in the Q_f curve (in-plane bending). Is the extrapolation of the result beyond, say A = 0.60 , too conservative? (for in-plane bending case)

B.3. AREAS THAT WERE CLEAR / UNCLEAR

1) 17.3.2 Environmental Impact

17.3.2a Significant Environmental Impact

(p.3) Last paragraph

Except for those cases in which release of hydrocarbons or sour gas would *not* occur, no one factor should be considered alone when performing an environmental impact review.

Comment : "not occur" implies that belongs to other category (insignificant environmental impact). Is my presumption correct?

2) 17.5.3 Assessment for Seismic Loading

6. Ultimate Strength Analysis :

(p. 9) ----- that do not meet the *screening criteria* -----.

Comment : "screening criteria" is not specifically defined in the text except that the term "screening" is appeared in Section 17.5 ASSESSMENT PROCESS (p.4). However, in the commentary C17.7.1 General (p. 37), the term "screening" is explained explicitly. Is the "screen criteria" for seismic loading different from that for metocean? If so, probably some further explanation on the "screen criteria" in the text or commentary would be helpful.

3) 17.5.4 Assessment for Ice Loading

4. Design Level Analysis :

(p. 10) The term "screen criteria" appeared twice in this section.

Comment : Same comment mentioned in item 2) above for seismic loading should be applied to the ice loading.

B.4. ERRATA / ENHANCEMENTS (API RP 2A SECTION 17.0 (DRAFT))

1) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS
17.6.2a Gulf of Mexico Criteria
4. Design Level and Ultimate Strength Analyses :
b. Insignificant Environmental Impact / Manned - Evacuated.

Comment : (p. 22) In the last sentence of third paragraph " ---- a directional spreading factor of 0.88 ---- " should read as " ------ a wave kinematics factor of 0.88 ----- ".

2) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS
17.6.2b West Coast Criteria
1. Metocean System :

Comment : (p. 23) In the 2nd sentence " ---- a directional spreading factor of 1.0 ---- " should read as " ------ a wave kinematics factor of 1.0 ----- ".

3) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS
17.6.2b West Coast Criteria
2. Deck Height Check :

Comment : (p. 23) In the 1st sentence " ---- on the same basis as prescribed in Section 17.6.2a.5 ---- " should read as " ------ on the same basis as prescribed in Section 17.6.2a.**2** ----- ".

4) 17.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS
17.6.3 Seismic Criteria / Loads
(p. 24) 3. Ultimate strength criteria is set at -----

Comment : It is suggested that the term "Ultimate strength criteria" be replaced by "Ultimate strength **seismic** criteria". This applies to the last sentence in this paragraph too.

5) 17.7 STRUCTURAL ANALYSIS FOR ASSESSMENT
(p. 25) 17.7.3 Ultimate Strength Analysis Procedures

Comment : In the first sentence " ----- , to insure adequacy for ----- " be more appropriate to read as " ----- , to ensure adequacy for ----- ".

6) C17.5 ASSESSMENT PROCESS

(p. 33) C17.5.3 Assessment for Seismic Assessment

Comment: The heading " Assessment for Seismic Assessment " should read as " Assessment for Seismic **Loading** ".

B.5. ERRATA / ENHANCEMENTS (API RP 2A-LRFD SECTION R (DRAFT))

The same errata / enhancements shown above should be applied to the API RP 2A- LRFD version (Section R (draft)).

1) R.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

R.6.2a Gulf of Mexico Criteria

4. Design Level and Ultimate Strength Analyses :

b. Insignificant Environmental Impact / Manned - Evacuated.

Comment: (p. 12) In the last sentence of third paragraph " ---- a directional spreading factor of 0.88 ---- " should read as " ------ a wave kinematics factor of 0.88 ----- ".

2) R.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

R.6.2b West Coast Criteria

1. Metocean System :

Comment: (p. 13) In the 2nd sentence " ---- a directional spreading factor of 1.0 ---- " should read as " ------ a wave kinematics factor of 1.0 ----- ".

3) R.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

R.6.2b West Coast Criteria

2. Deck Height Check :

Comment: (p. 13) In the 1st sentence " ---- on the same basis as prescribed in Section R.6.2a.5 ---- " should read as " ------ on the same basis as prescribed in Section R.6.2a.**2** ----- ".

4) R.6 METOCEAN, SEISMIC AND ICE CRITERIA / LOADS

R.6.3 Seismic Criteria / Loads

(p. 14) 3. Ultimate strength criteria is set at -----

Comment : It is suggested that the term "Ultimate strength criteria" be replaced by "Ultimate strength **seismic** criteria". This applies to the last sentence in this paragraph too.

5) COMM R.5 ASSESSMENT PROCESS

(p. 24) Comm R.5.3 Assessment for Seismic Assessment

Comment : The heading " Assessment for Seismic Assessment " should read as " Assessment for Seismic **Loading** ".

PLATFORM ASSESSMENT PROCESS - METOCEAN LOADING

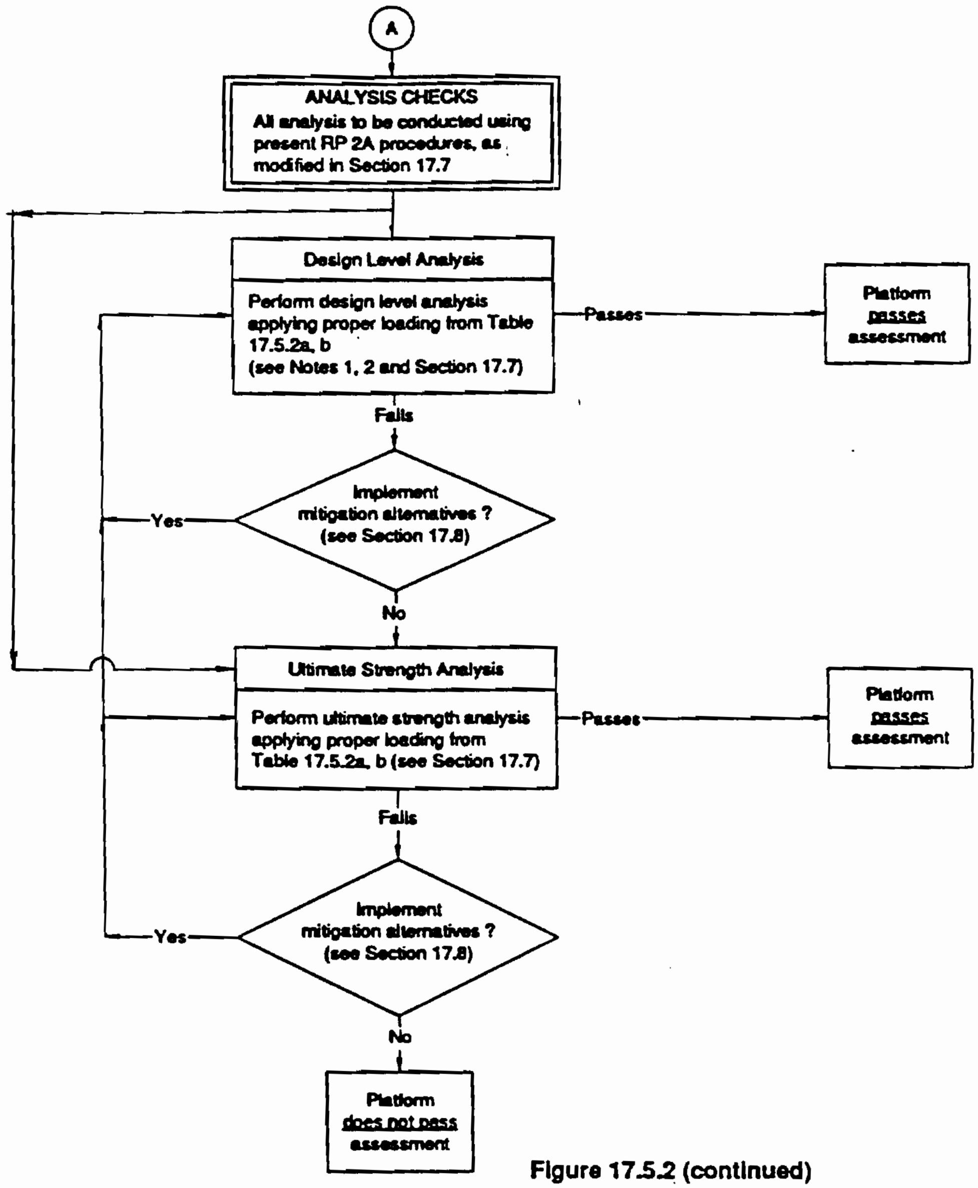


Figure 17.5.2 (continued)

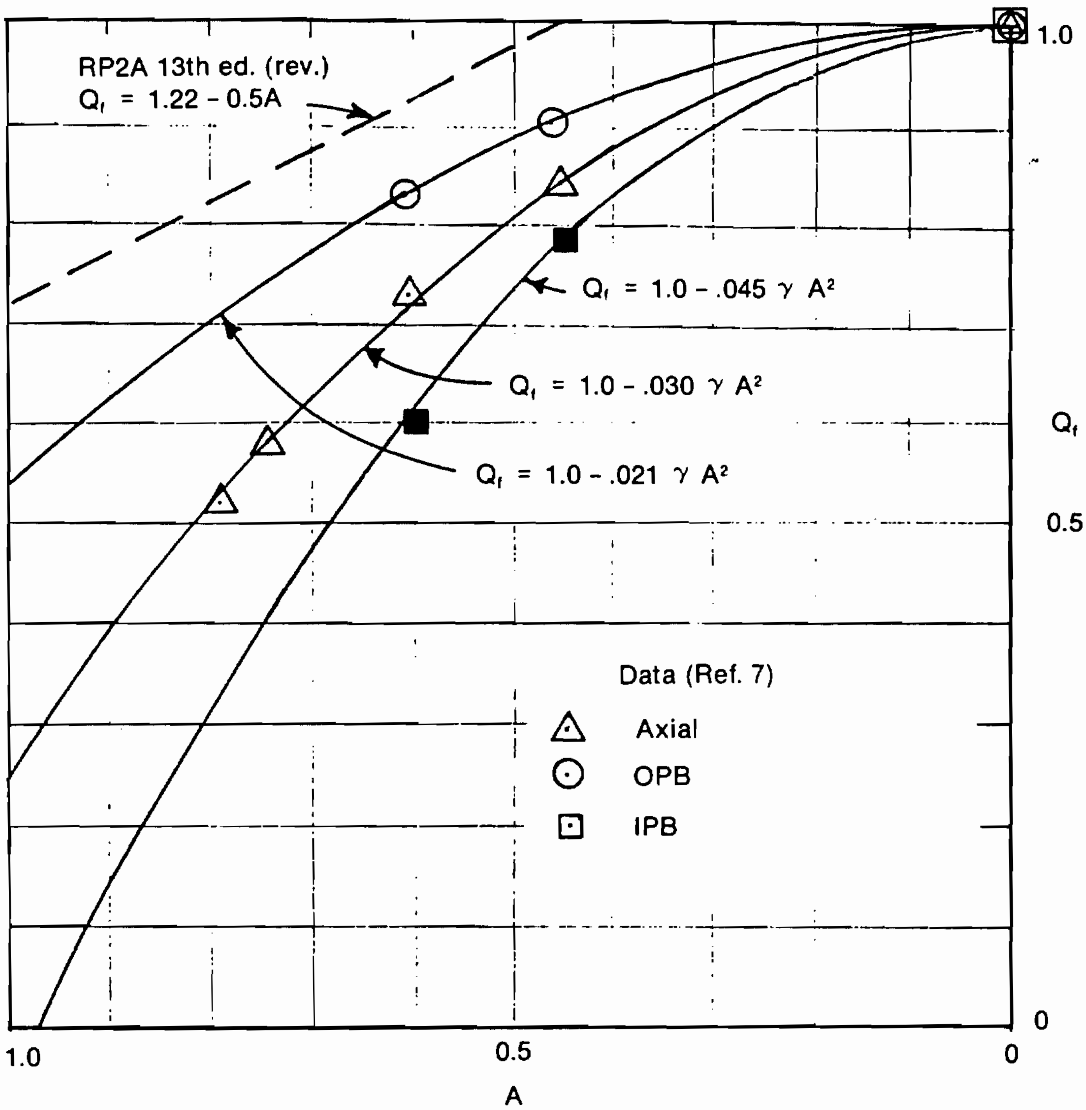


FIG. C4.3-3
CHORD STRESS REDUCTION EFFECTS FOR ALL BRANCH LOAD TYPES WITH SAFETY FACTOR REMOVED

PLATFORM "D"

1.0 PLATFORM INFORMATION

This section provides a brief description of ; Grand Isle Field Platform jacket and deck structures.

1.1 GENERAL CHARACTERISTICS

Grand Isle Field Platform is located in the Gulf of Mexico in a water depth of 88 feet. The platform is composed of Jacket "A" and Jacket "B", joined by a deck structure over the two jackets. Each eight-legged jacket has a foundation system consisting of eight piles, one at each leg. The two jackets are also connected to each other by multiple cables.

The platform was designed in 1957 and successfully resisted operating cyclic and extreme environmental loads for over 30 years.

Some of the pertinent general characteristics of the platform are summarized on Table 1.1-1. As indicated on the Table, while the drilling rig was removed following the completion of drilling program platform strength was increased by grouting of the piles in 1967.

1.2 SPECIFIC CHARACTERISTICS

Component Jackets A and B were lift installed and oriented to make the Platform North to be 45 deg. East of the True North (See Figure 1.2-1). All platform legs are 33" OD x 0.50" with 30" OD x 0.625" grouted pin piles. Each jacket has two rows (1 & 2 for Jacket A and 3 & 4 for Jacket B) of 4 vertical legs (i.e., rows A, B, C and D) held together by braces and cables. Since the cable bracing directly affects the analysis approach the cable arrangement is shown on Figure 1.2-2.

Some of the other specific information directly applicable to the trial application of Section 17 of API RP 2A, including original design basis and applied loading, are summarized on Table 1.2-1.

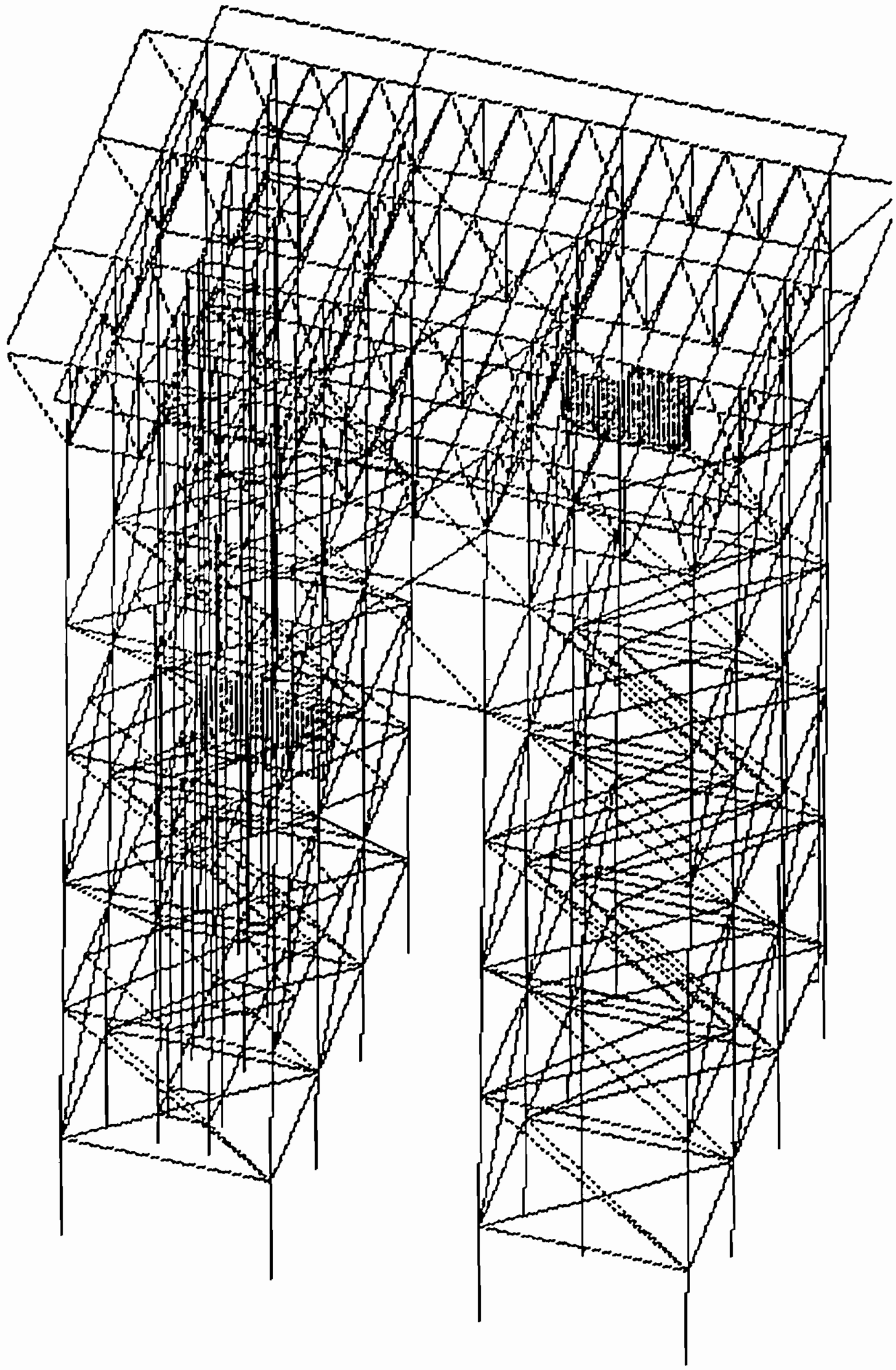


Figure 1.2-1
Platform Perspective

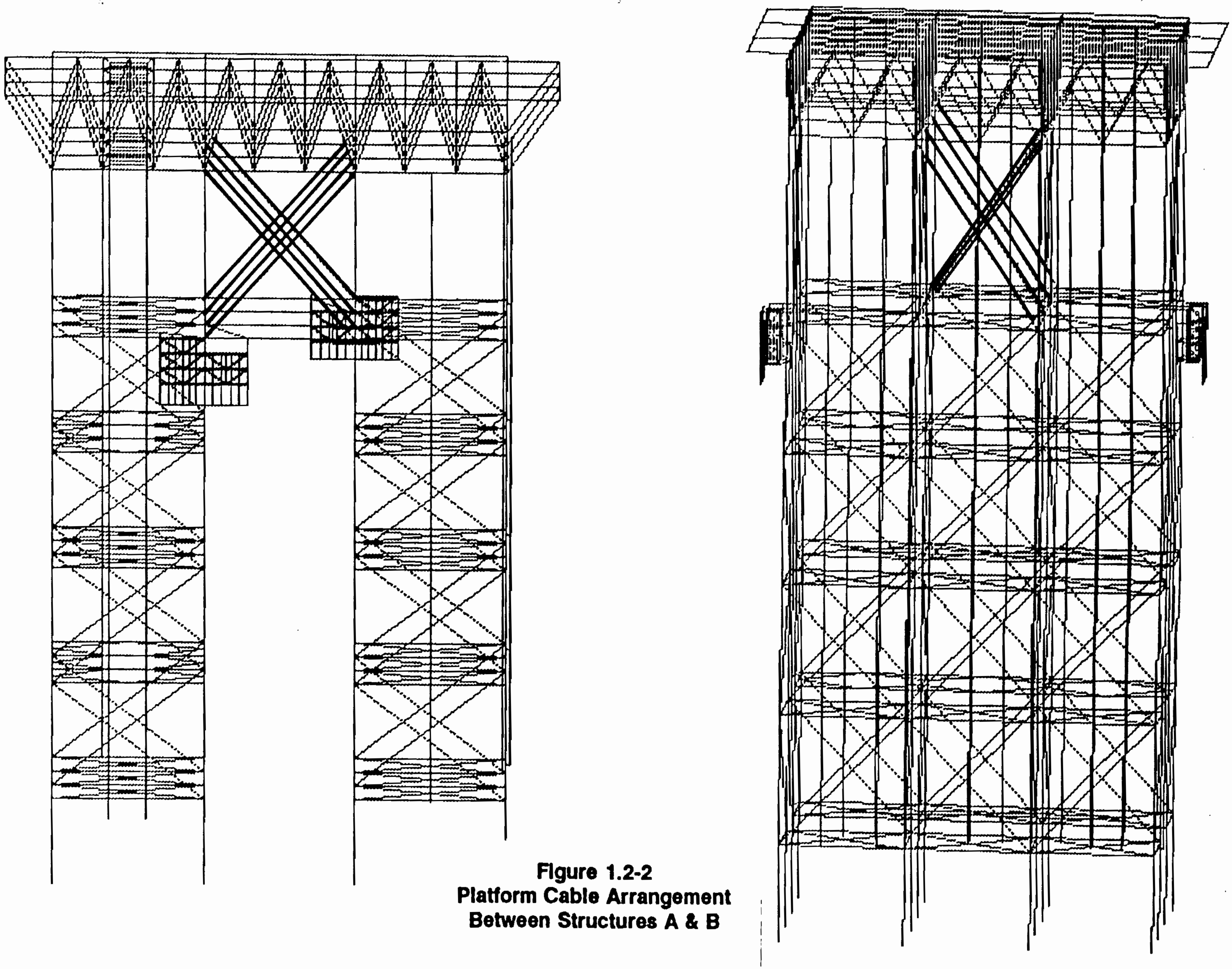


Figure 1.2-2
Platform Cable Arrangement
Between Structures A & B

TABLE 1.1-1 GENERAL CHARACTERISTICS

CHARACTERISTIC	DESCRIPTION	COMMENTS
1. GENERAL INFORMATION		
Owner - Original		
- Current		
Function	Drilling	
Location	Gulf of Mexico	In 88 feet water
Configuration	2x8-legged platforms	A & B Jackets Joined
No. of Wells	14	
Manning Levels	?	Unknown
Performance History	Good	
2. ORIGINAL DESIGN		
Design Contractor	Brown & Root	Lift Installed
Design Drwgs/Specs.	Yes	
Design Code	Unknown	
Design Criteria - Wave	520 psf pressure	At +36 feet crest ht.
- Wind	40 psf pressure	At 100 mph
Deck Clearance Elev.	+44.06 feet	to lower chord bottom
Operational Criteria	?	
Soil Data	?	
Pile Size & Penetration	16 - 30" to ML-150'	
Conductor Size & Penetration	14 - 30"	
Appurtenances	Fenders (4) & Boat Landings (2)	
3. CONSTRUCTION		
Fabr/Install Contractors/Date	Brown & Root	
"As Built" Drawings	?	
Construction Specifications	On Drawings	
Material Traceability Records	?	
Pile & Cond. Driving Records	?	
Pile Grouting Records	?	
4. PLATFORM HISTORY		
Environmental Loads		
Operating Loads		
Survey/Maint. Records		
Repair Descriptions & Dates		
Modification Descr. & Dates		
5. PRESENT CONDITION		
Deck Configuration		
Deck Loads		
Deck Clearance Measurements		
Prod. & Storage Inventory		
Appurtenances		
Wells - No., Size etc.		
Level I Surveys (Above Water)		
Level II Surveys (Below Water)		

NOTE: See Section 1.2 for specific data

TABLE 1.2-1 SPECIFIC CHARACTERISTICS

CHARACTERISTIC	DESCRIPTION	COMMENTS
1. ENVIRONMENTAL LOADING		
Wind Loading - Derrick,		
Setback	21.0	ORIGINAL DESIGN
& Deck	37.7	LOADS
Sub-Total (kips)	90.9	FOR JACKET "A"
Wind Overturning Moment (k-ft)	18,156	About mudline
Wave Loading (kips)	453.2	Basic jacket loading
Wave Overturning (k-ft)	35,653	About mudline
Wind + Wave Load (kips)	544.1	Max. Base Shear
Wind + Wave O.M. (k-ft)	53,809	Max. O.M.
2. OTHER LOADING		
Weight of Jacket (A & B)	776.3	386.2 Jacket A
Weight of Boat landing (2)	35.4	17.7 ea landing
Weight of Fenders (4)	29.8	14.9 two fenders
Sub-Total (Kips)	841.5	418.8 Jacket A
Weight of Piles - above ML	304.36	35% of ea. pile above ML.
- below ML	869.60	Total of 16 piles
Weight of Decks	378.44	
Total Steel - above ML	1524.3	
- inclusive	2089.5	
3. MEMBER SIZES		
Legs with Pin Piles	33.0" OD x 0.50"	
Pin Piles	+ 30.0" OD x 0.625"	
Diagonal Braces	12.75" OD x 0.35"	43.8 lbs/ln ft.
Plan Level Braces	12.75" OD x 0.35"	43.8 lbs/ln ft.
Braces Between A & B Jackets	10.75" OD x 0.35"	40.5 lbs/ln ft.

PART A: PLATFORM ASSESSMENT

A.1 PLATFORM SELECTION

An existing platform is to undergo the assessment process if any of the conditions listed occurs:

- Addition of personnel
- Addition of facilities
- Increased loading on the structure
- Damage found during inspections

Several platforms were reviewed to identify one for the trial application API RP 2A, Section 17. Grand Isle platform in the Gulf of Mexico was chosen for two reasons:

(1) The review of available data has shown that none of the platform assessment initiators listed above has occurred. However, applicable platform design criteria, methodology for the computation of applied loads and the formulations for determining component member and joint capacities have changed substantially in the last thirty years. Thus, the Grand Isle platform, representing the 1950s technology, was considered suitable for trial application of Section 17.

(2) Although in shallow water and not subjected to dynamic response, the Grand Isle platform consists of two jackets with a single deck and interconnected with braces and cables. The jackets are standard eight leg units with pin piles welded on to the legs. Thus, the structure being both unique and standard is suitable for trial application.

A.2 CONDITION ASSESSMENT

A.2.1 General

Assessment of the platform was made to document all pertinent parameters that were grouped under the categories of:

- General information
- Original design
- Construction
- Platform history
- Present condition

The summary findings on the above categories are presented on Table 1.1-1. While the structure appears to be in good condition for a unit to have been in operation for over 35 years, however, it has been subjected to modifications over the years.

A.2.2 Surveys

Surveys carried out over the years and the current assessment indicate the following:

- Pin piles within each leg, originally welded to the legs at elev. +89 feet, were grouted in 1967. Thus, the platform capacity is increased.

- Drilling rig and some of the associated equipment were removed. Thus, the overall design functional loads are in excess of current deck loading.

A.3 CATEGORIZATION

The exposure categories applicable to the Grand Isle platform are as follows:

A.3.1 Life Safety

Of the three categories for life safety, the one applicable "**manned, evacuated**" is underlined:

- Manned, non-evacuated
- **Manned, evacuated**
- Unmanned

A.3.2 Environmental Impact

Environmental impact is defined either as "significant" or "insignificant." This platform is identified to have "**insignificant**" impact due to its operational characteristics.

This categorization results in the selection of the following assessment criteria:

For Design Level (**DL**) Analysis:	Sudden hurricane design level analysis loading per Figure 17.6.2-3 of draft Section 17.
For Ultimate Strength (**US**) Analysis:	Sudden hurricane ultimate strength analysis loading per Figure 1.6.2-3 of draft Section 17.

NOTE: For the US analysis a higher upper bound loading was considered desirable to evaluate platform performance. Thus, platform category was switched from "insignificant" to "significant".

A.4 DESIGN BASIS CHECKS

The first query on determining the design basis checks is the location of the platform. The Grand Isle platform is in the Gulf of Mexico. It also was not designed in accordance with the 9th Edition (or later) of the API RP 2A.

Thus, the platform requires sequential analysis checks as discussed in the following section.

A.5 ANALYSIS CHECKS

A.5.1 Metocean Criteria and Loads

Metocean criteria used during the original design is:

- Maximum wind pressure of 40 psf
- Maximum wave pressure (at the crest) of 520 psf on equivalent legs

The 40 psf wind pressure due to 100 mph wind velocity is substantially higher than the 63 mph currently used for this site for the **DL** analysis and approximately equal to the 98 mph currently used for this site for the **US** analysis.

The 520 psf wave pressure compares quite well with applicable wave pressures based on draft Section 17 recommendations. Applicable **DL** wave height of 41 feet at 11.3 second period will yield about 100 psf pressure on a one foot diameter column. While the 520 psf design pressure was applied only on the legs of the platform (i.e., neglecting braces and conductors) and the marine growth was unaccounted, the overall design loading computed on the platform (i.e., 453 kips on Jacket A) appear to be reasonably close to the wave loading based on draft Section 17 requirements.

Application of draft Section 17 requirements resulted in the selection of wave heights, wave periods, current velocities and the computation of a series of parameters. A summary of metocean criteria in five directions (North, South, Northeast, Southwest and West) is presented in Appendix AA.

A complete computer model of the platform was developed and the environmental loads generated based on the metocean criteria summarized in Appendix A. Base shear and overturning moments determined are summarized on Tables A.5.3-2 and A5.4-2.

A.5.2 Screening

During the development of draft Section 17 an approach defined as "screening" to allow passing of the platform based on the assessment of applied loading was considered. Establishing the adequacy of a platform based on such a simplified procedure require thorough understanding of applied loads on the platform and the response of the platform to these applied loads.

Since substantial uncertainties exist as to the design basis, applied loading, stress distribution within the platform as well as the condition of the platform components, the "screening" procedure is considered not applicable.

A.5.3 Design Level Analysis

Deck Height Criteria:

Based on Figure 17.6.2-3b of Section 17, applicable to sudden hurricane deck height criteria, for a platform in a 88 ft. water depth site the minimum deck height should be **38.5**

feet. Since the lower chord of the deck is at 44 ft. elevation, the deck height criteria is satisfied for the **DL** analysis.

Other Parameters:

Other parameters to be used in the determination of the applied environmental loads are summarized as follows:

DESCRIPTION	MAGNITUDE	COMMENTS
Storm Tide (ft)	4.0	
Marine Growth from		
Inspections h < 5'	2.0	Inches on diameter
0' > h > -40'	4.0	
-40' > h > -88'	1.0	
Drag Coefficient		
C_d @ h > 5'	0.65	
C_d @ h < 5'	1.05	
Inertia Coefficient		
C_m @ h > 5'	1.6	
C_m @ h < 5'	1.2	
Wave Spreading		
Factor	0.88	

Applied Loads:

Environmental loads generated for the **DL** analysis range from a low of 1134 kips to a high of 1261 kips (See Table A5.3.1). Application of the 20th edition of API RP 2A to generate the environmental loads result in the platform being subjected to substantially higher loads. Table A5.3.2 provides a summary of the applied loads on the platform.

The design environmental load of 997 kips (i.e., 453 kip wave load on each jacket and 91 kip wind loading) based on very approximate methodology compare very well with present technology. The summary of wind and wave loads providing base shear and overturning moment data in five wave directions for the **DL** analysis is provided in Table A.5.3-1.

TABLE A.5.3-1 ENVIRONMENTAL LOADING BASED ON DESIGN LEVEL ANALYSIS

DIRECTION / DESCTIPTION	NORTH	SOUTH	WEST	NE	SW
WAVE DATA					
Height (ft)	41.0	37.7	41.0	41.0	41.0
Period (sec)	11.3	12.5	11.3	11.3	11.3
BASE SHEAR DUE TO					
Wind Load (kips)	82	139	64	74	74
Wave:Cellar (kips)	0	0	0	0	0
Wave:Jacket (kips)	1179	1063	1070	1123	1111
Total (kips)	1,261	1,202	1,134	1,197	1,185
OVERT. MOMENT					
Wind (k-ft)	11,600	19,600	9,000	10,400	10,400
Wave:Cellar (k-ft)	0	0	0	0	0
Wave:Jacket (k-ft)	72,800	62,800	61,600	66,700	65,700
Total (k-ft)	84,400	82,400	70,600	77,100	76,100

TABLE A.5.3-2 ENVIRONMENTAL LOADING BASED ON 20th EDITION OF API RP 2A

DIRECTION / DESCTIPTION	NORTH	SOUTH	WEST	NE	SW
WAVE DATA					
Height (ft)	45.2	45.8	53.5	38.5	52.6
Period (sec)	12.2	13.9	13.7	12.1	14.0
BASE SHEAR DUE TO					
Wind Load (kips)	178	178	134	158	158
Wave:Cellar (kips)	0	0	0	0	0
Wave:Jacket (kips)	1624	1422	1839	1187	1795
Total (kips)	1,802	1,600	1,973	1,261	1,953
OVERT. MOMENT					
Wind (k-ft)	25,900	25,900	19,400	22,900	22,900
Wave:Cellar (k-ft)	0	0	0	0	0
Wave:Jacket (k-ft)	100,200	85,000	108,500	68,900	107,000
Total (k-ft)	126,100	110,900	127,900	91,800	129,900

PART A: PLATFORM ASSESSMENT

Analysis Validation:

The computer model was first validated by applying symmetrical quality control loads at the deck level in the x-, y-, z-axes and verifying the load path by reviewing the reactions. The applied functional and environmental loads were validated and the platform reactions reviewed.

Platform distortions at the jacket/pile interface and the deck level were then reviewed for their compatibility with the applied loads and the range of viable foundation response. Pile reactions were within the foundation parametric study range and adequate capacity is considered available. While the maximum axial load was determined to be 720 kips, the pile axial capacity for 30-inch diameter pile was estimated to be 1600 kips (Note: 3000 kip pile capacity at 141 feet penetration was downgraded for skin friction and end bearing components of a 30-inch diameter pile).

Initially all platform joints and members were checked to determine utilization levels. While large number of joints do not meet the API punching shear requirements, these joint are considered to meet the API requirements, provided the grouting was performed according to the specifications and the integrity of grouted joints have not deteriorated.

Member Utilizations:

Platform response to the Design Level loading meets the design criteria. As-designed jacket member utilizations, with the exception of several braces, are within the target utilization ratios (i.e., less than 1.0). Several diagonal brace members are marginally overutilized. However, applying member slenderness ratios based on in-place member end rigidities (i.e., grouted legs) these members also meet the required utilization ratios.

Considering the age of the platform, further analysis was performed by reducing the member wall thicknesses due to corrosion and pitting over the years. This analysis showed that the major diagonal braces running from one leg to another would be overutilized. Thus, a series of members are identified for closer review during next platform inspection. Further assessment of actual condition of these braces would provide better information on the robustness of the platform.

A.5.4 Ultimate Strength Level Analysis

Platform category change allows determination of wave heights based on full population of hurricane waves (rather than sudden hurricane waves) and wave loading on the cellar deck level. This approach may result in platform passing the **DL** analysis and fail the **US** level analysis. However, if the platform can resist twice the **DL** loads without collapsing it will be considered to have passed the **US** level analysis.

Deck Height Criteria:

The deck height criteria will be met based on sudden hurricane deck height criteria (Figure 17.6.2-3b) since the deck lower chord at elev. +44 feet is higher than the minimum required height of +38.5 feet. However, based on full population deck height criteria (Figure 17.6.2-2b), the required deck height is 48 feet. Thus, the cellar deck is subject to wave loading action.

Other Parameters:

Other parameters to be used in the determination of the applied environmental loads are summarized on Table A.5.4-1.

TABLE A.5.4-1

DESCRIPTION	MAGNITUDE	COMMENTS
Storm Tide (ft)	5.0	
Marine Growth from		
Inspections h < 5'	2.0	Inches on diameter
0' > h > -40'	4.0	
-40' > h > -88'	1.0	
Drag Coefficient		
C_d @ h > 5'	0.65	
C_d @ h < 5'	1.05	
Inertia Coefficient		
C_m @ h > 5'	1.6	
C_m @ h < 5'	1.2	
Wave Spreading Factor	0.88	

Environmental Loads:

Environmental loads computed for the **US** level analysis are substantially higher than the **SL** analysis loads. The applied base shear values for the **US** level loading range from a low of 2,340 to a high of 2,581 kips in the five directions investigated.

Comparing the **US** and **DL** base shears, a **US** to **DL** loading ratio range of 2.05 to 2.15 is obtained as shown below:

- North R = (2,581/1,261) = 2.05
- South R = (2,581/1,202) = 2.15
- West R = (2,340/1,134) = 2.06
- NE R = (2,456/1,197) = 2.05
- SW R = (2,451/1,185) = 2.07

Thus, the **US** to **DL** loading ratio is larger than the traditional **US** to **DL** ratio considered desirable, namely ability to resist twice the design level load without collapsing. Table A.5.4-2 presents the applicable loads for the **US** level analysis.

TABLE A.5.4-2 ENVIRONMENTAL LOADS BASED ON ULTIMATE STRENGTH ANALYSIS

DIRECTION DESCTIPTION	NORTH	SOUTH	WEST	NE	SW
WAVE DATA					
Height (ft)	58.0	58.0	58.8	58.0	58.0
Period (sec)	14.6	14.6	14.6	14.6	14.6
BASE SHEAR DUE TO					
Wind Load (kips)	178	178	134	158	158
Wave:Cellar (kips)	282	282	211	249	249
Wave:Jacket (kips)	2,121	2,121	1,995	2,049	2,044
Total (kips)	2,581	2,581	2,340	2,456	2,451
OVERT. MOMENT					
Wind (k-ft)	25,900	25,900	19,400	22,900	22,900
Wave:Cellar (k-ft)	37,400	37,400	28,000	33,000	33,000
Wave:Jacket (k-ft)	131,100	131,000	124,000	124,000	123,400
Total (k-ft)	194,400	194,300	179,900	179,900	179,300

Analysis Validation:

Ultimate strength level analysis model used for the Push Over study is essentially same as the strength level analysis model and further discussed in Section 7. Applied load paths and platform deformations were reviewed and found to be valid.

Platform Response and Member Capacities:

Push Over analysis was performed not for the "pristine" condition but for a condition representing most likely "as is" condition. The assumptions made are as follows:

- Pitting and corrosion is likely to have affected many members. Thus, instead of using the original as-built thickness of 0.375 inches, all diagonal braces were analyzed based on 0.25 inch wall thickness.

- While some leg joints may have reduced capacities due to improper grouting or deterioration of grouted joint over the years, all leg joints were assumed to have full capacity. This assumption was justified as any strength reduction is likely to be small in a sheltered environment and the effect of such reductions will have minimal impact on a platform with numerous joints.

Platform was then shown to have redundant load paths and able to withstand additional environmental loads even after many of its members reach their capacity.

While many of the brace members reach their capacity at environmental loads levels below the strength level loading, the two-structure platform is able to resist full ultimate strength level environmental loading without overall collapse. Further discussion on the findings is presented in Section 7.

A.6 MITIGATION ALTERNATIVES

The structure is able to resist environmental loads more than twice the strength level loading. This conclusion is reached for a platform that was modeled to represent current "as is" condition where the diagonal brace member thicknesses were reduced to account for corrosion and pitting.

Thus, while no mitigation alternatives are considered, key diagonal members are identified for a closer review during next platform inspection to better estimate the actual condition of such brace members.

A.7 SUMMARY OF FINDINGS AND CONCLUSIONS

A.7.1 Design Level Analysis

Platform Deformations

The deck displacements are linear with increase in the applied loads for the design level analysis. At full design level loading the deck lateral displacements average about 7 inches.

Member Utilizations

Several members were found to be overutilized for the design level loading conditions. These members consist of the vertical frame compression diagonals between the first and third plan levels on frame lines A, B, C, and D between frame lines 1 and 2. Four braces had utilizations in excess of 1.3 with a maximum of 3.4. These high utilizations were due to the moment magnification effect under high axial stress. The corresponding diagonal bracing between frame lines 3 and 4 may also be subject to overutilization if the wave crest was positioned to maximize loading on these members.

Implication of Findings - Conclusions

Member thicknesses were reduced over as-built thickness for the purposes of analysis and code checking to account for corrosion. Member inspection might indicate more or less corrosion than assumed and thus justify or prohibit platform requalification based on a design level analysis.

A.7.2 Ultimate Strength Level Analysis

An ultimate strength level analysis consisted of pushover analyses of the platform in two storm directions (platform west and south-west). The pushover analysis consisted of the full application of the stillwater loading condtion (dead, live, and buoyancy) and an incremental application of the ultimate strength level storm loading condition (storm wave, current, and wind, including wave loading on the deck structure). The pushover load was applied in increments of 5 percent of the ultimate strength level load until the platform collapsed.

First Component to Reach Capacity

The first components to reach capacity (note: at reduced 0.25 inch wall thickness) were the same for both wave directions. These members (1020 and 1022) are vertical frame bracing diagonals located between the first and second plan levels on frame line B.

For the west direction pushover analysis, these components reached capacity at a platform base shear load of 902 kips (39 percent of the full ultimate strength level base shear). For the southwest direction pushover analysis, these components reached capacity at a platform base shear load of 1,420 kips (61 percent of the full ultimate strength level base shear). This outcome is reasonable as these component members were overutilized for the design level analysis, with 1.00 utilization occuring at a platform base shear load of 756 and 1,054 kips, for the west and southwest storm directions, respectively (Note: these members would not be overutilized if in pristine condition; reduced wall thicknesses were used to represent member degradation due to corrosion).

Tracking of Components Reaching Capacity

Each pushover analysis was performed until the structure collapsed. Each member component's "capacity ratio" was tracked for each incremental load step. Members reaching capacity are noted on the attached sketches with the member number, axial load compression (C) or tension (T), and the incremental load step number at which each member end reached capacity indicated.

The platform responses were similar for both analyses. Vertical frame diagonals first reach capacity in compression, with the lowest diagonal members reaching capacity first. This is as expected since all diagonals are of the same length and size. As the load increases, some vertical diagonals reach capacity in tension and a few vertical frame horizontals also reach capacity in either compression or tension. Finally, the leg sections immediately below the lowest plan level form plastic hinges, followed by platform failure.

Platform Response

The platform response was linear up to collapse since the platform legs were able resist the applied loads through frame action once other component members reached capacity. The average deck lateral displacement at collapse was just over 17 inches.

Platform Reserve Capacity

The platform collapsed at a base shear load level of 2.68 and 1.94 times the design level base shear. The collapse can also be expressed to have occurred at 1.30 and 0.94 times the ultimate strength level base shear for the west and southwest storm directions, respectively. However, the platform shows little or no nonlinearity in its response up to failure.

Conclusions

For the purposes of this trial application only, the ultimate strength level analysis indicates that this platform would pass the requalification assessment based on the Draft API RP2A Section 17 requirements. The platform joints were checked but were considered to be adequate as all leg joints are grouted. However, the integrity of the grouted joints require validation. Foundation data provided by indicates that the platform foundations will have adequate capacity to resist the most likely upper bound loads.

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	2.8	400	None		
4	4.2	651	None		
6	5.6	902	1020 1022	Buckling-DH Buckling-SH	DH Step 8
8	6.9	1,154	1026 1028	Buckling-DH Buckling-DH	
10	8.3	1,406	1017 1019	Tension-DH Tension-SH	DH Step 12
12	9.7	1,657	1023 1025 2017 2019 2025	Tension-DH Tension-DH Buckling-SH Buckling-DH Buckling-DH	DH Step 14
14	11.0	1,909	1030 2023 3022 3028	Buckling-DH Buckling-DH Buckling-DH Buckling-DH	Horizontal
16	12.4	2,160	1031 4019 4025	Buckling-DH Buckling-DH Buckling-DH	Horizontal
18	13.8	2,412	1131 1134	Tension-SH Tension-DH	Single Hinge
20	15.2	2,667	1035 1036	Tension-DH Tension-DH	
22	16.6	2,908	107 113 131 185 1130 1132/1133 2020 2022 3019	Buckling-SH Buckling-SH Tension-SH Tension-SH Tension-DH Buckling-DH Tension-DH Tension-DH Tension-DH Tension-DH	Leg/Pile Leg/Pile Leg/Pile Leg/Pile
23	Collapse	3,040			130% USL

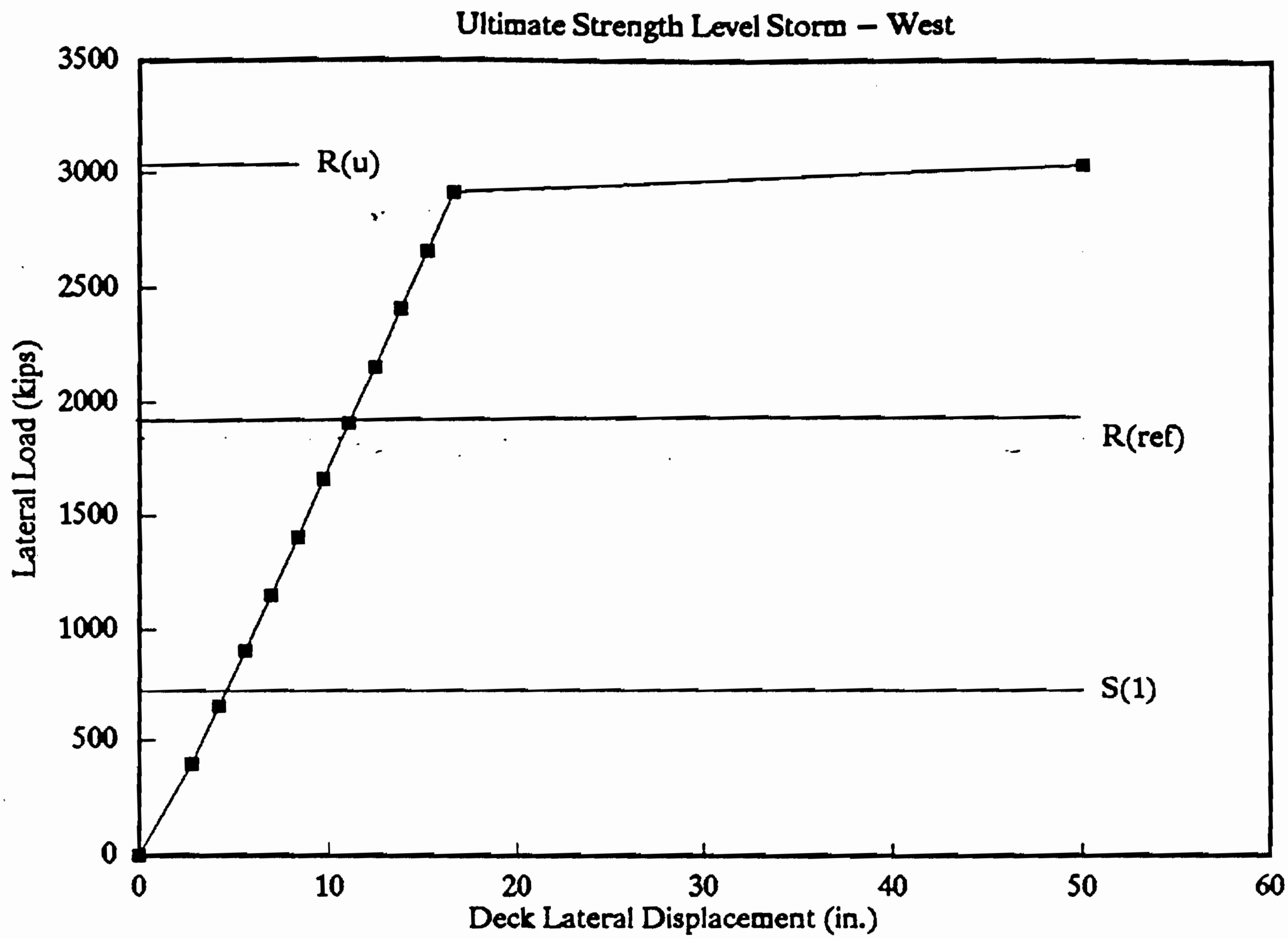
SH = Single Hinge, DH = Double Hinge USL load = 2,340 k

Table A7.2-1
Ultimate Strength Analysis Results - West Direction

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	3.0	439	None		
4	4.1	635	None		
6	5.8	831	None		
8	6.3	1,027	None		
10	7.4	1,234	None		
12	8.5	1,420	1001 1003 1005 1020 1022 1026 1028	Buckling-SH Buckling-SH Buckling-DH Buckling-DH Buckling-DH Buckling-SH Buckling-SH	DH Step 14 DH Step 14 DH Step 14 DH Step 14
14	9.7	1,616	None		Some 2nd Hinges Form
16	10.8	1,812	1006	Tension-SH	DH Step 18
18	11.9	2,008	1002 1004 1017 1023 1040	Tension-SH Tension-SH Tension-SH Tension-SH Buckling-DH	DH Step 20 DH Step 20 DH Step 20 DH Step 20 Horizontal
20	13.0	2,204	101 131 1007 1019 1025 1037 2019	Buckling-SH Tension-SH Buckling-SH Tension-DH Tension-DH Tension-DH Buckling-DH	Leg/Pile Leg/Pile Horizontal
21	Collapse	2,302			94% USL

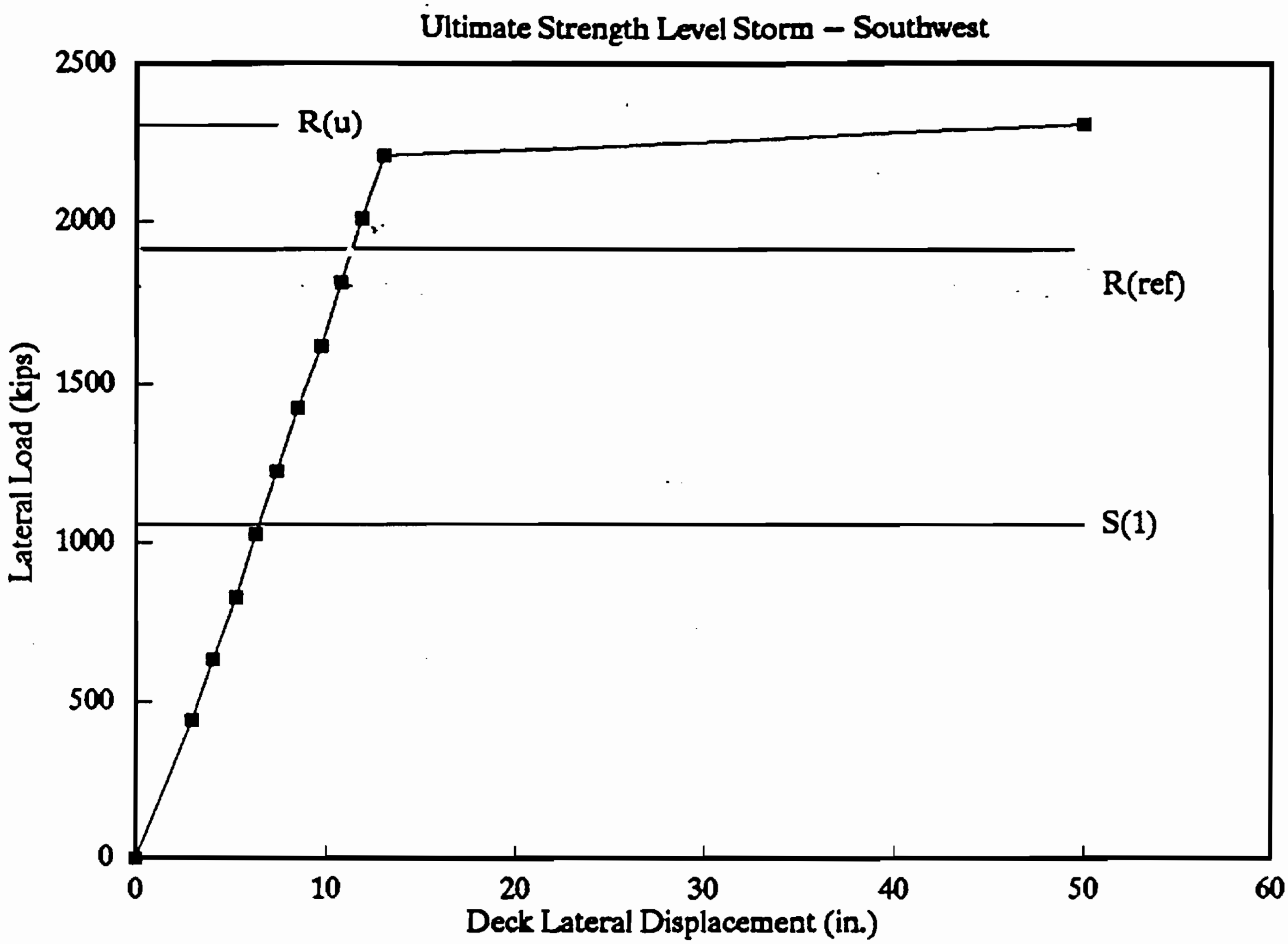
SH = Single Hinge, DH = Double Hinge **USL load = 2,451 k**

Table A7.2-2
Ultimate Strength Analysis Results - Southwest Direction



Load Level at which First Component Reaches I.R. of 1.0, S(1)	726 kips
Reference Level Load, (S_{ref})	1,973 kips
Design Level Load (DLL)	1,134 kips
Ultimate Strength Level Load (USL)	2,340 kips
Ultimate Capacity (R_u)	3,040 kips
Reserve Strength Ratio (RSR) = R_u / DLL	2.68
Platform Failure Mode: Jacket, Pile, Soils, etc.	Leg/Pile

Figure A7.2-1
Ultimate Strength Level Load-Displacement Results - West Direction



Load Level at which First Component Reaches I.R. of 1.0, S(1)	1,054 kips
Reference Level Load, (S_{ref})	1,953 kips
Design Level Load (DLL)	1,185 kips
Ultimate Strength Level Load (USL)	2,451 kips
Ultimate Capacity (R_u)	2,302 kips
Reserve Strength Ratio (RSR) = R_u / DLL	1.94
Platform Failure Mode: Jacket, Pile, Soils, etc.	Leg/Pile

Figure A7.2-2
Ultimate Strength Level Load-Displacement Results - Southwest Direction

APPENDIX AA

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
NORTH				
Wave Height, H(ft)	41.0	58.0	45.2	
Wave Period, T (sec)	11.3	13.5	13.0	
Current				
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.2	2.3	2.1	
Velocity (fps)	2.0	3.9	3.5	
Direction	North	North	215 deg	
Wave dir V (fps)	2.0	3.9	-2.9	
Computed Parameters				
V / gT				
d / gT²	0.005	0.009	-0.007	
T_s / T	0.021	0.015	0.016	
Apparent Period, T_s	1.04	1.08	0.94	
H / gT_s^2	11.8	14.6	12.2	
d / gT_s^2	0.009	0.008	0.009	
Wind (1 hr at 10m)	0.020	0.013	0.018	
velocity (knots)				
velocity (fps)	55	85	85	
	63	98	98	
SOUTH				
Wave Height, H(ft)	37.7	58.0	45.0	
Wave Period, T (sec)	12.5	13.5	13.0	
Current				
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.8	2.3	2.1	
Velocity (fps)	3.0	3.9	3.5	
Direction	South	South	215 deg	
Wave dir V (fps)	3.0	3.9	2.9	
Computed Parameters				
V / gT				
d / gT²	0.004	0.009	0.007	
T_s / T	0.017	0.015	0.016	
Apparent Period, T_s	1.04	1.08	1.07	
H / gT_s^2	13.0	14.6	13.9	
d / gT_s^2	0.007	0.008	0.007	
Wind (1 hr at 10m)	0.016	0.013	0.014	
velocity (knots)				
velocity (fps)	70	85	85	
	81	98	98	

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
NORTHEAST				
Wave Height, H(ft)	41.0	58.0	38.5	
Wave Period, T (sec)	11.3	13.5	13.0	
Current				
Blockage Factor	0.85	0.85	0.85	
Velocity (knots)	1.2	2.3	2.1	
Velocity (fps)	2.0	3.9	3.5	
Direction	NE	NE	215 deg	
Wave dir V (fps)	2.0	3.9	-3.5	
Computed Parameters				
V / gT	0.005	0.009	-0.008	
d / gT^2	0.021	0.015	0.016	
T_s / T	1.04	1.08	0.93	
Apparent Period, T_s	11.8	14.6	12.1	
H / gT_s^2	0.009	0.008	0.008	
d / gT_s^2	0.020	0.013	0.019	
Wind (1 hr at 10m)				
velocity (knots)	55	85	85	
velocity (fps)	63	98	98	
SOUTHWEST				
Wave Height, H(ft)	41.0	58.0	52.6	
Wave Period, T (sec)	11.3	13.5	13.0	
Current				
Blockage Factor	0.85	0.85	0.85	
Velocity (knots)	1.2	2.3	2.1	
Velocity (fps)	2.0	3.9	3.5	
Direction	SW	SW	215 deg	
Wave dir V (fps)	2.0	3.9	3.5	
Computed Parameters				
V / gT	0.005	0.009	0.008	
d / gT^2	0.021	0.015	0.016	
T_s / T	1.04	1.08	1.08	
Apparent Period, T_s	11.8	14.6	14.0	
H / gT_s^2	0.009	0.008	0.008	
d / gT_s^2	0.020	0.013	0.014	
Wind (1 hr at 10m)				
velocity (knots)	55	85	85	
velocity (fps)	63	98	98	

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DL	SEC. 17 US	API RP 20TH	COMMENTS
WEST				
Wave Height, H(ft)	41.0	58.0	53.5	
Wave Period, T (sec)	11.3	13.5	13.0	
Current				
Blockage Factor	0.80	0.80	0.80	
Velocity (knots)	1.2	2.3	2.1	
Velocity (fps)	2.0	3.9	3.5	
Direction	WEST	WEST	215 deg	
Wave dir V (fps)	2.0	3.9	2.0	
Computed Parameters				
V / gT	0.005	0.009	0.005	
d / gT^2	0.021	0.015	0.016	
T_s / T	1.04	1.08	1.05	
Apparent Period, T_s	11.9	14.6	13.7	
H / gT_s^2	0.009	0.008	0.009	
d / gT_s^2	0.020	0.013	0.015	
Wind (1 hr at 10m)				
velocity (knots)	55	85	85	
velocity (fps)	63	98	98	
Wave Height, H(ft)				
Wave Period, T (sec)				
Current				
Blockage Factor				
Velocity (knots)				
Velocity (fps)				
Direction				
Wave dir V (fps)				
Computed Parameters				
V / gT				
d / gT^2				
T_s / T				
Apparent Period, T_s				
H / gT_s^2				
d / gT_s^2				
Wind (1 hr at 10m)				
velocity (knots)				
velocity (fps)				

PART A: PLATFORM ASSESSMENT

APPENDIX AAA

ANALYSIS SPECIFICS

PART A: PLATFORM ASSESSMENT

APPENDIX AAA: ANALYSIS SPECIFICS

AAA1. INTRODUCTION

OBJECTIVES

The purpose of the in-place ductility level "Push Over" analyses and design is to validate the design by showing that although the structure subjected to Ultimate Strength (**US**) design level environmental loading (typically a 1000-year recurrence interval event) may become non-operational, it will not collapse. In addition to the direct application of **US** level loading to assess platform performance, this premise can also be validated by illustrating that the structure subjected to ever increasing Push Over loading finally collapses at either:

- An applied lateral load equal to or greater than twice that of the Design Level (**DL**) load associated with a 100-year recurrence interval event, or

- At an applied lateral load compatible with a predefined ultimate strength level condition associated with a substantially greater recurrence interval.

SCOPE OF WORK

The scope of work for the in-place **US** level analyses will be divided into the following tasks:

- Modify the existing model to inactivate all appurtenances and some of the secondary members not contributing to the in-place capacity of the platform.

- Modify the existing strength level analysis computer stiffness model of the platform jacket, deck, and foundation systems. All primary jacket and foundation members will be defined as non-linear (inelastic) "Strut-Elements" and "Beam-Columns". All secondary jacket members and all deck members will be defined as linear (elastic) elements.

- Revise, as necessary, the permanent in-place static stillwater loads used in conjunction with **US** conditions.

- Perform stiffness analysis for static **US** level load conditions.

- Generate environmental loads compatible with both **DL** and **US** parameters.

- Verify the validity of applied environmental loads, the load paths within the platform and transmission of the applied loads on to the foundation.

- Apply the **US** load incrementally in one direction and check all member capacities. Define all component member loadings, deformations and utilized capacities. Rearrange the data to list the members by utilized capacities.

- Determine a new stiffness matrix and continue the analysis for the next load increment. Continue the process until reaching **US** level loading or system collapse.

The specifics of modeling and nonlinear elements are discussed in the following sections.

AAA2. COMPUTER MODEL

The platform jacket, deck and foundation structure and applicable appurtenances, including conductors, caissons and risers, were modelled using the structure analysis and design software ASADS (Advanced Structures Analysis and Design System).

TOPOLOGY

A three-dimensional space frame computer model generated for the **DL** analyses was revised to meet the **US** level "Push Over" analysis objectives. Conductor, caisson and riser members were modelled only for the purpose of load generation. The conductors offer substantial resistance to applied environmental loading and were included in the analysis. Member sizes were taken from the latest revision of the available platform drawings with reduced brace member wall thicknesses to account for corrosion effects.

There are no joint cans in this structure; joint chord thicknesses being equal to member thickness. However, all of the legs were grouted in 1967.

PROPERTIES

Tubular members were typically input as one-segment prismatic tubulars with specified outside diameter and thickness. Built-up plate or wide flange girder members were input as one-segment prismatic girder members unless specific size variations are indicated on the drawings. Input will be further revised to reflect on-going design effort.

Equivalent pile-soil matrices were determined for a range of nonlinear foundation response to **DL** and **US** analysis loadings. The validity of the assumed matrices was confirmed prior to the initiation of "Push Over" analysis. At each step of the **US** analysis, platform stiffness matrices were recomputed.

Jacket appurtenances consist of conductors, caissons, and risers. These are all included in the model for load generation. Only the structural members remain active for the Push Over analyses.

AAA3. ENVIRONMENTAL DATA

HYDRODYNAMIC DATA

Hydrodynamic data used in the analyses is in accordance with the Draft Section 17 of the API RP 2A. Summary of all input data is presented in the computer input and output.

Marine growth was based on inspection data for the platform and considered to represent present condition of the platform accurately.

METOCEAN DATA

Environmental data used for the **DL** and **US** analyses is based on both Client input and the draft Section 17 of the API RP 2A document. Tabulated data is presented in Appendix AA.

The ductility level **US** Push Over analysis were based on environmental loads generated for **US** level loading, applying a fraction of this load distribution and incrementally increasing the load until either platform collapse or exceeding the full **US** level loading.

AAA4. BASIC LOAD CASES

STILLWATER LOAD CASES

This section summarizes dead, live, buoyancy and ballast load conditions implemented on completed **DL** analysis and the revisions introduced for **US** level analysis.

Dead Load

Dead loading was generated by ASADS for all beam-type members based on input cross-sectional areas and input or assumed weight densities. Dead load for facilities, equipment, non-modelled appurtenances and other significant structure (e.g. mudmats) was hand-calculated and explicitly input at the appropriate location.

Generalized area loads were developed based on equipment weights applied to the deck structure.

Live Load

Deck area live loads were modelled per the Design Basis with appropriate reductions for the primary platform structure in recognition of the fact that not all areas will be loaded simultaneously to their design live load. Deck area live loads were modelled as linear live loads on adjacent modelled girders as a function of deck beam framing orientation. Deck area live load conditions were developed for each deck level and may be further subdivided to allow subsequent load combinations utilizing varying combinations of live load.

Buoyancy Load

Buoyancy loads were generated by ASADS for all tubular members below MSL.

PART A: PLATFORM ASSESSMENT

US LEVEL "PUSH OVER" LOAD COMBINATIONS

Stillwater condition loads were combined with lateral Push Over loads applied first along one axis and then along diagonal axis (i.e., X- and XY-axes). The **US** level load in a given direction was applied as discussed in the following section.

Load Factors and Stillwater Condition

A stillwater loading combination developed consisting of the following load conditions for **DL** analysis will be used for the **US** level Push Over analysis:

- Deck facilities and equipment and other load
- Deck live load
- Jacket and deck dead (self weight) load
- Jacket buoyancy load

US Level Load Combination

The ductility level Ultimate Strength (**US**) Push Over loading combinations consist of the stillwater load condition an incrementally increasing lateral load condition as follows:

- Stillwater condition + up to 125% of the Northwest **US** storm wave, current and wind.

- Stillwater condition + up to 125% of the West **US** storm wave, current and wind.

AAA5. ANALYSIS METHOD AND VALIDATION

GENERAL METHODOLOGY

US Level analyses may be performed by implementing either one of the following methods:

- Push-Over Method
- Time-Domain Method
- Equivalent Method

The Time-Domain Method will accurately capture structural dynamic response due to excitational loads. An alternate Equivalent Method (such as Serrahn's FOURDYN, Reference 4), utilizing a Frequency-Domain method that incorporates dynamic modes (each with its own participating factor through the use of Fourier Series) can be effectively used to capture dynamic response of the structure to excitational loads. Since the Grand Isle platform dynamic response to excitational loads will be negligible, both of the methods described are not applicable for such platforms.

A Push-Over Method was used in the project to effectively track the performance of platform by incrementally increasing the applied loads. The **US** analysis results are directly applicable for Push Over analysis. A fraction of this loading was applied on the platform and incrementally increased after each step. The following generalized steps were taken:

- Apply stillwater load condition; reformulate component stiffness and iterate if any component reaches capacity.
- Incrementally apply the pushover loading, reformulate the component stiffnesses for components reaching capacity.
- Iterate to find the solution for each increment of load application.
- Conclude the analysis at structure collapse or at the application of predefined loading level **(US or higher)**, whichever occurs first.

To conservatively assess the overall reserve capacity of the platform an initial Push Over analysis was first performed for the actual **US** loading. This effort indicated total number of components reaching capacity and overall load absorption capacity of the platform.

<u>LINEAR VERSUS NONLINEAR BEHAVIOR</u>

All jacket tubular members were modelled as nonlinear beam-column members. Deck non-tubular members were not expected to reach capacity; thus for the purposes of analysis efficiency, all non-tubular deck members were modelled as linear truss (strut) members. Piles were modelled as non-linear beam-columns with additional orthogonal linear strut elements to properly model the pile head loads and resultant pile response during the analysis.

Understressed component members and members contributing little to overall resistance of the platform to increased Push-Over loading could have been defined as linear members since such members are expected to perform elastically throughout the ultimate strength level Push Over analysis. However, all jacket members were defined as non-linear beam-columns to validate the assumption. Deck girders and beams, secondary members, and non-structural members were considered to be good candidates for such a definition and some of these were modeled as linear struts. Analysis validation included review of all members to ensure that the elastic behavior assumption for these members were maintained.

Those members defined to have nonlinear behavior can be defined by material stress-strain relationship in resisting applied axial compression or tension and bending. When a member reaches its yield load capacity, its post-yield capacity is determined. Initially, post failure may be defined to be zero capacity. However, since such an approach is conservative and will lead to erroneous tracking of any potential collapse mechanism, components reaching their capacities were defined as a function of their properties and deformations. Two basic types of linear/nonlinear elements can be used to define member behavior:

- Strut Elements: Platform braces expected to fail primarily in axial tension/yielding or axial compression/buckling may be modeled with strut-type elements which account for reductions in strength and stiffness after yielding/buckling. Typically, member slenderness and D/t ratios influence performance of such members.

The assumed post-yield/buckling capacity is the yield/buckling capacity. This is equivalent to a strain-hardening ratio of 0.0.

- Beam-Column Elements: Platform legs, piles and other members with low slenderness ratios and high bending stresses will be defined as Beam-Column Elements. These members are primarily expected to fail due to high bending stresses with increases in the applied loading. Beam-Column elements will effectively account for axial and bending interaction and facilitate definition of reduction in capacity with progressive elasto-plastic hinge formation. Th post-buckling capacity of all components with compact sections (i.e., D/t ratio of less than $1500/F_y$) were assumed to have a strain hardening ratio of 1.2%. Review of test data for various D/t ratios indicate the appropriate range of strain hardening to be 1.15 to 1.50%.

STATIC ANALYSIS

A standard stiffness analysis was preformed for all applicable static loading conditions. This analysis included substructuring of various platform components such as the deck and jacket structures in order to allow for a more time efficient stiffness analysis. The interconnecting cables were specifically excluded from any substructure.

Once substructuring was completed, six separate stiffness analyses were performed, one for stillwater load condition, and one each for each of the five storm directions considered. If a connecting cable would be loaded in tension for a given analyzed storm direction, it was excluded from that particular stiffness analysis.

ANALYSIS VALIDATION

A linear analysis is not considered to be valid unless it is compatible with the nonlinear foundation system. At the beginning of DL analysis, pile top reactions and displacements were obtained for the highest utilized pile and compared with the nonlinear pile response. Typically, if adequate compatibility is not obtained, revised equivalent pile stiffnesses are determined. The global stiffness analysis will then be rerun and another compatibility check performed until adequate pile compatibility is obtained.

Same approach is implemented in the Push-Over analysis. Since static equivalent loading is applied, the pile-structure interaction can be automatically accounted for and compatibility achieved at each step of the analysis. However, an automated pile-structure interaction option was not used during the analysis of the platform and instead, compatibilities checked at predefined load increments manually.

Other validation efforts include, but are not limited to the following:

- The assumption of elastic behavior for selected members was verified for these members at the end of the US level analysis.

- The effect of member post-failure capacities on the overall platform behavior and reserve strength was reviewed to assess their impact on failure path.

AAA6. MEMBER AND JOINT DESIGN

Member forces and moments obtained from the **DL** loading combinations previously discussed was used for member strength and stability checks. In general, all tubular members and joint chords were checked against the requirements of API RP 2A, 20th Edition (Reference 2). Wide flange shapes and truss connections were generally checked against the requirements of AISC, 9th Edition (Reference 3).

LIMITING STRESSES

The **DL** design loading combinations were checked against 1.33 times basic allowable stresses; i.e. 33 percent increase in basic allowable stresses. The stillwater loading was also checked at basic allowable stresses.

JACKET MEMBERS AND JOINTS

Tubular Member Design Check

All tubular members were checked for adequacy against the following failure modes:

- yield
- local buckling
- column buckling
- bending alone
- shear
- external hydrostatic pressure
- combined axial and bending
- combined axial, bending and external hydrostatic pressure

Column buckling effective length factors (K) for all members were based on API RP 2A recommendations.

For code checking, unbraced lengths were taken as the joint-to-joint (work point to work point) length for all members provided that they are adequately braced in orthogonal directions. For bracing members which may be supported in only one direction, two member unbraced lengths were computed, each length being the actual unbraced length in that direction.

Bending moment reduction factors (Cm) for all members were based on the recommendations of API RP 2A. In general, utilization ratios were determined at the two ends and at midspan. The target utilization ratio for members reaching capacity was defined as 1.0.

Member utilization ratios were summarized by member and by utilization ratio for all utilizations at each **US** step to facilitate review of potential failure paths.

PART A: PLATFORM ASSESSMENT

Joint Design Check

Unstiffened simple tubular joints were checked for punching shear in accordance with API recommendations. The target joint utilization for failure definition was 1.00.

The punching shear check utilized API RP 2A's "nominal loads method" to determine punching shear utilizations. It should be noted that all joints were checked to meet the requirements of API RP 2A, Section 4.1.1-1. Since all joints were checked as a part of the punching shear check to resist brace capacity, no joint is expected to fail during **US** level analysis.

FOUNDATIONS

Foundation members consist of piles and the pile/jacket interfaces. The piles were modeled as non-linear "Beam-Column" elements and checked against degradation of their load carrying capacity due to elasto-plastic hinge action.

Since the plastic hinge formation occurs due to combined axial compression and bending effects, elasto-plastic capacity of piles were determined for a range of load combinations and deformations.

The maximum pile top loads and deformations for the following two piles are as follows:

ITEM (UNITS)	CORNER PILE	ADJACENT PILE
Axial Load P (Kips)	679	720
Shear Load V (Kips)	172	199
Bending Moment M (K-in)	-23,600	-23,960
Lateral Deflection (in)	13.0	12.8
Rotation (Radians)	-0.015	-0.013

Maximum pile axial load of 720 kips is substantially smaller than the estimated pile foundation capacity of 1600 kips derived from skin friction (1,125 kips) and end bearing (475 kips). However, at mudline the axial stress utilizes 22% of the total pile load capacity. Since full plastic moment capacity of the pile is about 30,300 kip-inches, the applied moment results in the development of a full plastic hinge.

PART A: PLATFORM ASSESSMENT

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3. American Institute of Steel Construction, "Manual of Steel Construction - Allowable Stress Design," Ninth Edition, 1989.

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PLATFORM "E"

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

TABLE OF CONTENTS

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

EXECUTIVE SUMMARY

This report presents a summary of the analysis work and field inspections conducted by The platform is an 8-leg, 18-slot, self-contained drilling and production platform in 247 feet of water at Main Pass Block 293, Gulf of Mexico.

The analyses were performed as part of a Joint Industry Project entitled "Trial Application of the Draft API RP 2A Guidelines of Existing Platforms." The purpose of the analyses is to determine if the platform can withstand the environmental forces defined in API RP 2A, 20th Edition, Section 17 (draft version). Three analyses were performed: design-level analysis, linear elastic ultimate-strength analysis, and static nonlinear pushover analysis. The 3 analyses show that the platform can satisfactorily support the loads described in API RP 2A, 20th Edition, Section 17 (draft version), for a platform categorized as manned, evacuated with an insignificant environmental impact. The only concern left for a more detailed investigation are the joints supporting K-brace members located at Row 1, Row 2, Row 3, and Row 4, which the design-level and linear elastic ultimate-strength analyses show to be overstressed. Additional analyses are recommended since the current joint check procedure has some conservatism built into its equations.

The field inspections (Level I and Level II inspections) show that there is no significant structural damage to the platform.

Based on the global analysis behavior (excluding the above joints), platform inspection, and 25 years in-service with no significant damage, the structure can support the load defined in API RP 2A, 20th Edition, Section 17 (draft version).

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth

TRIAL DOCUMENT

1.0 PLATFORM INFORMATION
1.1 As-Is Condition Details

1.1.1 PHYSICAL FEATURES

The platform is an 8-leg, 18-slot, self-contained drilling and production platform in 247 feet of water at Main Pass Block 293, offshore Louisiana in the Gulf of Mexico.

The platform is composed of deck, jacket, and piles. The deck consists of a drilling and production deck level 136' long by 72' wide. The production deck top-of-steel is at El (+)47'-6", and the top of the drilling deck is at El (+)65'-11". The spacing between longitudinal rows is 40'-0" and 35'-0", with 40'-0" and 35'-0" spacing between lateral rows. A 13'-0" wing is typical for the longitudinal frames, and a 16'-0" wing is typical for the lateral frames.

The jacket which serves as a template for the eight 42" diameter piles is 262'-6" tall and has the following configuration: 40' x 110' at top-of-jacket, 115'-6" x 105'-6"' at the base, two longitudinal frames (Row A and Row B), four transverse frames (Row 1, Row 2, Row 3, and Row 4), and six horizontal levels at El (+)10', El (-)40', El (-)90', El (-)139'-7-3/8", El (-)189'-2-3/4", and El (-)247'-0".

Each of the eight main 42" diameter piles has a wall thickness at the mudline of 1.375" and a penetration below mudline of 240'. The pile to jacket connection is by means of six 3/4" plates per leg.

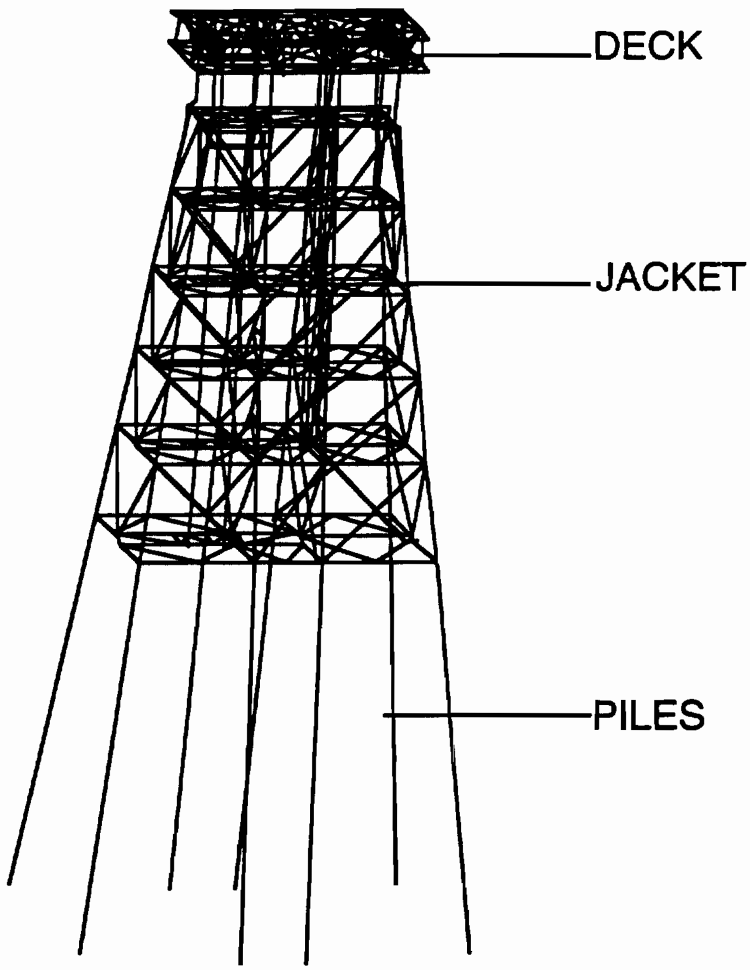
1.1.2 OPERATIONAL INFORMATION

The facilities on the platform consist of a 16-man quarters, 2 gas compressors, and equipment to process 7,000 bpd of oil, 20 MMscfd gas, and 17,000 bpd of water. There are presently 14 conductors installed with 11 producing wells: 10 on the platform and 1 from a subsea well. The platform is presently manned with 4 men.

1.1.3 PERTINENT INSPECTION INFORMATION

The platform is in good operational condition. A Level II inspection of the platform was recently completed: no significant structural damage was found, and the cathodic protection system was functioning properly.

Platform Perspective View

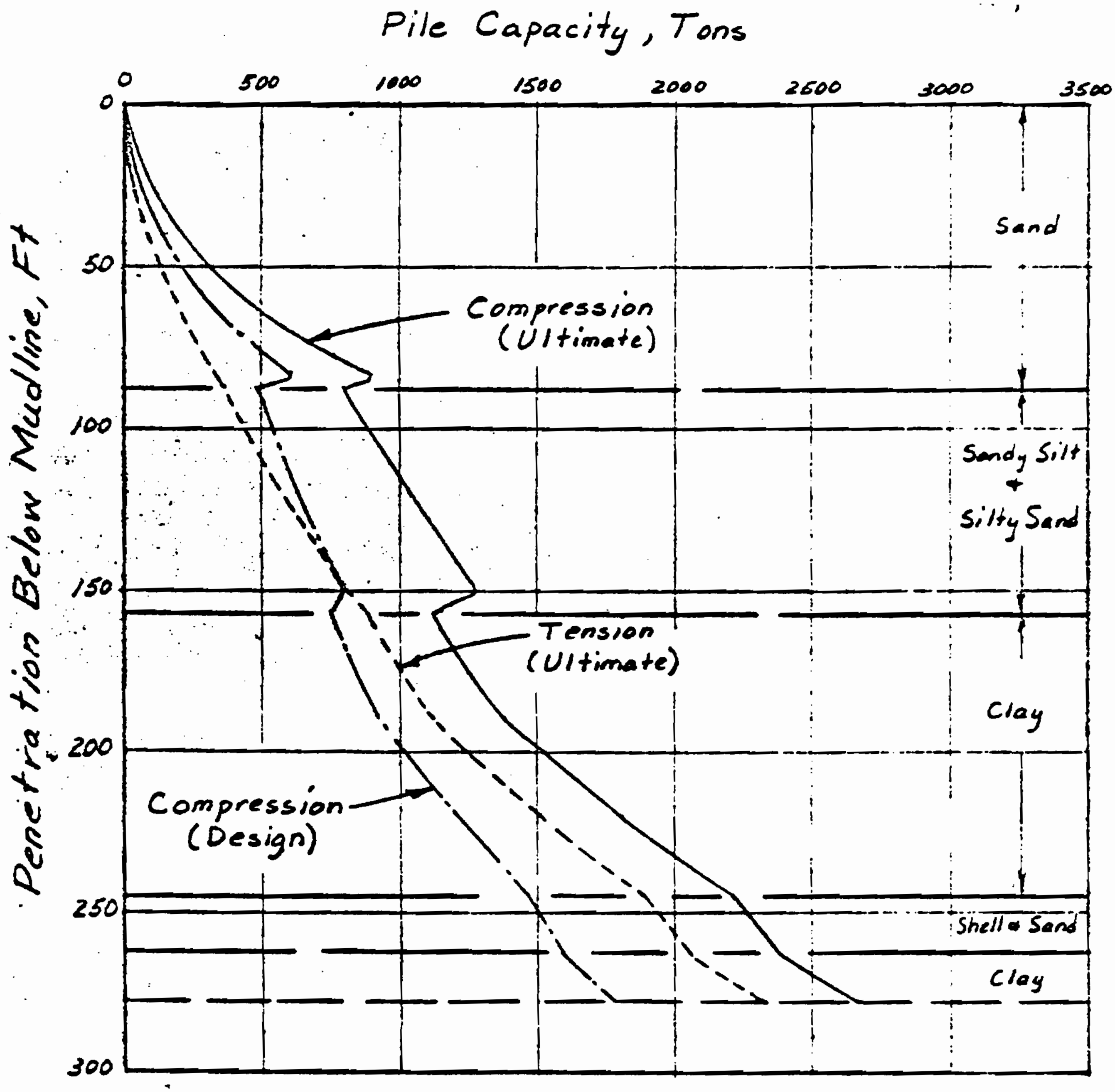


1.1.4 STRUCTURAL ASSESSMENT DATA

to evaluate the strength of the platform due to criteria outlined in API RP 2A-WSD, 20th Edition, Section 17.0 (draft version). Elf provided a full set of drawings describing the platform as it was built and updates done to the platform.

1.1.5 SOIL BORING AND SHEAR STRENGTH PROFILE

provided McClelland Engineering's soil report, dated November 15, 1968. The soil report includes P-Y curves and skin friction and end bearing information. In addition, a log of boring and test results were provided.

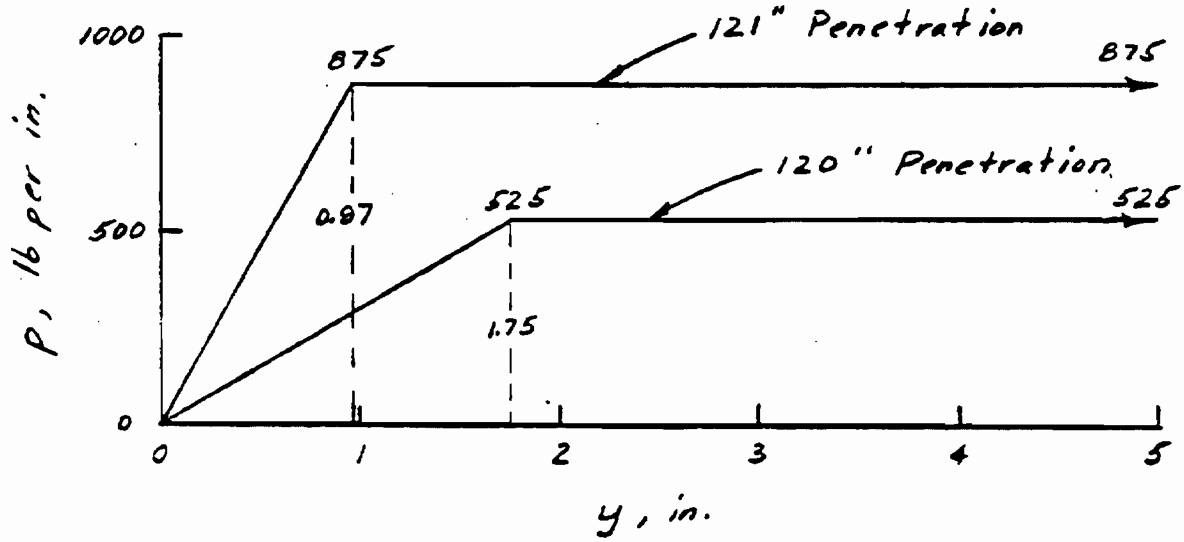


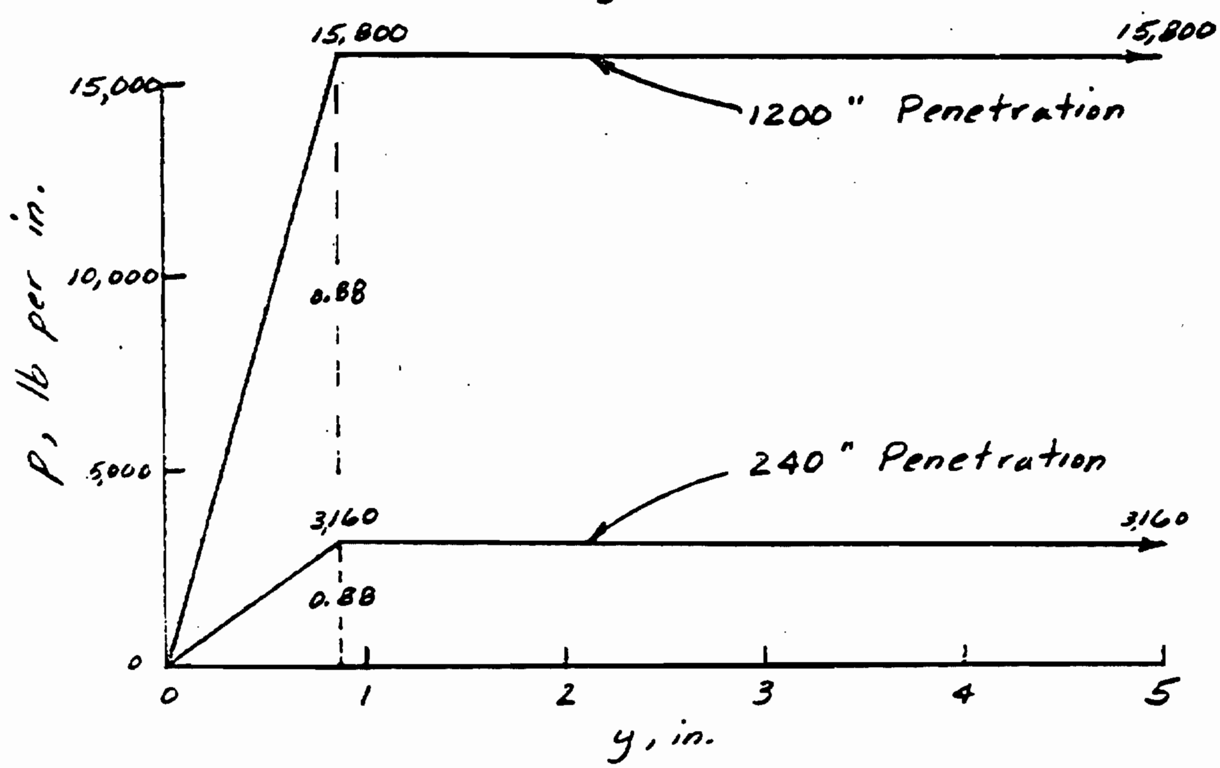
PILE CAPACITY CURVES

42-in. Pipe Piles

At mudline, p = zero for all deflections, y

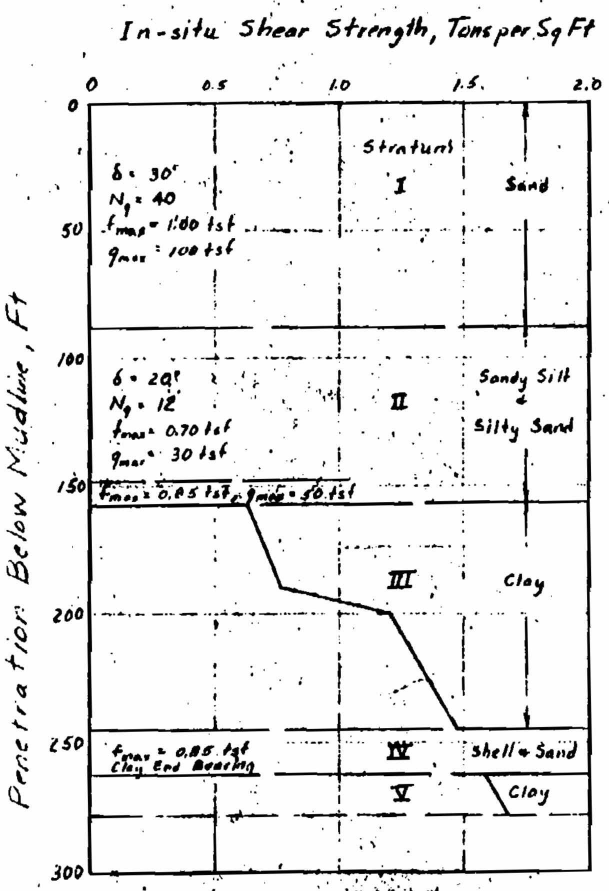
Assuming 5 ft scour, p = zero for all deflections, y, at 60" penetration.



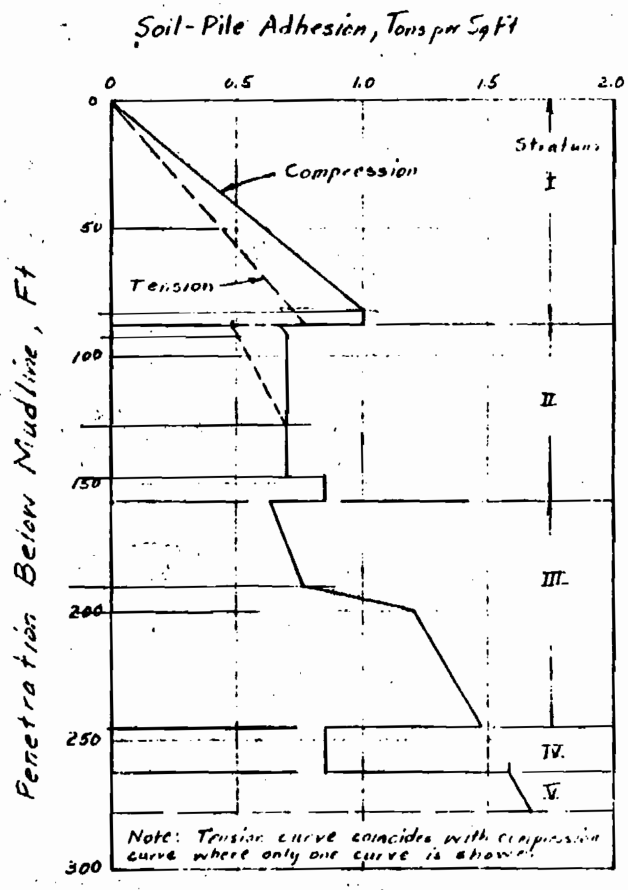


Note: All penetrations measured from original mudline.

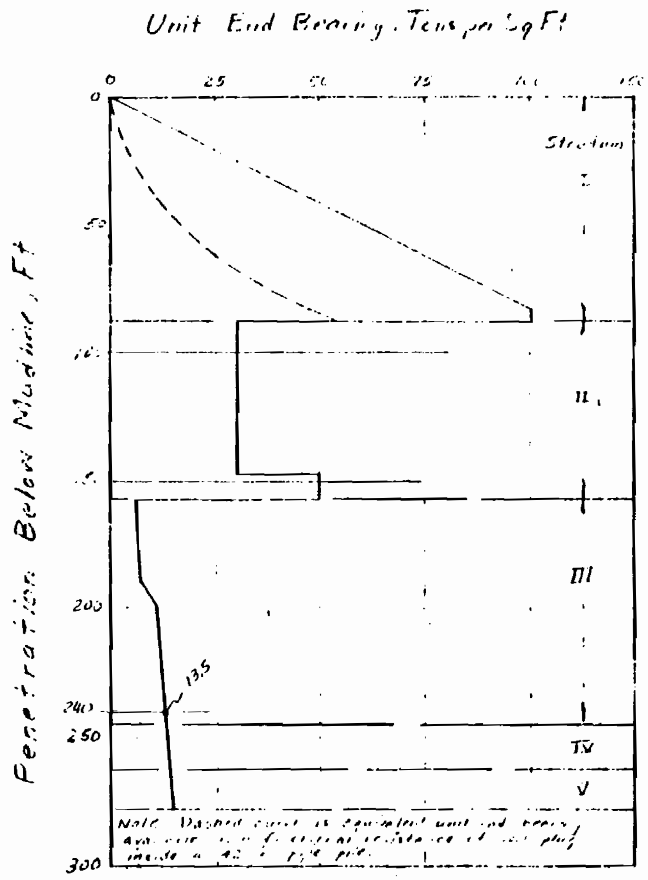
P-Y CURVES
42-in. Pipe Piles



STRENGTH PARAMETERS



SOIL-PILE ADHESION



UNIT END BEARING

INTERPRETATION OF DATA

Form 108-11A Job No. 65-209

25 50 75 100 0.2 0.4 0.6 0.8 1.0 1.2 1.4

0.1 0.2 0.3

Main Pass Area
Block 293

Mudline at El −297

Depth (ft)	Description	Blow Count*
0	Gray fine sand	7
	– very soft gray clay, slightly sandy, loose	15
5	– w/shell fragments & clay pockets, 0.5'-5'	25/21
		30
	– w/clayey sand pockets, 6'-12'	22
	– light gray, 6'-12'	
10		22
	– tan [illegible]	30/12
	– fine-to-medium at 13'	30/12
15		30/10
		30/12
20	– trace of shell fragments, 18'-22'	30/11
		30/10
25		30/10
		30/10
		30
30	– w/clay seams & layers, 30'-32'	30
		30/12
		30/6
35		30/6
		30/6
40		

* [illegible] per foot [illegible] hole on sampling [illegible] [illegible] continuation of boring [illegible] penetration

PLASTIC LIMIT — WATER CONTENT, % — LIQUID LIMIT

20 40 60 80

Non-Plastic

COHESIVE SHEAR STRENGTH
TONS PER SQ FT

0.1 0.2 0.3 0.4 0.5 0.6 0.7

PENETRATION BELOW MUDLINE IN FEET

0 5 10 15 20 25 30 35 40

LEGEND FOR SHEAR STRENGTH PLOT
● Unconfined Compression ▲ Unconsolidated-Undrained Triaxial
◆ Miniature Vane (motorized) ■ Consolidated-Undrained Triaxial
(Open symbols designate remolded tests)

LOG OF BORING AND TEST RESULTS

McCLELLAND ENGINEERS

PLATE

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth

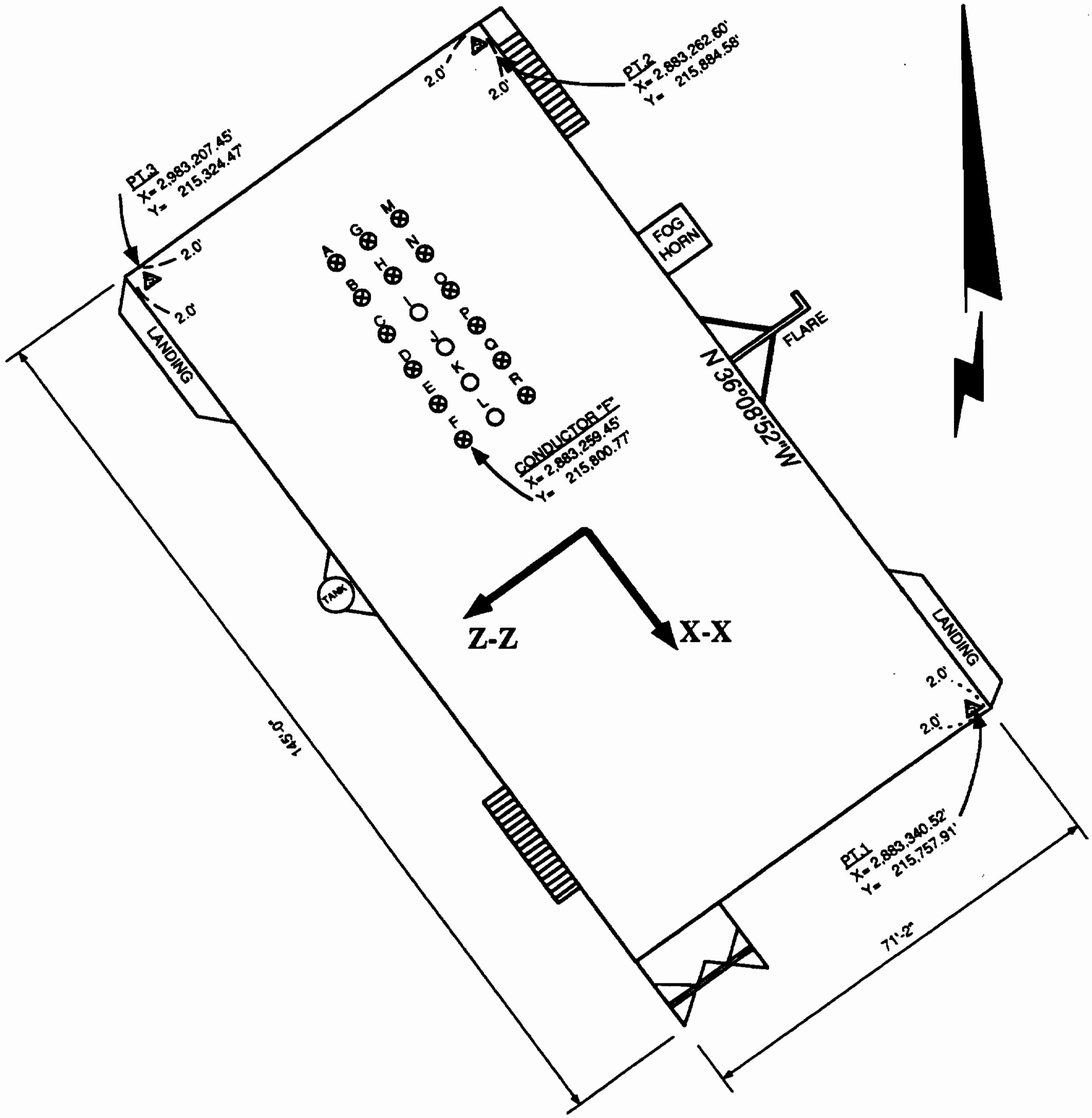
TRIAL DOCUMENT

1.0 PLATFORM INFORMATION

1.2 Platform Sketches

1.2.1 PLATFORM ORIENTATION

The platform is located at Main Pass Block 293, offshore Louisiana in the Gulf of Mexico. The platform orientation is N 36° 09' 52" W. The origin of the global structural system is located at the center of the bottom horizontal level. That is, the X-axis is along the platform's longitudinal direction, the Z-axis is along the platform's transverse direction, and the Y-axis is vertically upward with origin at the mudline. See Figure 1.2.1 for a graphical representation of the platform orientation and coordinate system convention.



NOTE:

POI NTS 1,2 & 3 ARE CHISLED ▲
⊗ DENOTES CONDUCTOR

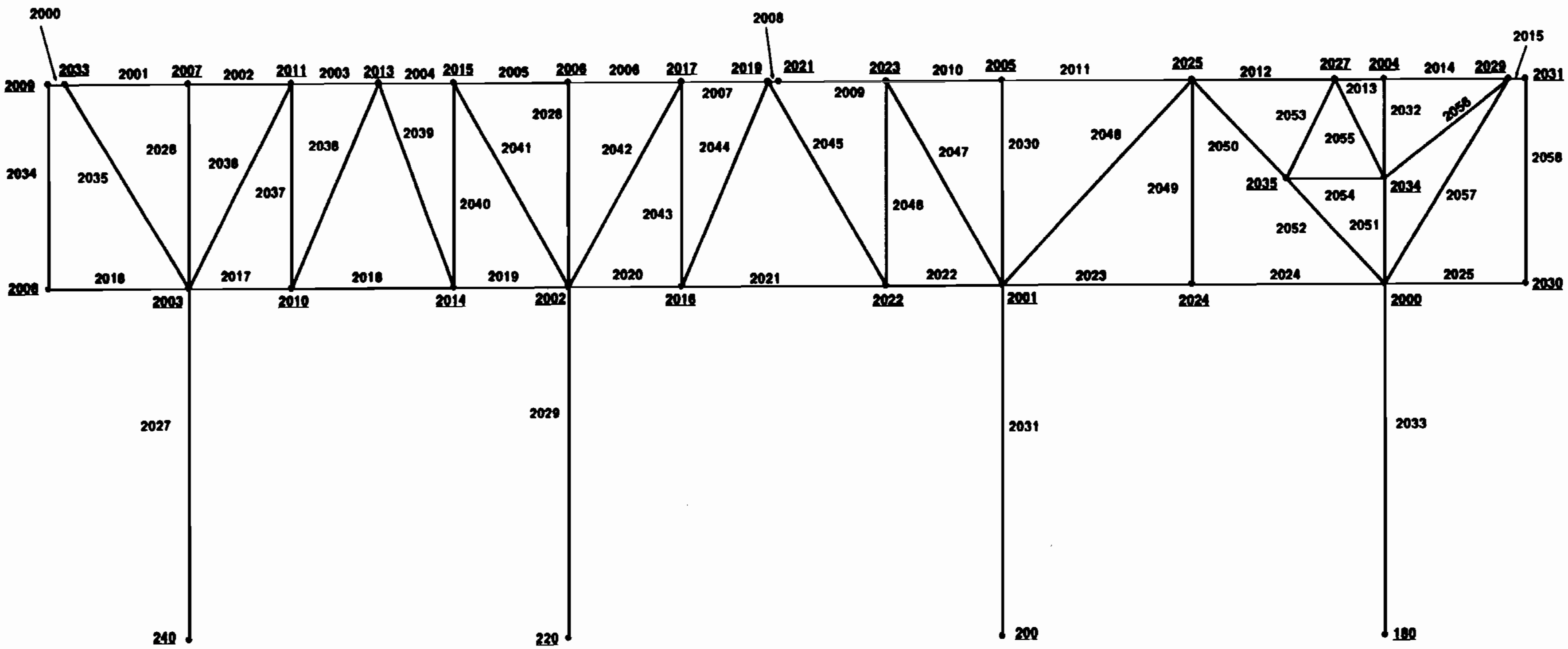
FIGURE 1.2.1

MAIN PASS AREA
BLOCK 293 STR."A"

1.0 PLATFORM INFORMATION
1.2 Platform Sketches

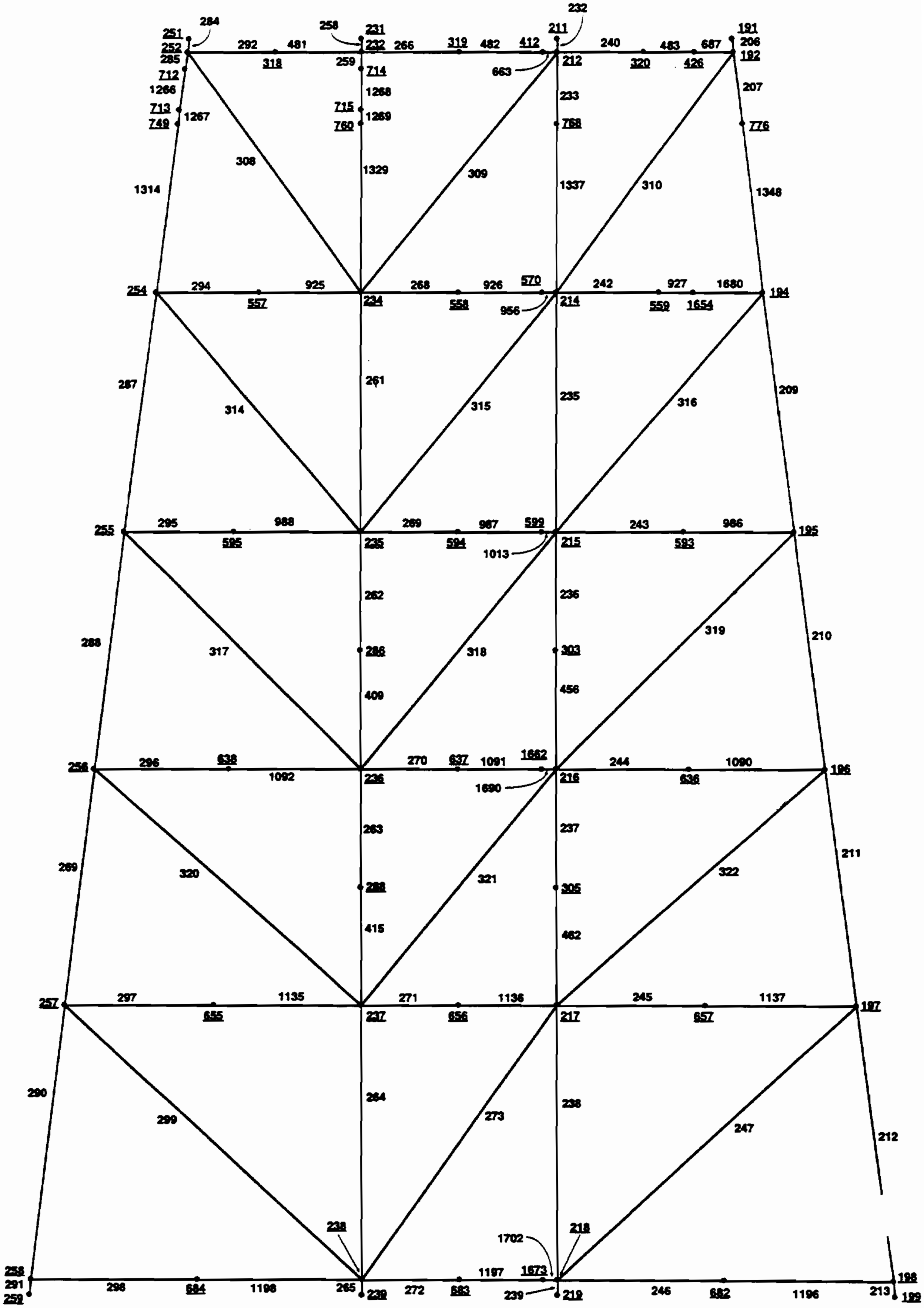
1.2.2 VERTICAL FRAMING ELEVATIONS

The structural vertical framing system consists of 2 longitudinal frames (Row A and Row B) and four transverse frames (Row 1, Row 2, Row 3, and Row 4). The sketches included in this section are separated into vertical elevations for the deck and jacket. Each sketch shows member and joint numbers for the computer model used for the analyses. Members are labeled in plain text; joints are shown underlined text.



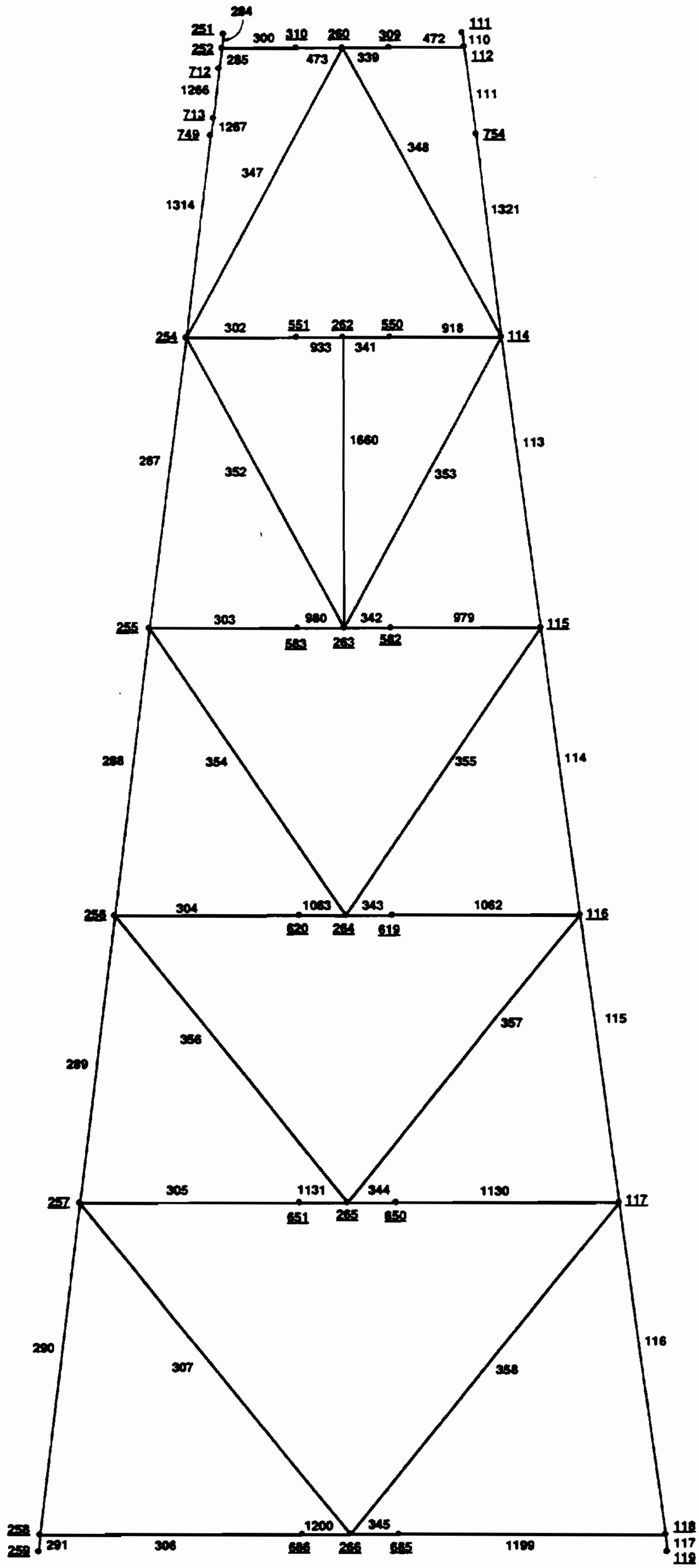
DECK
ROW A

4/30/94

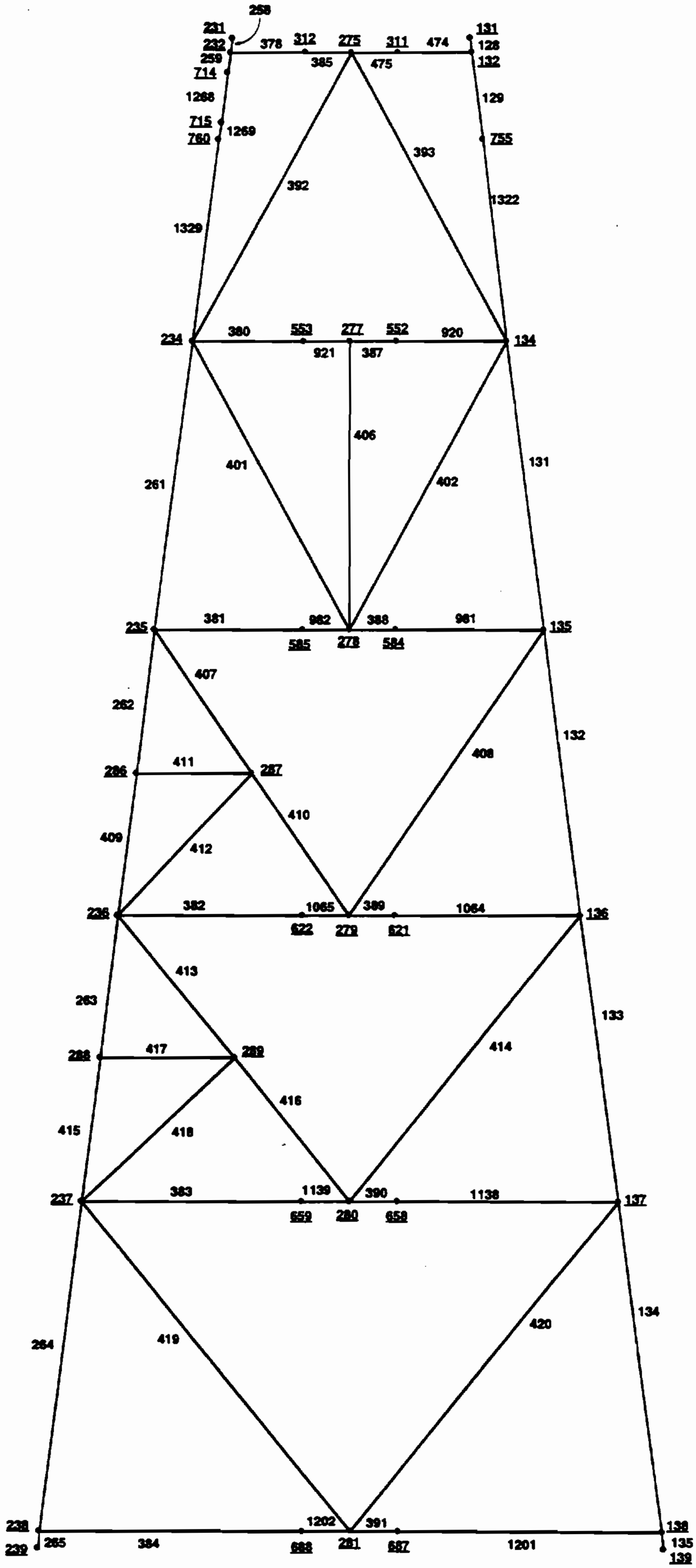


284 258 231 232 211 191
251 252 292 481 232 266 319 482 412 212 240 483 687 206 192
285 318 259 714 663 320 426
712 1266 1268 233 207
713 1267 715 1269
749 760 768 776
308 1329 309 1337 310 1348
1314
254 294 925 234 268 926 570 214 242 927 1680 194
557 558 956 559 1654
287 261 235 209
314 315 316
255 295 988 235 269 987 599 215 243 986 195
595 594 1013 593
262 236
288 317 286 318 303 319 210
409 456
256 296 638 1092 236 270 637 1091 1662 216 244 1090 196
1690 636
263 237
289 320 288 321 305 322 211
415 462
257 297 1135 237 271 1136 217 245 1137 197
655 656 657
290 299 264 273 238 247 212
238 218 1702
258 1197 1673 198
291 298 684 1198 265 239 272 683 239 219 246 682 1196 213 199
259

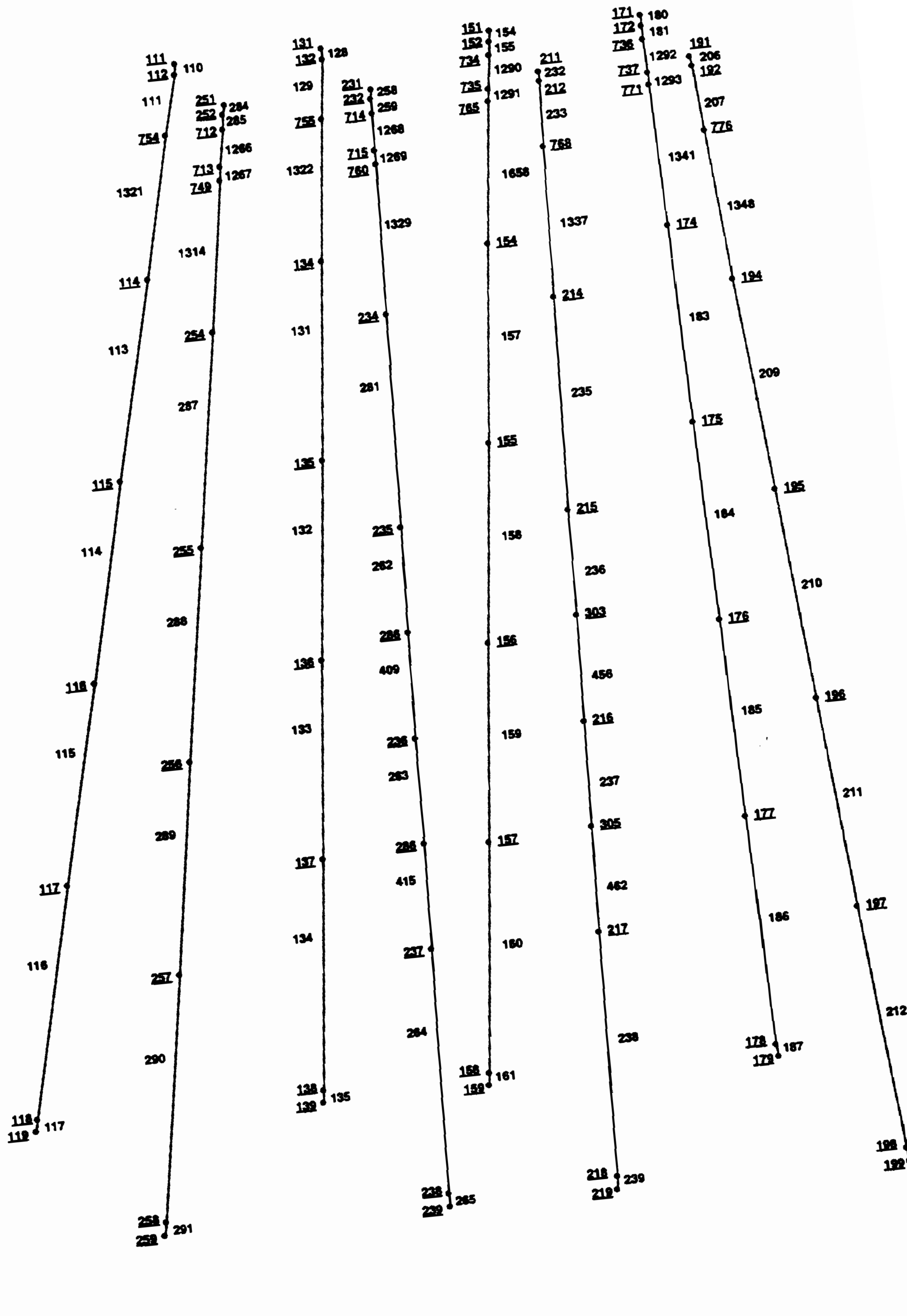
JACKET
ROW A


284
251
252
300
310
260
309
472
111
110
112
285
473
339
712
1266
111
713
1267
749
754
348
347
1314
1321
254
302
551
262
550
918
114
933
341
1660
287
352
353
113
255
303
980
342
979
115
583
263
582
288
354
355
114
256
304
1063
343
1062
116
620
264
619
289
356
357
115
257
305
1131
344
1130
117
651
265
650
290
307
358
116
258
1200
345
118
259
291
306
686
266
685
1199
117
119

JACKET
ROW 1


258
231
232
259
714
378
312
275
385
311
475
474
131
128
132
1268
715
1269
760
129
755
392
393
1322
1329
234
380
553
277
552
920
134
921
387
406
261
401
402
131
235
381
982
278
388
981
135
585
584
407
262
132
286
411
287
408
410
409
412
236
382
1065
279
389
1064
136
622
621
413
263
133
288
417
289
414
416
415
418
237
383
1139
280
390
1138
137
659
658
264
419
420
134
238
1202
281
391
136
239
265
384
688
687
1201
135
139

JACKET
ROW 2



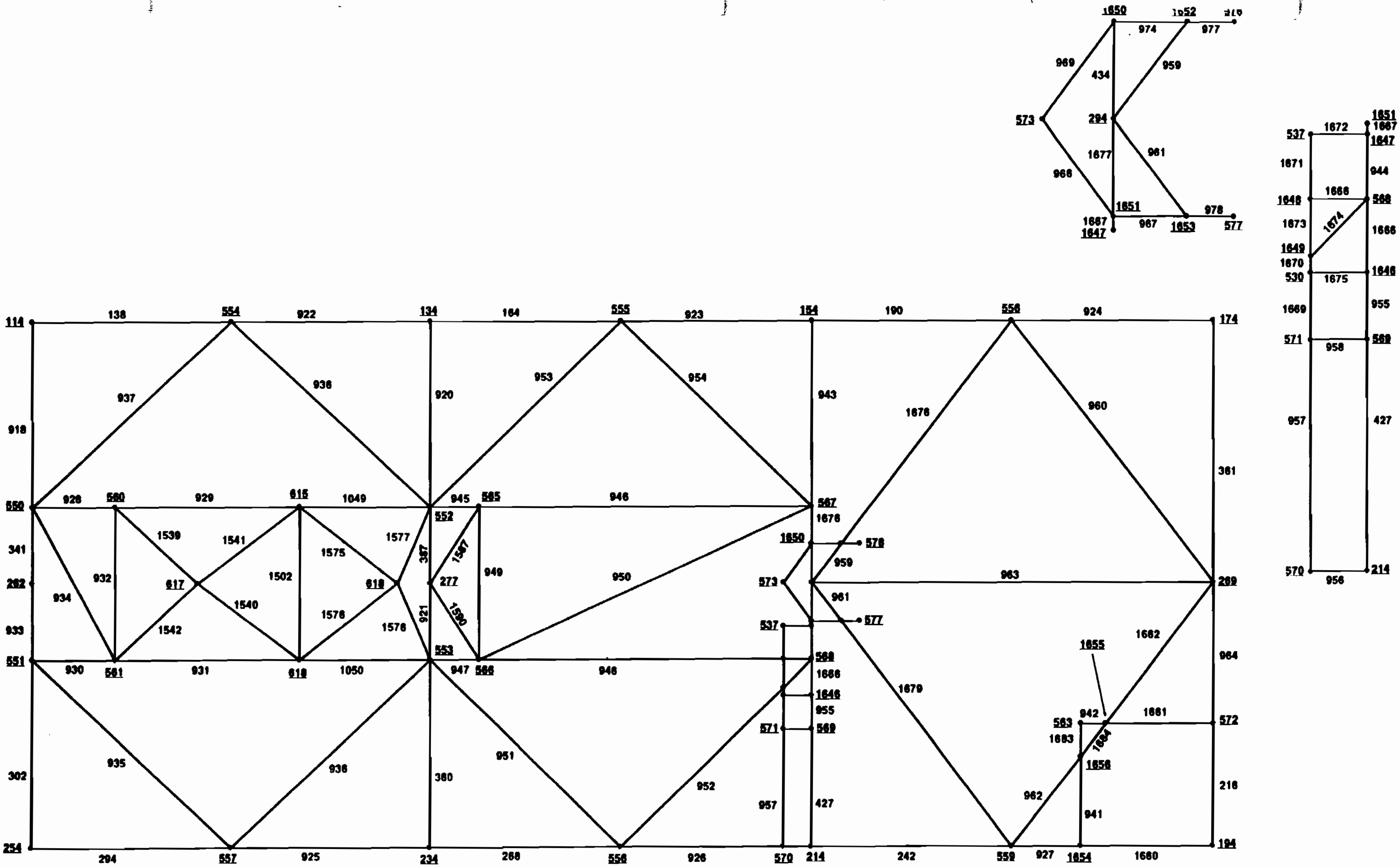
JACKET LEGS

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

1.0 PLATFORM INFORMATION
1.2 Platform Sketches

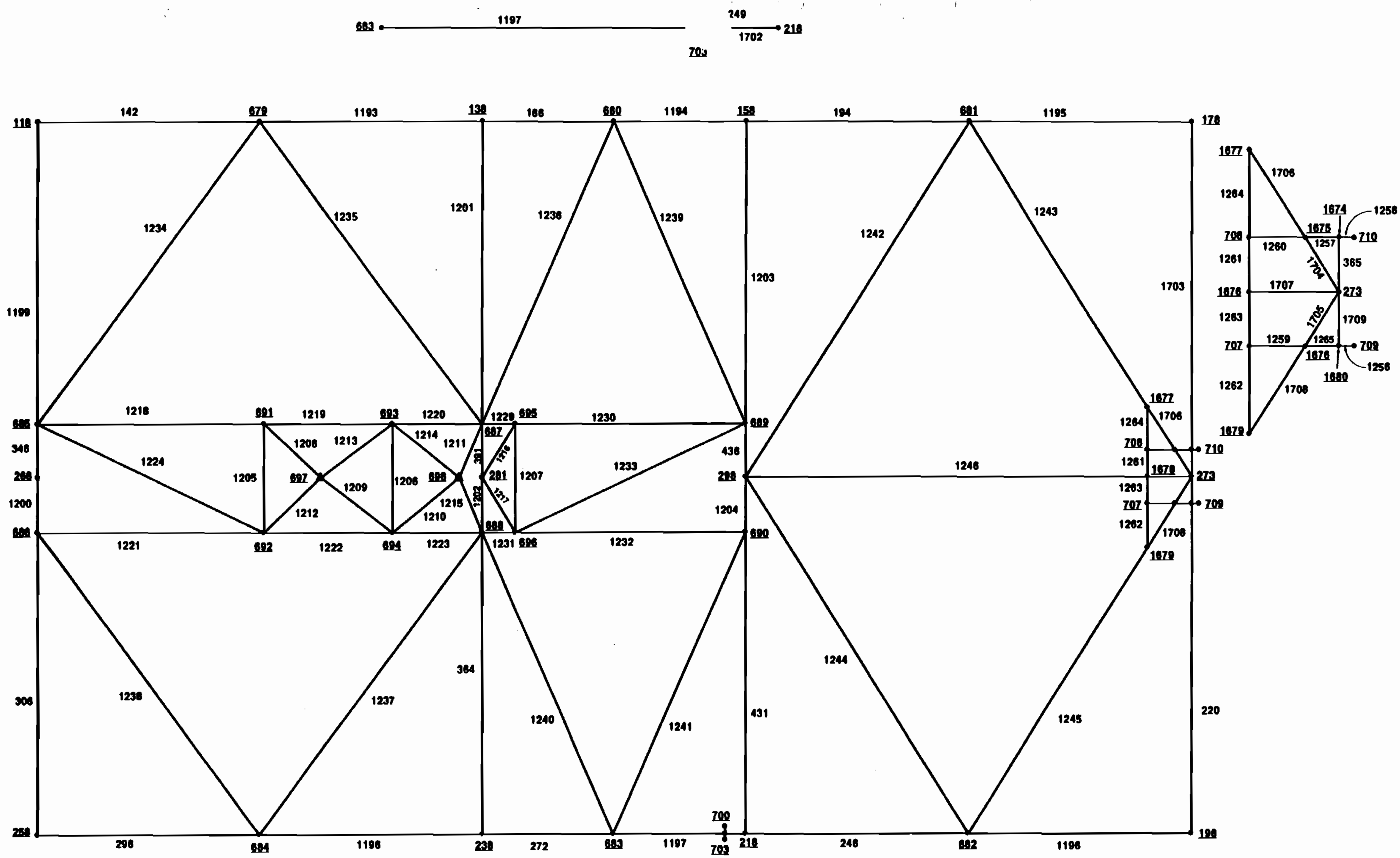
1.2.3 HORIZONTAL FRAMING PLANS

The structural horizontal framing system consists of 2 deck levels and 6 horizontal jacket levels. The deck horizontal levels are located at El (+)47'-6" (the drilling deck level) and at El (+)65'-11" (the production deck level). The jacket horizontal levels are located at El (+)10', El (-)40', El (-)90', El (-)139'-7-3/8", El (-)189'-2-3/4" and El (-)247'-0". The sketches included in this section are separated into horizontal elevations for the deck and jacket. Each sketch shows member and joint numbers for the computer model used in the analyses. Members are labeled in plain text; joints are shown underlined.



EL (-) 40' - 0''

4/30/94



EL (-) 247' - 0"

4/30/94

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

1.0 PLATFORM INFORMATION
1.2 Platform Sketches

1.2.4 PILE MAKE-UP AND DETAILS

There are 2 types of pile make-ups defining the configuration of the 8 main piles. A Type A pile-test pile assigned to Pile A4 and a Type B pile assigned to the remaining piles (A1, A2, A3, B1, B2, B3, and B4). Both piles are similar above the mudline. Below the mudline, the piles differ in wall thickness segment lengths.

- **Test Pile (Type A)**

Pile diameter:................................ 42" Ø
Pile penetration:...............................240'
Pile material:............... 36 Ksi (all segments)
Pile make-up:...............From below mudline

Wall thickness (inches)	Segment length (feet)	
1.375	80.0	mudline level
1.125	40.0	
1.000	20.0	
0.875	95.0	
1.250	5.0	

- **Test Pile (Type B)**

Pile diameter:................................ 42" Ø
Pile penetration:...............................240'
Pile material:............... 36 Ksi (all segments)
Pile make-up:................from below mudline

Wall thickness (inches)	Segment length (feet)	
1.375	50.0	mudline level
1.125	40.0	
1.000	20.0	
0.875	125.0	
1.250	5.0	

The sketches included in this section are separated into the pile members located above mudline (sketch labeled "PILES") and below mudline (sketch labeled "PILES BELOW MUD"). Each sketch shows member and joint numbers for the computer model used in the analyses. Members are labeled in plain text; joints are shown underlined.

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

1.0 PLATFORM INFORMATION
1.2 Platform Sketches

1.2.5 OTHER INFORMATION

This section contains sketches for members other than the steel considered main steel: that is, conductors (2 conductors are modeled to account for the existing 14 conductors), J-tubes, casings, barge bumpers, and boat landings. Each sketch shows member and joint numbers for the computer model used in the analyses. Members are labeled in plain text; joints are shown underlined.

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
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PART A: PLATFORM ASSESSMENT
A.1 - A.4

A.1 PLATFORM SELECTION

Based on platform assessment initiators of API RP 2A-WSD, 20th Edition, Section 17.0 (draft version), this platform is not subject to the assessment process; it was selected, however, for the following reasons:

- The platform has been in production for 25 years, and the potential for an additional 10+ years of production exists.
- Tentative development plans call for additional sidetracks and workovers of existing wells.
- The general age of the platform.

A.2 CONDITION ASSESSMENT

A Level II survey was completed this summer on the platform. The results show that there is no significant structural damage to the platform and that the sacrificial anodes are providing adequate cathodic protection.

A.3 CATEGORIZATION

The platform is categorized as manned, evacuated with an insignificant environmental impact.

A.4 DESIGN BASIS CHECKS

___ 8-pile platform located at Main Pass Block 293 was designed in 1968 and installed in 1969. The API RP 2A 9th Edition was issued in November, 1977. Therefore, we concluded that analysis checks should be performed for this platform.

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
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PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks (A.5.1 - A.5.2)

A.5.1 METOCEAN CRITERIA / LOADS

The following information describes the environmental conditions considered in the analysis. All wave, current, and wind information is based on Table 17.6.2-1 of API RP 2A-WSD, 20th Edition, Section 17.0 (draft version). As stated in Section 17.6.2, the metocean criteria is based on the 100-year force due to sudden hurricane. For the design level analysis, omni-directional criteria is utilized in which the in-line current is assumed to be acting in the same direction as the wave. For the ultimate strength analysis, the directionality of the waves and currents is taken into account.

Gulf Of Mexico Metocean Criteria
Insignificant Environmental Impact / Manned-Evacuated

Criteria	Design Level Analysis	Ultimate Strength Analysis
Wave height & storm tide, ft	46.9	59.4
Deck height, ft	36.3	36.3
Wave & current direction	**omni-dir	Fig. 17.6.2-4
Current speed, kts	1.2	1.8
Wave period, sec	11.3	12.5
Wind speed (1hr @ 10 m), kts	55.0	70.0

** If the wave height or current vs. direction exceeds that required for ultimate strength analysis, then the ultimate strength criteria will govern.

A.5.2 SCREENING

In accordance with Section 17.5, the screening of platforms to determine if detailed analysis is to be performed depends upon the results from assessing the first 4 components of the assessment process (i.e., platform selection, categorization, condition assessment, and design basis checks). The results of assessing the 4 components are outlined in Section A.1 through Section A.4 of this report. These results indicate that detailed analysis should be performed to determine if the platform is capable of resisting the environmental conditions as specified in API RP 2A-WSD, 20th Edition, Section 17.0 (draft version).

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.3 Design-Level Analysis

A.5.3.1 INTRODUCTION

This section contains the results for the design level analysis. The design level analysis procedure follows the recommendations specified in API RP 2A-WSD, 20th Edition, Section 17.0 (draft version). The analysis utilizes the working strength approach which accounts for the elastic properties of the members composing the platform. The analysis is designed to predict the platform's response to the operational and environmental conditions to which it will be subjected throughout its remaining life. These conditions include the occurrence of site hurricanes and winter storms, loading produced by platform equipment, and loading produced by the platform's self-weight. See Figure A.5.1 for a perspective view of the computer model used in the design level analysis.

To complete the platform's design level analysis, the following steps were completed:

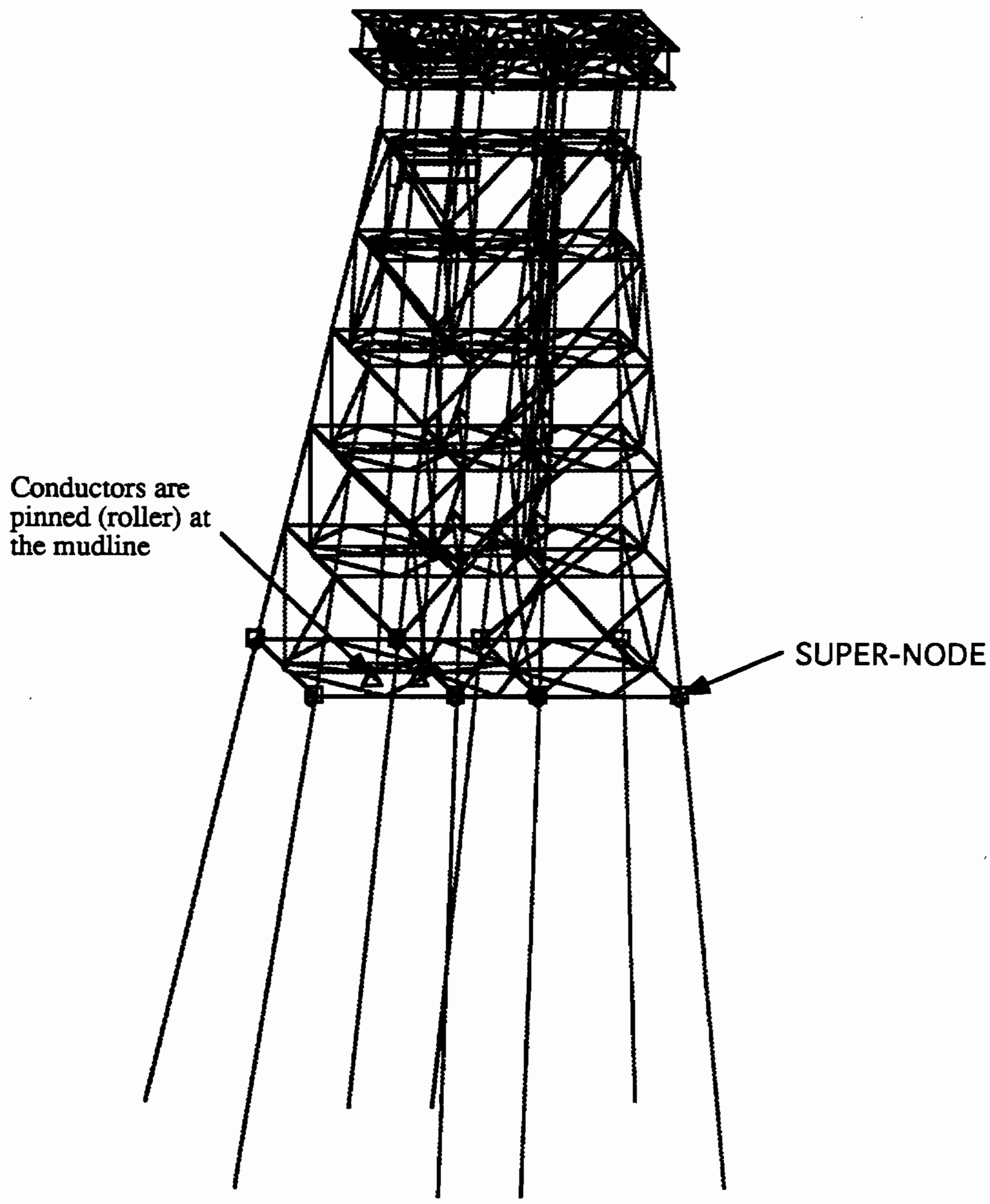
1. Development of platform model
2. Development (and application) of platform loads
3. Characterization of soil properties
4. Development (and condensation) of stiffness matrix
5. Nonlinear soil/pile/structure foundation analysis
6. Displacement back-substitution (calculation of member loads)
7. Member and joint checks.

This section is followed by a description of the metocean criteria/loads and analysis results.

Analysis Assumptions

The analysis was completed assuming the 14 conductors in operation are pinned at the mudline. Therefore, the piles and the soil system are the elements responsible for bearing equipment weights, the platform's self weight, and storm loads. We modeled 2 conductors to simulate the 14 conductors. The lateral load (wave) transfer from the conductors to the horizontal levels was achieved assuring that the connection between horizontal levels and conductor can only distribute lateral loads. The vertical load due to the conductor weight was assumed to be supported by the conductor.

DESIGN LEVEL ANALYSIS
ANALYTICAL MODEL



PERSPECTIVE VIEW

FIGURE A.5.1

A.5.3.2 METOCEAN CRITERIA

The criteria for the design level analysis is based on the 100-year force due to combined sudden hurricane and winter storm population. The corresponding wave force is applied using the omni-directional criteria with the associated in-line current acting constant for all directions. For some noncritical directions, the omni-directional criteria exceed the ultimate strength analysis values, in which case the ultimate strength analysis values will govern the design for those directions. See Table A.5.1 (Wave Base Shear Comparison) and Figure A.5.3.

A.5.3.3 LATERAL LOAD LEVEL

This section describes the loads used for the design level analysis. These are broken down as follows:

- **Gravity load**: includes weights of all modeled members.
- **Live, dead, equipment, and open area deck loads**: includes deck loadings during operating conditions.
- **Miscellaneous jacket dead loads**: accounts for non-modeled jacket dead loads, such as anode weights, conductor guide weights, barge bumper/boat landing weights, mudmat weights, etc.
- **Buoyancy**: accounts for the buoyancies of all modeled members.
- **Miscellaneous buoyancies**: accounts for the buoyancies of non-modeled elements, such as anode weights, conductor guide weights, barge bumper/boat landing weights, mudmat weights, etc.
- **Storm wave / current loads**: describe wave/current storm loadings, as defined by API RP2A-WSD, 20th Edition, Section 17.0 (draft version), Table 17.6.2-1 (Insignificant Environmental Impact/Manned-Evacuated). For the design level analysis, the wave height (H_w) is constant for all directions. For this study the $H_w = 46.9'$ is used. The current direction is in-line with the wave directions. Typically, designs use 8 wave approach directions to compute the lateral wave forces applied to the platform. This study has selected 11 wave directions (0, 45, 90, 135, 169, 180, 214, 225, 244, 270, and 315 degrees). See Figure A.5.2 (Wave Direction) for a description of the conventional and wave directional approach with respect to the platform. The wave direction of 169 degrees corresponds to the worst wave direction (wave height factor = 1.0) for the platform classified as insignificant environmental impact/manned-evacuated case. The wave direction of 214 degrees corresponds to the worst wave direction (wave height factor = 1.0) in API RP 2A, 20th edition. The reason for including the 214-degree wave is that in the pushover analysis, the reserve strength ratio is defined in reference to API RP 2A, 20th edition, guidelines. The wave direction of 244 degrees corresponds to the case where the current is in-line with the wave for the insignificant environmental impact/manned-evacuated.

TABLE A.5.1

WAVE BASE SHEAR COMPARISON
INSIGNIFICANT ENVIRONMENTAL IMPACT
AND API 20TH (NEW DESIGN)

WAVE	ULTIMATE STRENGTH		DESIGN LEVEL ANALYSIS		SELECTED	API 20TH (New Design)	
ANGLE (DEG)	WAVE HEIGHT (FT)	SHEAR (K)	WAVE HEIGHT (FT)	SHEAR (K)	LOADING CRITERIA	WAVE HEIGHT (FT)	SHEAR (K)
0	41.6	1134.7	46.9	1572.2	ULTIMATE	46.9	1481.2
45	43.8	1111.1	46.9	1737.2	ULTIMATE	46.9	1273.9
90	51.9	1819.9	46.9	1863.1	ULTIMATE	49.4	1605.7
135	57.2	2245.9	46.9	1733.8	DESIGN LEVEL	58.6	2390.3
169	59.4	2624.3	46.9	1584.2	DESIGN LEVEL	63.7	3143.5
180	57.9	2594.2	46.9	1556.1	DESIGN LEVEL	64.5	3360.9
214	53.5	2481.5	46.9	1664.1	DESIGN LEVEL	67.0	4044.3
225	51.3	2351.6	46.9	1728.6	DESIGN LEVEL	65.4	3947.1
244	47.5	2120.2	46.9	1803.0	DESIGN LEVEL	62.5	3750.2
270	43.8	1840.0	46.9	1862.7	ULTIMATE	57.8	3253.7
315	41.6	1389.1	46.9	1743.0	ULTIMATE	49.4	2015.7

NOTES:

1. The conductor shielding factor was interporlated for the varying conductor spacing. Since the shape factors cannot be changed easily for each waveload an constant value of 0.875 will be used.
2. For 0, 45, 90, 270 and 315 wave directions the design level criteria base shear exceeds the ultimate strength criteria base shear. Therefore, the ultimate strength values are selected for those directions. ?
3. The wave forces are calculated using a directional spreading factor of 0.88 since the criteria referenced the sudden hurricane population.

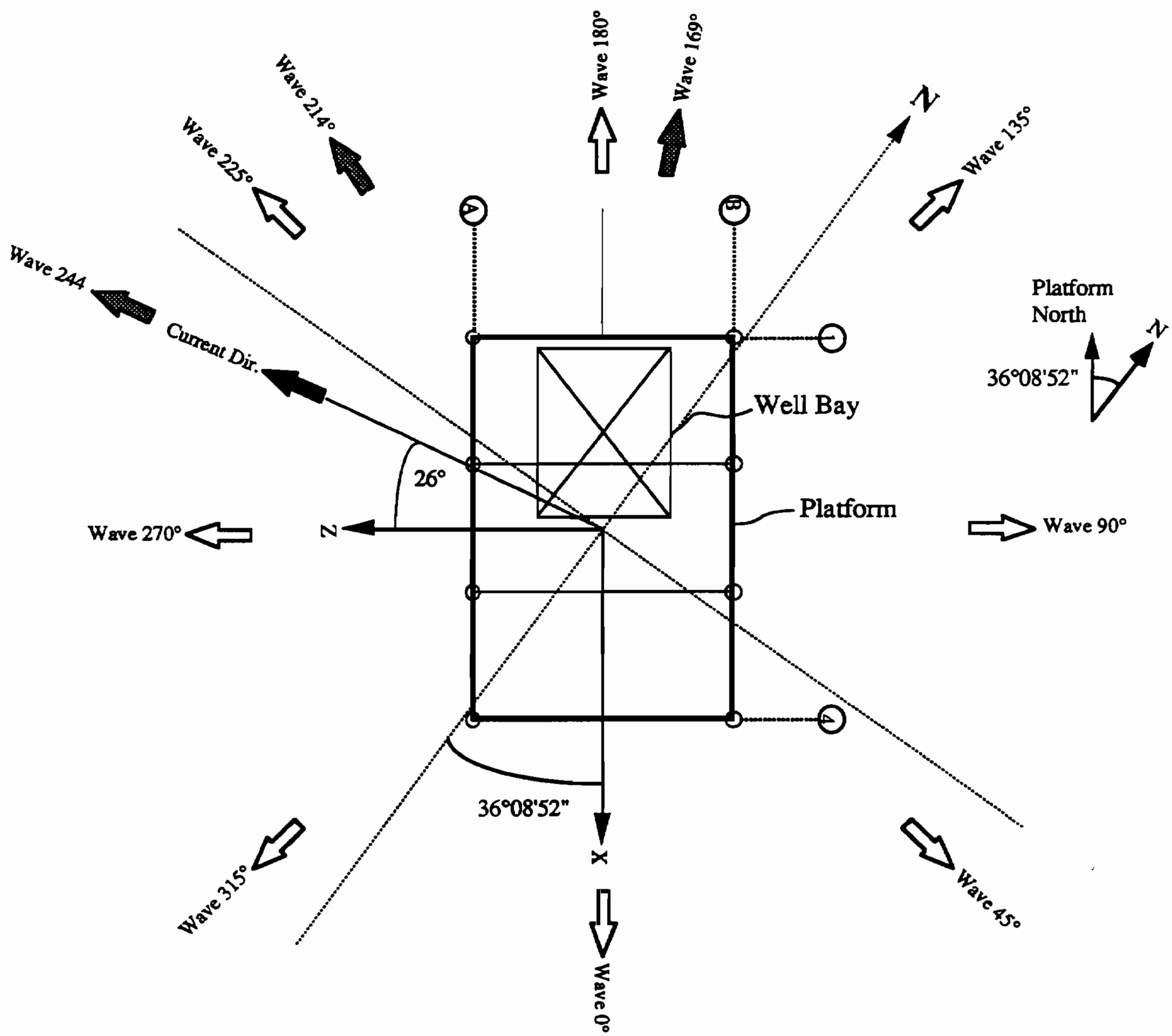


Figure A.5.2 Wave Directions

WAVE BASE SHEAR COMPARISON

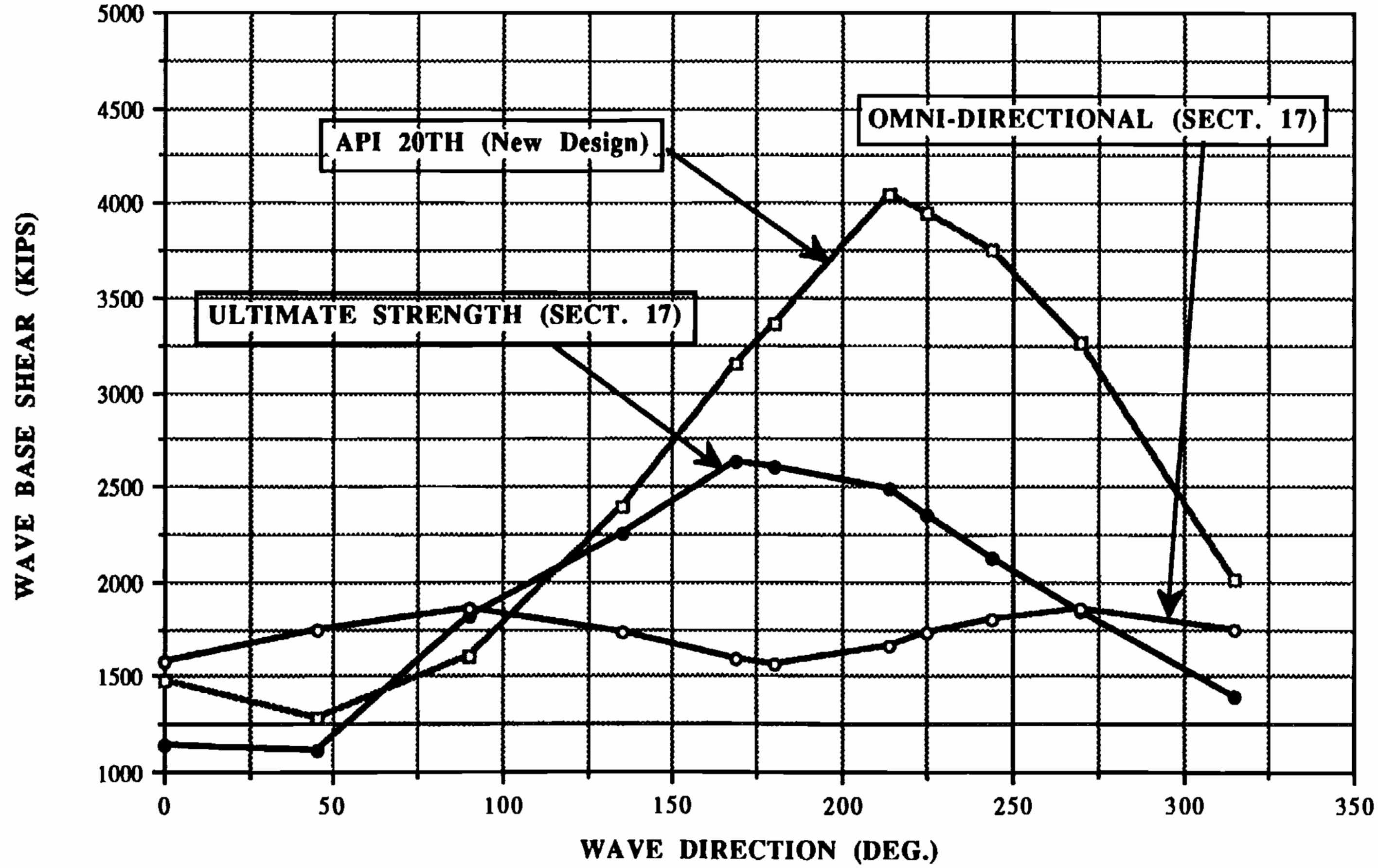


FIGURE A.5.3

The lateral load level due to the wave and current effects are summarized in Table A.5.1 of this report. The maximum lateral load is 1863.1 kips in the 90-degree wave direction. See Figure A.5.3 for a comparison of base shear resulting from different wave directions (design level analysis, ultimate strength analysis, and API RP 2A, 20th edition) and Figure A.5.4 for a comparison of the wave heights.

- Wind loads: describes wind storm loads as defined by API RP2A-WSD. 20th Edition. Section 17.0 (draft version), Table 17.6.2-1 (Insignificant Environmental Impact/Manned-Evacuated).

WAVE HEIGHT COMPARISON

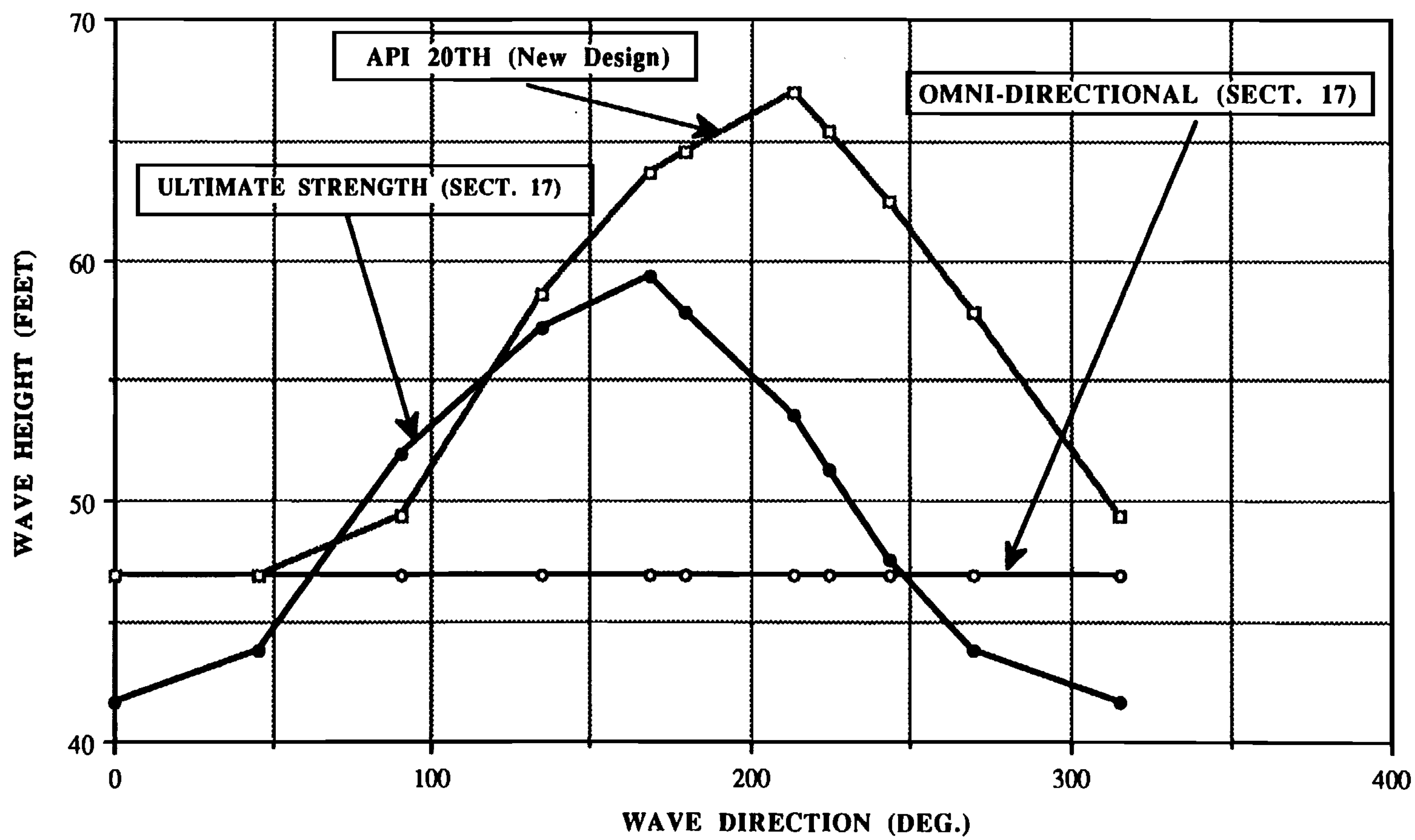


FIGURE A.5.4

Drilling and Production Platform
Main Pass Block 293
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TRIAL DOCUMENT

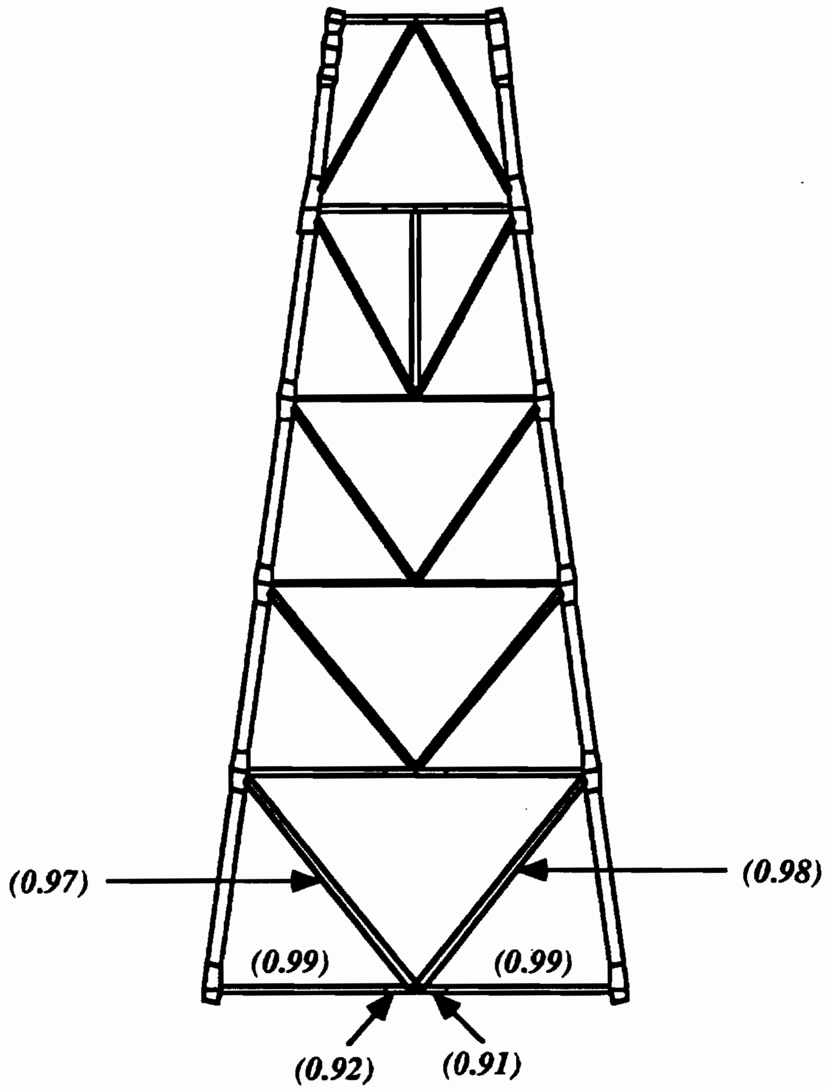
PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.3 Design-Level Analysis

A.5.3.4 MEMBER CHECK RESULTS WITH STRESS RATIOS > 0.85

This section contains sketches depicting members whose stress ratios exceed 0.85 (See Figure A.5.5 through Figure A.5.8). All checks were performed in accordance with API RP 2A, 20th Edition, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms. Note that the results have been divided into stress ratios due to strength check and stress ratios due to hydrostatic loads. In addition, the material yield stress used throughout the analysis is 36 ksi. The supplied platform drawings do not indicate material other than a 36 ksi steel.

The pile strength check results showed that the highest stress ratio is 0.60. This stress ratio occurs at Pile A1, 20 feet below the mudline. The low stress ratio indicates that the piles have sufficient strength and are still in the linear mode.

STRESS RATIO SKETCHES
MEMBER CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 1

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS

TRIAL APPLICATION
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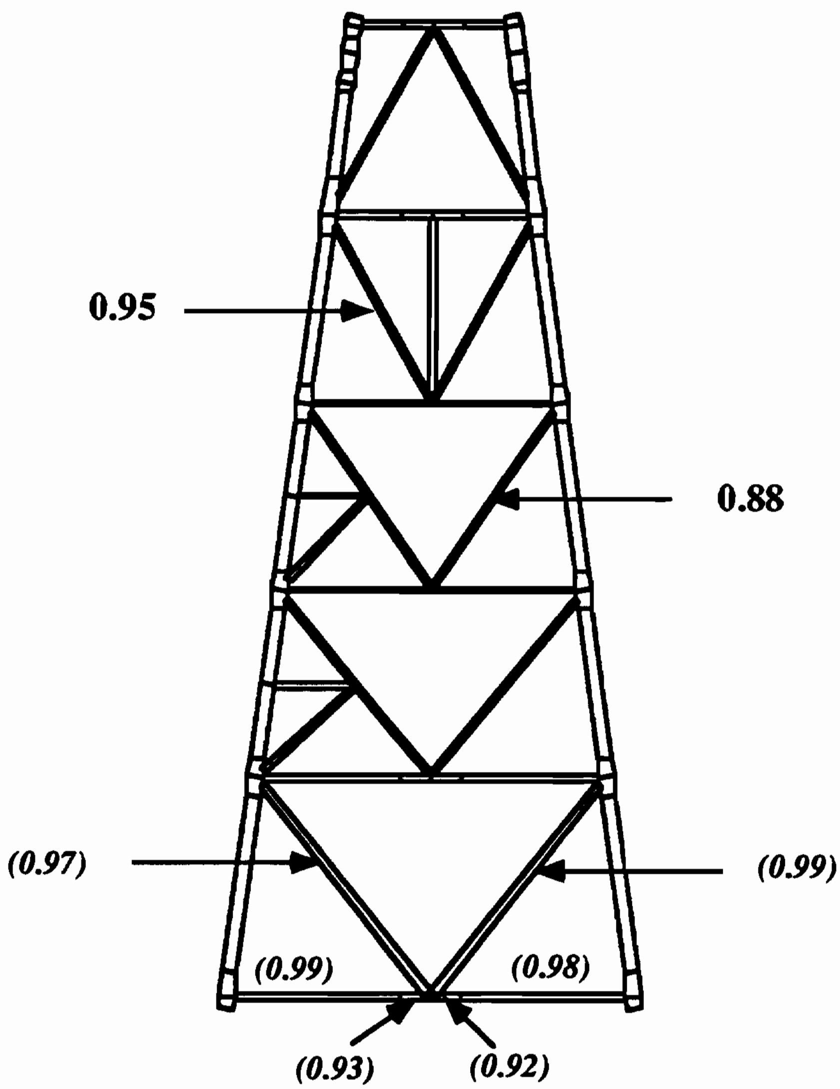
FIGURE A.5.5

- STRESS RATIO SKETCHES
MEMBER CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 2

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS

TRIAL APPLICATION
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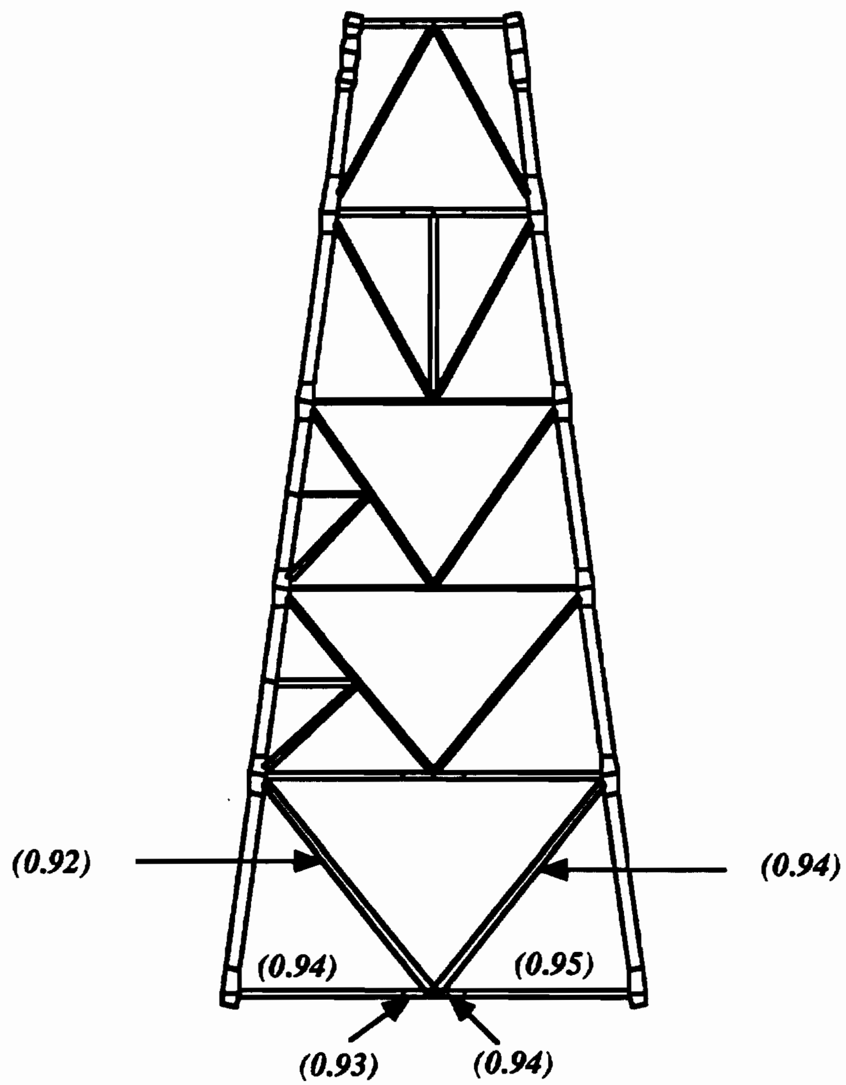
FIGURE A.5.6

STRESS RATIO SKETCHES
MEMBER CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 3

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS

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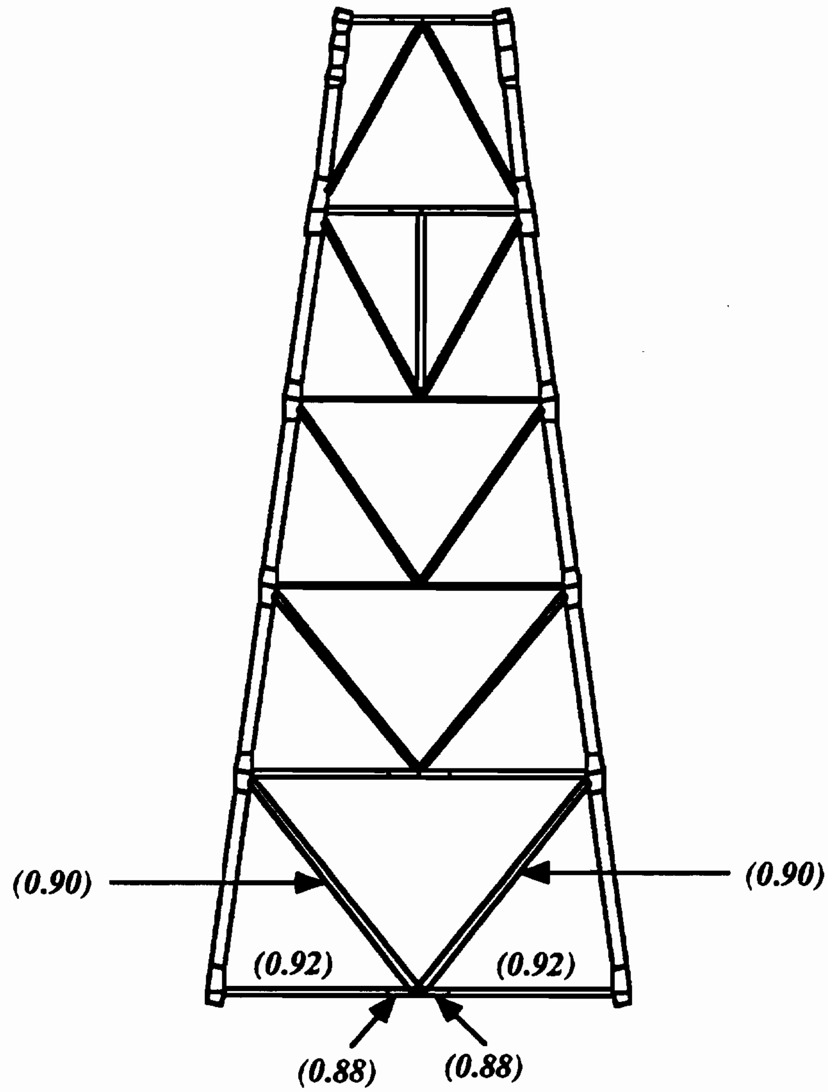
FIGURE A.5.7

- STRESS RATIO SKETCHES
MEMBER CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 4

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS

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FIGURE A.5.8

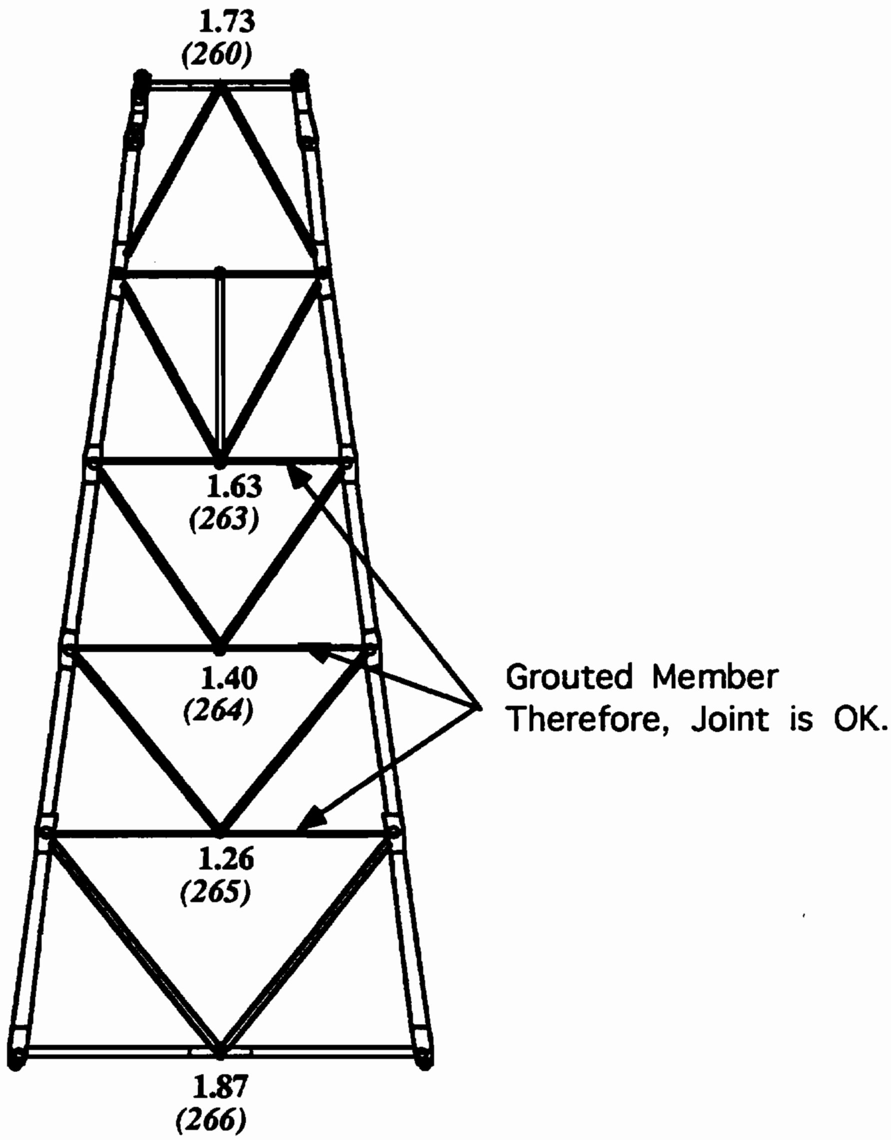
PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.3 Design-Level Analysis

A.5.3.5 NOMINAL JOINT CHECK RESULTS WITH STRESS RATIOS > 0.85

This section contains sketches depicting joints whose stress ratios exceed 0.85 (See Figure A.5.9 through Figure A.5.12). All of the joint with stress ratios greater that 1.0 are K-joints. The reason is because the wall thickness and material of the framing members (K-braces) are the same as the wall thickness and material of the chord section.

All checks were performed in accordance to API RP 2A, 20th Edition, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms. Note that the sketches show only the results due to nominal joint check criteria. The results due to the 50% effectiveness check are disregarded as stated in API RP 2A, 20th Edition, Section 17.7.2c (draft version). In addition, the material yield stress used throughout the analysis is 36 ksi. The supplied platform drawings do not indicate material other than 36 ksi steel.

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 1

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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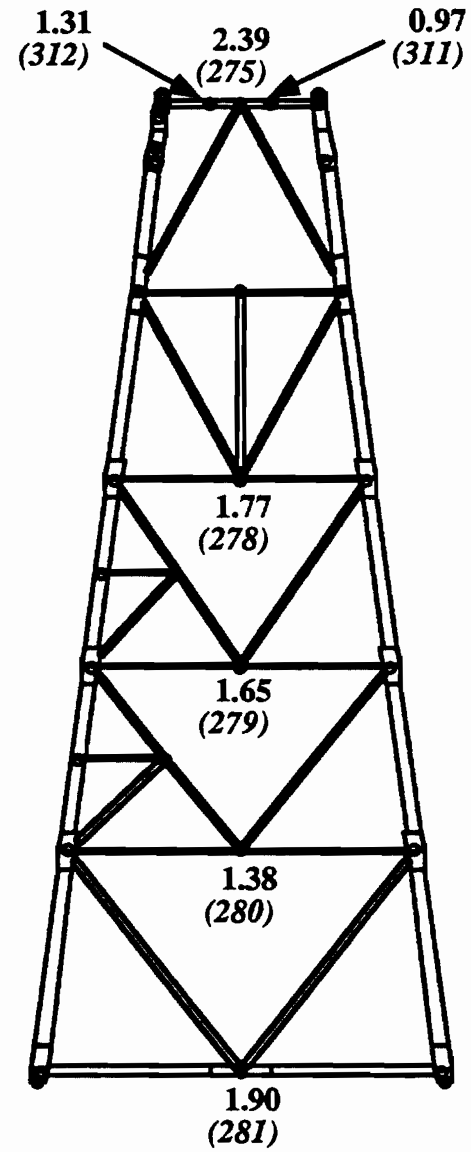
FIGURE A.5.9

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 2

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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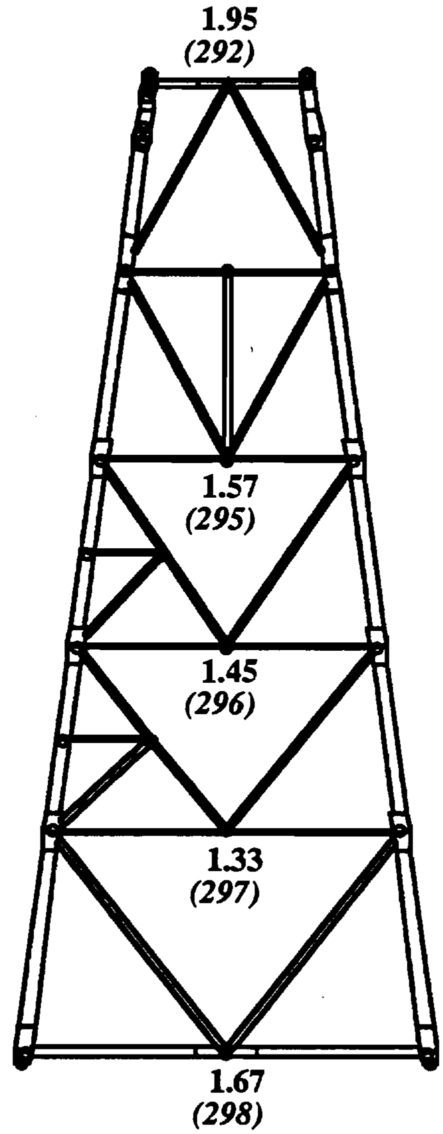
FIGURE A.5.10

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

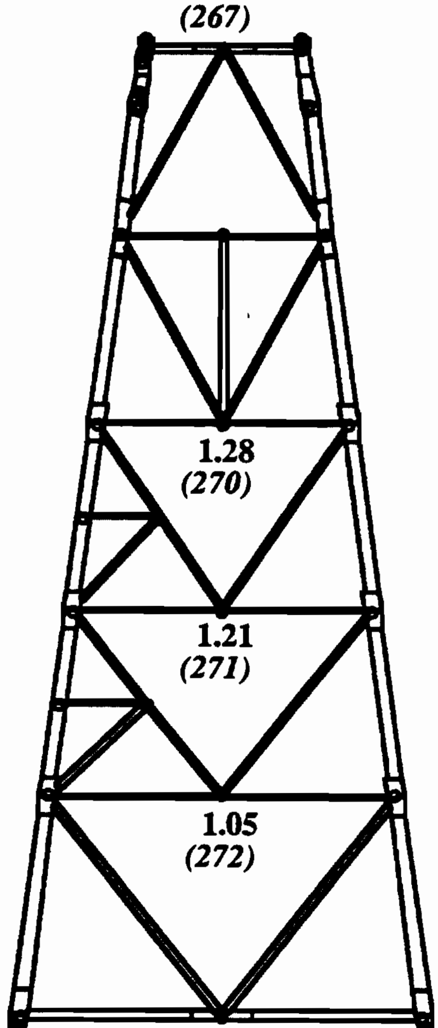
VERTICAL
ROW 3

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

TRIAL APPLICATION
MAIN PASS BLOCK 29?

FIGURE A.5.11

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
DESIGN LEVEL ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 4

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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FIGURE A.5.12

Drilling and Production Platform
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PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.4 Ultimate-Strength Analysis

A.5.4.1 INTRODUCTION

This section contains the results for the ultimate-strength analysis. The ultimate strength of the platform is assessed by using 2 separate procedures:

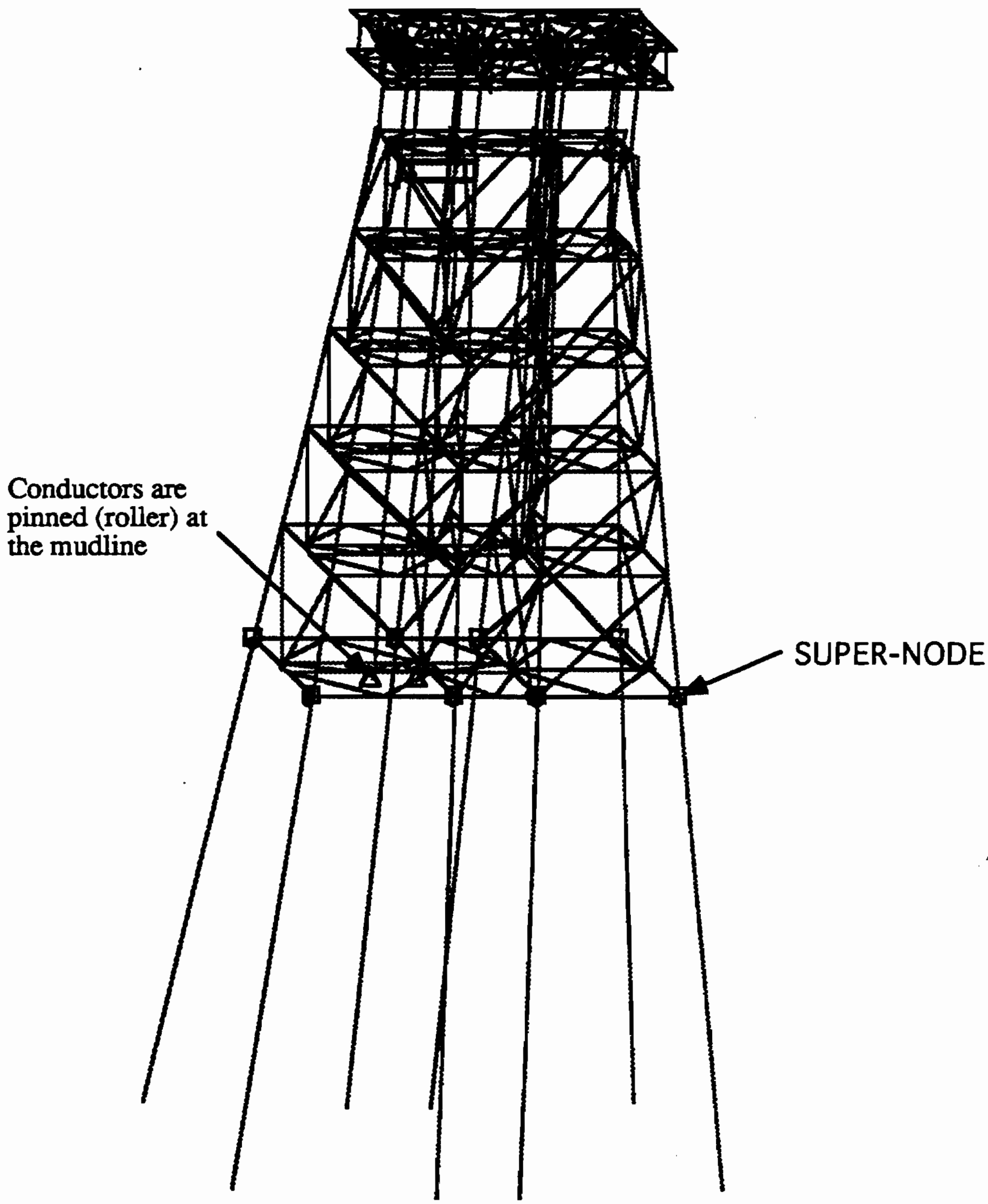
- First, the platform is checked following the same procedure as for the design-level analysis with the exception that the load criteria used corresponds to the ultimate criteria specified as described in Section A.5.1, and members are checked using equations with all safety factors and sources of conservatism removed. This analysis is referred to in this report as the linear ultimate strength analysis. This approach provides a conservative estimate of ultimate strength.
- Second, the platform's ultimate capacity is assessed using the inelastic, static pushover analysis.

Model

Two models were used to determine the ultimate capacity of the platform. For the first analysis (linear ultimate strength analysis), the same model as for the design-level analysis is used. See Figure A.5.13 for a perspective view of the computer model used in this analysis.

For the second analysis (static pushover analysis), the model used in the design-level analysis is revised. A discretized pile model is added below the mudline level to represent the pile and soil system. Each discretized pile is composed of 16 tubular elements (7 at 10', 5 at 20', 2 at 25', 1 at 15', and 1 at 5'). Nonlinear springs at each joint along the pile model the soil properties according to P-Y and T-Z curves. A single nonlinear spring at the end of each pile models the tip-load/tip-movement behavior. The 2 modeled conductors, representing the 14 existing conductors, are supported by a "roller" boundary condition at the mudline. See Figure A.5.14 for a perspective view of the computer model used in the pushover analysis.

ULTIMATE STRENGTH ANALYSIS
ANALYTICAL MODEL



<u>PERSPECTIVE VIEW</u>

FIGURE A.5.13

STATIC PUSHOVER MODEL

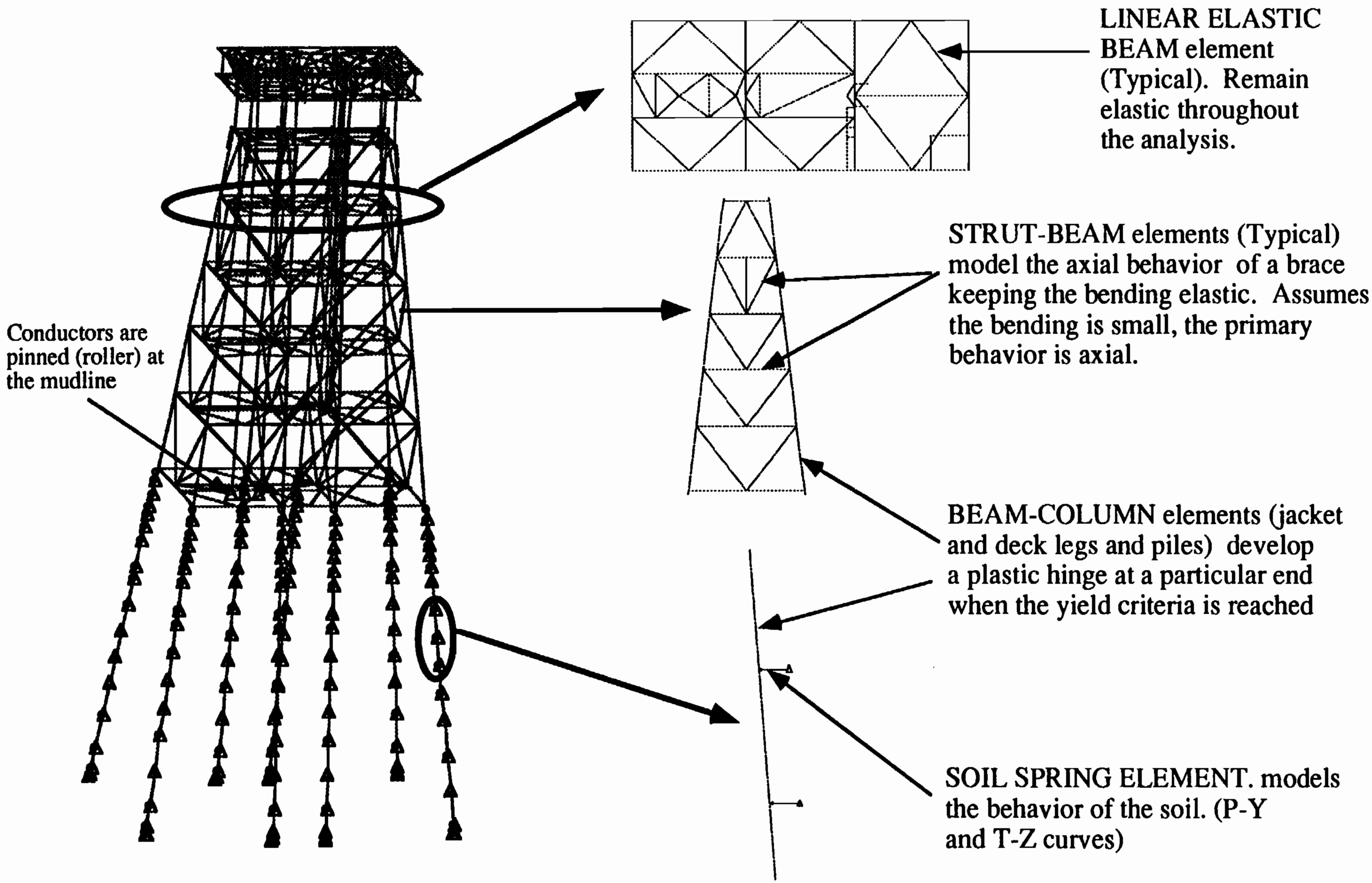


FIGURE A.5.14

Tubular members are classified into groups depending upon their expected ultimate behavior:

Nonlinear beam-column:...........Piles, jacket and deck legs

Strut-beam: ....Jacket diagonal and K-braces, horizontals in main trusses and deck braces

Linear (secondary):.................Conductors, J-tubes, other horizontals, etc.

Analysis Assumptions

The analysis was completed assuming the 14 conductors in operation were pinned at the mudline. Therefore, the piles and the soil system are the elements responsible for bearing the equipment weights, platform's self weight and storm loads. Two conductors were modeled to simulate the 14 conductors. The lateral load (wave) transfer from the conductors to the horizontal levels was achieved by assuring that the connection between horizontal levels and conductor can only distribute lateral loads. The vertical load due to the conductor weight is assumed to be supported by the conductor.

A.5.4.2 METOCEAN CRITERIA

The criteria for the ultimate-strength analysis is based on the 100-year force due to combined sudden hurricane and winter storm population. The corresponding wave force is applied using the directionality of the waves and the currents. The wave height, associated current, and profile (as a function of direction) is calculated in the same manner as described in API RP2A, 20th Edition, Section 2.3.4c4., except that the directional factors are in accordance with Figure 17.6.2-4 (API 20th Edition, Section 17). The required wave height, wave period, and current magnitude are described in Section A.5.1. See Figure A.5.3 for a comparison of wave base shears (base shear for omni-directional, ultimate-strength, and API RP 2A, 20th edition, criteria).

A.5.4.3 LOAD LEVEL AT ULTIMATE CAPACITY OF PLATFORM

The loads utilized during the ultimate-strength analysis are the same loads described in Section A.5.3.3, with the exception of the storm wave/current loads and wind loads. For the ultimate-strength analysis, the wave height (H_w), current value, and current profile are functions of wave directions. Reference Section A.5.1 for the wave criteria used in the ultimate-strength analysis. This study has selected 11 wave directions (0, 45, 90, 135, 169, 180, 214, 225, 244, 270, and 315 degrees). See Figure A.5.2 (Wave Direction) for a description of the conventional and wave directional approach with respect to the platform. The reasons for the selection of wave directions of 169, 214 , and 244 degrees are clearly explained in Section A.5.3.3.

The lateral load level due to the wave and current effects are summarized in Table A.5.1 of this report. The maximum lateral load for the ultimate-strength analysis is 2,624.3 kips in the 169-degree wave direction. See Figure A.5.3

for a comparison of wave base shear resulting from different wave directions (design-level analysis, ultimate-strength analysis, and API RP 2A, 20th edition).

A.5.4.4 RESULTS FROM LINEAR ULTIMATE-STRENGTH ANALYSIS

The next two sections present results corresponding to the linear ultimate-strength analysis. The results are presented in the form of sketches depicting members and joints with stress ratios greater than 0.85. The purpose of this analysis is to determine if overload is local or global. The intent is to determine which members or joints have exceeded their buckling or yield strength. As determined from the member check sketches, there are no members with stress ratios greater than 1.0. As determined from the joint check sketches, there are several joints with stress ratios greater than 1.0. All of the joints with stress ratios greater than 1.0 are joints supporting K-braces located at Row 1, Row 2, Row 3, and Row 4.

These results give us the indication that the structure's global response is adequate and can support the environmental forces described in API RP 2A, 20th Edition, Section 17. In addition, it indicates that a more detailed analysis should be performed to study the behavior of the overstressed joints due to the inherent conservatism built into the joint check formulation.

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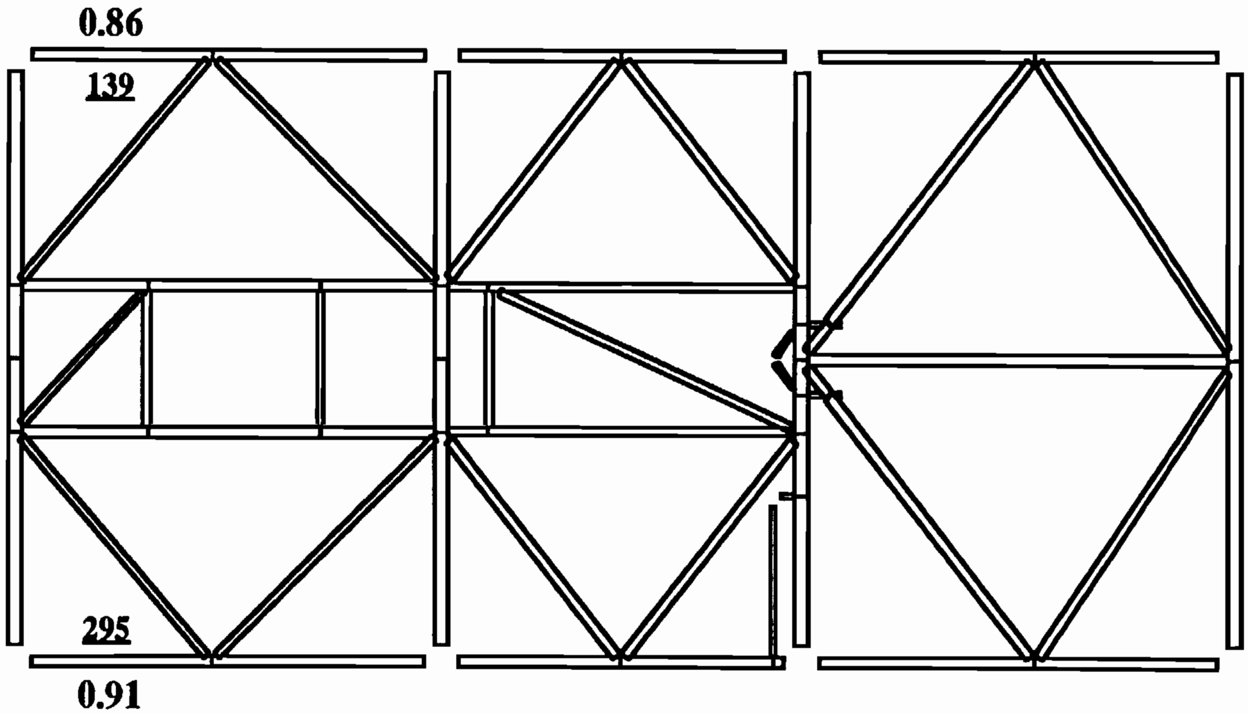
PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.4 Ultimate-Strength Analysis

A.5.4.5 MEMBER CHECK RESULTS WITH STRESS RATIOS > 0.85

This section contains sketches depicting members whose stress ratios exceeded 0.85 (see Figures A.5.15 through A.5.18). All checks were performed in accordance to API RP 2A, 20th Edition, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms. The checks were performed using equations with all safety factors and sources of conservatism removed. Note that the results have been divided into stress ratios due to strength check and stress ratios due to hydrostatic loads. In addition, the material yield stress used throughout the analysis is 36 ksi. The supplied platform drawings do not indicate material other than a 36 ksi steel.

- STRESS RATIO SKETCHES
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

HORIZONTAL
EL.(-)90'-0"

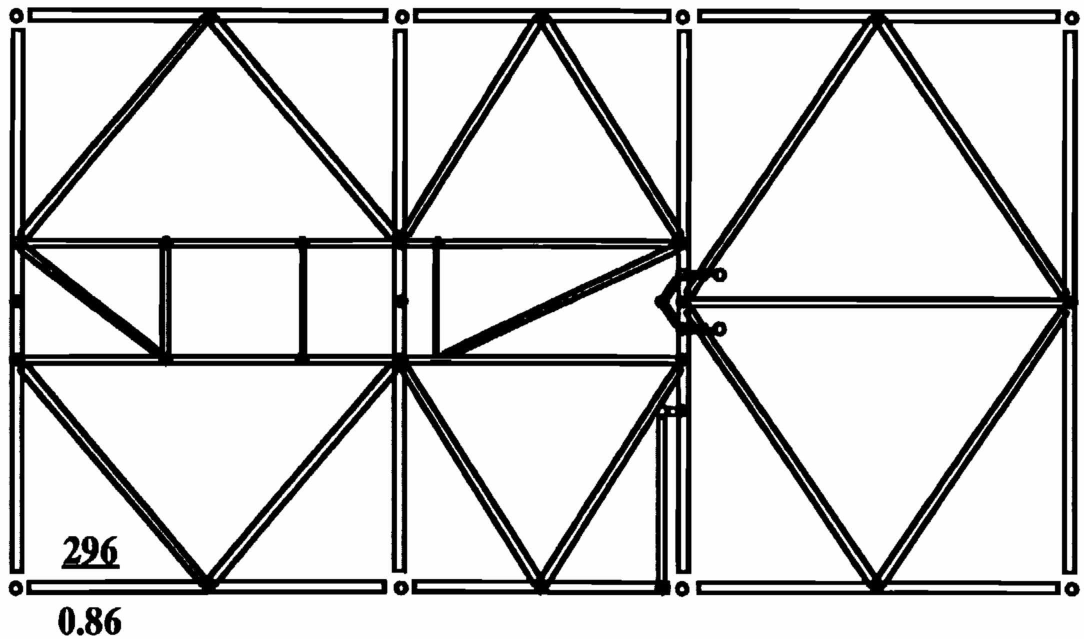
LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS
XXX Member No.

TRIAL APPLICATION
MAIN PASS BLOCK 293

FIGURE A.5.15

- STRESS RATIO SKETCHES

<u>ULTIMATE STRENGTH ANALYSIS</u>



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

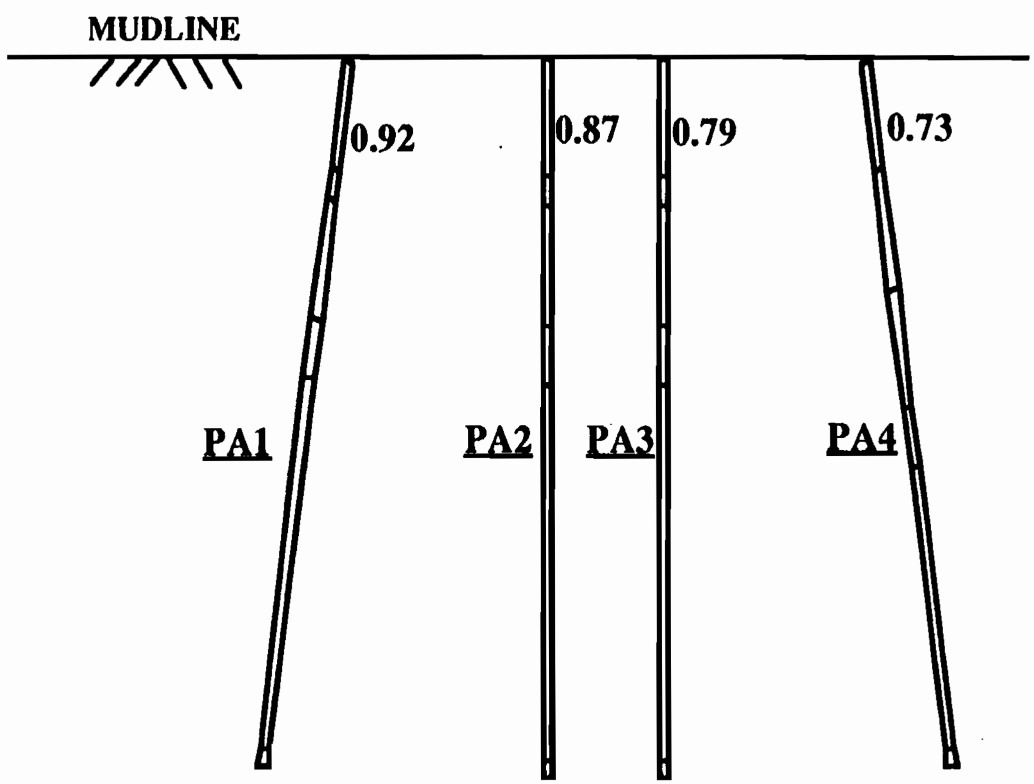
HORIZONTAL
EL.(-)139'-7 3/8"

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS
<u>XXX</u> Member No.

TRIAL APPLICATION
MAIN PASS BLOCK 293

FIGURE A.5.16

- STRESS RATIO SKETCHES
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

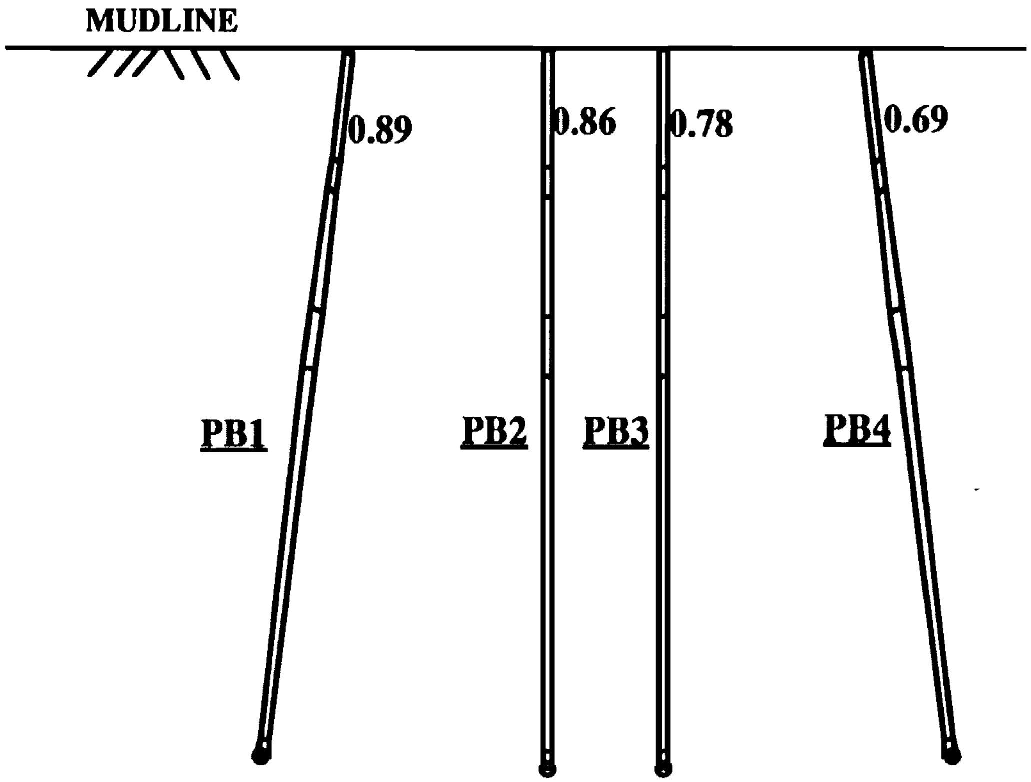
PILES BELOW MUDLINE
ROW A

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS
XXX Member No.

TRIAL APPLICATION
MAIN PASS BLOCK 293

FIGURE A.5.17

- STRESS RATIO SKETCHES
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

PILES BELOW MUDLINE
ROW B

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
HYDROSTATIC STRESS RATIO - ITALICS AND PARENTHESIS
XXX Member No.

TRIAL APPLICATION
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FIGURE A.5.18

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

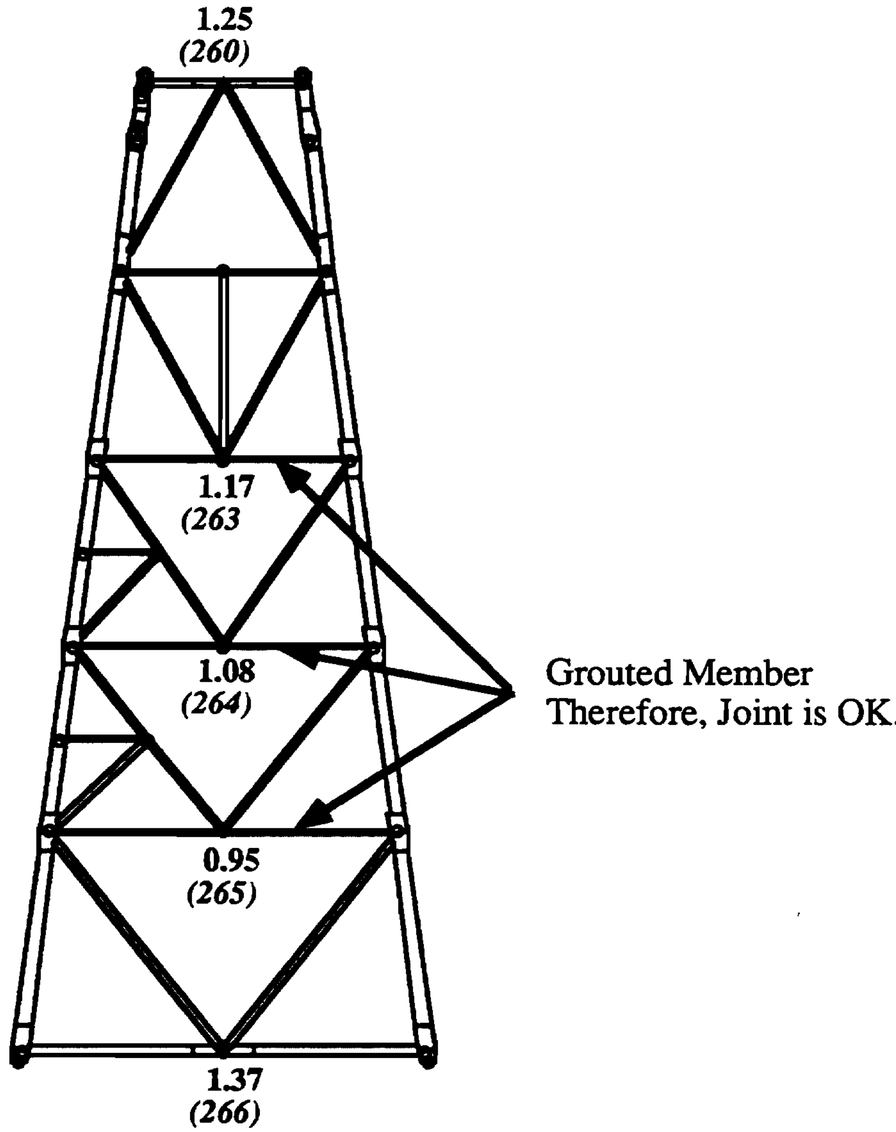
PART A: PLATFORM ASSESSMENT
A.5 Analysis Checks
A.5.4 Ultimate-Strength Analysis

A.5.4.6 NOMINAL JOINT CHECK RESULTS WITH STRESS RATIOS > 0.85

This section contains sketches depicting joints whose stress ratios exceed 0.85 (see Figures A.5.19 through A.5.22). All of the joints with stress ratios greater than 1.0 are K-joints. The reason is because the wall thicknesses and materials of the framing members (K-braces) are the same as the wall thicknesses and materials of the chord section.

All checks were performed in accordance with API RP 2A, 20th Edition, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms. The checks were performed using equations with all safety factors and sources of conservatism removed. Note that the sketches show only the results due to nominal joint check criteria. The results due to the 50% effectiveness check are disregarded, as called for in API RP 2A, Section 17.7.2c (draft version). In addition, the material yield stress used throughout the analysis is 42 ksi, which corresponds to the expected mean yield stress of the cans.

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 1

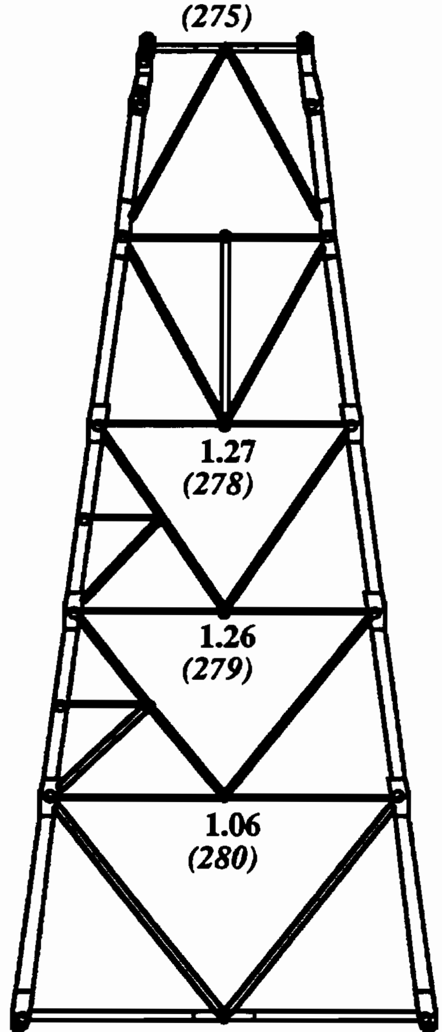
MEAN YIELD STRESS USED = 42 ksi

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

TRIAL APPLICATION
MAIN PASS BLOCK 293

FIGURE A.5.19

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 2

MEAN YIELD STRESS USED = 42 ksi

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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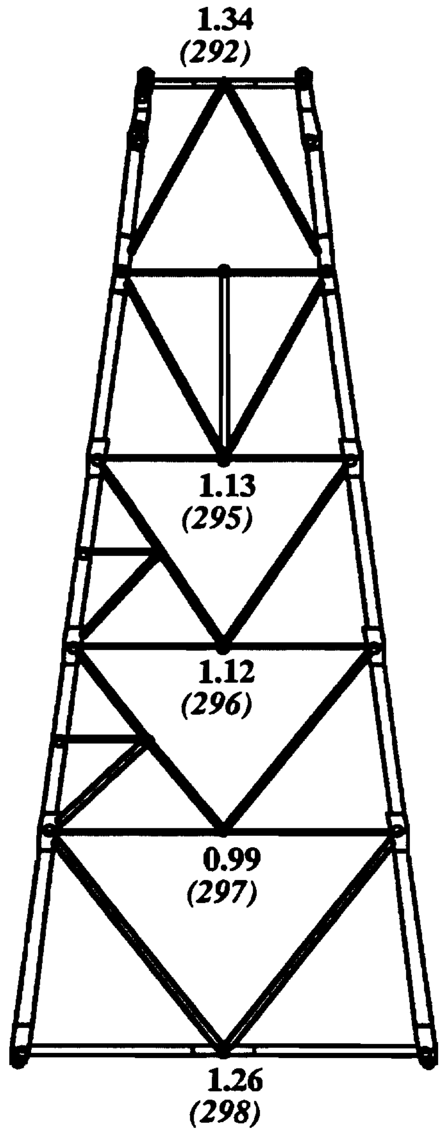
FIGURE A.5.20

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 3

MEAN YIELD STRESS USED = 42 ksi

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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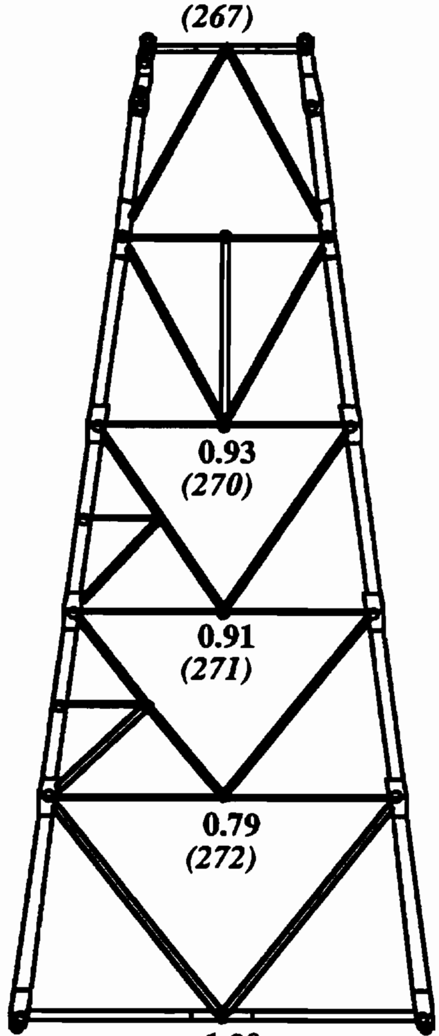
FIGURE A.5.21

- STRESS RATIO SKETCHES
NOMINAL JOINT CHECK
ULTIMATE STRENGTH ANALYSIS



NOTE: ONLY STRESS RATIOS GREATER THAN 0.85 ARE SHOWN

VERTICAL
ROW 4

MEAN YIELD STRESS USED = 42 ksi

LEGEND:
STRENGTH STRESS RATIO - PLAIN TEXT
JOINT NUMBER - ITALICS AND PARENTHESIS

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FIGURE A.5.22

A.5.4.7 PUSHOVER LOAD LEVEL

After inspecting the results of the linear ultimate-strength analysis (soil-pile interaction analysis [SPIA], member check, and joint check), it is concluded that the 169-degree and 214-degree wave directions are the dominant directions to the response of the platform. The static pushover analysis considers 4 wave directions (169, 180, 214, and 270 degrees). The 180-degree direction is in the longitudinal axis of the platform, and the 270-degree direction is in the transverse direction. See Figure A.5.2 (Wave Direction) for a description of the conventional and wave directional approaches with respect to the platform.

The pushover load in each wave direction is incrementally increased by a load factor by multiplying the load factor against the lateral load corresponding to the linear ultimate-strength analysis. The pushover load level for the wave directions considered is summarized below:

Wave Direction (Degrees)	Base Ultimate Lateral Load — Wave & Current (Kips)	Pushover Lateral Load (Kips)**
169	2,624	3,592
180	2,594	3,719
214	2,482	4,470
270	1,840	4,876

** The pushover lateral load is the same as the ultimate capacity of the platform.

A.5.4.8 LOAD - DEFLECTION PLOTS

Four plots are included to illustrate the global response of the platform. Each plot corresponds to the 4 wave directions selected (169-, 180-, 214-, and 270-degree wave directions) for pushover analysis. The displacement in the above plots corresponds to Joint No. 2007, located in the drilling deck at the A1 deck leg. The 2007 joint was selected after inspecting the displacement of the remaining 7 joints, corresponding to the other deck leg joints, and finding that the displacement difference between them is very small. Therefore, it is concluded that the load-deflection plot for Joint No. 2007 does not present a substantial variation in global response when compared to the other 7 joints. See Figure A.5.23 through Figure A.5.26 for an illustration of load-deformation for the different wave directions.

The results indicate that the ultimate capacity (Ru) of the platform is 1.37 times the ultimate lateral load in the 169-degree direction, 1.43 in the 180-degree direction, 1.80 in the 214-degree direction, and 2.65 in the 270-degree direction.

A.5.4.9 LOAD LEVEL AT WHICH FIRST COMPONENT REACHES IR=1.0

The load level at which the first component reaches IR=1.0 is defined as the load at which the first member buckles or yields in the static pushover analysis.

The following discussions illustrate the load level at which the first member buckles per wave direction selected.

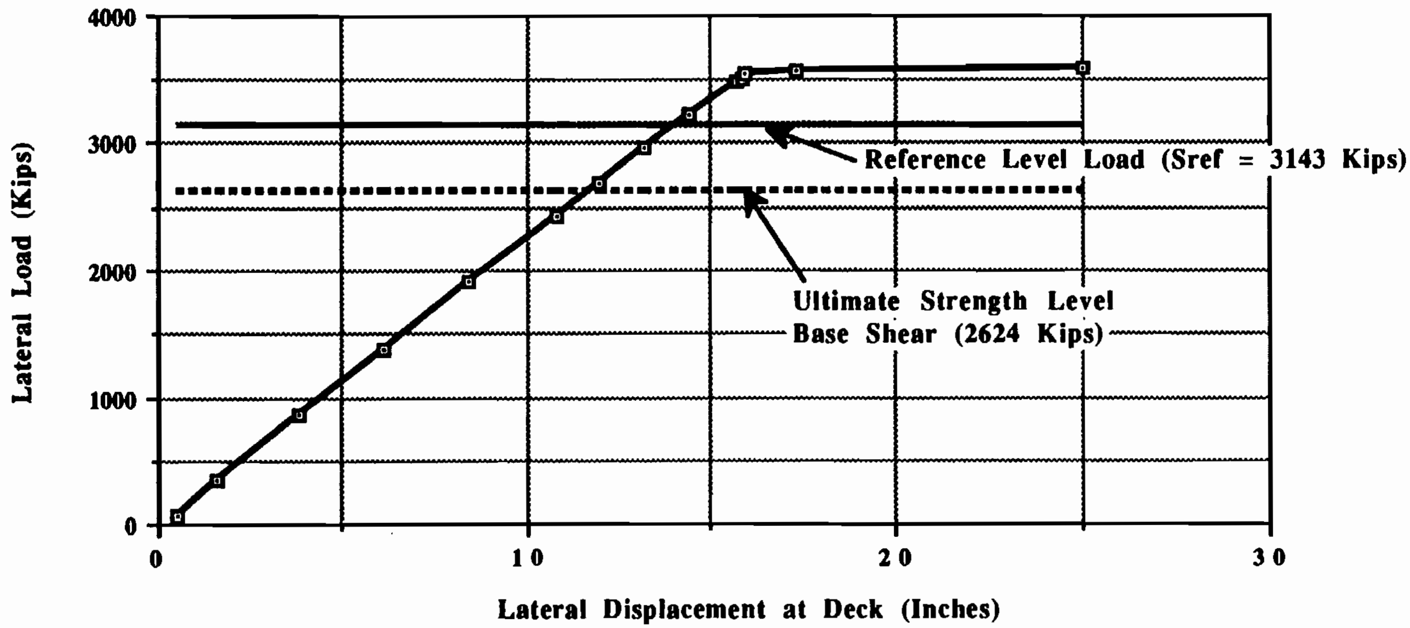
- **Wave direction — 169 degrees**: The ultimate strength level lateral load was increased to a load factor of **1.323** (load step #13) when member #295 (14" diameter x 0.375" wall thickness), a horizontal member in Row A at El (-)90'- 0", buckled. The corresponding lateral load for this load step is **3,547 kips**. See Figure A.5.23 and Table A.5.2 for a detailed description of the lateral load for each step before and after member #295 buckled.

- **Wave direction — 180 degrees**: The ultimate strength level lateral load was increased to a load factor of **1.38** (load step #14) when member #295 (14" diameter x 0.375" wall thickness), a horizontal member in Row A at El (-)90'- 0", buckled. The corresponding lateral load for this load step is **3,641 kips**. See Figure A.5.24 and Table A.5.3 for a detailed description of the lateral load for each step before and after member #295 buckled.

- **Wave direction — 214 degrees**: The ultimate strength level lateral load was increased to a load factor of **1.71** (load step #14) when member #140 (16" diameter x 0.375" wall thickness), a horizontal member in Row B at El (-)139'-7-3/8", buckled. The corresponding lateral load for this load step is **4,339 kips**. See Figure A.5.25 and Table A.5.4 for a detailed description of the lateral load for each step before and after member #140 buckled.

- **Wave direction — 270 degrees**: The ultimate strength level lateral load was increased to a load factor of **1.92** (load step # 16) when members #408 and #414 (18" diameter x 0.375" wall thickness), diagonal members at Row 2 buckled. The corresponding lateral load for this load step is **4,861 kips**. See Figure A.5.26 and Table A.5.5 for a detail description of the lateral load for each step before and after members #408 and #414 buckled.

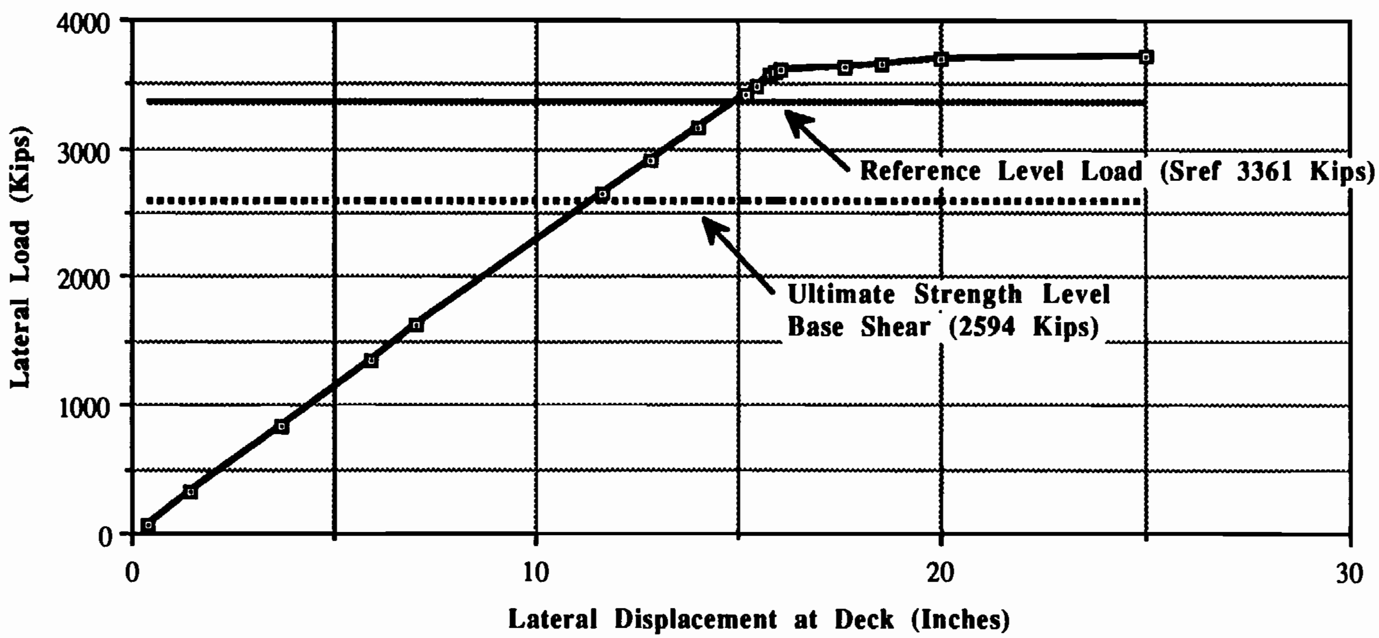
STATIC PUSHOVER ANALYSIS - STORM in 169 DEG. DIRECTION



Load Level which First Component Reaches I.R. of 1.0 (Si)	3547 Kips
Reference Level Load (Sref)	3143 Kips
Ultimate Capacity (Ru)	3592 Kips
Reserve Strength Ratio (RSR)	1.143
Platform Failure Mode: Jacket, Piles, Soils, etc.	JACKET

FIGURE A.5.23

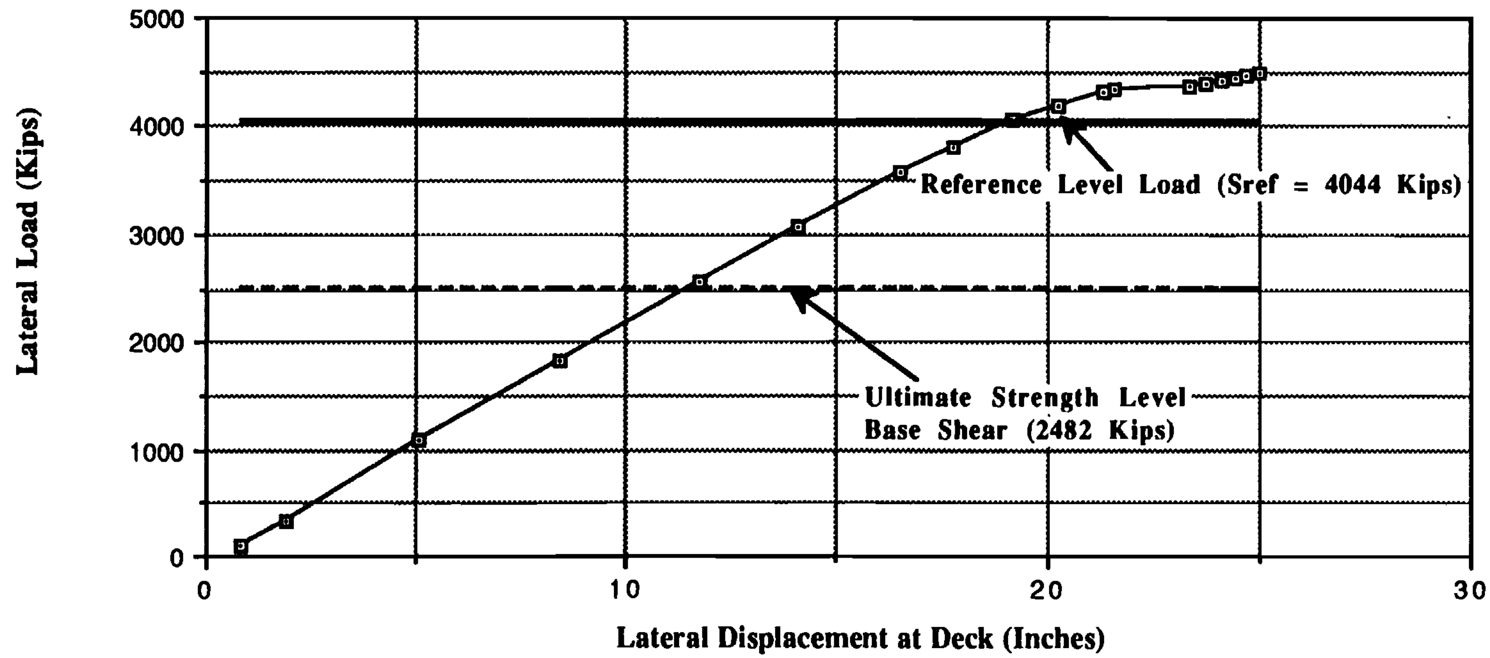
STATIC PUSHOVER ANALYSIS - STORM in 180 DEG. DIRECTION



Load Level which First Component Reaches I.R. of 1.0 (Si)	3641 Kips
Reference Level Load (Sref)	3361 Kips
Ultimate Capacity (Ru)	3719 Kips
Reserve Strength Ratio (RSR)	1.11
Platform Failure Mode: Jacket, Piles, Soil, etc.	JACKET

FIGURE A.5.24

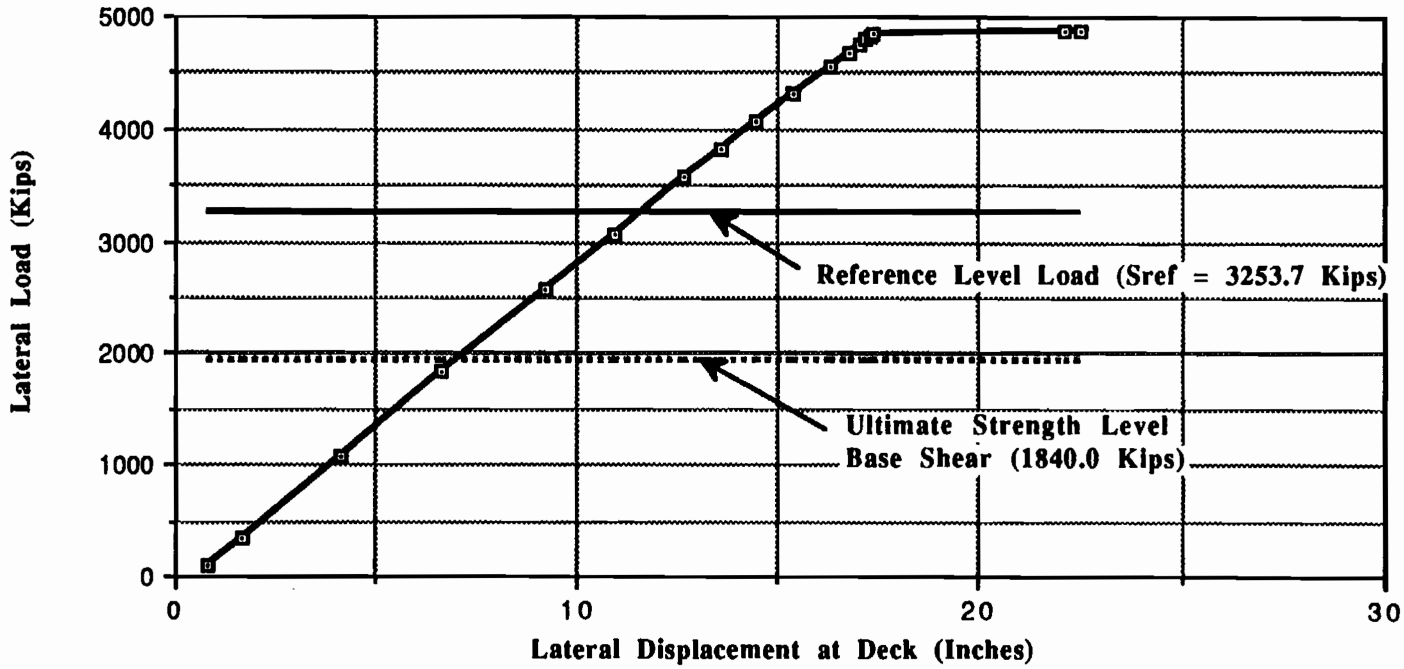
STATIC PUSH-OVER ANALYSIS - STORM in 214 DEG. DIRECTION



Load Level which First Component Reaches I.R. of 1.0 (Si)	4339 Kips
Reference Level Load (Sref)	4044 Kips
Ultimate Capacity (Ru)	4470 Kips
Reserve Strength Ratio (RSR)	1.11
Platform Failure Mode: Jacket, Piles, Soils, etc.	JACKET

FIGURE A.5.25

STATIC PUSHOVER ANALYSIS - STORM in 270 DIRECTION



Load Level which First Component Reaches I.R. of 1.0 (Si)	4861 Kips
Reference Load Level (Sref)	3254 Kips
Ultimate Capacity (Ru)	4876 Kips
Reserve Strength Ratio (RSR)	1.50
Platform Failure Mode: Jacket, Piles, Soils, etc.	JACKET

FIGURE A.5.26

TABLE A.5.2

PUSHOVER ANALYSIS

Storm in 169 Deg. Direction

LATERAL LOAD LEVEL FOR FIRST MEMBER WITH UNITY CHECK = 1.0 3547 Kips **

Load Step	Load Factor	Lateral Displacement at Deck Level (Feet)	Lateral Load (Kips)	Element Failure	Component Failure Mode
1	0.00	0.47	72		
2	0.10	1.57	335		
3	0.30	3.83	860		
4	0.50	6.11	1385		
5	0.70	8.44	1911		
6	0.90	10.80	2436		
7	1.00	12.00	2699		
8	1.10	13.21	2961		
9	1.20	14.44	3224		
10	1.30	15.69	3486		
11	1.31	15.83	3513		
12	1.32	15.96	3539		
13	1.323	15.99	3547	294, 295	Buckle Horizontals
14	1.326	17.05	3555	294, 139, 295, 140, 296, 141 , 297	Buckle Horizontals
15	1.330	17.29	3565	138, 294, 139, 295, 140, 296, 141, 297	Buckle Horizontals
16	1.34	25.00	3592	136, 138, 294, 139, 295, 140, 296, 141, 297 315, 316, 318, 319, 321, 322, 273, ...	Buckle Horizontals and diagonals

** The load level at which first component reach IR=1.0 is defined as the load at which the first member buckle or yield.

TABLE A.5.3

PUSHOVER ANALYSIS

Storm in 180 Deg. Direction

LATERAL LOAD LEVEL FOR FIRST MEMBER WITH UNITY CHECK = 1.0 3641 Kips **

Load Step	Load Factor	Lateral Displacement at Deck Level (Feet)	Lateral Load (Kips)	Element Failure	Component Failure Mode
1	0.00	0.41	58		
2	0.10	1.47	318		
3	0.30	3.66	837		
4	0.50	5.89	1356		
5	0.60	7.02	1616		
6	1.00	11.62	2655		
7	1.10	12.80	2914		
8	1.20	13.99	3174		
9	1.30	15.20	3433		
10	1.32	15.45	3485		
11	1.35	15.82	3563		
12	1.36	15.95	3589		
13	1.37	16.08	3615		
14	1.38	17.64	3641	295	Buckle Horizontal
15	1.39	18.52	3667	138, 294, 139, 295, 140, 296, 141, 297	Buckle Horizontals
16	1.40	20.00	3693	136, 138, 294, 139, 295, 140, 296, 141, 297, 315	Buckle Horizontals and diagonal
17	1.41	25.00	3719	136, 138, 294, 139, 295, 140, 296, 141, 297, 308 310, 315, 316, 318, 319, 321, 322, 273, 247, .. 1314	Buckle Horizontals, and diagonals Yield

** The load level at which first component reach IR=1.0 is defined as th- 'oad at which the first member buckle or yield.

TABLE A.5.4

PUSHOVER ANALYSIS

Storm in 214 Deg. Direction

LATERAL LOAD LEVEL FOR FIRST MEMBER WITH UNITY CHECK = 1.0 4339 Kips **

Load Step	Load Factor	Lateral Displacement at Deck Level (Feet)	Lateral Load (Kips)	Element Failure	Component Failure Mode
1	1.00	0.83	92		
2	0.00	0.00	0		
3	0.00	0.83	92		
4	0.10	1.87	341		
5	0.40	5.08	1086		
6	0.70	8.39	1831		
7	1.00	11.78	2576		
8	1.20	14.11	3073		
9	1.40	16.51	3569		
10	1.50	17.79	3818		
11	1.60	19.16	4066		
12	1.65	20.26	4190		
13	1.70	21.30	4314		
14	1.71	21.59	4339	140	Buckle horizontal
15	1.72	23.36	4364	138, 139, 295, 140, 141	Buckle horizontals
16	1.73	23.71	4389	138, 294, 139, 295, 140, 296, 141, 297	Buckle Horiz &diagonals
17	1.74	24.14	4414	138, 294, 139, 295, 140, 296, 141, 297	Buckle Horiz &diagonals
18	1.75	24.40	4438	138, 294, 139, 139, 295, 140, 296, 141, 297 1341	Buckle Horiz. , diagonals & Jacket leg yield
19	1.76	24.69	4463	138, 294, 139, 139, 295, 140, 296, 141, 297 ... 1341	Buckle Horiz. , diagonals & Jacket leg yield

** The load level at which first component reach IR=1.0 is defined as the load at which the first member buckle or yield.

TABLE A.5.5

PUSHOVER ANALYSIS

Storm in 270 Deg. Direction

LATERAL LOAD LEVEL FOR FIRST MEMBER WITH UNITY CHECK = 1.0 4861 Kips **

Load Step	Load Factor	Lateral Displacement at Deck Level (Feet)	Lateral Load (Kips)	Element Failure	Component Failure Mode
1	0.00	0.82	92		
2	0.10	1.64	341		
3	0.40	4.12	1086		
4	0.70	6.64	1831		
5	1.00	9.20	2576		
6	1.20	10.95	3073		
7	1.40	12.71	3569		
8	1.50	13.60	3818		
9	1.60	14.49	4066		
10	1.70	15.40	4314		
11	1.80	16.30	4563		
12	1.85	16.76	4687		
13	1.88	17.03	4761		
14	1.90	17.22	4811		
15	1.91	17.31	4836		
16	1.92	17.40	4861	408, 414	Buckle diagonals
17	1.923	22.12	4868	408, 414, 357, 402, 449, 455, 461	Buckle diagonals
18	1.93	22.50	4876	408, 414, 357, 402, 449, 455, 461, 353, 449, 373 375, 377 102, 104, 120, 122, ...	Buckle diagonals and Horizontals and Jacket leg yields

** The load level at which first component reach IR=1.0 is defined as ʼad at which the first member buckle or yield.

The table below summaries the load level at which the first member buckle per wave direction selected.

Wave direction (degrees)	Lateral load level (kips)*	Load factor **
169	3,547	1.35
180	3,641	1.40
214	4,339	1.75
270	4,861	2.64

* Lateral load at which first member buckled.

** Load factor = lateral load level / ultimate-strength load level

A.5.4.10 RESERVE STRENGTH RATIO

The reserve strength ratio (RSR) is defined as the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present API RP 2A, 20th Edition, procedures. In this study, a static pushover analysis is used to calculate the ultimate lateral load-carrying capacity of the platform for each wave direction selected.

The local ultimate strength of the platform might vary depending on the selection of the type of members (such as strut, beam-column, linear-beam, etc.) used to model the platform, the numerical solution technique used by the nonlinear computer program, and the selected tolerances for unbalanced forces and displacements. However, the global response should be similar if the same guidelines are followed.

The results below represent values of RSR for the platform being studied:

Ultimate wave direction (degrees)	Reference capacity (kips)	Reserve lateral load (kips)	Strength ratio (RSR)
169	3,592	3,143	1.14
180	3,719	3,361	1.11
214	4,470	4,044	1.11
270	4,876	3,254	1.50

A.5.4.11 RESULTS FROM PUSHOVER NONLINEAR ANALYSES

The basic results of the nonlinear pushover analysis are summarized in Tables A.5.2 to A.5.5 and in Figures A.5.23 to A.5.26. The results are presented in terms of a factored load. The lateral load corresponding to the linear ultimate-strength analysis is incrementally increased by a load factor.

The results show that the platform behaves linearly up to the first member failure, followed by a significant reduction in strength due to extensive brace failures. In the case of storms in the 169- and 180-degree directions, the first member to buckle is a horizontal member at El (+)90'-0". Additional loads cause members in adjacent horizontal levels to buckle, leading to a decrease in strength-buckling of diagonals between levels and a significant reduction in the platform's strength. A similar behavior occurs for the 214-degree direction, with the exception that the first member to buckle is a member at El (+)139'-7-3/8".

The behavior of the platform in the case of the storm in the 270-degree direction is different in the sense that the first member to buckle is a diagonal member. Additional loads cause adjacent diagonal members to buckle. In the next stage, a portal frame mechanism is developed in the legs over adjacent elevations, and, finally, a significant reduction in platform's strength occurs.

For all 4 wave directions, the lateral load corresponding to the platform's ultimate load-carrying capacity is higher than the referenced lateral load. The referenced lateral load is defined as the load corresponding to the 100-year environmental condition lateral loading, computed using present API RP 2A, 20th Edition, procedures. The ratio between the two loads is defined as the reserve strength ratio (RSR). For end-on loading, a storm in the 180-degree direction, the RSR is 1.11. For diagonal loading, a storm in the 214-degree and 169-degree directions, the RSR are 1.14 and 1.11, respectively. For the broadside storm in the 270-degree direction, the RSR is 1.50.

A comparison, between the platform's ultimate load-carrying capacity and the load corresponding to the linear ultimate-strength load level, yields ratios in the order of 1.35, 1.40, 1.75, and 2.64 for the 169-, 180-, 214-, and 270-degree wave directions. These ratios indicate that the platform's capacity is higher than the lateral load levels specified in API RL 2A, 20th Edition, Section 17 (draft version).

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

PART A: PLATFORM ASSESSMENT
A.6 Consideration of Mitigation

In order to determine if the Main Pass Block 293 platform is adequate to withstand the environmental forces defined in API RP 2A, 20th Edition, Section 17 (draft version), analytical and field checks were performed. The analytical checks consisted of 3 analyses: design-level analysis, linear elastic ultimate-strength analysis, and static nonlinear pushover analysis. The field checks consisted of Level I and Level II surveys.

The 3 analyses show that the structure can satisfactorily support the loads described in API RP 2A, 20th Edition, Table 17.6.2-1 (draft version), for a platform categorized as manned, evacuated with an insignificant environmental impact. The only concern left for a more detailed investigation are the joints supporting K-brace members located at Row 1, Row 2, Row 3, and Row 4 which the design-level and ultimate-strength analyses show to be overstressed. Additional analyses are recommended since the current joint check procedure has some conservatism built in its equations.

The results of the Level I and II surveys show that there is no significant structural damage to the platform.

Based on the global analysis behavior (excluding the above joints), platform inspections, and the platform's 25 years of service with no significant damage, the structure can support the load level defined in API RP 2A, 20th Edition, Section 17 (draft version) — load level defined for a platform categorized as manned, evacuated with an insignificant environmental impact.

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

PART A: PLATFORM ASSESSMENT
A.7 Summary Note - Part A

This section presents a summary of the results for the analysis work conducted for the subject platform. The platform is an 8-leg, 18-slot, self-contained drilling and production platform in 247 feet of water at Main Pass Block 293, Gulf of Mexico.

The analyses were performed as part of a Joint Industry Project entitled "Trial Application of the Draft API RP 2A Guidelines of Existing Platforms." The purpose of the analyses was to determine if the platform can withstand the environmental forces defined in API RP 2A, 20th Edition, Section 17 (draft version). Three analyses were performed: design-level analysis, linear elastic ultimate-strength analysis, and static nonlinear pushover analysis.

Design-Level and Linear Elastic Ultimate-Strength Analyses

The design-level and the linear elastic ultimate-strength analyses show that the platform can satisfactorily support the loads described in API RP 2A, 20th Edition, Section 17 (draft version), for a platform categorized as manned, evacuated with an insignificant environmental impact. The results are summarized in Figures A.5.5 to A.5.12 for the design-level analysis and in Figures A.5.15 to Figure A.5.22 for the linear elastic ultimate-strength analysis. The only concern left for a more detailed investigation affects the joints supporting K-brace members located at the transverse lateral frames (Row 1, Row 2, Row 3, and Row 4). Punching shear checks indicate that, for both analyses, these joints are overstressed. Therefore, additional analyses are recommended, since the current joint check procedure has some conservatism built into its equations.

Nonlinear Pushover Analysis

The basic results of the nonlinear pushover analysis are summarized in Tables A.5.2 to A.5.5 and Figures A.5.23 to A.5.26. The results are presented in terms of a factored load. The lateral load corresponding to the linear ultimate-strength analysis is incrementally increased by a load factor.

The results show that the platform behaves linearly up to the first member failure, followed by a significant reduction in strength due to extensive brace failures. In the case of storms in the 169- and 180-degree directions, the first member to buckle is a horizontal member at El (+)90'-0". Additional loads cause members in adjacent horizontal levels to buckle, leading to a decrease in strength-buckling of diagonals between levels and a significant reduction in the platform's strength. A similar behavior occurs for the 214-degree direction, with the exception that the first member to buckle is a member at El (+)139'-7-3/8".

The behavior of the platform in the case of a storm in the 270-degree direction is different in the sense that the first member to buckle is a diagonal member. Additional loads cause adjacent diagonal members to buckle. In the next stage, a portal frame mechanism is developed in the legs over adjacent elevations, and, finally, a significant reduction in platform's strength occurs.

For all 4 wave directions, the lateral load corresponding to the platform's ultimate load-carrying capacity is higher than the referenced lateral load. The referenced lateral load is defined as the load corresponding to the 100-year environmental condition lateral loading, computed using present API RP 2A, 20th Edition, procedures. The ratio between the 2 loads is defined as the reserve strength ratio (RSR). For end-on loading, a storm in the 180-degree direction, the RSR is 1.11. For diagonal loading, a storm in the 214- and 169-degree directions, the RSR are 1.14 and 1.11, respectively. For a broadside storm in the 270-degree direction, the RSR is 1.50.

A comparison, between the platform's ultimate load-carrying capacity and the load corresponding to the linear ultimate-strength load level, yields ratios in the order of 1.35, 1.40, 1.75, and 2.64 for the 169-, 180-, 214-, and 270-degree wave directions. These ratios indicate that the platform's capacity is higher than the lateral load levels specified in API RP 2A, 20th Edition, Section 17 (draft version).

Based on the global analysis behavior (excluding the above joints), platform inspection, and 25 years in-service with no significant damage, the structure can support the load defined in API RP 2A, 20th Edition, Section 17 (draft version).

Drilling and Production Platform
Main Pass Block 293
247-Foot Water Depth
TRIAL DOCUMENT

PART B: REVIEW AND FEEDBACK TO API TG 92-5

After completing the assessment process of ꞏ 8-leg platform located at Main Pass Block 293, there are several concerns in the draft version of API RP 2A, 20th Edition, Section 17, that need to be expanded or clarified:

1. Based on platform initiators of API RP 2A, 20th Edition, Section 17.2 (draft version), this platform is not subject to the assessment process. None of the conditions noted in Sections 17.2.1 through 17.2.5 exist. In addition, underwater inspection (Level II inspection) indicates that the platform is in satisfactory condition. That is, members, as well as joints, do not present any signs of being affected by the environmental conditions to which the platform has been subject during its 25 years of operation. Nevertheless, after completing the analytical platform assessment, the study found that the joints supporting the K-braces at Row 1, Row 2, Row 3, and Row 4 are overstressed. Moreover, the platform was designed, built, and installed before the release of API RP 2A, 9th edition. All of this discussion leads to the need to include guidelines to check these joints, taking into account that the current joint check procedure has some conservatism built into its equations. As it is understood from the JIP meeting of June 7, 1994, an API committee is currently reviewing the joint check design procedure. The committee performing this revision should consider assessment of existing platforms as one of their key evaluations.

2. The difference in lateral load level between a platform being classified as belonging to the Significant Environmental Impact category and a platform in the Insignificant Environmental Impact category is substantial. As Figure B.1, Figure B.2, and Figure B.3 indicate, the difference of load can be as high as a factor of 2.0. Nevertheless, the definitions in API RP 2A, Section 17.3.2 and Section C17.3.2, are not clear enough. Section 17.3.2b indicates "that a platform may have potential for liquid hydrocarbon or sour gas release and still be categorized as Insignificant Environmental Impact." The level of hydrocarbon or sour gas release required to still belong in the insignificant impact category must be defined.

3. The static pushover analysis calls for a description of the load level at which the first component reaches IR=1.0. This study has assumed that it means the load level at which the first member buckles or yields. A more expanded definition needs to be provided to the definition of this load level.

WAVE BASE SHEAR COMPARISON FOR SIGNIFICANT AND INSIGNIFICANT ENVIRONMENTAL IMPACT CONDITIONS

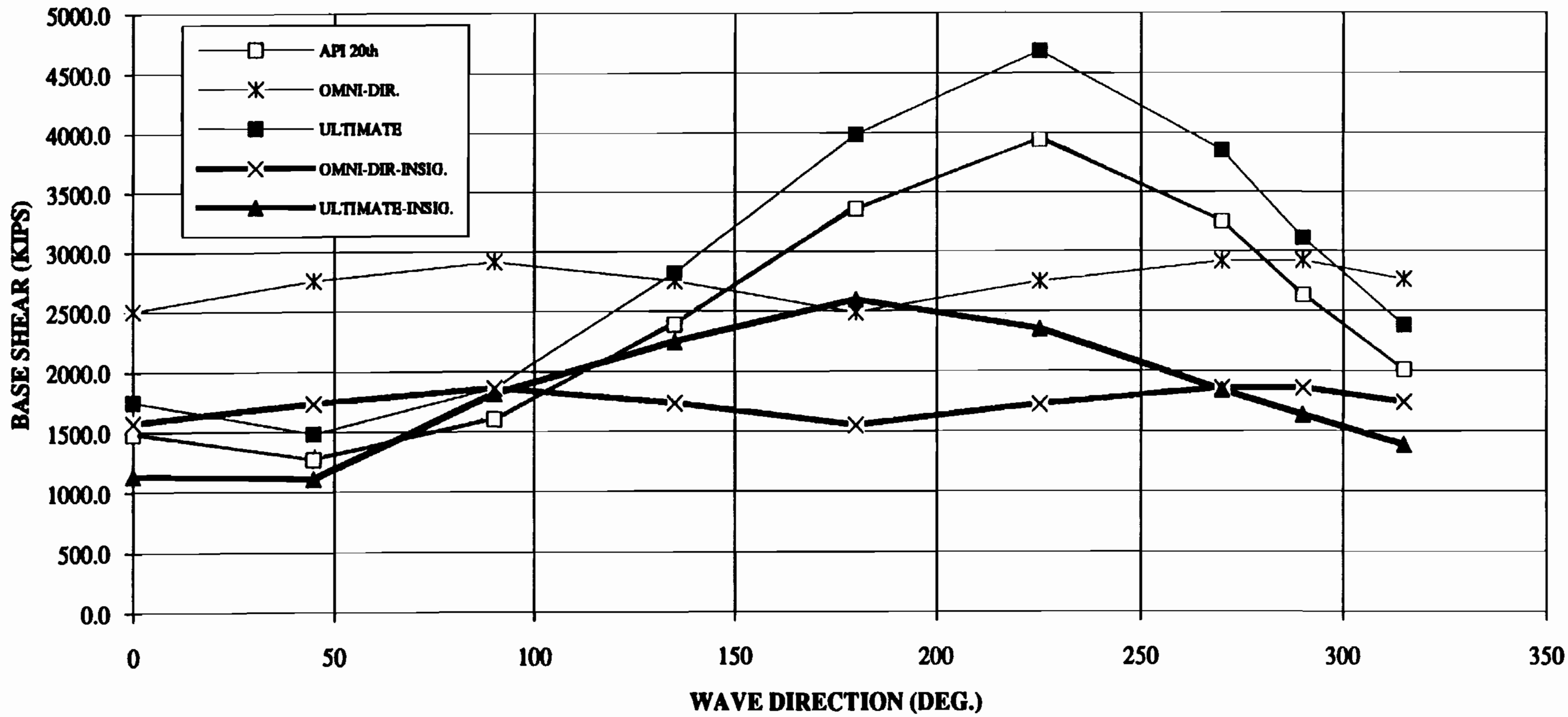


FIGURE B.1

WAVE BASE SHEAR RATIO BETWEEN SIGNIFICANT AND INSIGNIFICANT ENVIRONMENTAL IMPACT FOR OMNI-DIRECTIONAL CRITERIA

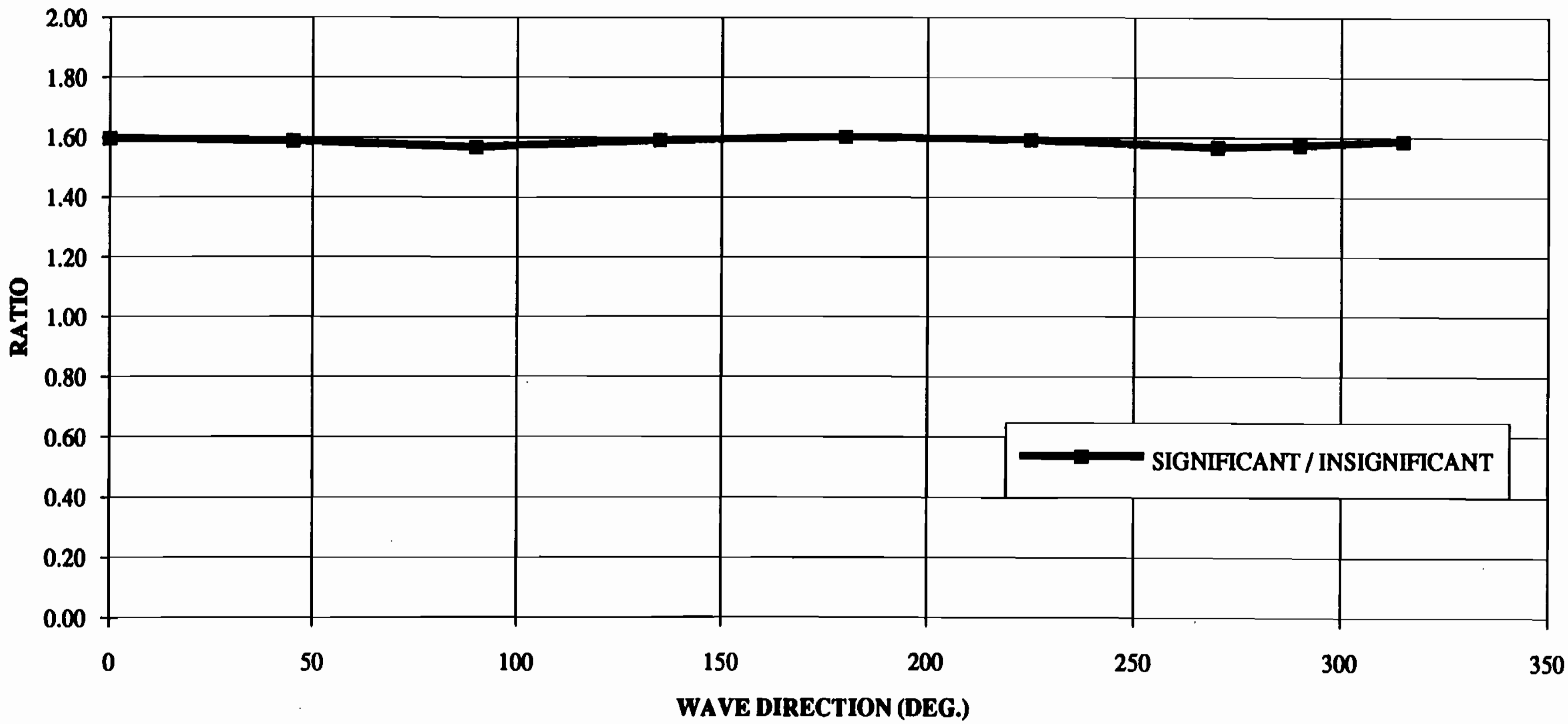


FIGURE B.2

WAVE BASE SHEAR RATIO BETWEEN SIGNIFICANT AND INSIGNIFICANT ENVIRONMENTAL IMPACT FOR ULTIMATE STRENGTH CRITERIA

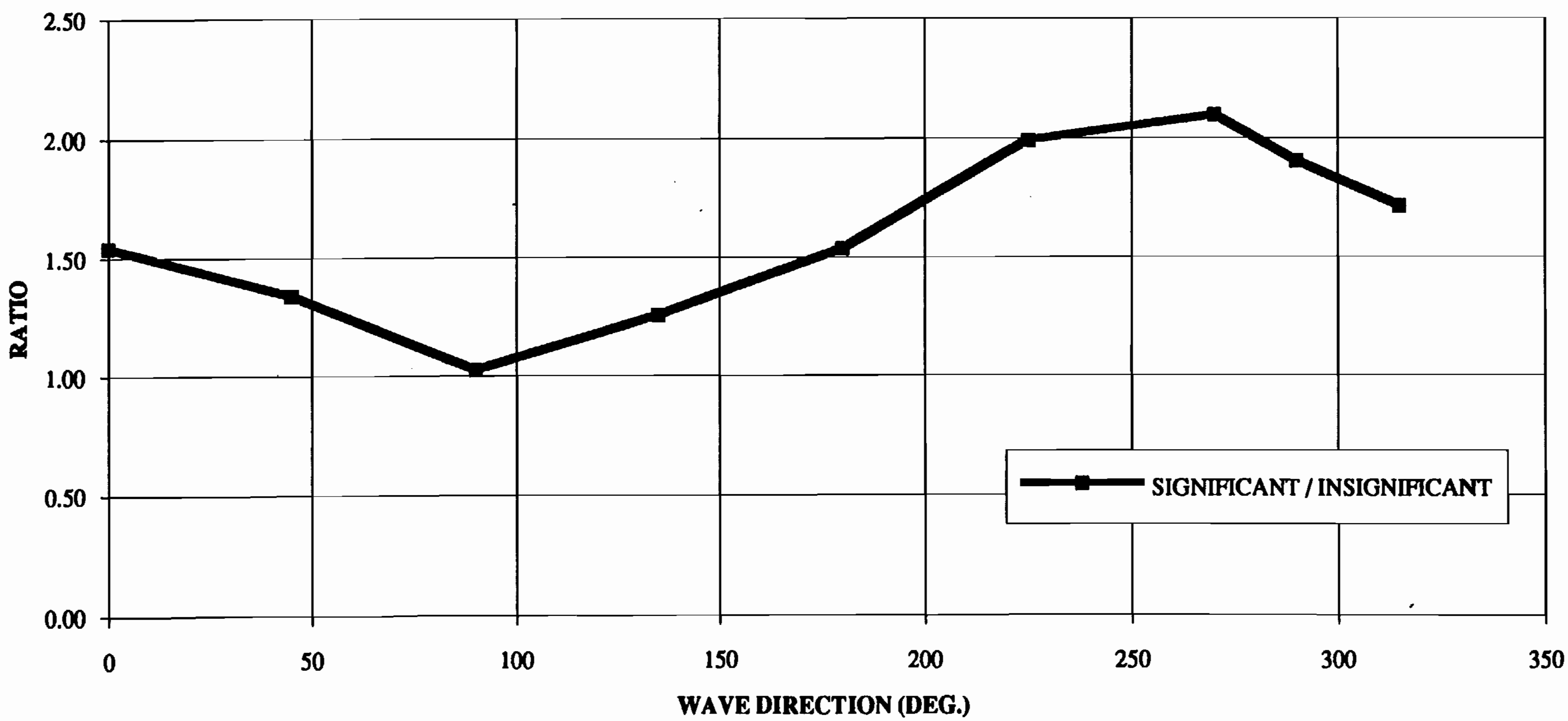


FIGURE B.3

PLATFORM "F"

A JOINT INDUSTRY PROJECT

TRIAL APPLICATION OF THE DRAFT API RP2A GUIDELINES FOR ASSESSMENT OF EXISTING PLATFORMS

PLATFORM "F"

I. General Platform Information

A. Physical Features

No. Legs: 4

No. Piles: 4 Insert

Chart Depth: 161'-8"

Original Design Water Depth: 157'-0"

No. Conductors: 4 Total, 3 @ 30"Ø and 1 @ 48"Ø

No Risers: 8 Risers located along 3 sides of the platform

Boat Landings: 2 full width landings, one on platform east and one on platform south.

Manning: Manned platform with a normal compliment of 3 men.

B. Surveys

A Level II Survey was performed in 1989. This survey reveled the following data.

- Cathodic protection system of sacrificial anodes was working at acceptable levels.

- A diver visual inspection was made of all members, with no abnormalities reported.

- No significant amount of scour has taken place, but the bottom of platform is at EL (-)162'-0", Not EL (-)157'-0".

II. Platform Assessment

A. Platform Selection

Addition of Personnel: No

Addition of Facilities: No. There has been some added equipment to the facilities.

Increased Loading: No. loading has not increased by 10% or more.

Damage: No known damage

Inadequate Deck Height: Yes. Required deck height by fig. 17.6.2-36 is 36.4 ft. The actual height above MLW to bottom of steel of lowest deck is 27'-9 5/8".

B. Categorization

1. Life Safety: Manned, evacuated

2. Environmental Impact: Insignificant environmental impact

   - Down hole safety valves are in place on producing wells.

   - Oil storage tank is pumped down in preparation for hurricanes.

   - Field is shut in prior to evacuation of platform.

   - Platform is not near to coastline.

C. Condition Assessment

1. Surveys (from page one)

2. Soil Data: A site specific soil boring is available. This boring was taken by McClelland Engineers in Sept. 1969 and has been updated by Fugro-McClelland in May, 1994.

3. Is platform damaged: No
   Is deck height Inadequate: Yes
   Has loading increased: No.

   Because of inadequate deck height, Analysis Checks will be required. Specifically, an ultimate strength analysis is required. For the JIP, both a Design Basis Check and Design Level Analysis will be performed also.

4. Is platform unmanned: No — Design Basis
   Insignificant Environmental Impact: Yes — Check Req'd

D. **Design Basis Check**

1. Platform location: Gulf of Mexico

2. Platform designed to 9th edition or later of API RP2A: No

   *Platform fails the design Basis Check.

E. **Summary of Design Level Analysis**

A computer model of the selected platform, designated as platform F, was developed using Engineering Dynamics, Inc. SACS program suite. A static elastic analysis of the three dimensional model was then performed. The analysis consisted of a wave loading program Seastate, structural analysis and code checks SACS, non-linear platform and foundation coupled interaction analysis PSI, and joint can punching shear check Joint Can.

Force level and patterns were developed using the API RP2A Section 17 design level criteria for insignificant environmental impact. Any storm criteria not explicitly given in Section 17 was taken from API RP2A 20th edition, Section 2. Although the platform is very near symmetrical in both structural arrangement and loading, the storm loads were applied in 8 directions at 45° increments starting with platform north.

E.1 Metocean Criteria for Sudden Hurricanes

Chart depth:	161'-8"
Storm tide:	3'-0"
Still water depth:	164'-8"
Wave height:	45'-0" (Omni-directional)
Wave period:	11.75 Secs.
Wind speed:	55 knots
Current speed:	1.2 knots (Omni-directional, full depth)
Current blockage factor:	End-on, broadside = 0.80 diagonal = 0.85
Wave kinematics factor:	0.88

	fouled	clean
Coefficient of Drag, C_D	1.05	0.65
Coefficient of Inertia, C_M	1.20	1.60

Marine growth: 1.5 inches, full depth

E.2 Summary of loadings

Wave Direction	Wind Load (Kip)	Wave-In-Deck Load	Wave Current Load (Kips)	Total Load (Kips)
N.E. to S.W. (End-On)	27.9	30.1	955.2	1013.3
N. to S. (Diagonal)	27.7	43.6	915.4	986.1

Maximum unity ratio in jacket and deck structure is 0.82 in the leg at approximate EL of (-)13'0" where the leg transitions from 39.5"Ø x 1.25" to 39.5"Ø 0.500"

Maximum unity ratio in pile foundation is 0.850 at the pile-jacket leg.

Minimum factor of safety with respect to axial capacity of pile is 1.70.

Maximum joint can unity ratio is 0.763 at K Brace joint on Elev. (-)97'-0".

*The Platform passes the Design Level Analysis check.

F. Summary of Ultimate Strength Analysis

A model was built of platform "F" for the JIP using PMB's CAP software. The model consists of a combination of different nonlinear and linear elements to represent the various components of the structure. The selection of the element is made on the basis of it local properties, the mode in which the member is likely to fail and the post-elastic response the member is likely to exhibit. The model included explicit representation of all 4 piles with PAS (pile-soil analysis system) elements to capture the coupled response of the platform and foundation.

The platform is classified as having insignificant environmental impact. Force patterns were developed using API Section 17 ultimate capacity criteria. The storm wave analyses were performed with drag and inertia coefficients per the 20th edition of API RP 2A. Due to the symmetry of the platform, two critical directions (South-North diagonal and Southeast-Northwest broadside) were chosen with respect to maximum wave height and expected failure mechanisms. In addition, API Section 2 loading based on the 100-year wave height was calculated in order to determine the reserve strength ratio (RSR) for each of the ultimate capacity analyses. The loads are summarized as follows:

Wave Direction & Criteria	Wind Load (Kip)	Wave-In-Deck Load (Kip)	Current Load (Kip)	Wave Load (Kip)	Total Load (Kip)
S-N (Ult. Cap)	60	76	9	827	972
SE-NW (Ult. Cap)	38	87	7	942	1074
S-N (100 yr)	33	72	9	841	956
SE-NW (100 yr.)	47	182	7	1091	1327

The load profiles established from the ultimate capacity criteria were used in the pushover analyses to determine ultimate strength. The results of these analyses are summarized as follows:

Wave Direction	Ultimate Capacity (Base Shear-kip)	Reference Level Load (Base Shear - kip)	Reserve Strength Ratio
S-N (Diagonal)	2035	956	2.13
SE-NW (Broadside)	2320	1327	1.75

In both of the analyses, the controlling mechanism occurs in the foundation with the plunging of the piles followed by hinges forming in the piles due to excessive deformation.

F.1 Metocean Criteria

A. Environmental Loads - API 20th Ed Ultimate Capacity Wave (Sudden Hurricane)

Two separate wave directions have been chosen for analysis: 315 deg. and 360 deg.

Property	315 Deg.	360 Deg.
Orientation WRT Plat. North	SE-NW (Broadside)	S-N (Diagonal)
Wave Height (ft.)	56.5	53.68
Apparent Period (s)	13	12.63
Storm Tide (ft.)	3	3
Current Heading (deg)	277	277
Current Speed (kts)	1.8	1.8
Current Blockage Factor	0.8	0.85
Crest Height (ft.)	41.8	39.75
Crest Velocity (ft/s)	21.303	20.224
Wave-in-deck load (kip)	87	76
Wind Load (kip)	38	60
Current Load (kip):	7.2	9.3
Pushover Wave Load (kip)	942	827
Reference Level Load In Direction of Wave (kip)	1074.2	972.3

Notes:
Water depth = 161.67 ft.
Wave Kinematics factor = 0.88
Wind Speed @ 10m above msl = 118.3 ft/s
Section 17 C17.6.2 used to determine wave/current deck forces

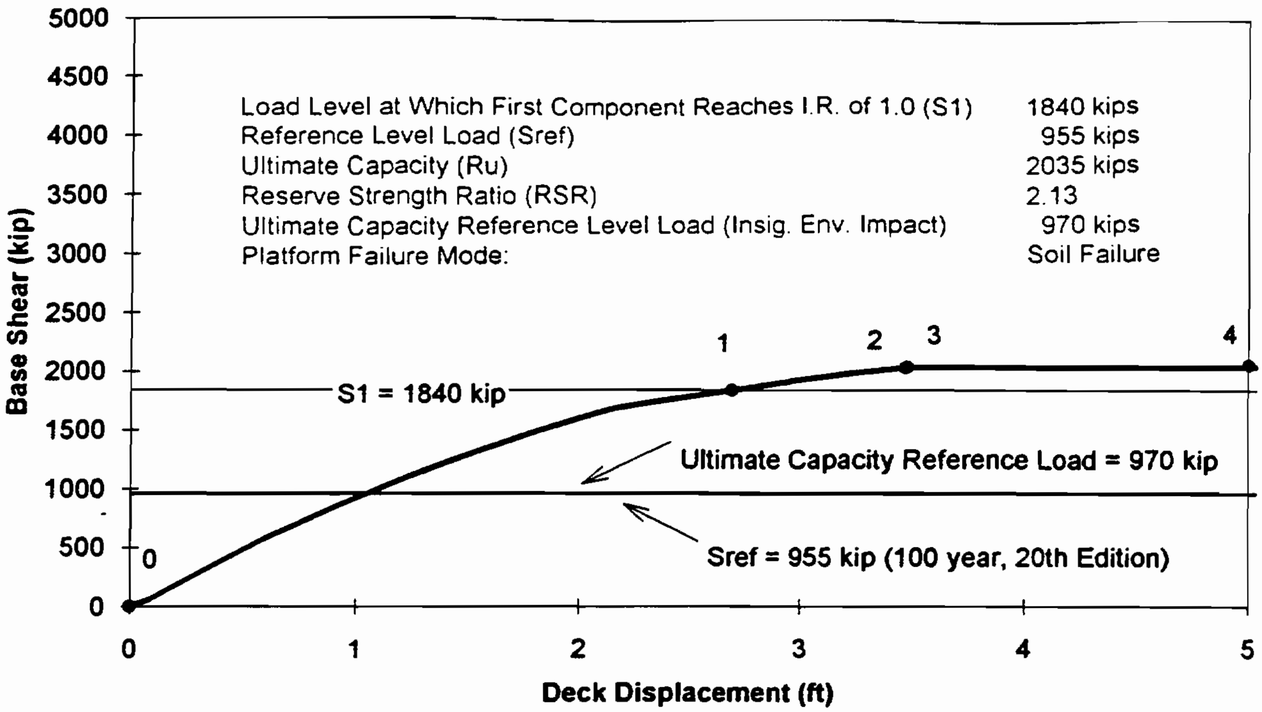
B. Environmental Loads - API 20th Ed Design Wave

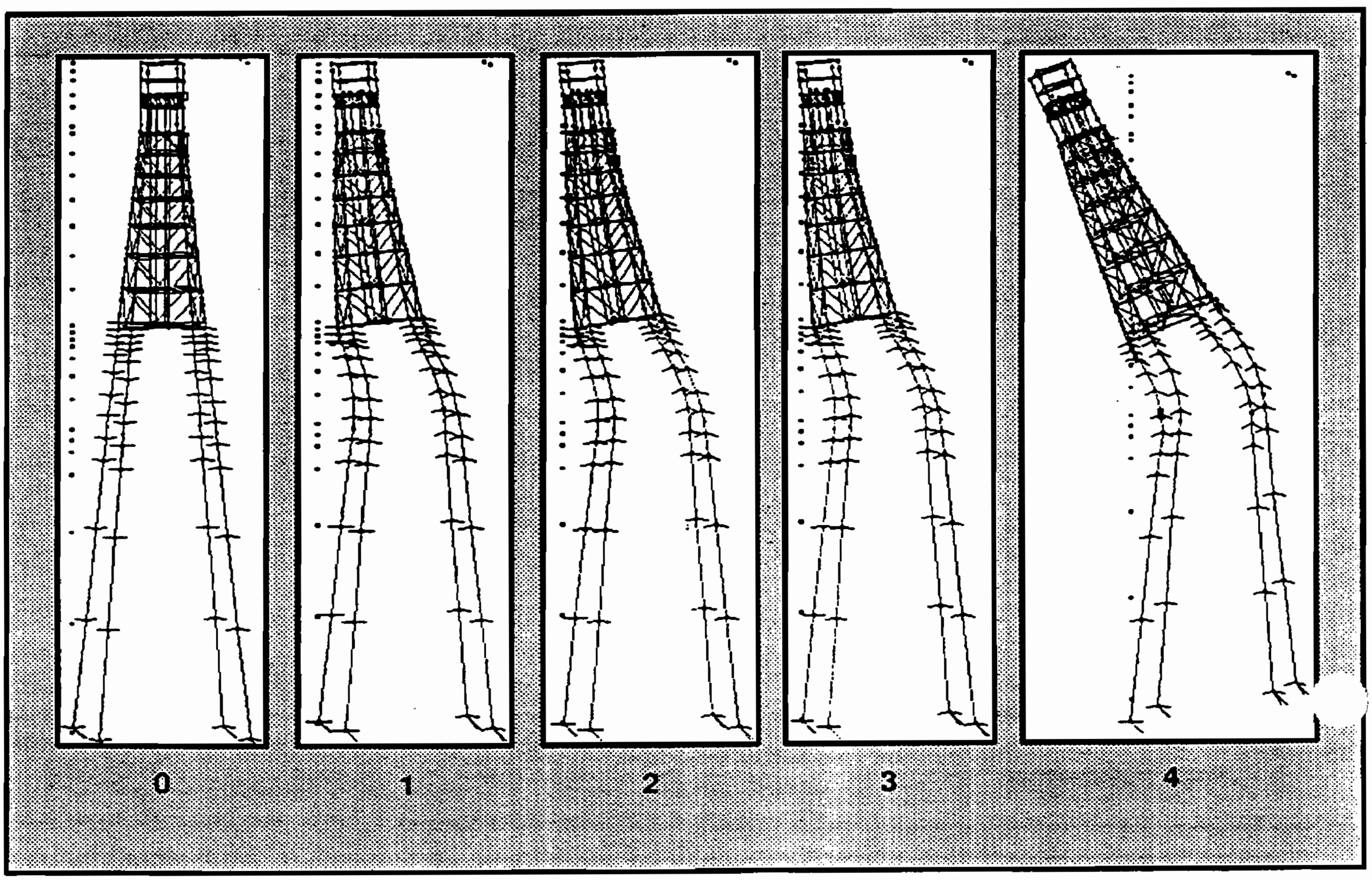
Two separate wave directions have been chosen for analysis: 315 deg. and 360 deg.

Property	315 Deg.	360 Deg.
Orientation WRT Plat. North	SE-NW (Broadside)	S-N (Diagonal)
Wave Height (ft.)	59.85	53.56
Apparent Period (s)	13.65	13.13
Storm Tide (ft.)	3.5	3.5
Current Heading (deg)	277	277
Current Speed (kts)	2.1	2.1
Current Blockage Factor	0.8	0.85
Crest Height (ft.)	44.84	40.19
Crest Velocity (ft/s)	22.43	19.69
Wave-in-deck load (kip)	182	72
Wind Load (kip)	47	33
Current Load (kip):	7.2	9.2
Pushover Wave Load (kip)	1091	841
Reference Level Load In Direction of Wave (kip)	1327	956

Notes:
Water depth = 161.67 ft.
Wave Kinematics factor = 0.88
Wind Speed @ 10m above msl = 135.2 ft/s
Section 17 C17.6.2 used to determine wave/current deck forces

Trials JIP
Diagonal Pushover



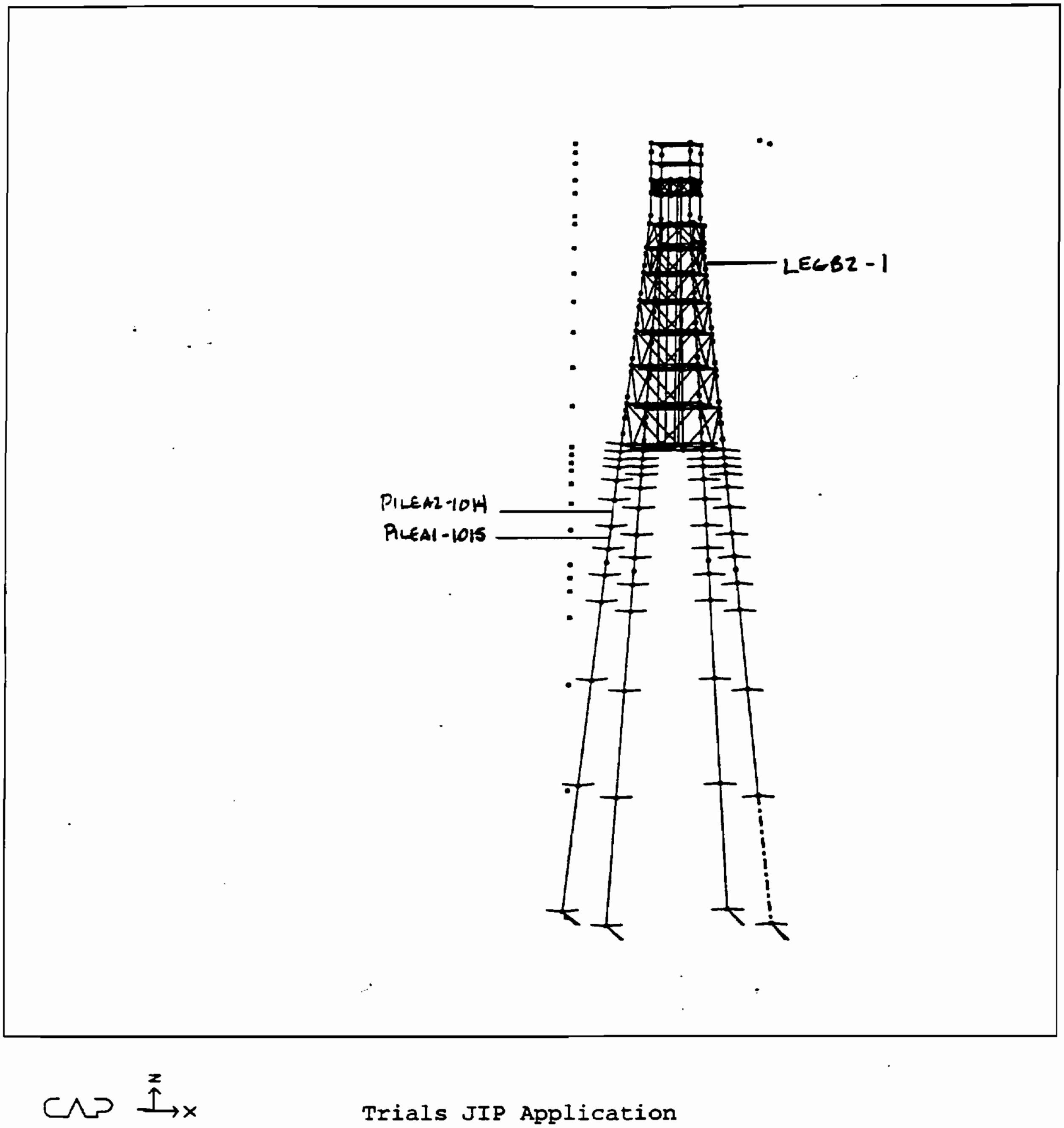


Analysis Case: Diagonal Loading

Lateral Load for First Member with Unity Check = 1.0 is 1840 kips

Load Step	Lateral Displacement at Deck Level (ft)	Lateral Load (kips)	Element Failures	Component Failure Mode	Remarks
15	2.68	1840	LegB2-1	Axial + Bending	
84	3.48	2035	Soil	Plunging	
93	8.15	2055	PileA1-1014	Bending	
93			PileA1-1015	Bending	

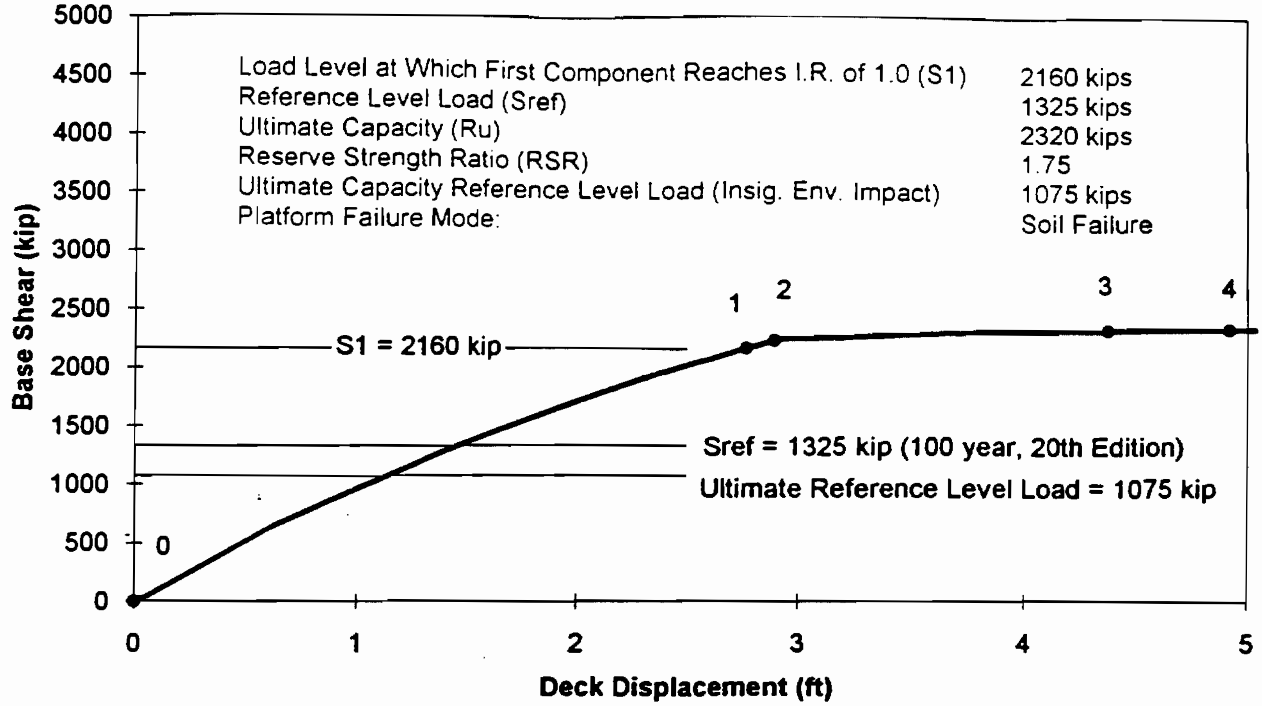
Project: Trial Model: 315 Version: 2   Fri Jul 1 17:22:41 1994 *

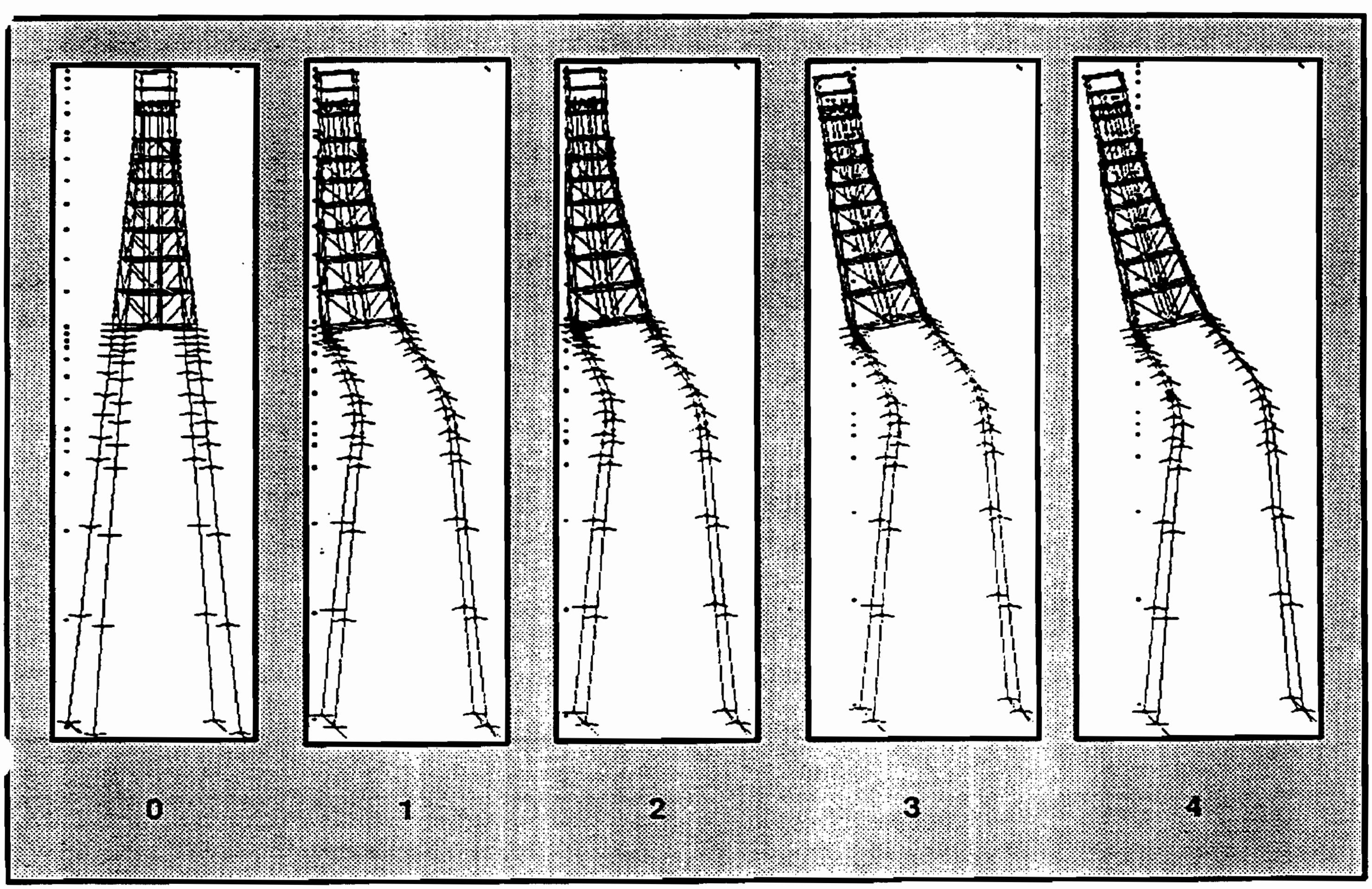


Trials JIP Application

DIAGONAL PUSH

Trials JIP
Broadside Pushover



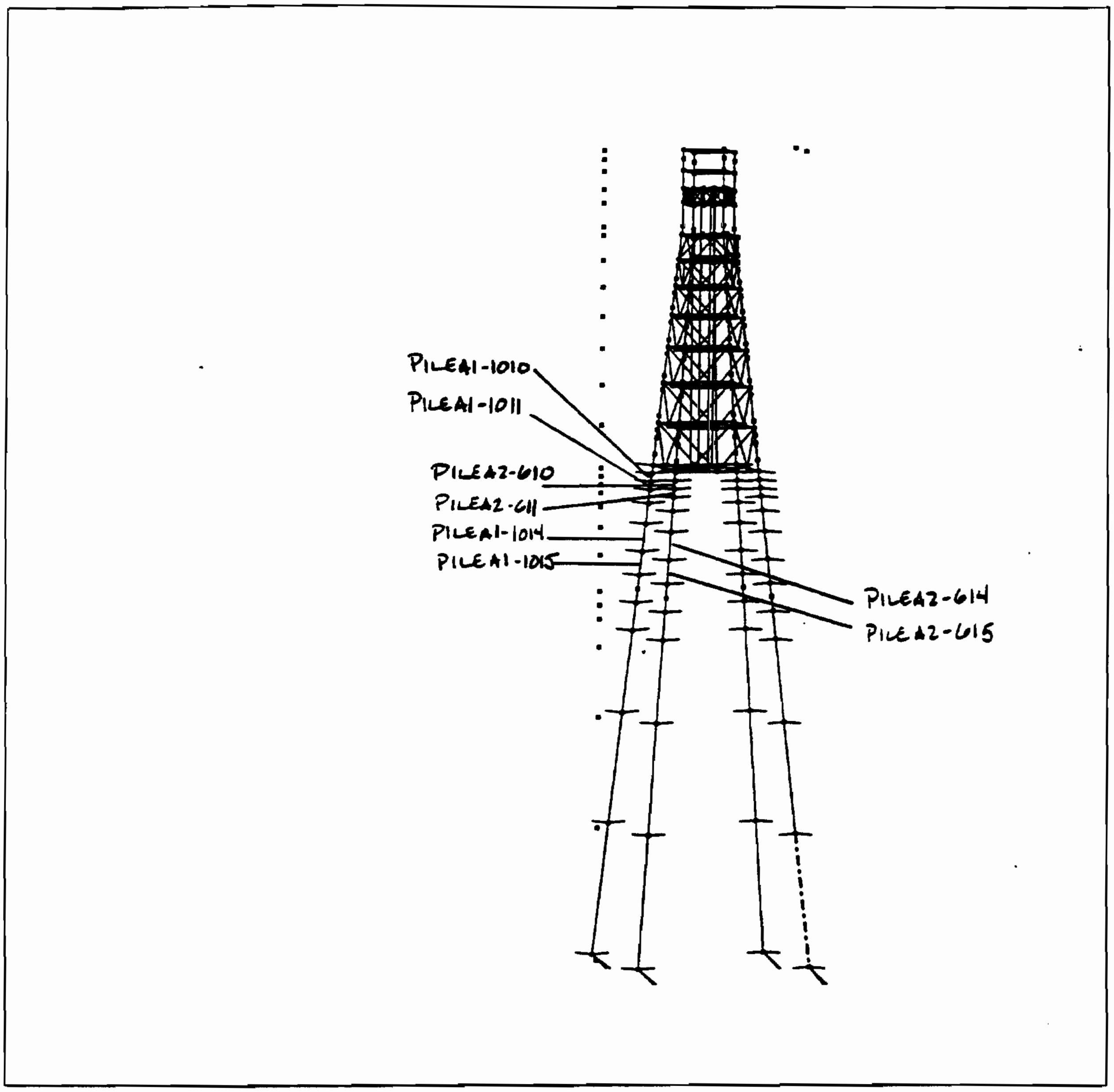


Analysis Case: Broadside Loading

Lateral Load for First Member with Unity Check = 1.0 is 2160 kips

Load Step	Lateral Displacement at Deck Level (ft)	Lateral Load (kips)	Element Failures	Component Failure Mode	Remarks
15	2.77	2160	PileA1-1010	Bending	
15			PileA1-1011	Bending	
17	2.90	2225	PileA2-610	Bending	
17			PileA2-611	Bending	
32	4.37	2320	Soil	Plunging	
33	4.92	2320	PileA1-1014	Bending	
33			PileA1-1015	Bending	
34	5.34	2323	PileA2-614	Bending	
35	5.71	2325	PileA2-615	Bending	

Project: Trial Model: 315 Version: 2 Fri Jul 1 17:22:41 1994 *



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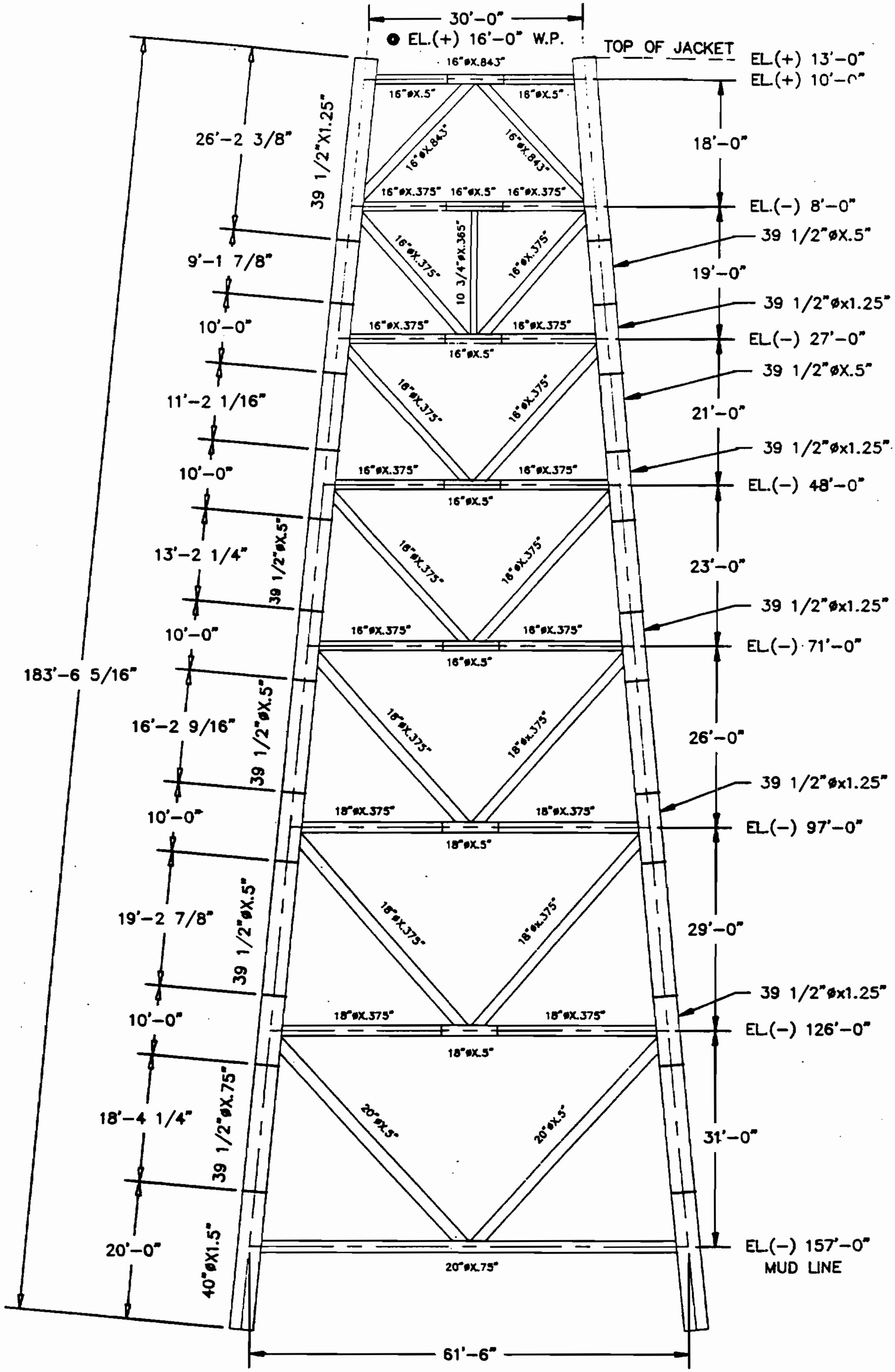
Trials JIP Application

BROADSIDE PUSH

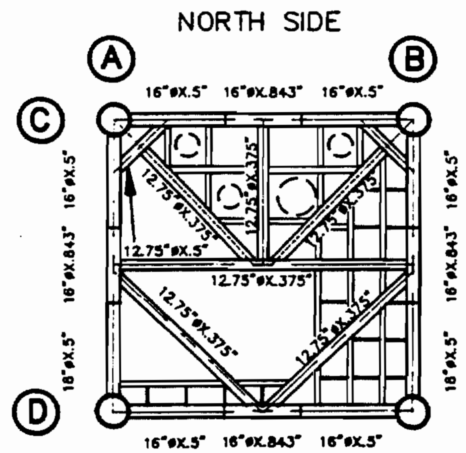
F
PRODUCTION PLATFORM



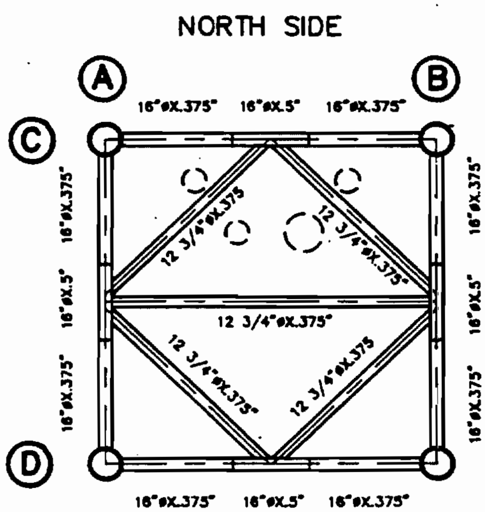
TYPICAL ELEVATION

BOAT LANDINGS & BARGE BUMPER NOT SHOWN

"F"
PRODUCTION PLATFORM
JACKET HORIZONTAL FRAMING

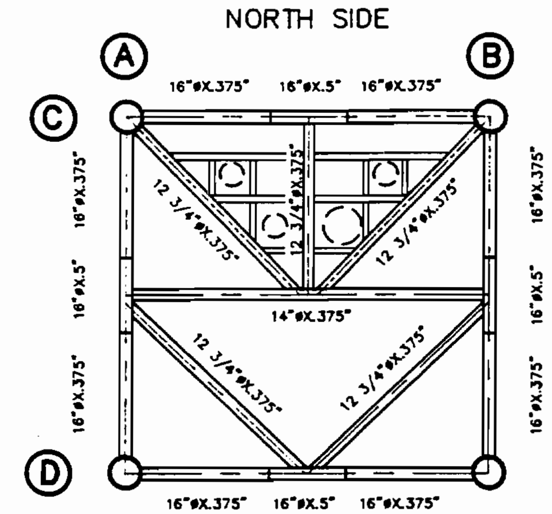


PLAN @ EL.(+) 10'–0"

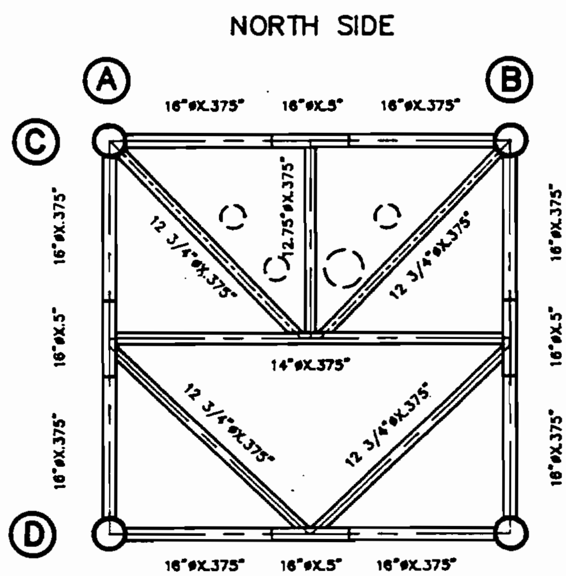


PLAN @ EL.(–) 8'–0"

"F"
PRODUCTION PLATFORM
JACKET HORIZONTAL FRAMING

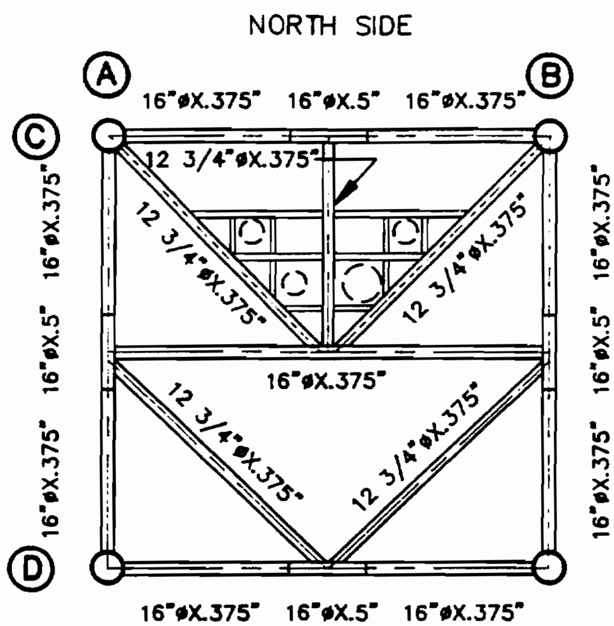


PLAN @ EL.(–) 27'–0"

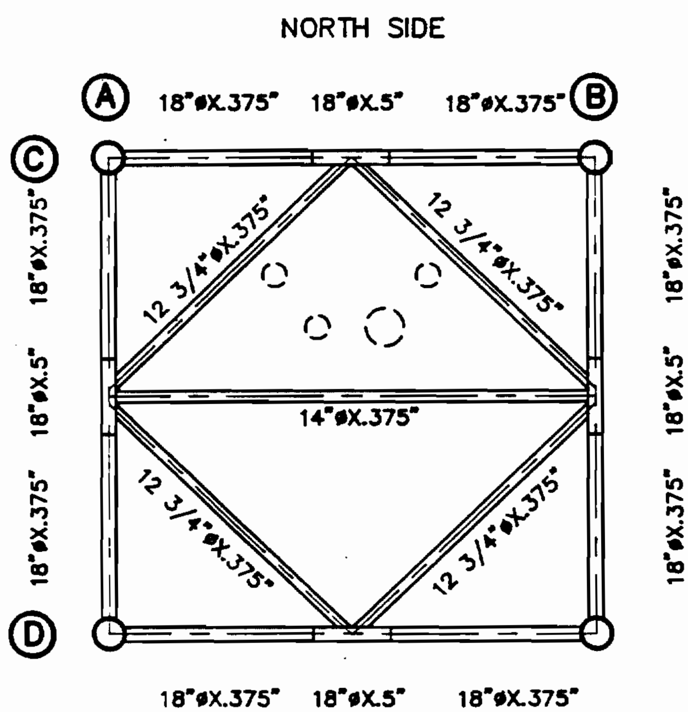


PLAN @ EL.(–) 48'–0"

"F"
PRODUCTION PLATFORM
JACKET HORIZONTAL FRAMING

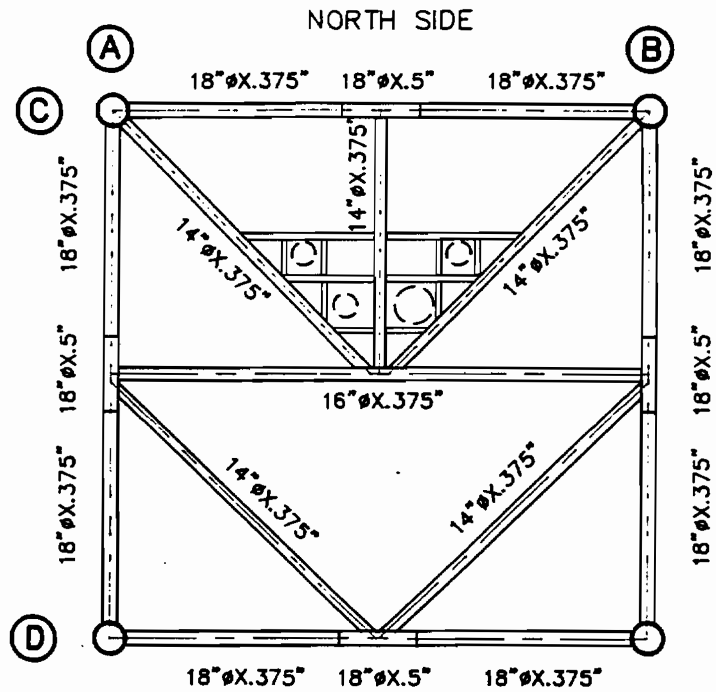


PLAN @ EL.(–) 71'–0"

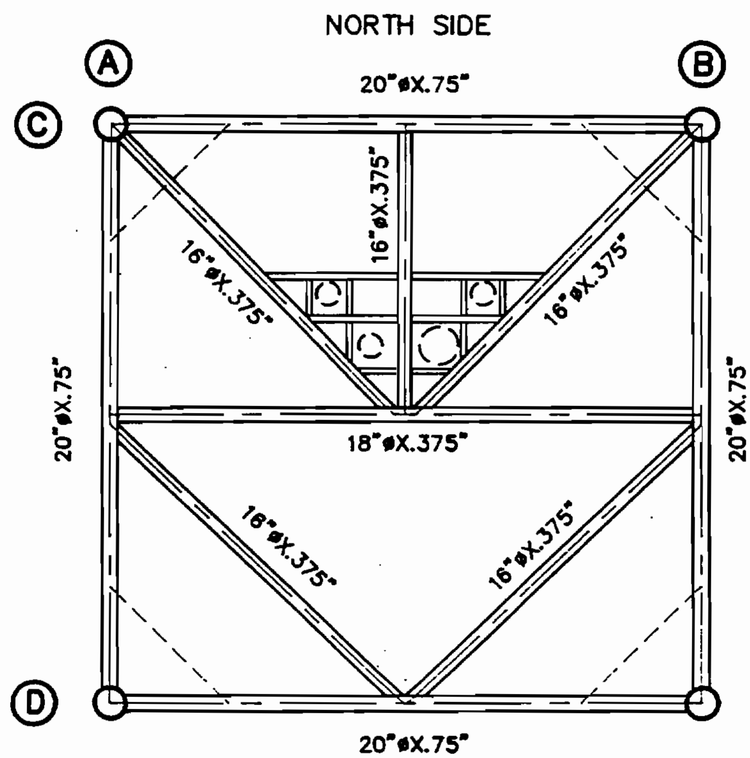


PLAN @ EL.(–) 97'–0"

"F"
PRODUCTION PLATFORM
JACKET HORIZONTAL FRAMING



PLAN @ EL.(−) 126'−0"



PLAN @ EL.(−) 157'−0"

PART B

1. Section 17.5.2 Assessment for Metocean Loading

   This Section makes the following statement

   "For the Gulf of Mexico, design level and ultimate strength Metocean Criteria are explicitly provided, including wave height vs. water depth curves.

   Section 17.6.1 makes a similar but less confusing statement of the criteria given in Section 17.

   " The criteria/loads to be utilized in the assessment of existing platforms should be in accordance with section 2.0 with the exceptions, modifications and/or additions noted herein as a function of exposure category defined in Section 17.3 and applied as outlined in Section. 17.5."

There may be less confusion if after the statement in Section 17.5.2 there was a reference made to see Section 17.6.1.

PLATFORM "G"

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

TABLE OF CONTENTS

API RP 2A PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS
PROPOSED TRIAL PLATFORM

Platform Identification

Operator:	
Platform Location:	Gulf of Mexico Ship Shoal Block 322 Lat: 28° 09' 43.9" Long: 91° 11' 37.2"
Water Depth:	310'

Operational Characteristics

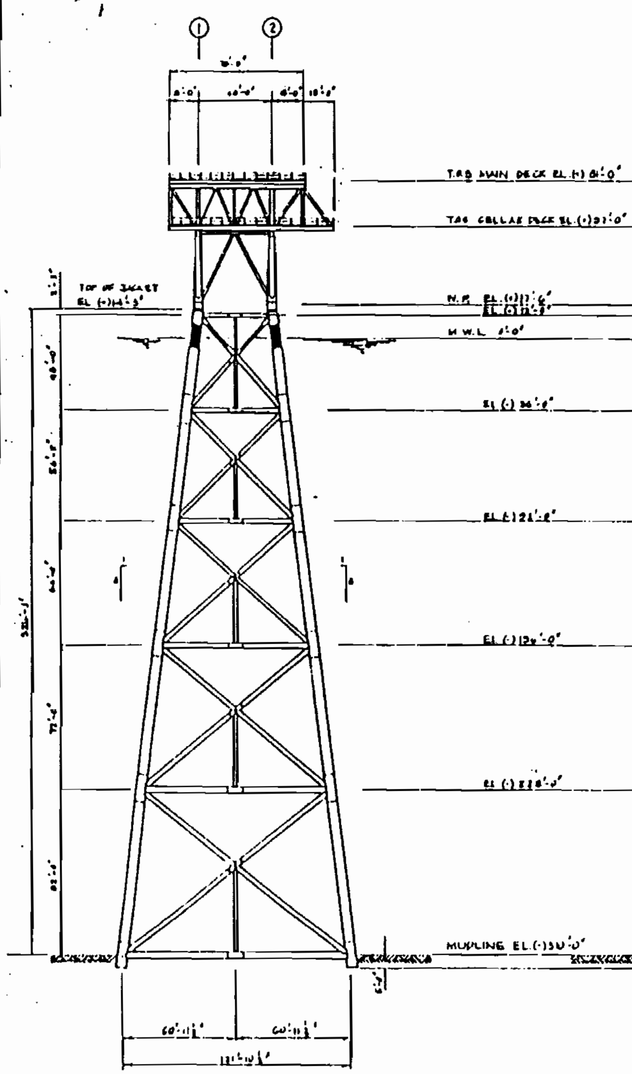
Type of Facility:	Drilling Production, Quarters Manned Evacuated during Storm
No. of Wells:	10
No. of Risers:	2 - 4 inch 1 - 6 inch 2 - control umbilicals

Design Basis

No. of Legs/Piles:	8 Main Piles
Vertical Framing:	Longitudinal - Diagonal Bracing Transverse - X-Bracing
Leg/Pile Annulus:	Ungrouted
Lower Deck Elevation:	(+) 57'-0"

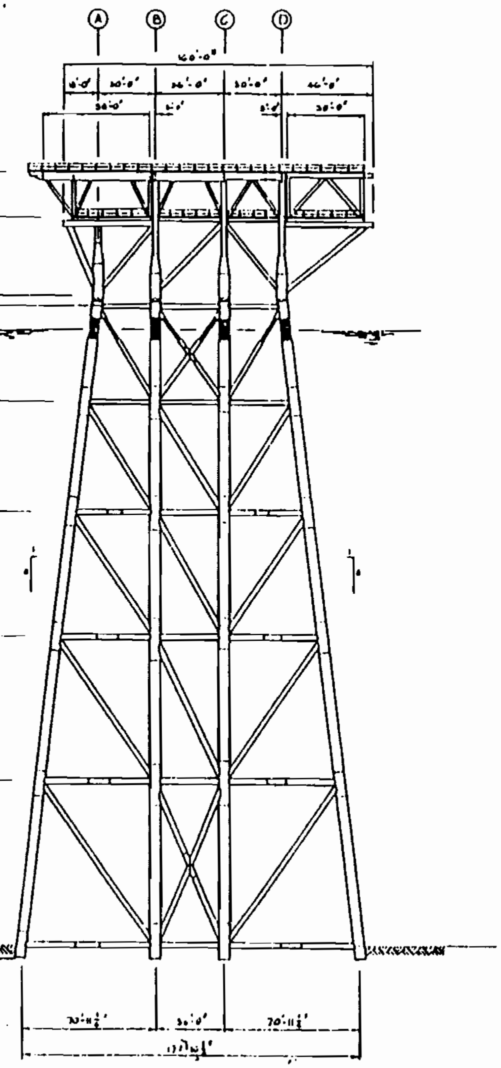
Platform Physical State

Damage to Primary Jacket Members:	None
Modifications to Platform from As-Design State:	Add 2 - 20" Conductors Add 1 - 8" Riser



SOUTH ELEVATION

SCALE: 1"=30'

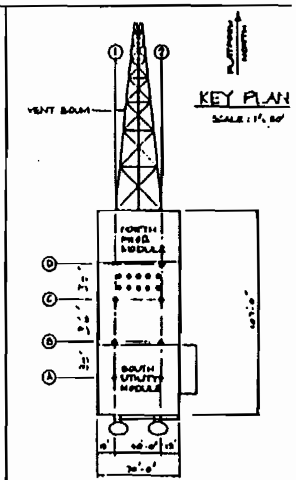


EAST ELEVATION

SCALE: 1"=30'

REFERENCE

- FOR DECK UNIT SEE HQE-312, HQE-315, HQE-314
- FOR VENT BOOM SEE ES-2340
- FOR FENDER SEE ES-2375



KEY PLAN

SCALE: 1"=30'

DRAWING INDEX

1. SOUTH & EAST ELEVATIONS, KEY PLAN AND INDEX
2. JACKET FRAMING ELEVATION AT COLUMN ROW '1'
3. JACKET FRAMING ELEVATION AT COLUMN ROW '2'
4. JACKET FRAMING ELEVATION AT COLUMN ROW 'A'
5. JACKET FRAMING ELEVATION AT COLUMN ROW 'B'
6. JACKET FRAMING ELEVATION AT COLUMN ROW 'C'
7. JACKET FRAMING ELEVATION AT COLUMN ROW 'D'
8. HORIZONTAL FRAMING PLAN AT ELEVATION (+)12'-0"
9. DETAILS & SECTIONS OF HORIZ. FRG. PLAN AT ELEVATION (+)12'-0"
10. HORIZONTAL FRAMING PLAN AT ELEVATION (-)36'-0"
11. HORIZONTAL FRAMING PLAN AT ELEVATION (-)92'-0"
12. HORIZONTAL FRAMING PLAN AT ELEVATION (-)156'-0"
13. HORIZONTAL FRAMING PLAN AT ELEVATION (-)228'-0"
14. HORIZONTAL FRAMING PLAN AT ELEVATION (-)310'-0"
15. PILE TO JACKET CONNECTIONS
16. JOINT CRITERIA AND MISCELLANEOUS DETAILS
17. △ PLACEMENT OF ANODE, REFERENCE ELECTRODE AND SUB-SEA MARKER SYSTEMS.
18. MAIN PILES AND CONDUCTORS
19. PILE AND CONDUCTOR DETAILS
20. LAUNCHING WAYS ARRANGEMENT
21. LAUNCH CRADLE AND DETAILS
22. PULLING LUG & LIFTING EYE DETAILS
23. JACKET LEG FLOODING SYSTEM
24. CURVED CONDUCTOR DETAIL
25. MUDMAT DETAILS
26. △ REFERENCE ELECTRODE SYSTEM DETAILS
27. △ REFERENCE ELECTRODE SYSTEM DETAILS

NOTES

1. MATERIAL: UNLESS NOTED, STEEL SHALL BE ASTM A36, EXCEPT HEAVY WALL JOINTS WHICH SHALL BE API 2H GR. 42 STEEL.
2. ALL WELDS SHALL BE FULL STRENGTH CONTINUOUS BEVELED WELDS UNLESS NOTED.
3. THE NUMBER FOLLOWING A SECTION OR DETAIL DESIGNATION DENOTES THE SHEET NUMBER ON WHICH IT APPEARS. EXAMPLE: A-5 A-5 (LOCATED ON SHT 5)

SOUTH & EAST ELEVATIONS, KEY PLAN & INDEX

NO.	DATE	REVISIONS	BY	CHK.	APPD.	NO.	DATE	REVISIONS	BY	CHK.	APPD.

JACKET UNIT
310' WATER, 10 WELL
... 2 LEVEL DECK
FLUORITE PROSPECT

HQE-311-1

Ship Shoal 322 A
Inspection Summary

The Ship Shoal 322 A platform, an eight (8) leg design sitting in 312 feet of water, was inspected by Global Divers & Contractors, Inc. on August 16, 1993. There are three (3) risers and nine (9) conductors on this platform. The diving support vessel used was the M/V "Global Diver 111", a 110 foot utility boat. The company representative for Diving operations were supervised by Byron Gray under s specifications for Level II Inspection Surveys.

Underwater Visual Inspection

A general visual survey of the structure from the waterline to the mudline was performed by the divers. Each leg, riser, and support member was visually inspected. All riser clamps were intact and tight. No areas of underwater structural damage were reported.

Cathodic Protection

Cathodic potential (CP) readings were taken, and they ranged from -0.910 volts to -1.030 volts. The structures anodes were approximately 10% depleted. Voltage readings less than -.800 volts indicate less than adequate protection by the anodes.

Bottom Survey

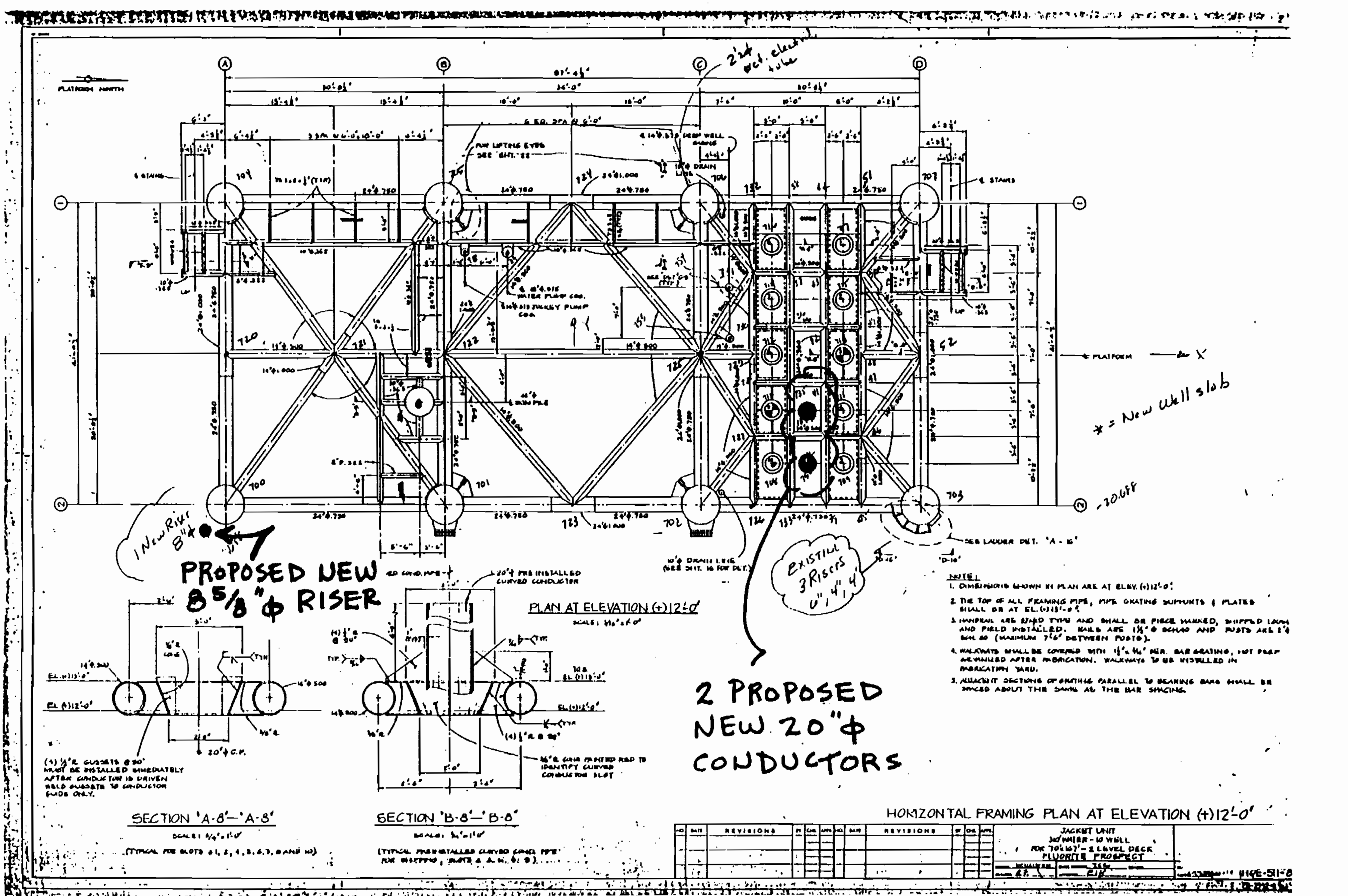
A bottom survey was conducted and depth readings were taken at the bottom of each leg. During the bottom survey and visual swim-by, a debris survey was performed. There is a considerable amount of debris at the base, and hung up in the structure. See **Debris Surveys** for detailed findings.

Marine Growth

Marine growth was also surveyed; and the types of growth were recorded along with the depths at which they occur on the structure.

Topside Inspection

A visual and topside coating inspection from the waterline to the first production level was conducted. The boat landings are in good condition, however, the rubber bumpers are missing. There are several stair treads (D1 - D2 Stairway) that may need replacement in the near future. The overall platform condition is good, and the topside coating looked good. The swing ropes need to be five (5) feet longer.



PLATFORM NORTH
PROPOSED NEW 8 5/8"ϕ RISER
1 New Riser 8"ϕ
2 PROPOSED NEW 20"ϕ CONDUCTORS
Existing 3 Risers 6", 4", 4"
2 1/2"ϕ elect. tube
* = New Well slab
PLATFORM
SEE LADDER DET. 'A-16'
PLAN AT ELEVATION (+)12'-0"
SECTION 'A-8'—'A-8'
SECTION 'B-8'—'B-8'
NOTES
1. DIMENSIONS SHOWN IN PLAN ARE AT ELEV. (+)12'-0".
2. THE TOP OF ALL FRAMING PIPE, PIPE GRATING SUPPORTS & PLATES SHALL BE AT EL. (+)13'-0".
5. ADJACENT SECTIONS OF GRATING PARALLEL TO BEARING BARS SHALL BE SPACED ABOUT THE SAME AS THE BAR SPACING.
HORIZONTAL FRAMING PLAN AT ELEVATION (+)12'-0"
JACKET UNIT
FLUORITE PROSPECT

TRIAL APPLICATION OF API RP 2A-WSD,
20th ED, DRAFT SECTION 17,
PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM SHIP SHOAL 322-A; 310' W.D.

2.1 ASSESSMENT INITIATION (Section 17.2)

When a modification is proposed for an existing platform the following assessment initiators must be evaluated:

- Addition of Personnel - If additional personnel are proposed to a more restrictive level, the platform must be assessed.

- Addition of Facilities - If significant additional facilities loading is proposed, the platform must assessed.

- Increased Lateral Loading - If the proposed modification causes significant additional lateral load, the platform must be assessed.

- Inadequate Deck Height - If the platform has an inadequate deck height for its exposure category and was not designed for wave load on the deck, an assessment must be performed whether or not a modification is planned.

- Damage Found During Inspection - If significant damage to a primary structural member is discovered during an inspection, an assessment must be performed whether or not a modification is planned.

The above initiators are evaluated, as they relate to this selected platform, in the following Sections 2.1.1, 2.1.2, 2.1.3, 2.1.4, and 2.1.5.

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.1.1 Addition of Personnel

This initiator suggests there is no need to assess the selected platform because no additional personnel are planned.

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.1.2 Addition of Facilities

This initiator suggests there is no need to assess the selected platform because no additional facilities are planned.

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.1.3 Increased Loading on Platform

The modifications planned for the selected platform include the addition of two (2) new 20" dia. conductors and one (1) new 8-5/8" dia. riser. This modification indicates the need to determine if the increased lateral load on these additions is significant (greater than 10%). The method used in this study to make this determination was to use API RP 2A-WSD, 20th ED guidelines to develop and compare environmental load on models of the platform without and then with the proposed modifications.

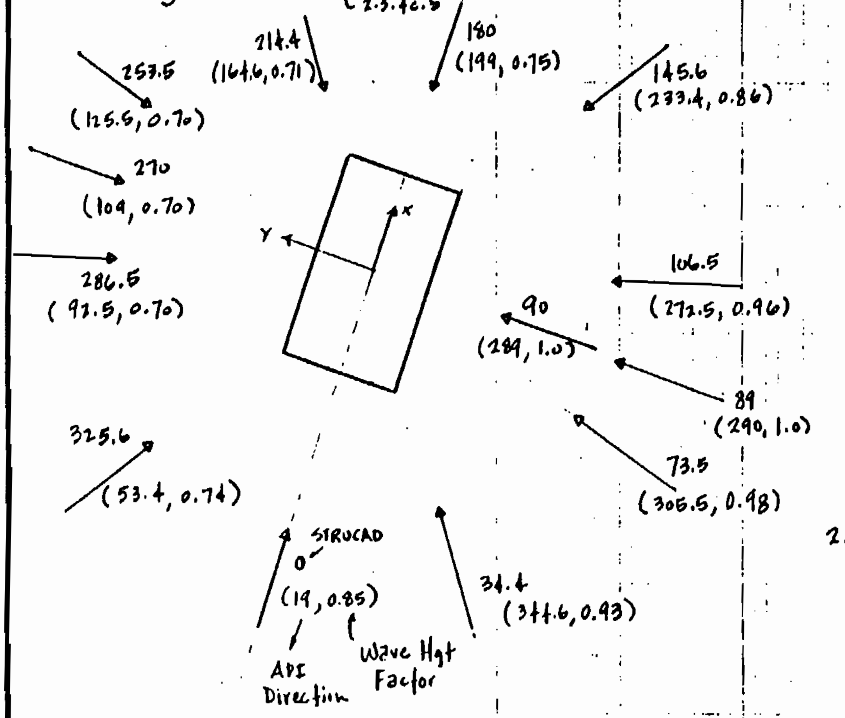
The following pages present information related to the determination of the lateral load on the platform without and with the proposed modifications. The information is as outlined below:

- Development of API RP 2A-WSD, 20th ED environmental data.
- Input description and load results for platform w/o modifications.
- Input description and load results for platform w/two (2) additional 20" dia. wells and one new 8-5/8" dia. riser.
- Summary of results of environmental loads.

The summary of results of environmental loads on the platform without and with the proposed modifications show a maximum increase in base shear of 3.41%. Since the maximum increase is less than 10% this indicator suggests there is no need to assess the structure further.

1.0 Determine apparent wave period.

$d/gT^2 = \frac{(310+3)}{32.2\,(13)^2} = 0.058 < .04 < .10$ Fig. 2.3.4-7 / 2.3.4c.5



STRUCAD Wave Direction	V	Wave Hgt Factor	V_I	V_I/gT	T_{app}/T	T_{app} ✓
0	3.547	0.85	3.015	.007	1.044	13.58
34.4		0.93	3.298	.008	1.05	13.65
73.5		0.98	3.476	.008	1.05	13.65
89		1.0	3.547	.008	1.05	13.65
90		1.0	3.547	.008	1.05	13.65
106.5		0.96	3.405	.008	1.05	13.65
145.6		0.86	3.050	.007	1.044	13.58
180		0.75	2.660	.006	1.042	13.55
214.4		0.71	2.518	.006	1.042	13.55
253.5		0.70	2.483	.006	1.042	13.55
270		0.70	2.483	.006	1.042	13.55
286.5		0.70	2.483	.006	1.042	13.55
325.6		0.74	2.625	.006	1.042	13.55

2.0 Determine Wave Height

Guideline wave Hgt = 68.3' → Fig. 2.3.4-3

STRUCAD Wave Direction	Guideline Wave Hgt,	Wave Hgt Factor	Wave Hgt
0	68.3'	0.85	58.1
34.4		0.93	63.5
73.5		0.98	66.9
89		1.0	68.3
90		1.0	68.3
106.5		0.96	65.6
145.6		0.86	58.7
180		0.75	51.2
214.4		0.71	48.5
253.5		0.70	47.8
270		0.70	47.8
286.5		0.70	47.8
325.6		0.74	50.5

3.0 Determine Storm Water Depth

d = 310 + 3 = 313' Fig 2.3.4-7

CLIENT:

PROJECT: SS 32

JOB NO.: 1441

DRAWN BY: K. M. Dewat

SCALE:

TE: 7/1/9

4.0 Determine Wave Theory - Fig 2.3.1-3

STRUCAD Wave Direction	H	d	Tapp	H/gT_{app}^2	d/gT_{app}^2	Wave Theory
0	58.1	313	13.58	.010	.053	STREAM3
34.4	63.5		13.65	.011	.052	
73.5	66.9		13.65	.011	.052	
89	68.3		13.65	.011	.052	
90	68.3		13.65	.011	.052	
106.5	65.6		13.65	.011	.052	
145.6	58.7		13.58	.010	.053	
180	51.2		13.55	.009	.053	
214.4	48.5		13.55	.008	.053	
253.5	47.8		13.55	.008	.053	
270	47.8		13.55	.008	.053	
286.5	47.8		13.55	.008	.053	
325.6	50.5		13.55	.009	.053	

5.0 Wave Kinematics Factor - 0.88 per 2.3.4d.1.

6.0 Current Blockage Factor -

STRUCAD Wave Direction	Current Blockage Factor
0	0.70
34.4	0.85
73.5	0.85
89	0.80
90	0.80
106.5	0.85
145.6	0.85
180	0.70
214.4	0.85
253.5	0.85
270	0.80
286.5	0.85
325.6	0.85

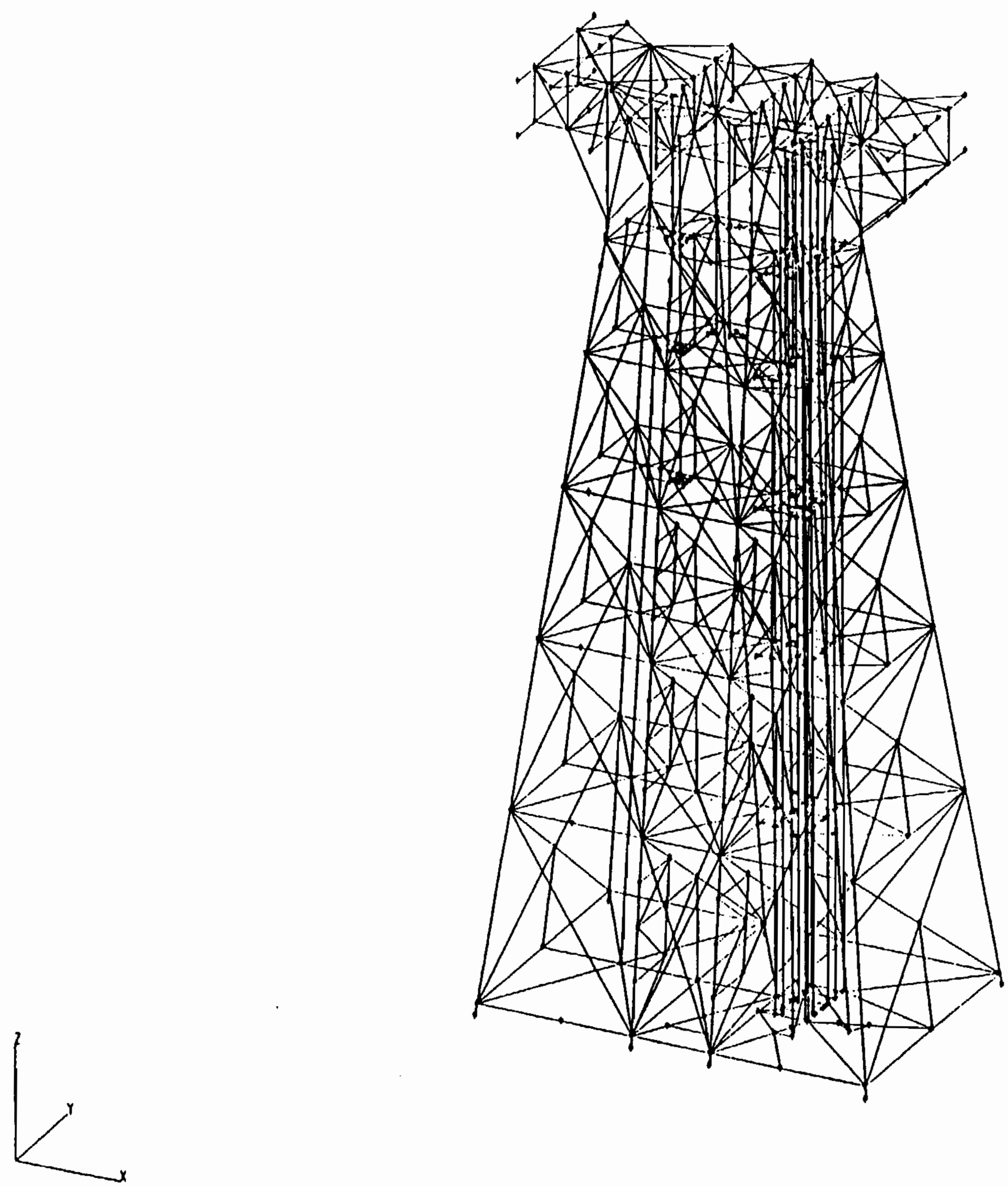
7.0 Determine Conductor Sheilding -

STRUCAD Wave Direction	S/D	Sheilding Factor
0	6	1.0
34.4	6+	
73.5	6+	
89	4.2	
90	> 4.0	
106.5		
145.6		
180		
214.4		
263.5		
270		
286.5		
325.6		

CLIENT:
PROJECT: SS 322 A
JOB NO.: 1441
TITLE: API RP 2A-WSD, 20th Ed. Environmental
DRAWN BY: K.M. Dewalt
SCALE:
DATE: 7/9/9
DWG NO.: 2 of 2

SHIP SHOAL 322-A

3-D VIEW - PLATFORM W/O MODIFICATIONS



STRUCTURAL SOFTWARE, INC

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             * * *    Echo Of Input Data - PREP    * * *

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            (P-SHIP SHOAL 322A- API 20TH/NO STRUCT MOD-INP20A-1441



               8-PILE DRILLING AND PRODUCTION PLATFORM
               SHIP SHOAL 322-A; 310' W.D.

      FILENAME:       S:\1441\ES\INP20A
      ENGINEER:       BLR, MCH, ABJ, KMD
      RUN DATE:       08-03-94

      GENERAL:   STUDY COMPARING STORM LOADS DEVELOPED USING API RP 2A-WSD,
                 20TH ED GUIDELINES FOR PLATFORM AS-IS WITHOUT ANY MODIFICATIONS
                 VS PLATFORM WITH TWO (2) ADDITIONAL 20" DIA CONDUCTORS (ONE (1)
                 BETWEEN SLOTS 4 AND 9, AND ONE (1) BETWEEN SLOTS 5 AND 10) AND
                 A NEW 8" RISER AT LEG A-2


                 1)  S:\1441\ES\INP20A <===== THIS MODEL

                     - PLATFORM AS-IS WITHOUT ANY MODIFICATIONS

                     - API RP 2A-WSD, 20TH ED GUIDELINES TO DEVELOP
                       ENVIRONMENTAL LOADS

                 2)  S:\1441\ES\INP20B

                     - PLATFORM WITH MODIFICATIONS AS FOLLOWS:

                       o TWO (2) ADDITIONAL 20" DIA CONDUCTORS; ONE (1)
                         BETWEEN SLOTS 4 AND 9, AND ONE (1) BETWEEN SLOTS
                         5 AND 10

                       o A NEW 8" RISER AT LEG A-2

                     - API RP 2A-WSD, 20TH ED GUIDELINES TO DEVELOP
                       ENVIRONMENTAL LOADS

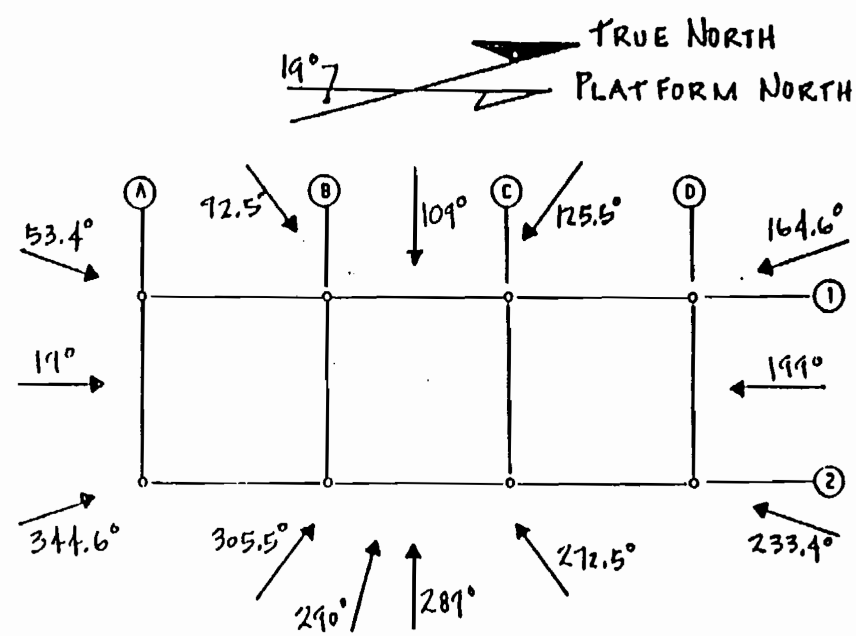


        MODEL S:\1441\ES\INP20A DATA
        ============================

                 1.  60" MAIN PILES
                 2.  10- 20" DIAMETER CONDUCTORS (EXISTING), TWO (2) OF THESE
                     BEING CURVED.
                 3.  THREE (3) EXISTING RISERS AT NORTHEAST CORNER - ONE (1)
```

SHIP SHOAL 322-A

API STORM DIRECTION	CASE I BASE SHEAR	CASE II BASE SHEAR	% INCREASE
19 DEG	3943	4059	2.94
344.6 DEG	4992	5153	3.23
305.5 DEG	5576	5766	3.41
290 DEG	5702	5896	3.40
289 DEG	5707	5901	3.40
272.5 DEG	5354	5536	3.40
233.4 DEG	4268	4404	3.19
199 DEG	3117	3205	2.82
164.6 DEG	2978	3065	2.92
125.5 DEG	2915	3004	3.05
109 DEG	2854	2940	3.01
92.5 DEG	2887	2975	3.05
53.4 DEG	3416	3520	3.04



NOTES:

1) CASE I LOADS ARE BASED ON THE FOLLOWING:

- PLATFORM AS-IS WITHOUT ANY MODIFICATIONS
- API RP 2A-WSD, 20TH ED GUIDELINES ARE USED TO DEVELOP ENVIRONMENTAL LOADS

2) CASE II LOADS ARE BASED ON THE FOLLOWING:

- PLATFORM WITH MODIFICATIONS AS FOLLOWS:
  - o TWO (2) ADDITIONAL 20" DIA CONDUCTORS; ONE (1) BETWEEN SLOTS 4 AND 9, AND ONE (1) BETWEEN SLOTS 5 AND 10
  - o A NEW 8" RISER AT LEG A-2
- API RP 2A-WSD, 20TH ED GUIDELINES ARE USED TO DEVELOP ENVIRONMENTAL LOADS

3) % INCREASE IS CALCULATED AS FOLLOWS:

([CASE II BASE SHEAR - CASE I BASE SHEAR] / CASE I BASE SHEAR) * 100

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.1.4 Inadequate Deck Height

The minimum deck height recommended by Draft Section 17 in 310' water depth is 36.5' for sudden hurricanes (Ref. Fig. 17.6.2-3b) and 44.5' for full population hurricanes (Ref. Fig. 17.6.2-2b). The deck height for the selected platform is 52.5', so this indicator suggests there is no need to assess the platform.

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.1.5 Damage Found During Inspections

This indicator suggests there is no need to assess the selected platform because a recent platform survey/inspection found no significant damage to any of the primary structural framing. A copy of the survey/inspection report is included in Section 1.5.

TRIAL APPLICATION OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17, PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.2 EXPOSURE CATEGORY (Section 17.3)

Exposure categories are established by Draft Section 17 for life safety and environmental impact. Available categories are as follows:

- Life Safety
  - Manned, Non-Evacuated
  - Manned, Evacuated
  - Unmanned
- Environmental Impact
  - Significant Environmental Impact
  - Insignificant Environmental Impact

The selected platform is normally manned except during a forecasted design environmental event and is so categorized for life safety as "Manned, Evacuated". The selected platform is not located near a populated area, has no significant liquid hydrocarbon storage facilities, and does not process sour gas. Accordingly, the platform is categorized for environmental impact as "Insignificant Environmental Impact".

TRIAL APPLICATION OF API RP 2A-WSD,
20th ED, DRAFT SECTION 17,
PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM SHIP SHOAL 322-A; 310' W.D.

2.3 PLATFORM SURVEYS (Section 17.4)

A Level II survey of the selected platform was performed in August 1993. The survey found the platform to be in good condition structurally. A copy of the survey is included in Section 2.1.5.

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.4 ANALYTICAL ASSESSMENT (Sections 17.5, 17.6, and 17.7)

An analytical assessment is necessary if a platform fails any of the five (5) assessment indicator tests stated in Draft Section 17.2 and as discussed previously herein in Section 2.1. The selected platform passes all the indicator tests but an analytical assessment has been performed to provide additional data to be used in the evaluation of Draft Section 17.

A fatigue analysis was not performed because no damage was detected during the platform survey. If a recent survey had not been performed then a fatigue analysis using API RP 2A-WSD, 20th Ed guidelines would have been necessary.

**TRIAL APPLICATION OF API RP 2A-WSD,
20th ED, DRAFT SECTION 17,
PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS**

PARTICIPANT
SELECTED PLATFORM SHIP SHOAL 322-A; 310' W.D.

2.4.1 Design Level Analysis (Section 17.5.2.3)

The pretense is made that an assessment of the selected platform is necessary to determine if the structure is adequate to support the proposed modifications. Accordingly, a design level analysis was performed using "Sudden Hurricane" environmental data for "Insignificant Environmental Impact/Manned-Evacuated" platforms as provided in Draft Section 17, Table 17.6.2-1. Data not given in this table was taken from API RP 2A-WSD, 20th Ed, Section 2 as recommended in Draft Section 17, Section 17.6.1 and 17.6.2.

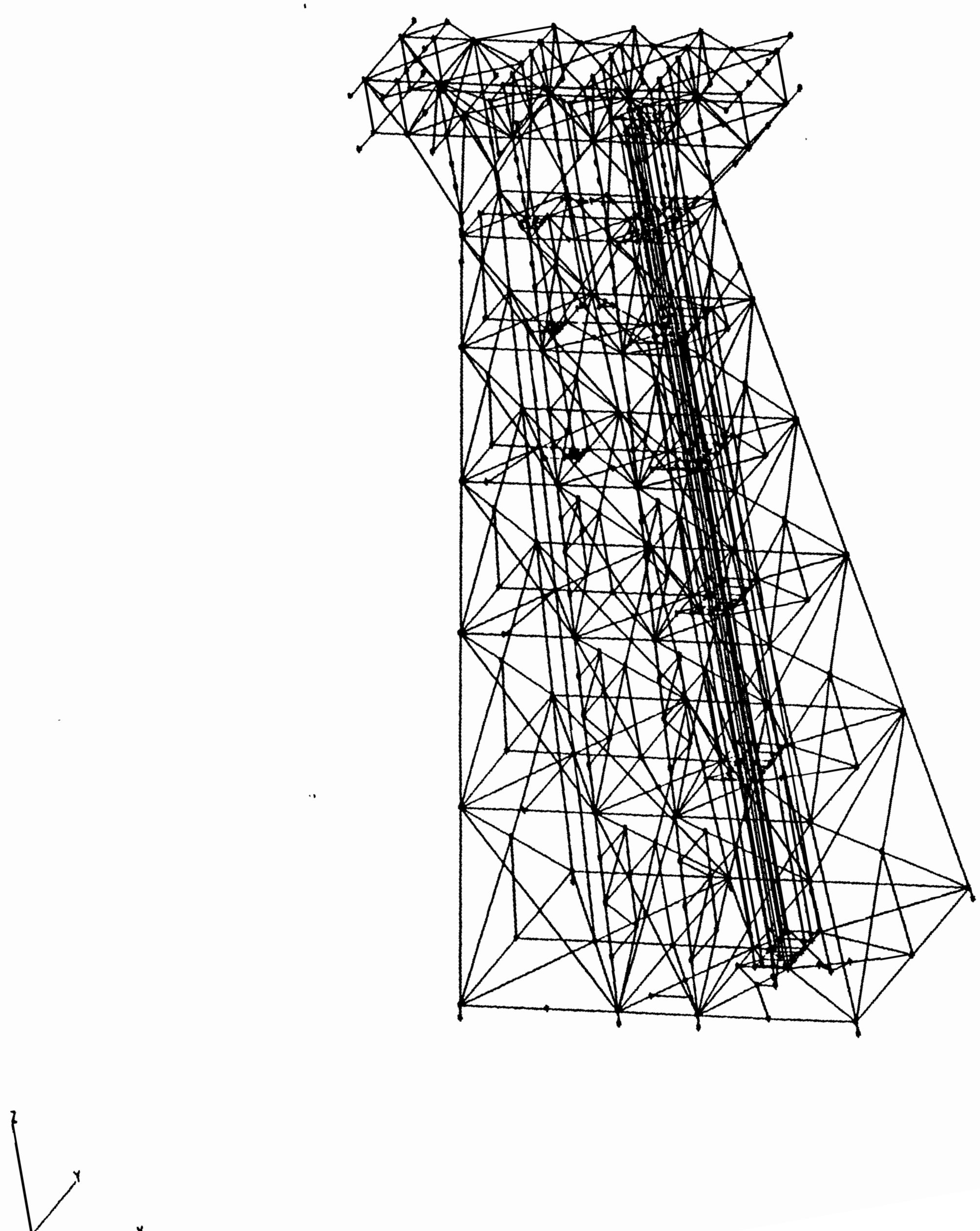
The following information pertaining to the design level analysis is included in this section:

- Computer Model Plots
- Description of Computer Input and Load Cases
- Load Summary Output
- Pile Load Summaries
- Member Check Summaries
- Joint Punching Shear Summary

The results of this analysis are that stress levels for members and joints are all relatively low and that the platform is adequate globally to support the proposed additions.

SHIP SHOAL 322-A

3D VIEW

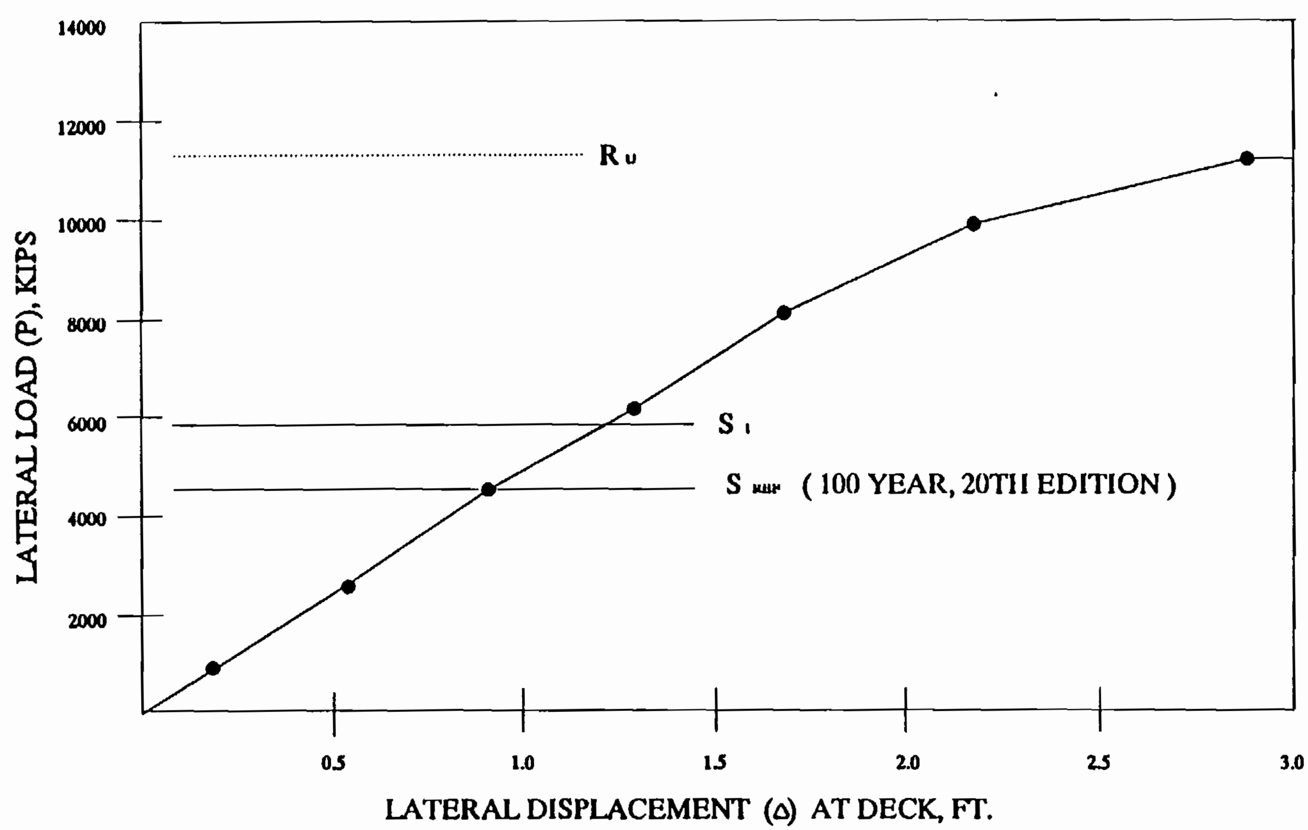


STRUCTURAL SOFTWARE, INC

LOAD STEP	LATERAL DISPLACEMENT AT DECK LEVEL FT.	LATERAL LOAD KIPS	ELEMENT FAILURES	COMPONENT FAILURE MODE	REMARKS
1	0.181	906			
2	0.362	1811			
3	0.543	2717			
4	0.724	3622			
5	0.910	4528			
6	1.094	5433			
7	1.280	6339	521-522	"PLASTICITY"	
8	1.478	7245			
9	1.684	8150	307-342	"PLASTICITY"	
10	1.916	9056	400-420, 386-186	"PLASTICITY"	
11	2.172	9962	526-575, 686-586, 382-182, 803-683, 405-406, 500-521, ETC.	"PLASTICITY"	
12	2.509	10867	423-426, 401-402, 501-502, 502-552, 602-603,582-482, ETC.	"PLASTICITY"	
13	2.880	11320	426-431, 707-787, 400-301, 404-305, 504-405, 10-187, ETC.	"PLASTICITY"	
14			STRUCTURE COLLAPSES!!		
15					

LATERAL LOAD FOR FIRST MEMBER WITH UNITY CHECK = 1.00 : 5886 KIPS

TABLE 1: ULTIMATE STRENGTH ANALYSIS RESULTS FOR 0 DEGREE WAVE



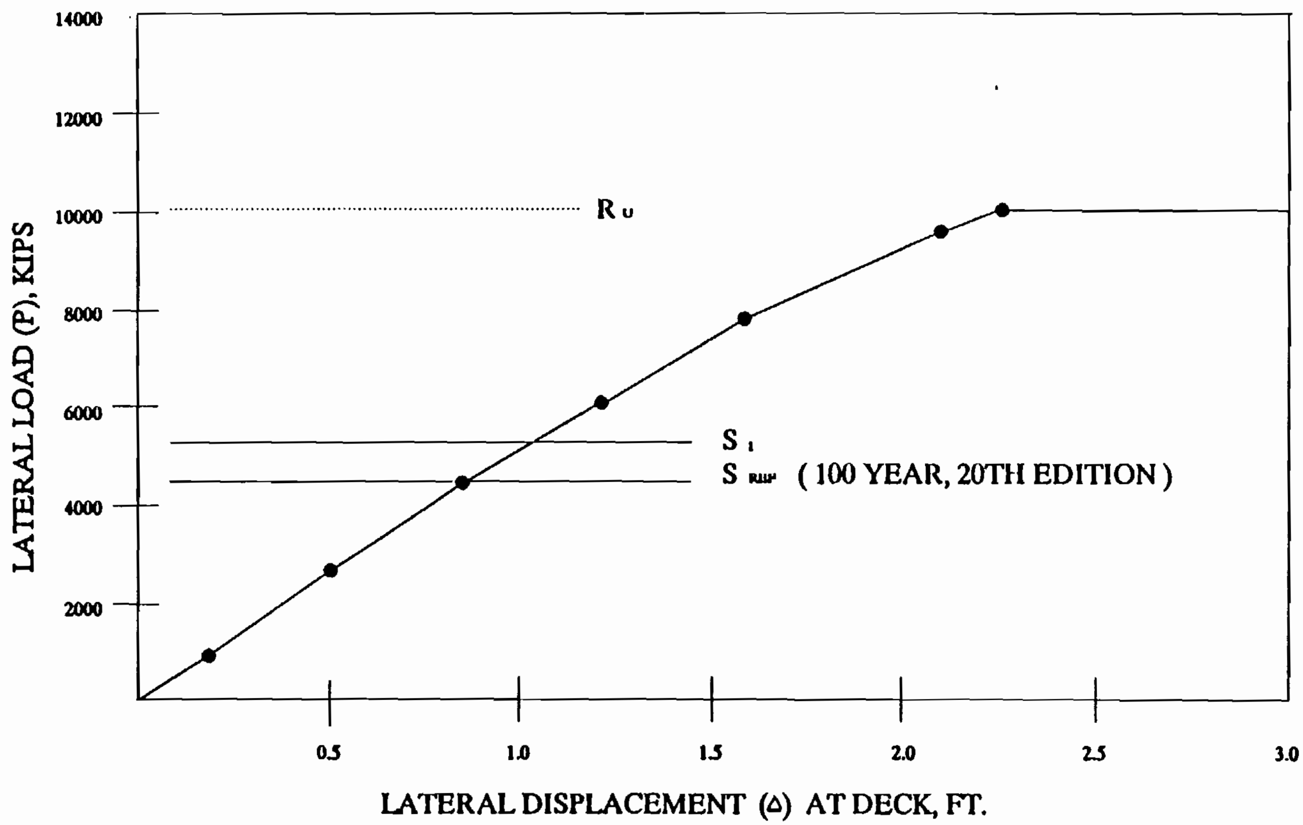
LOAD LEVEL AT WHICH FIRST COMPONENT REACHES I.R. OF 1.0 (S_I)	5886	KIPS
REFERENCE LEVEL LOAD (S_{REF})	4528	KIPS
ULTIMATE CAPACITY (R_U)	11320	KIPS
RESERVE STRENGTH RATIO (RSR)	2.500	
PLATFORM FAILURE MODE: JACKET, PILE, SOILS, ETC.	JACKET	

FIGURE 1: LATERAL DISPLACEMENT VS LATERAL LOAD - 0 DEGREE WAVE

LOAD STEP	LATERAL DISPLACEMENT AT DECK LEVEL FT.	LATERAL LOAD KIPS	ELEMENT FAILURES	COMPONENT FAILURE MODE	REMARKS
1	0.180	880			
2	0.348	1760			
3	0.517	2640			
4	0.686	3520			
5	0.855	4400			
6	1.030	5280	521-522	"PLASTICITY"	
7	1.205	6161			
8	1.390	7041			
9	1.586	7920	686-586, 386-186, 803-603,	"PLASTICITY"	
10	1.812	8801	586-486, 486-486,	"PLASTICITY"	
11	2.099	9681	307-342, 703-783, 783-688	"PLASTICITY"	
12	2.260	10120	400-420, 400-421, 526-575, 804-704, 704-784, 688-603, ETC.	"PLASTICITY"	
13			STRUCTURE COLLAPSES!!		
14					
15					

LATERAL LOAD FOR FIRST MEMBER WITH UNITY CHECK = 1.00 : 5280 KIPS

TABLE 2: ULTIMATE STRENGTH ANALYSIS RESULTS FOR 34 DEGREE WAVE



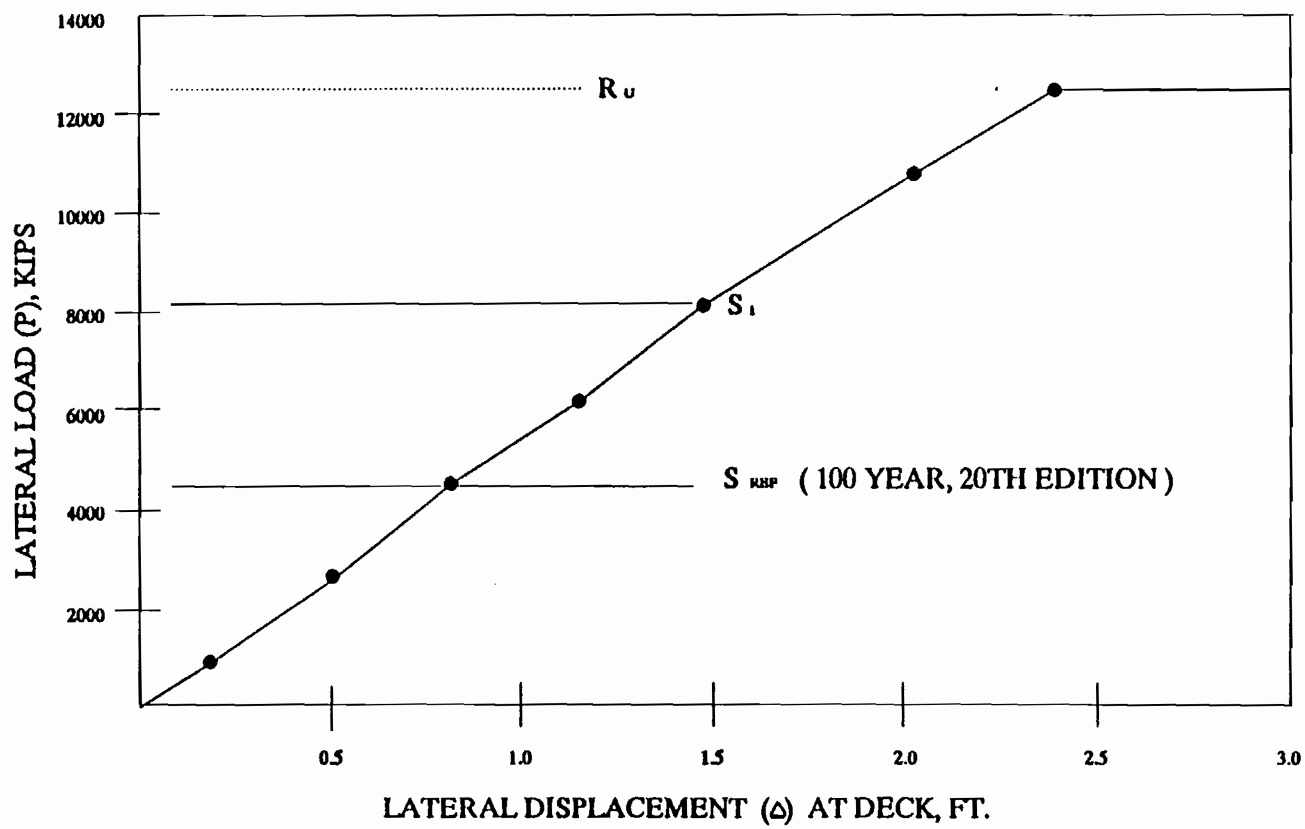
LOAD LEVEL AT WHICH FIRST COMPONENT REACHES I.R. OF 1.0 (S_I)	5280	KIPS
REFERENCE LEVEL LOAD (S_{REF})	4400	KIPS
ULTIMATE CAPACITY (R_U)	10120	KIPS
RESERVE STRENGTH RATIO (RSR)	2.300	
PLATFORM FAILURE MODE: JACKET, PILE, SOILS, ETC.	JACKET	

FIGURE 2: LATERAL DISPLACEMENT VS LATERAL LOAD - 34 DEGREE WAVE

LOAD STEP	LATERAL DISPLACEMENT AT DECK LEVEL FT.	LATERAL LOAD KIPS	ELEMENT FAILURES	COMPONENT FAILURE MODE	REMARKS
1	0.179	908			
2	0.339	1816			
3	0.498	2724			
4	0.658	3631			
5	0.818	4539			
6	0.978	5447			
7	1.142	6355			
8	1.305	7263			
9	1.474	8171	521-522	"PLASTICITY"	
10	1.644	9079			
11	1.823	9987			
12	2.019	10894	555-554, 449-446, 505-558,	"PLASTICITY"	
13	2.241	11802	686-586, 386-186, 803-603, 503-445, 507-445, ETC.	"PLASTICITY"	
14	2.375	12710	805-685, 685-585, 385-185, 452-450, 607-553, ETC	"PLASTICITY"	
15			STRUCTURE COLLAPSES!!		

LATERAL LOAD FOR FIRST MEMBER WITH UNITY CHECK = 1.00 : 8171 KIPS

TABLE 3: ULTIMATE STRENGTH ANALYSIS RESULTS FOR 90 DEGREE WAVE

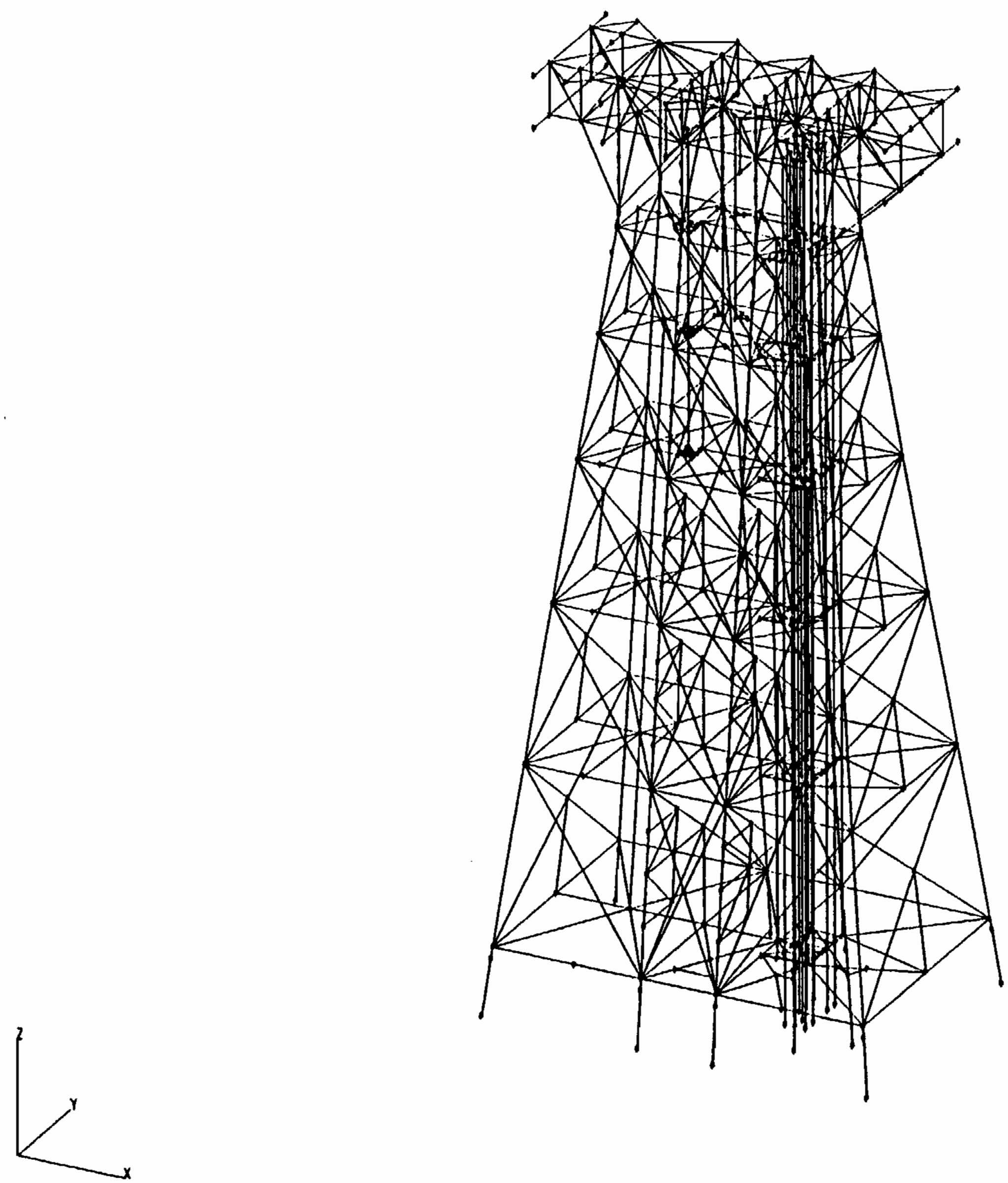


LOAD LEVEL AT WHICH FIRST COMPONENT REACHES I.R. OF 1.0 (S_I)	8171	KIPS
REFERENCE LEVEL LOAD (S_{RSF})	4539	KIPS
ULTIMATE CAPACITY (R_U)	12710	KIPS
RESERVE STRENGTH RATIO (RSR)	2.800	
PLATFORM FAILURE MODE: JACKET, PILE, SOILS, ETC.	JACKET	

FIGURE 3: LATERAL DISPLACEMENT VS LATERAL LOAD - 90 DEGREE WAVE

MODEL "ULTIMATE STRENGTH ANALYSIS"

3-D VIEW



TRIAL APPLICATION OF API RP 2A-WSD,
20th ED, DRAFT SECTION 17,
PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

2.5 TASK A - SUMMARY STATEMENT

The selected platform was evaluated for proposed additions of two (2) wells and one (1) riser against the given assessment initiators as follows:

- Addition of Personnel - No
- Addition of Facilities - No
- Increased Loading on Platform - < 10%
- Inadequate Deck Height - No
- Damage Found During Inspections - No

Based on the above evaluation the selected platform does not need to be further assessed. However, for the purposes of providing additional data to evaluate Draft Section 17, an analytical assessment of the platform was performed. A design level analysis was performed using sudden hurricane environmental data for "insignificant environmental impact/manned-evacuated" category structures. The results of the analysis showed all members and joints to be adequate to support the proposed modifications.

TRIAL APPLICATION OF API RP 2A-WSD,
20th ED, DRAFT SECTION 17,
PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

PARTICIPANT
SELECTED PLATFORM **SHIP SHOAL 322-A; 310' W.D.**

3. CRITICAL REVIEW OF API RP 2A-WSD, 20th ED, DRAFT SECTION 17

1) In general the method of comparing base shear for original vs. modified structure is a good method, particularly when the increase in base shear is less than 10%. It is questionable, however, since software is not yet available to the majority of engineering companies, whether normal lead time will permit the application of an ultimate strength analysis on a routine conventional platform when the indicators suggest it is necessary.

2) The use of the word "requirement(s)" should be limited and where possible changed to "recommendation(s)".

3) In Sections 17.2.1, 17.2.2, 17.2.3, and 17.2.4, is there any significance to interchangeably using the phrases "must be assessed", "shall be assessed", and "should be assessed"?

4) In Section 17.2.6, the third line, shouldn't the wording "cumulative damage and the increase in loading" be changed to "cumulative damage or the increased in loading"?

5) "Section 17.4.3. Soil Data." doesn't seem to belong in Section 17.4.

6) In Figure 17.5.2 there needs to be a mechanism in the flow chart which allows a termination to the assessment process when it is determined that no personnel or facilities are being added and there is no significant damage or load increase. A proposed revision to the flow chart is attached at the end of this section.

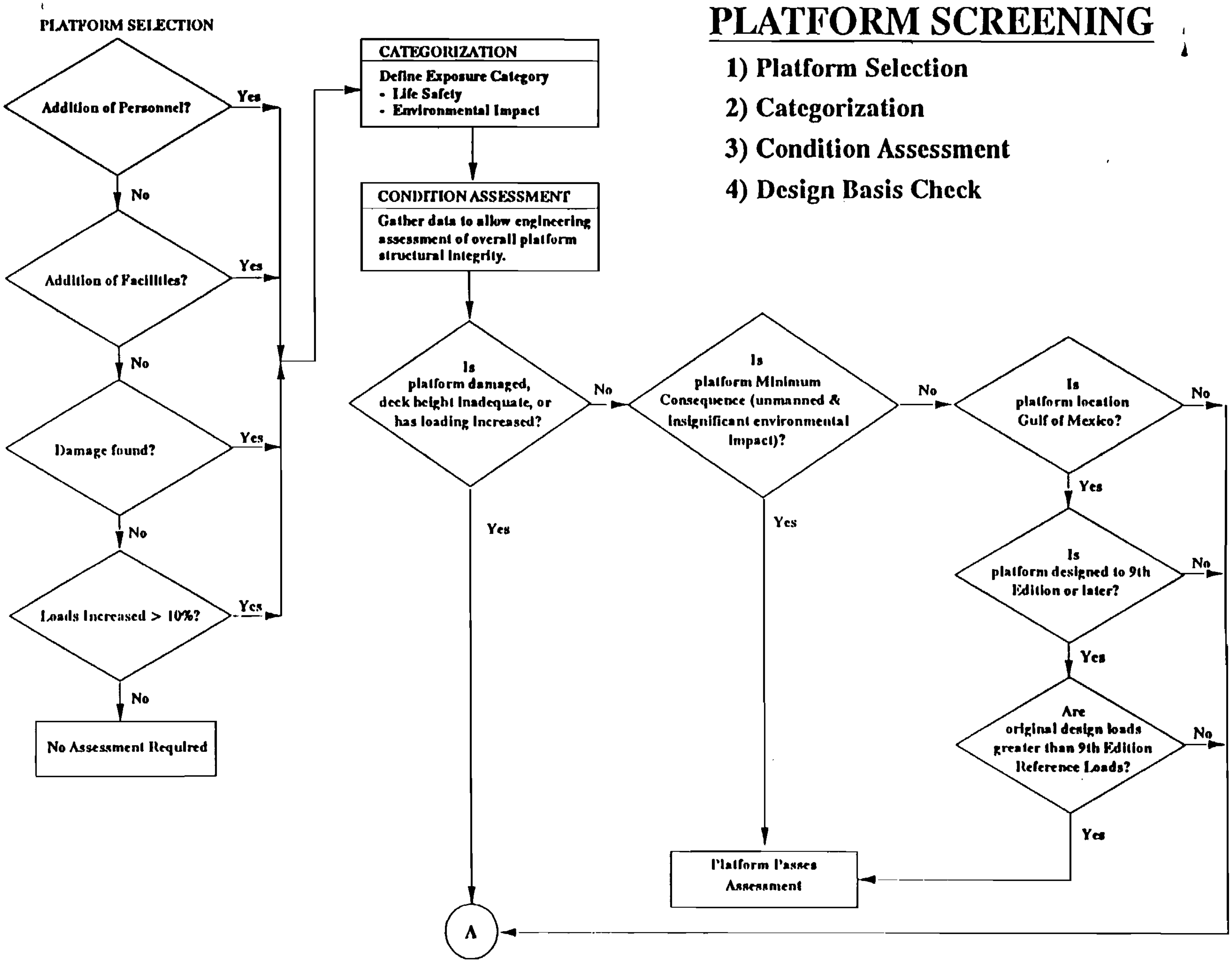
7) In Section 17.5.2.3, the third sentence says that "requirements are described in Section 17.7.2". Section 17.7.2 is entitled "Design Level Analysis Procedures". The nature of Section 17.7.2 seems to state neither requirements or procedures.

8) In Section 17.5.2.4, the fifth sentence says that "requirements are described in Section 17.7.3". Section 17.7.3 is entitled "Ultimate Strength Analysis Procedures".

9) Criteria and procedures are not discussed for structures in the cross hatched area in the Gulf of Mexico shown in Fig. 2.3.4.-2 in API RP 2A-WSD, 20th Ed.

PLATFORM SCREENING

1) Platform Selection
2) Categorization
3) Condition Assessment
4) Design Basis Check



PLATFORM "H"

Trial Application of The Draft API RP 2A Guidelines for Assessment of Existing Platforms

PLATFORM H

1.0 Platform Information

1.1 Physical Characteristics

- Water Depth: 95 feet
- Number of Leg/Piles: 4/4
- Vertical Framing Type: Combination K-brace and X-brace
- Leg/Pile Annulus: Ungrouted
- Lower Deck Elevation: T.O.S. EL. (+) 54'- 9

1.2 Design Basis

- Year Designed: Originally designed in 1977. Design for reuse in 1989.
- Year Installed: Originally installed in 1978. Salvaged, modified and reinstalled in 1989.
- API Edition: Original Design - 7th Edition. Re-use Design - 16th Edition.

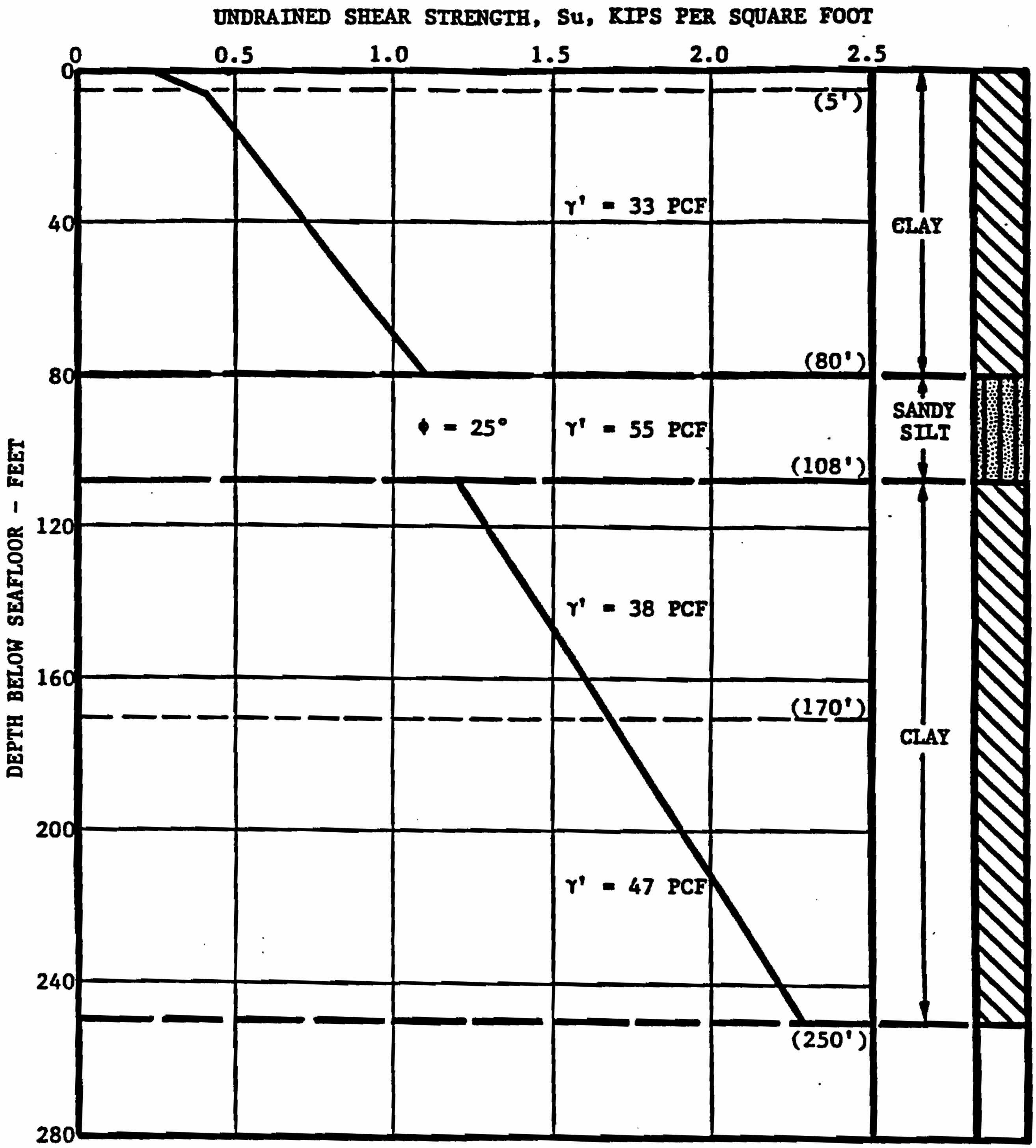
1.3 Operational Information

Production Rates:

- Gas: 65 MMSCFD
- Condensate: 4000 BBL/day
- Water: 2500 BBL/day
- Number of Wells: Two (2) at platform reinstallation (1989)
  Two (2) added since reinstallation
- Manned: Originally unmanned
  Now Manned, evacuated.

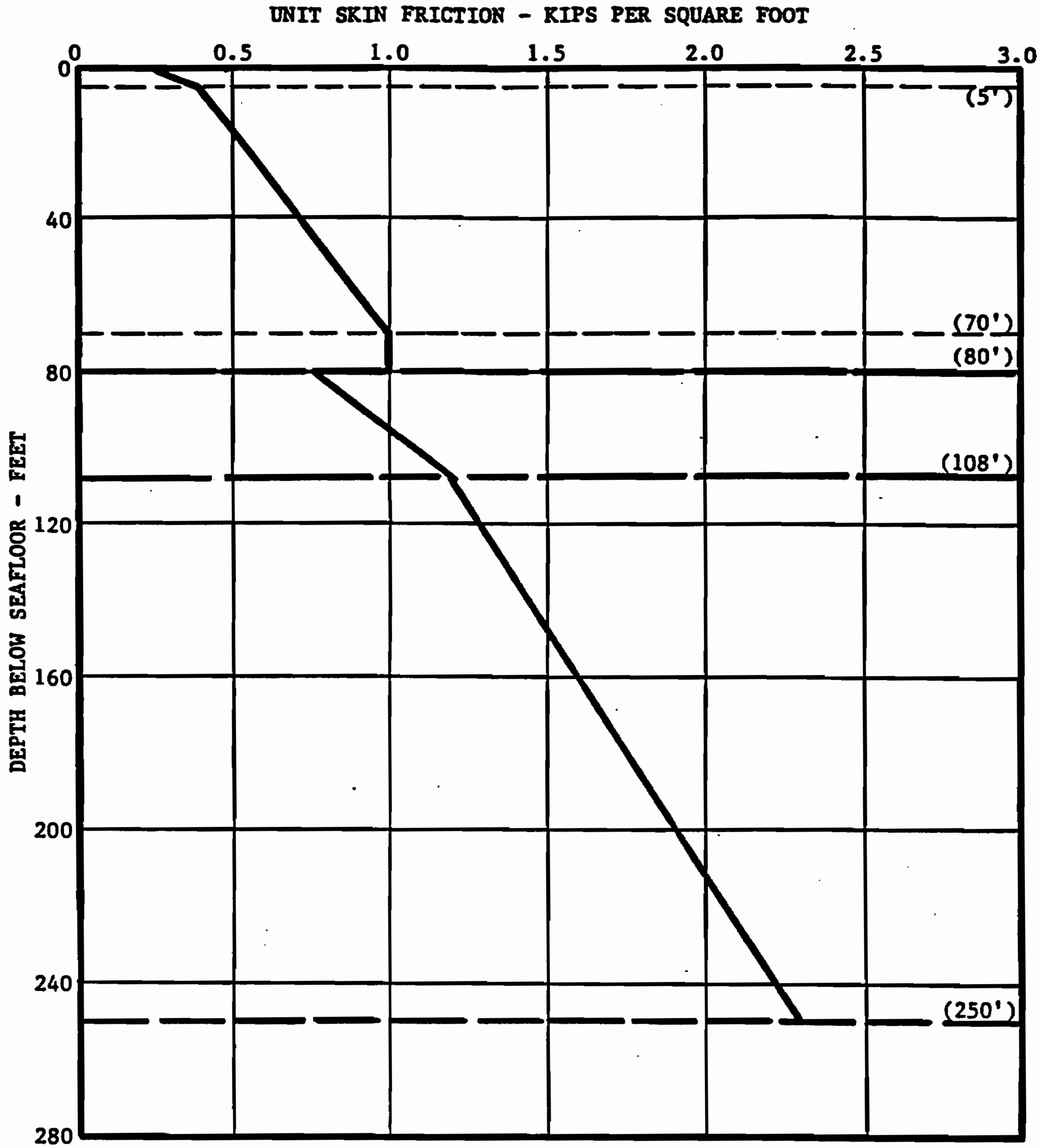
1.4 Platform Physical State

The platform has had regular surveys in accordance with Section 14. Latest inspection indicated that the platform is structurally sound and undamaged. Since being designed for reuse at this location, the platform has become manned, production has increased, two wells have been added, four (4) risers have been added, and a subsea cooling coil has been added.



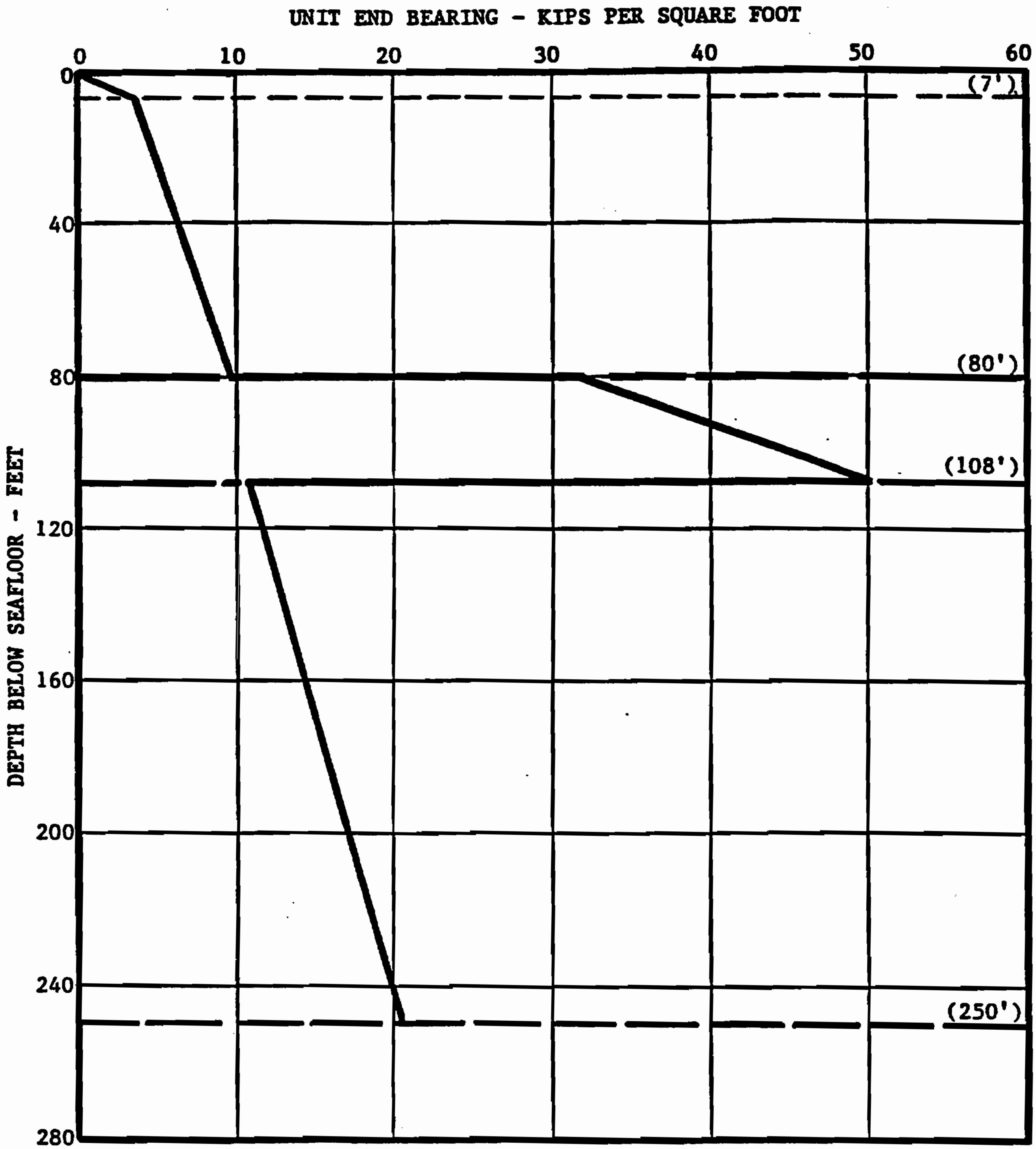
DESIGN SOIL PROPERTIES - API METHOD
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

PROJECT NO. EWA89110



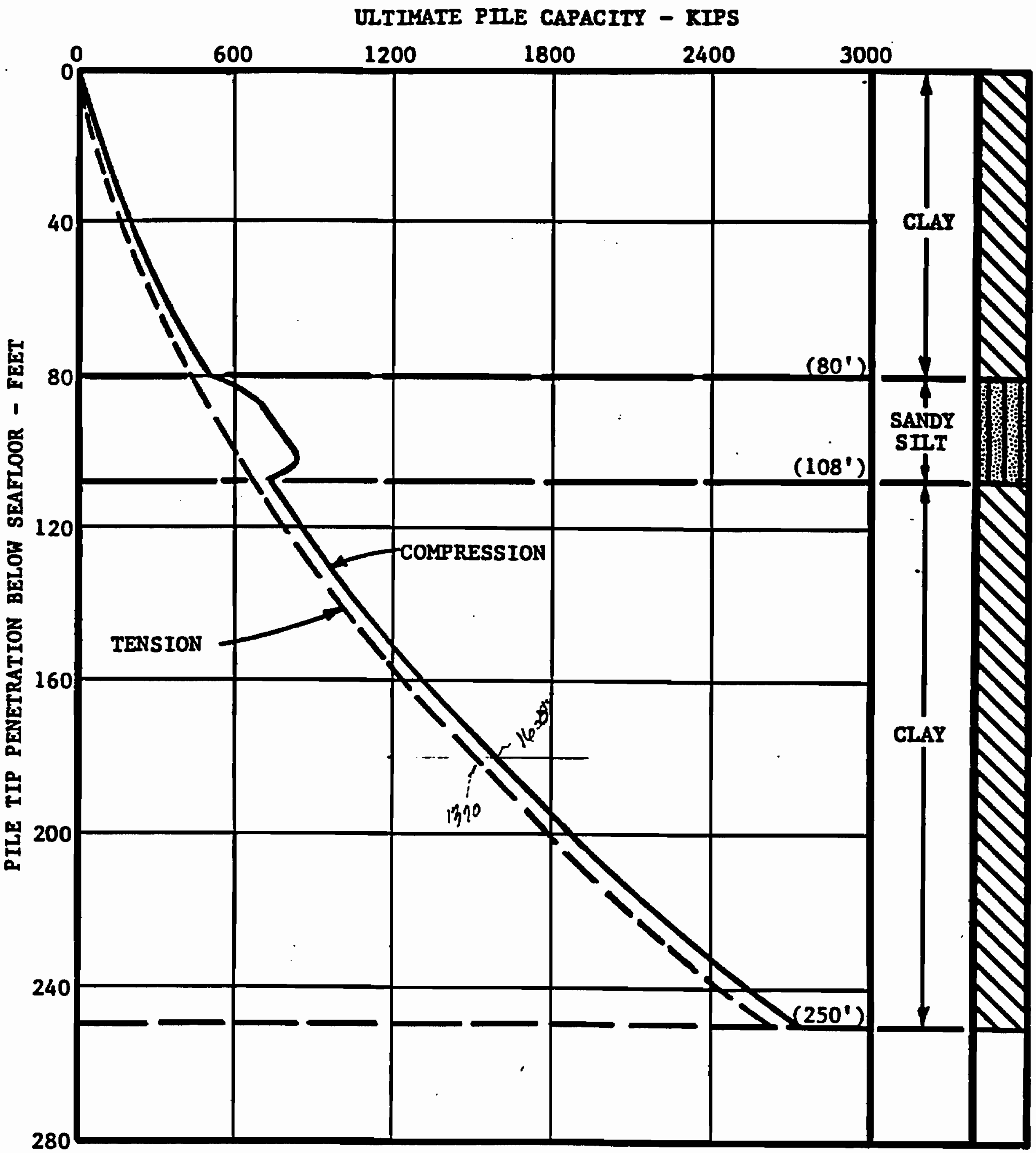
UNIT SKIN FRICTION - API METHOD
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

PROJECT NO. EWA89110



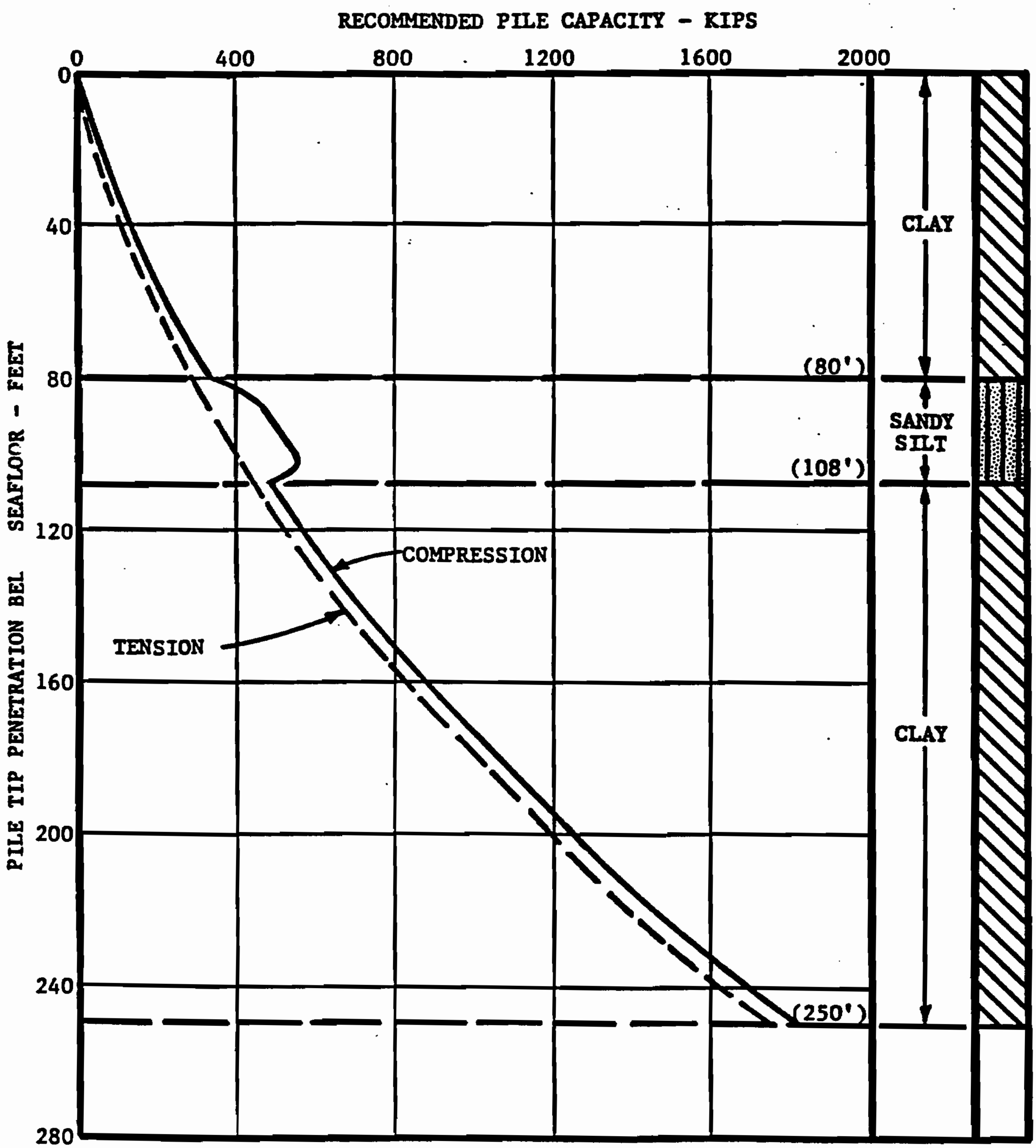
UNIT END BEARING - API METHOD
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

PROJECT NO. EWA89110



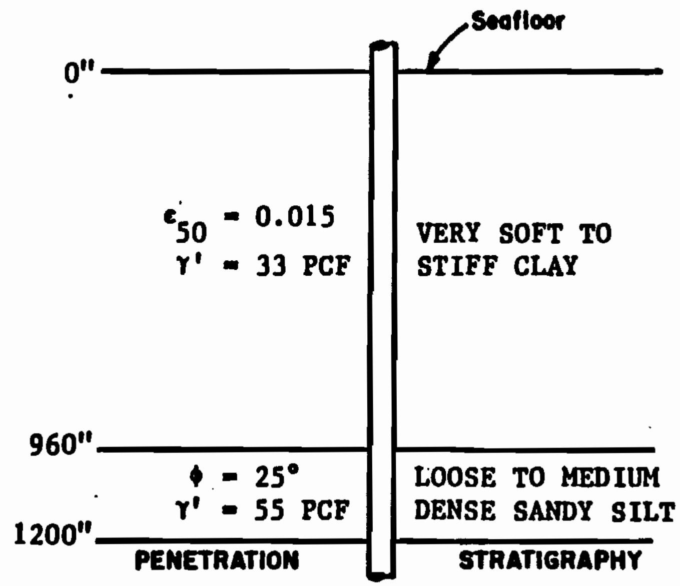
API METHOD
ULTIMATE PILE CAPACITY CURVES
30-INCH OD PIPE PILES
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

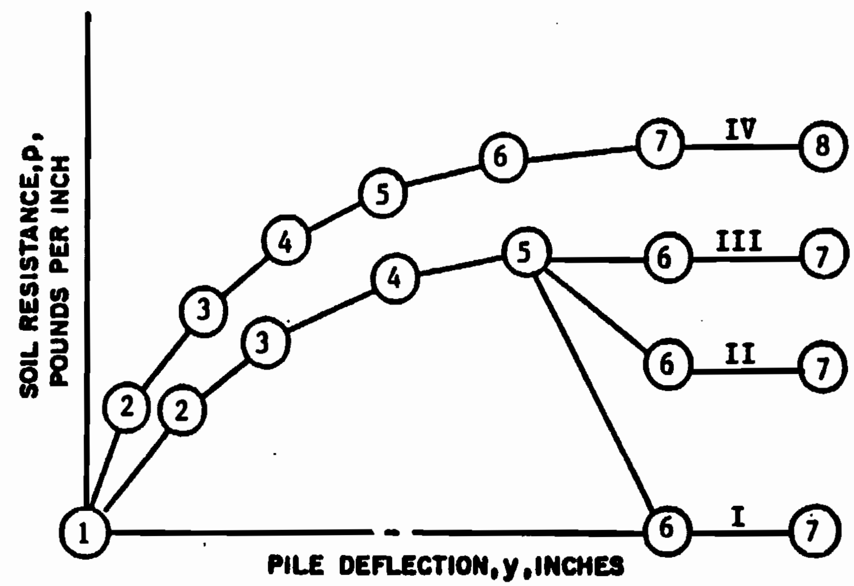
PROJECT NO. EWA89110



API METHOD
RECOMMENDED PILE CAPACITY CURVES
30-INCH OD PIPE PILES
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

PENETRATION IN INCHES	TYPICAL CURVE	COORDINATES OF CURVE POINTS															
		y_1	p_1	y_2	p_2	y_3	p_3	y_4	p_4	y_5	p_5	y_6	p_6	y_7	p_7	y_8	p_8
0	I	0	0	0.11	36	0.34	51	1.13	78	3.38	112	16.88	0	30.0	0		
15	II	0	0	0.11	47	0.34	67	1.13	102	3.38	146	16.88	10	30.0	10		
30	II	0	0	0.11	58	0.34	84	1.13	127	3.38	183	16.88	24	30.0	24		
60	II	0	0	0.11	85	0.34	121	1.13	184	3.38	265	16.88	62	30.0	62		
120	II	0	0	0.11	123	0.34	176	1.13	267	3.38	384	16.88	175	30.0	175		
279	III	0	0	0.11	246	0.34	353	1.13	535	3.38	770	16.88	770	30.0	770		
960	III	0	0	0.11	474	0.34	680	1.13	1031	3.38	1485	16.88	1485	30.0	1485		
960	IV	0	0	0.03	1207	0.06	1725	0.15	2673	0.30	3708	0.50	4743	1.13	7589	30.0	7589
1200	IV	0	0	0.03	1710	0.06	2443	0.15	3787	0.30	5253	0.50	6720	1.13	10751	30.0	10751

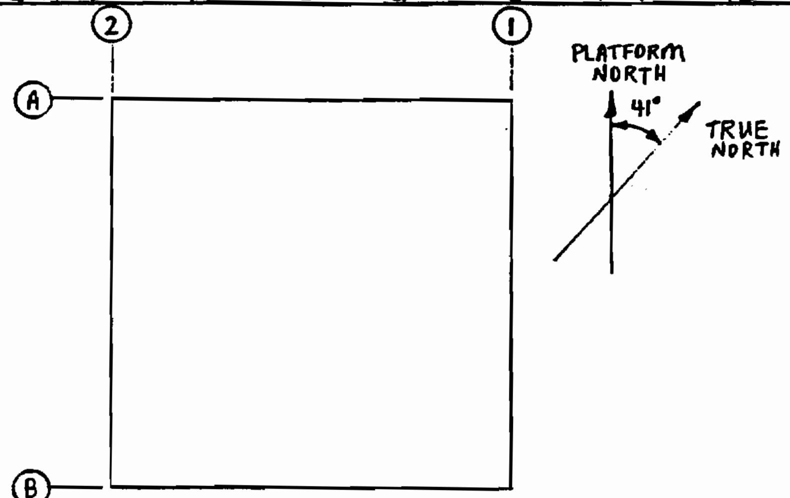




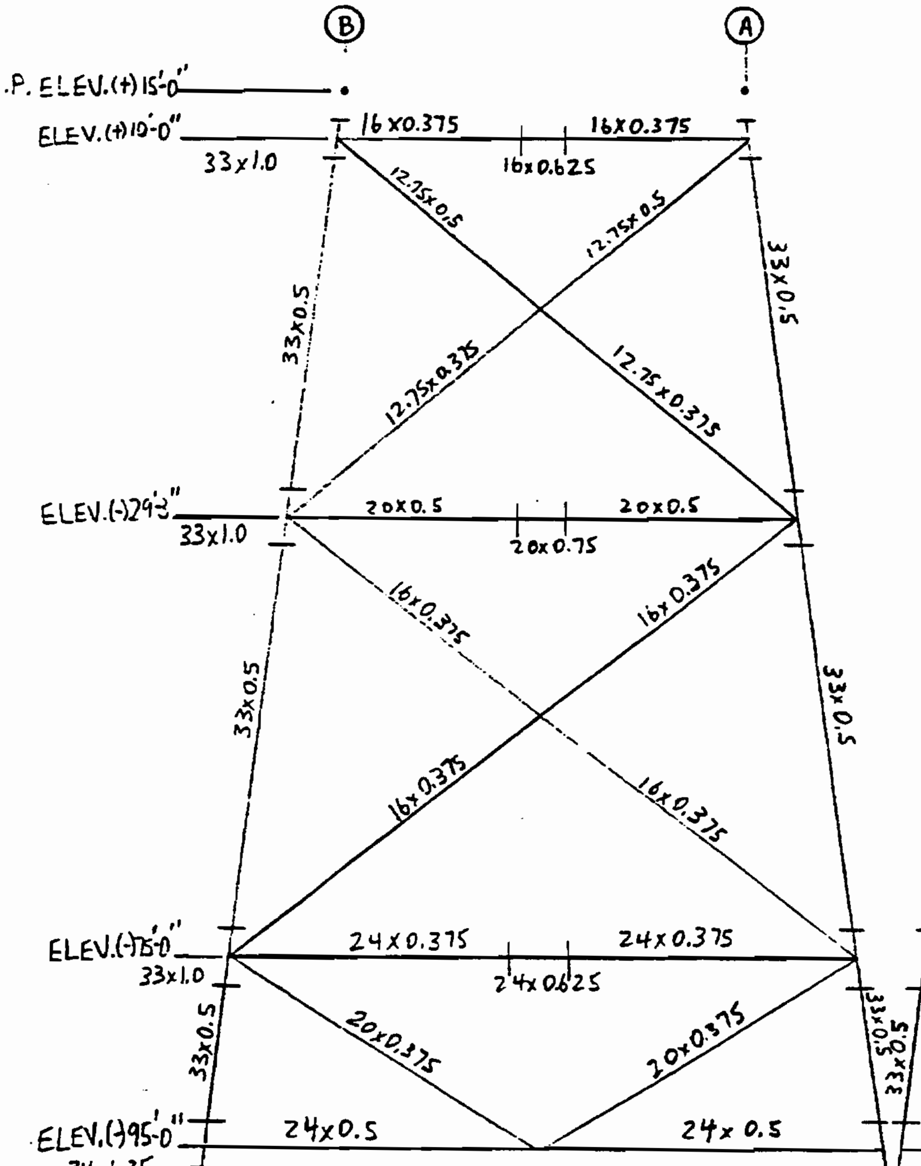
SOIL RESISTANCE - PILE DEFLECTION (p-y) DATA
30-ICH OD PIPE PILE
BLOCK 36, SOUTH MARSH ISLAND AREA
GULF OF MEXICO

PROJECT NO. EWA89110

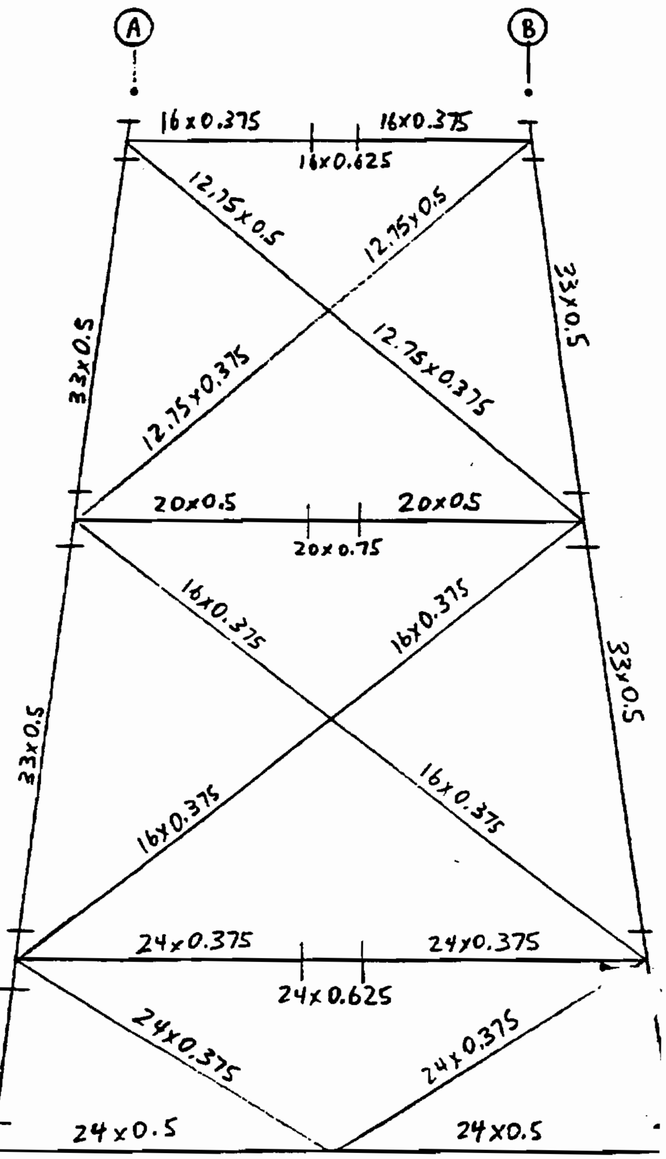
Client	Project TRIAL APPLICATION - SEC. 17	Project No. 111600	Sht. of
Subject PLATFORM H - KEY PLAN & JKT ROWS 1 & 2		By	Date



KEY PLAN

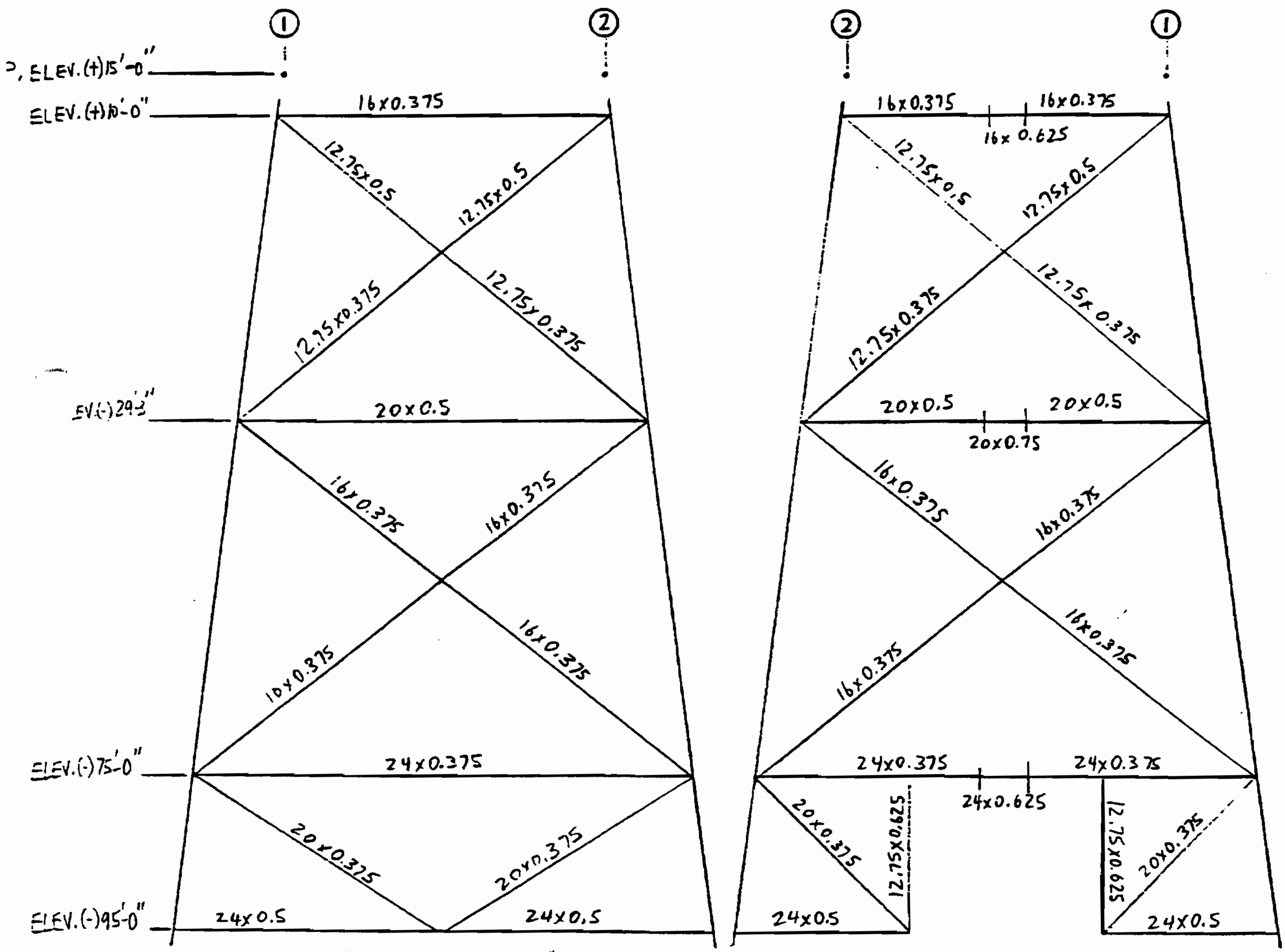


ELEVATION ROW "1"



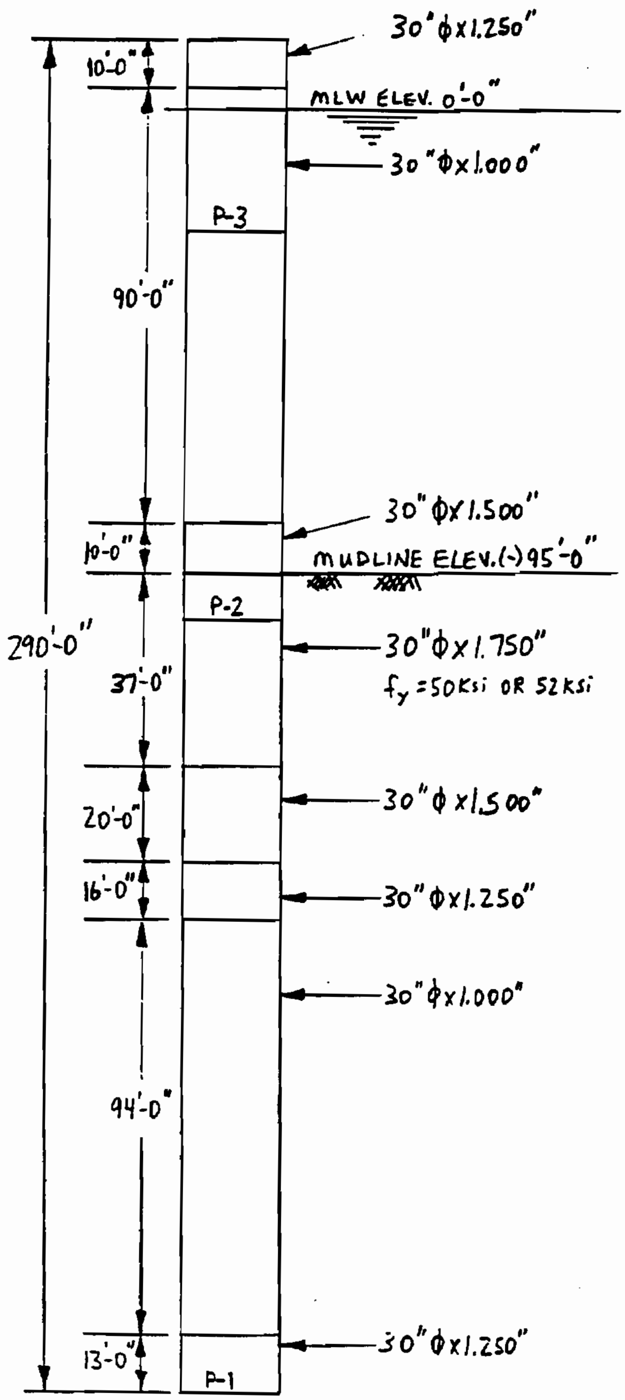
ELEVATION ROW "2"

Client	Project TRIAL APPLICATION - SEC 17	Project No. 111600	Sht. of
Subject PLATFORM 1 JACKET ROWS A & B		By	Date



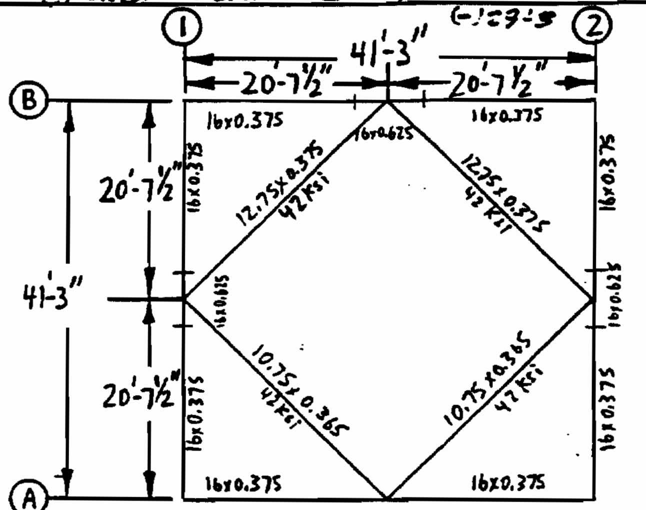
ELEVATION ROW "A" ELEVATION ROW "B"

Client	Project TRIAL APPLICATION -SECT 17	Project No. 711600	Sht. of
Subject PLATFORM H-PILE MAKE-UP		By	Date

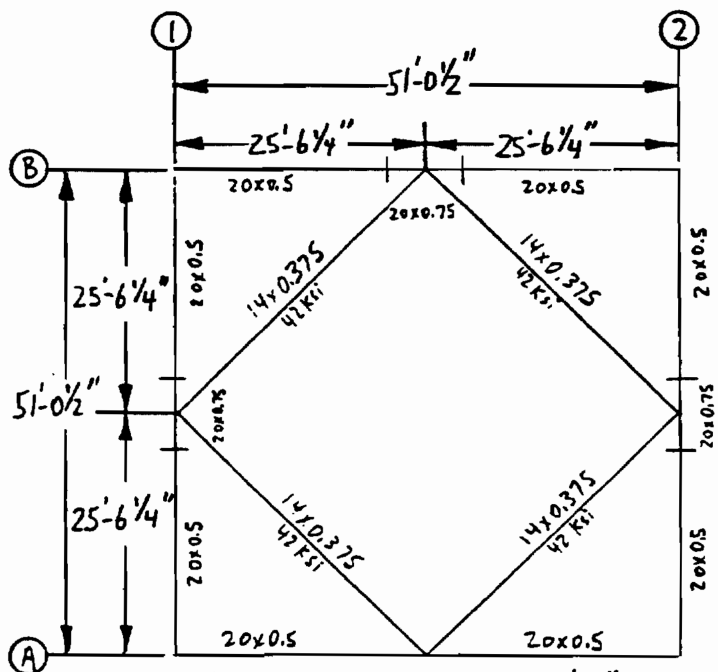


PILE MAKEUP

Client	Project TRIPLE AMPLICATION-SECTN	Project No. 111600	Sht. of
Subject PLATFORM H-JACKET PLANS @ ELEV (+)10-0 (-)29-3		By	Date

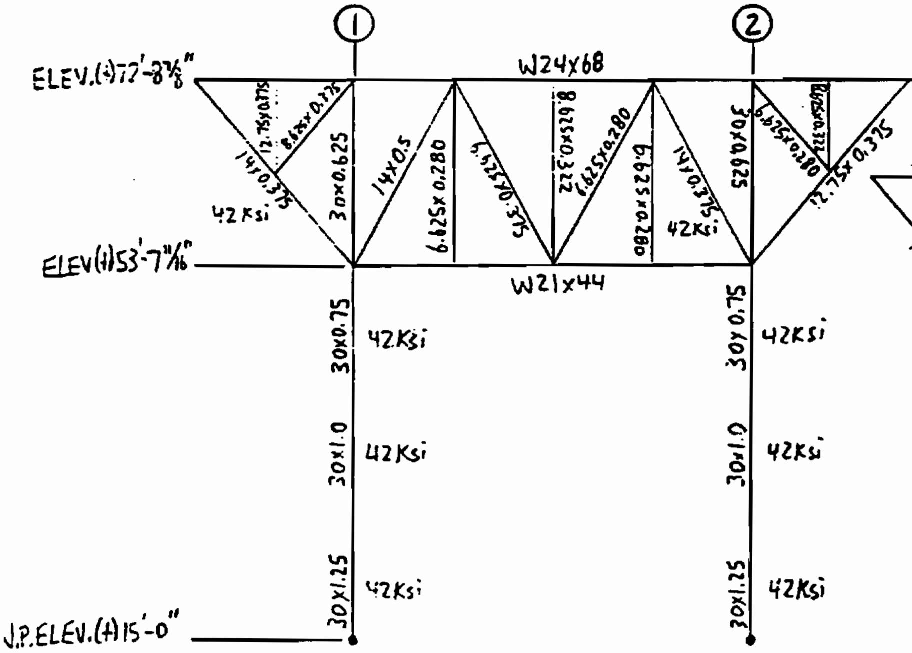


PLAN @ ELEV. (+) 10'-0"

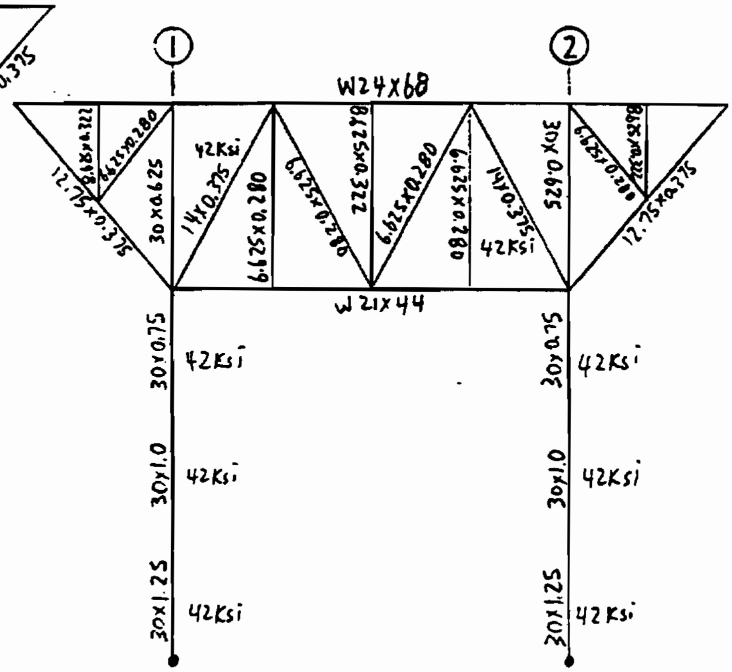


PLAN @ ELEV. (-) 29'-3"

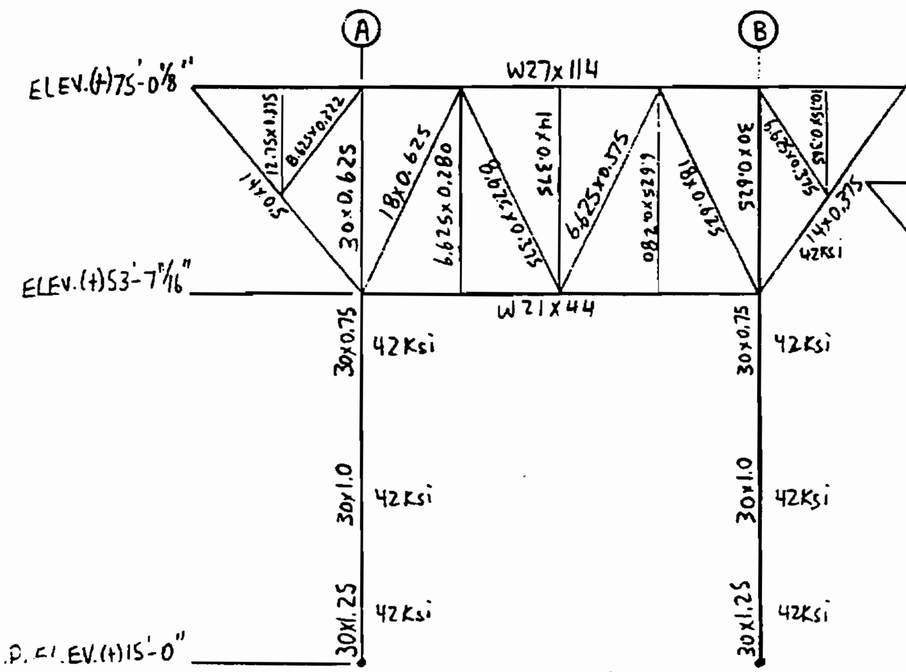
Client	Project TRIAL APPLICATION - SECT 17	Project No. 111600	Sht. of
Subject PLATFORM H - DECK ELEVATIONS		By	Date



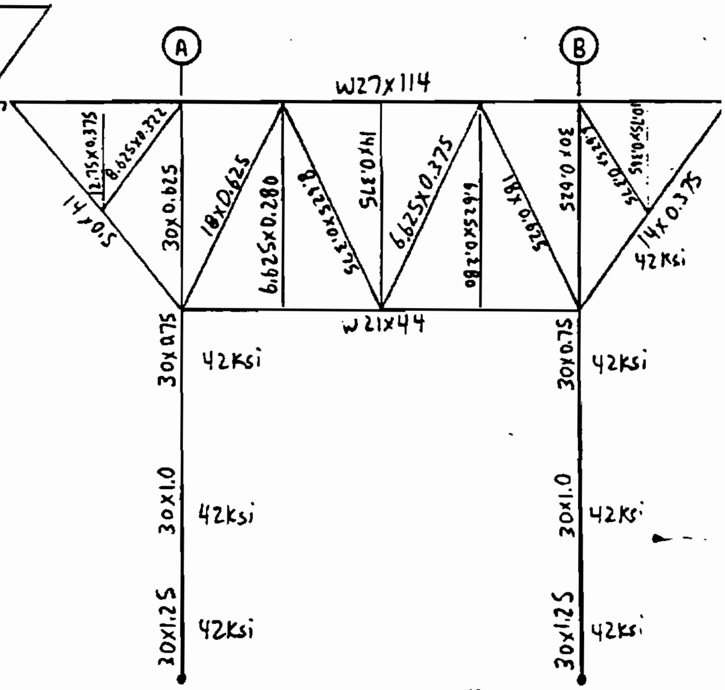
DECK ROW "A"



DECK ROW "B"

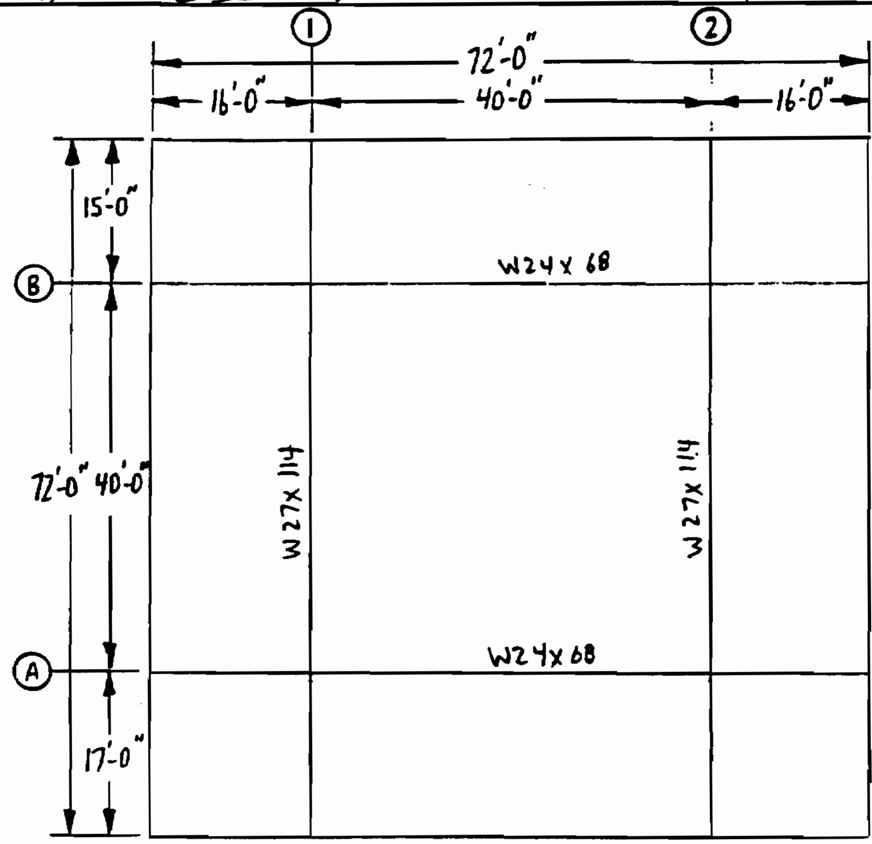


DECK ROW "1"

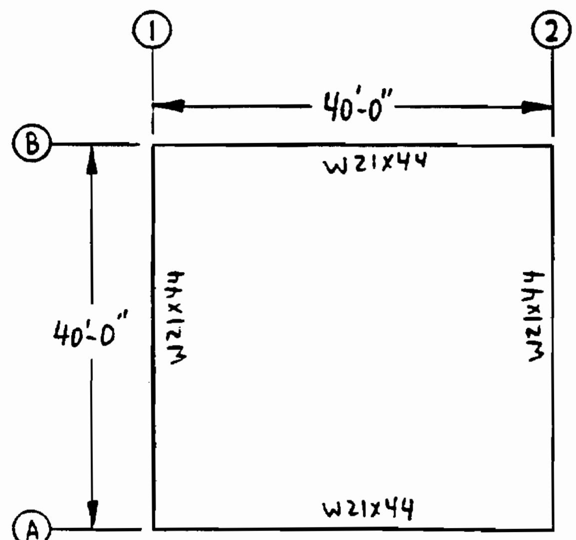


DECK ROW "2"

Client	Project TRIAL APPLICATION - SECT 17	Project No. 111600	Sht. of
Subject PLATFORM H- DECK PLANS		By	Date



PLAN @ MAIN DECK



PLAN @ LOWER DECK

Trial Application of The
Draft API RP 2A Guidelines for
Assessment of Existing Platforms

PLATFORM H

Part A. Platform Assessment

A.1 Platform Selection

A.1.1 Addition of Personnel - Since the platform reinstallation in 1989 the platform manning condition has changed from Unmanned to Manned - Evacuated.

A.1.2 Addition of Facilities - The platform, when reinstalled, supported two (2) wells. Since that time two (2) additional wells have been added. The number of risers has increased from two (2), when reinstalled, to six (6). Also, a subsea cooling coil has been added.

A.1.3 Increased Loading on Structure - The items located in Section A.1.2 have resulted in an increased in structural loading in excess of 10 percent greater than the original design loading.

A.1.4 Deck Height - The platform has an adequate deck height as defined by Figure 17.6. 2-3b.

A.1.5 Damage - No significant damage has been found during recent surveys.

A.2 Condition Assessment

A.2.1 Recent Level I and Level II surveys have indicated that the platform is structurally sound and undamaged.

A.3 Categorization

A.3.1 Life Safety - The platform is normally manned and evacuated during forecasted storms. Therefore, Manned - Evacuated.

A.3.2 Environmental Impact - If collapsed, the platform would not release hydrocarbon liquids nor sour gas. It is not in close proximity to a environmentally sensitive area. Therefore, Insignificant Environmental Impact.

Trial Application of The
Draft API RP 2A Guidelines for
Assessment of Existing Platforms

PLATFORM H

A.4 Design Basis Check

The platform, when reinstalled, was analyzed in accordance with API RP-2A 16th Edition. Since there has been significant change in the design premise, as described in Section A.1, this application proceeded with the Design Level Analysis.

A.5 Analysis Checks

A.5.1 Design Level Analysis

A.5.1.1 Metocean Criteria

Water Depth:	95.0	feet
Wave Height:	41.5	feet
Storm Tide:	3.21	feet
Wave Period:	11.3	secs.
Wave Kinematics Factor:	0.88	
Current Speed:	1.2	knots
Current is inline for all directions		
Current Blocking Factor:		
End on:	0.80	
Diagonal:	0.85	
Broadside:	0.80	
Drag and Inertia Coefficients:	Per 20th Edition	
Marine Growth:	1.5	inches on radius
Wind Speed:	55	knots
Required Deck Height:	38.3	feet

NOTE: Loading from the following directions reflect the less severe Ultimate Strength Analysis criteria.

94 degrees
139 degrees
184 degrees
229 degrees

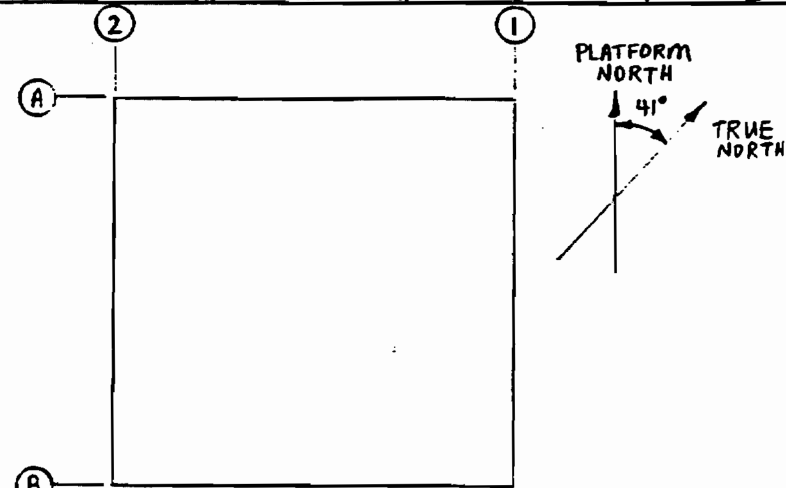
Trial Application of The Draft API RP 2A Guidelines for Assessment of Existing Platforms

PLATFORM H

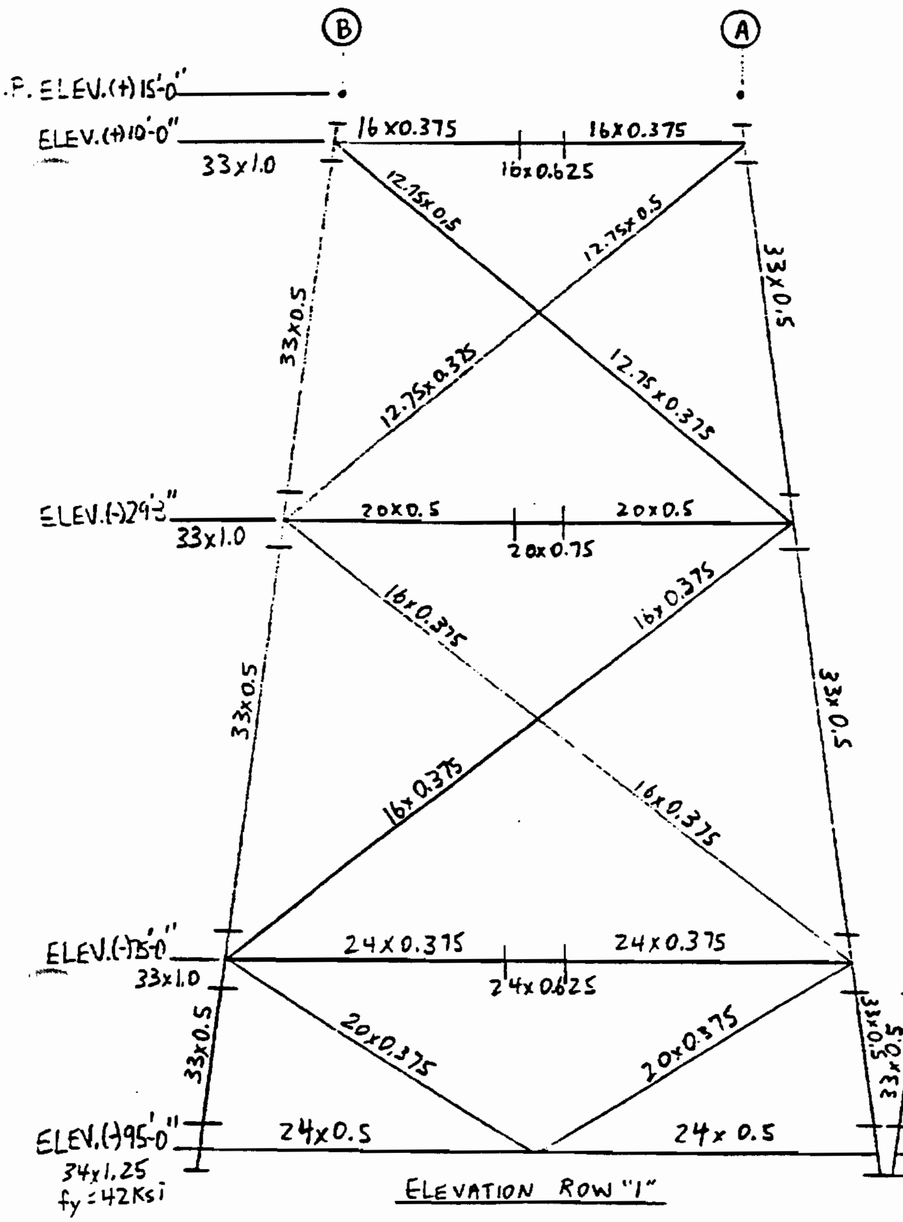
A.5.1.2 Design Level Analysis Base Shear Values

WAVE DIRECTION (Degrees)	BASE SHEARS (Kips)
4	915.2
49	954.7
94	563.9
139	591.8
184	628.9
229	881.1
290	908.5
319	935.0

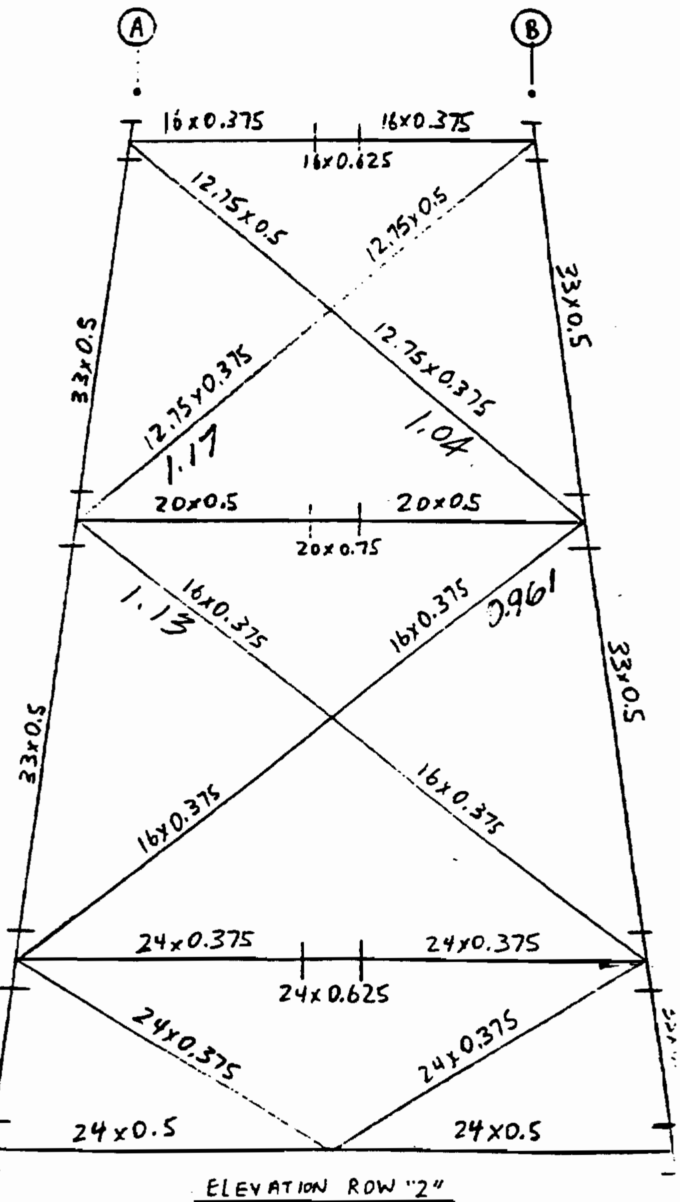
Client	Project TRIAL APPLICATION - SECT. 17	Project No. 111323	Sht. of
Subject PLATFORM H - KEY PLAN & JKT ROWS 1 & 2		By	Date



KEY PLAN

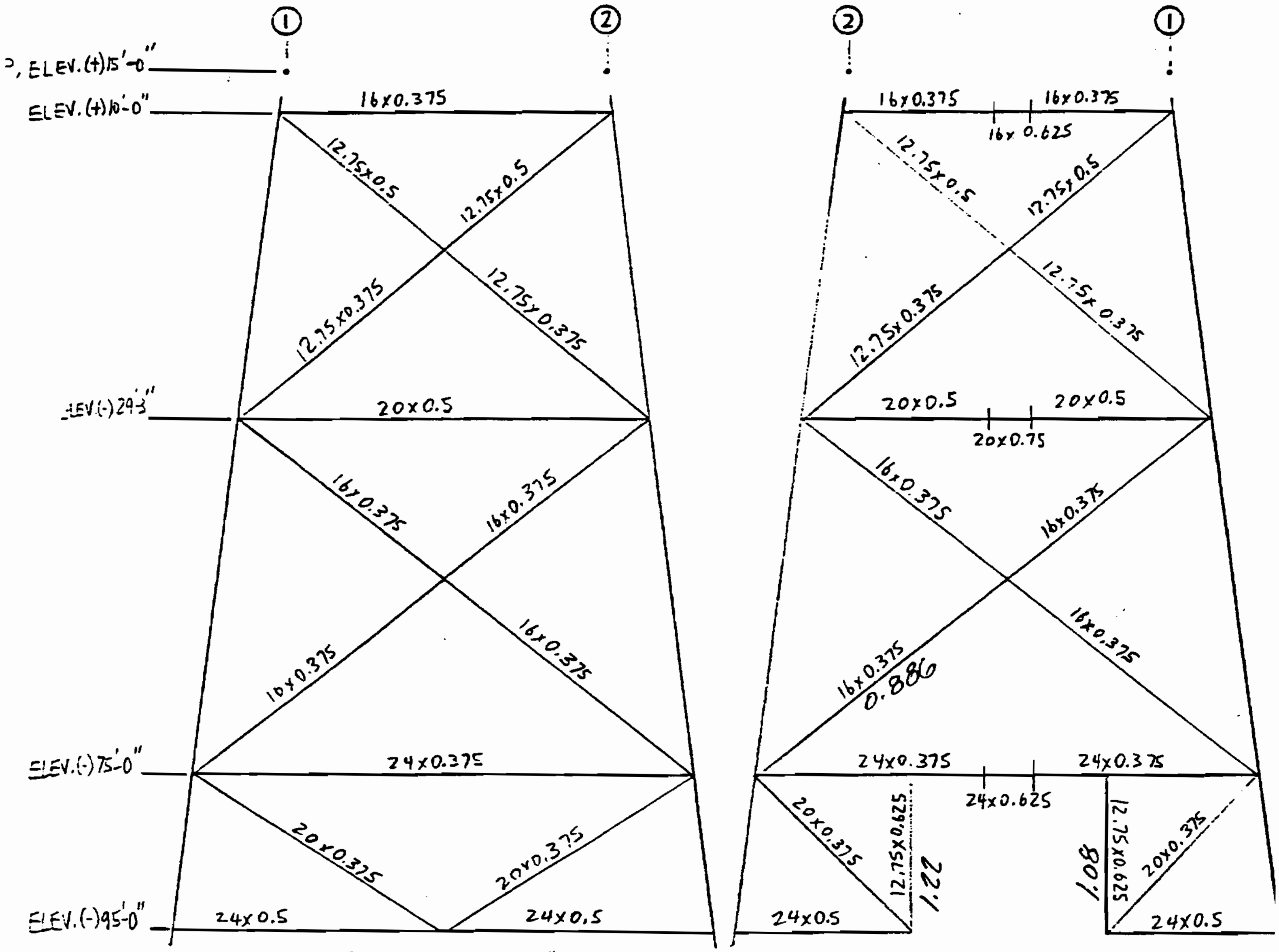


ELEVATION ROW "1"



ELEVATION ROW "2"

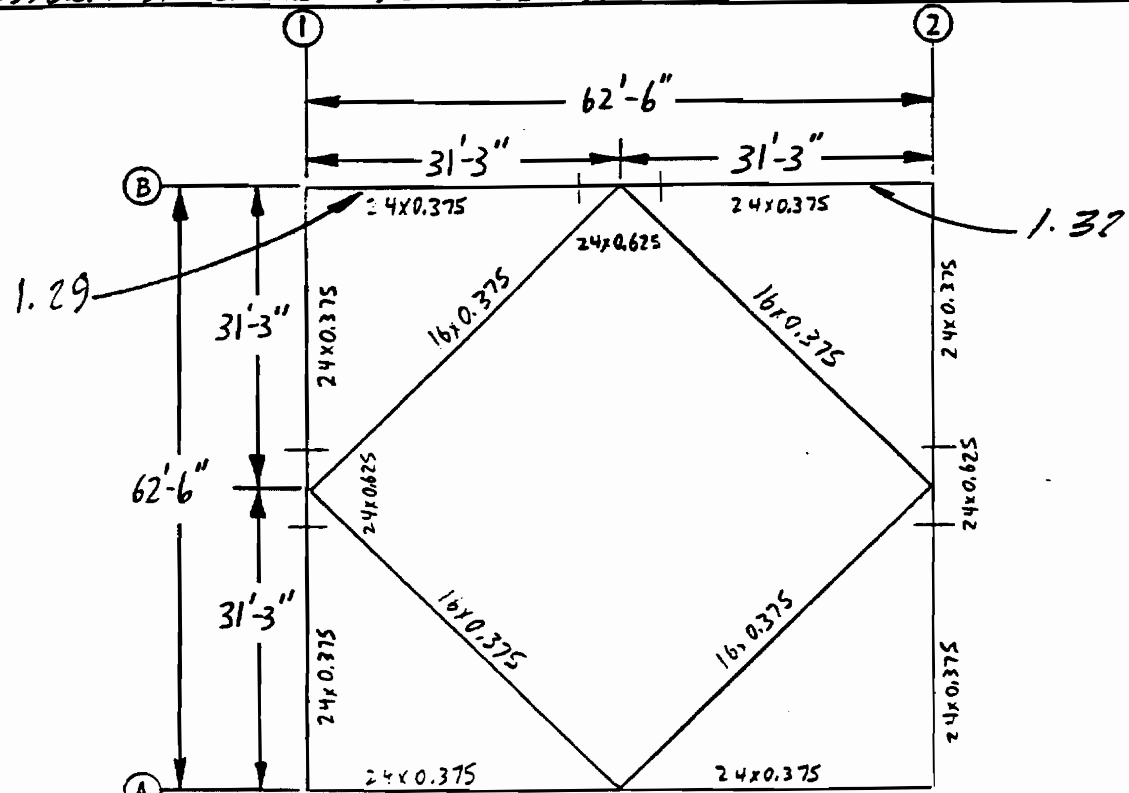
Client	Project [illegible] SEC 17	Project No. [illegible]	Sht. of
Subject PLATFORM H. JACKET ROWS A & B		By	Date



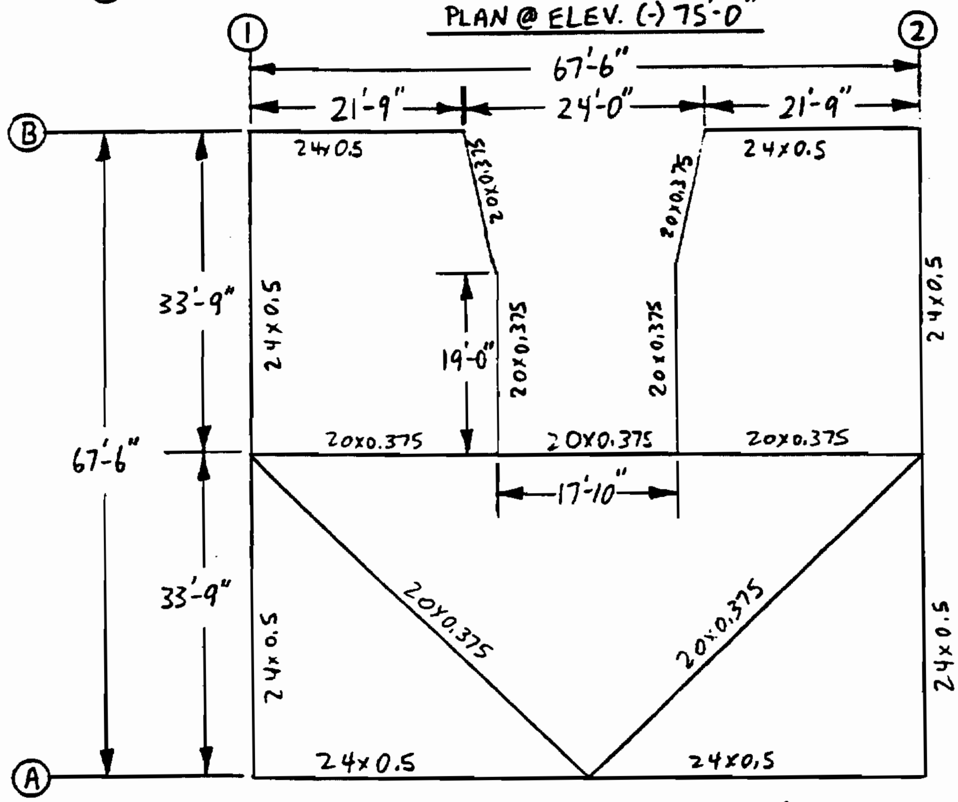
ELEVATION ROW "A"

ELEVATION ROW "B"

Client	Project TRIAL APPLICATION - SECT 17	Project No. 111600	Sht. of
Subject PLATFORM H - JACKET PLAN @ EL (-)75'-0" & 95'-0"		By	Date

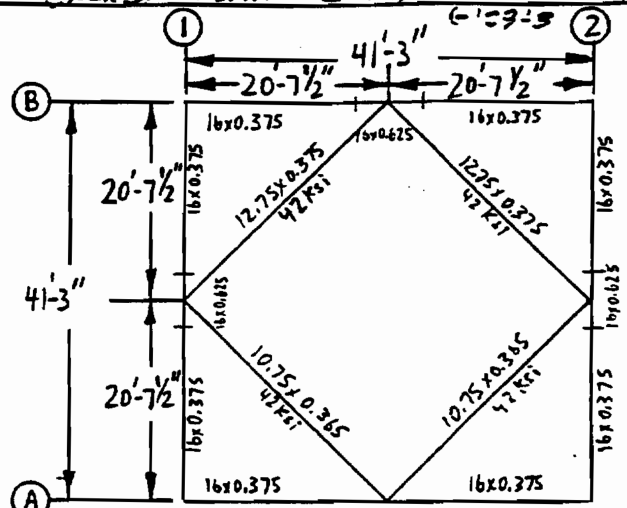


PLAN @ ELEV. (-) 75'-0"

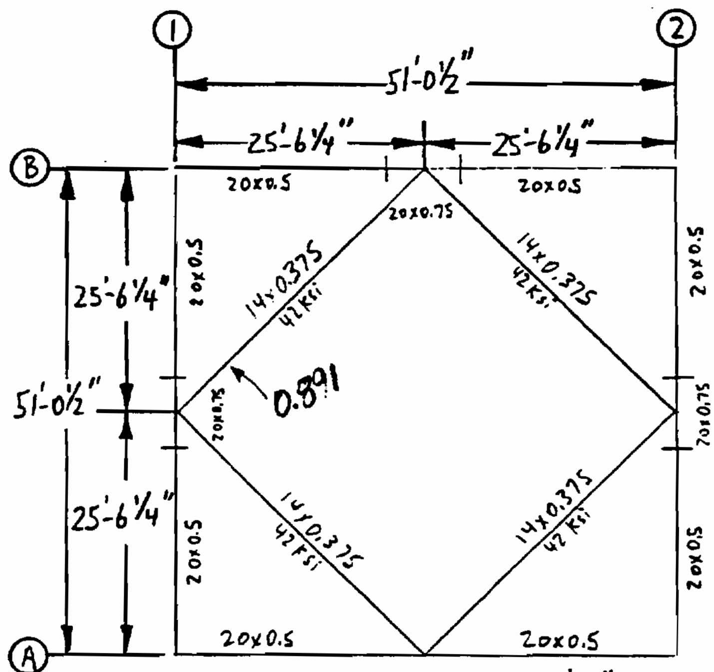


PLAN @ ELEV. (-) 95'-0"

Client	Project TRIPLE [illegible]	Project No. 111600	Sht. of
Subject: PLATFORM H- JACKET PLANS @ ELEV (+)10-0 (-)29-3		By	Date

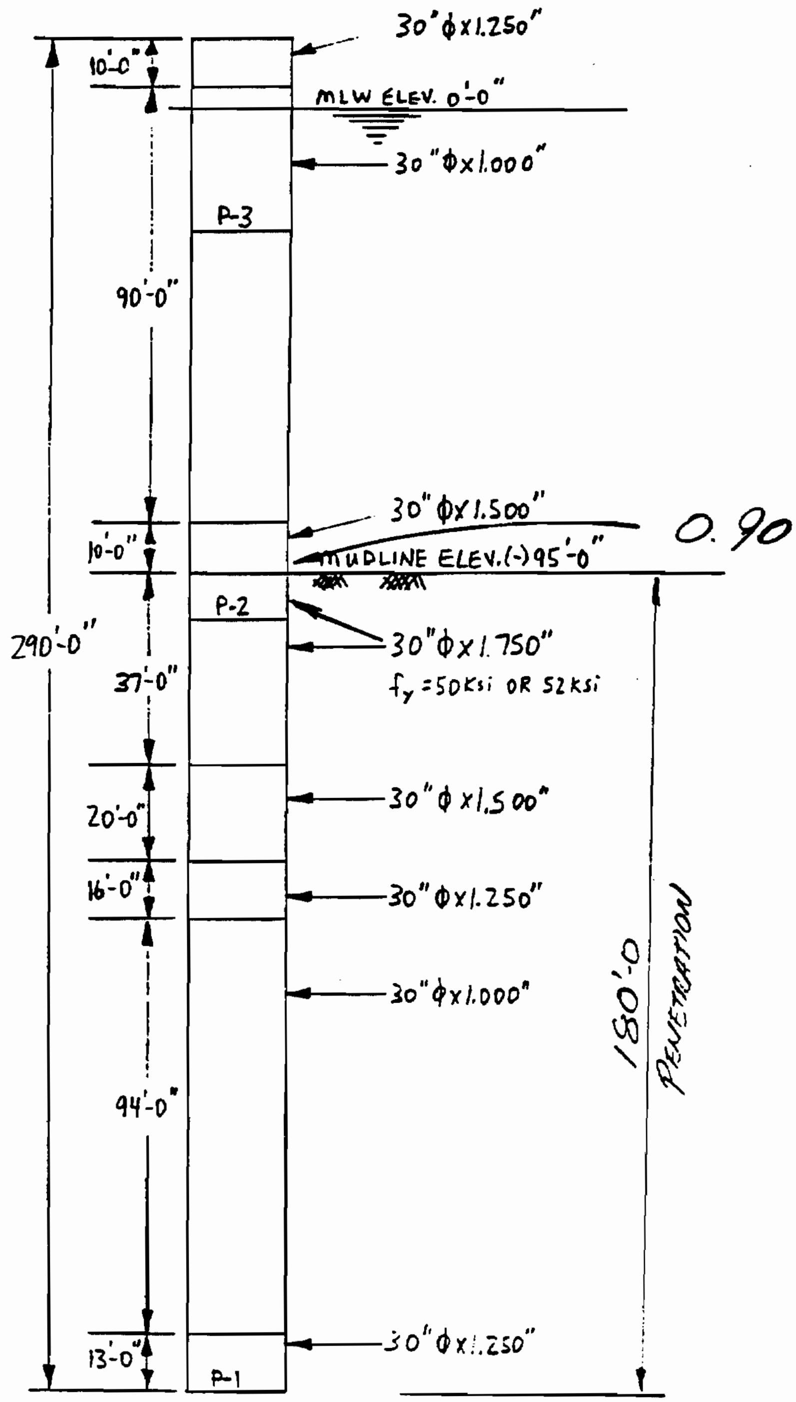


PLAN @ ELEV. (+)10'-0"



PLAN @ ELEV. (-) 29'-3"

Client	Project TRIAL APPLICATION - SEC 17	Project No. 111600	Sht. of
Subject PLATFORM H-PILE MAKE-UP		By	Date



PILE MAKEUP

MAX COMPRESSION = 1164^k

S.F. = 1.37

MAX TENSION = 303^k

S.F = 4.29

Trial Application of The Draft API RP 2A Guidelines for Assessment of Existing Platforms

PLATFORM H

A.5.2 Ultimate Strength Analysis

A.5.2.1 Metocean Criteria

Water Depth:	95.0	feet
Wave Height(Max):	51.1	feet
Storm Tide:	3.21	feet
Wave Period:	12.5	secs.
Wave Kinematics Factor:	0.88	
Current Speed:	1.8	knots
Current Direction:	285	degrees
Minimum In-Line Current:	0.2	knots
Current Blocking Factor:		
End on:	0.80	
Diagonal:	0.85	
Broadside:	0.80	
Drag and Inertia Coefficients:	Per 20th Edition	
Marine Growth:	1.5	inches on radius
Wind Speed:	70	knots

WAVE DIRECTION (Degrees)	WAVE HT FACTOR	WAVE HEIGHT (feet)
4	0.95	48.5
49	0.85	43.4
94	0.70	35.8
139	0.70	35.8
184	0.70	35.8
229	0.75	38.3
290	0.90	46.0
319	1.00	51.1

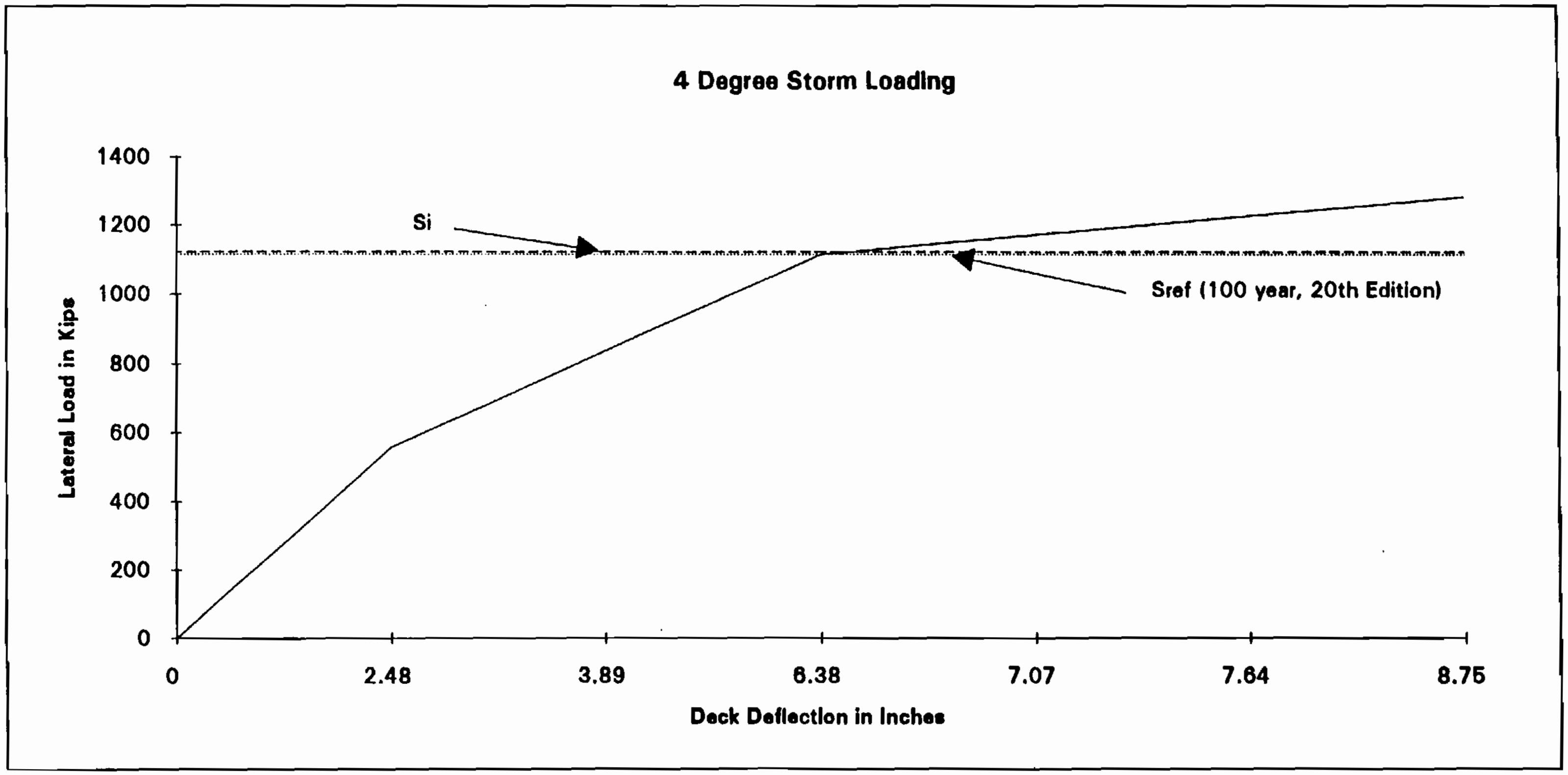
Trial Application of The Draft API RP 2A Guidelines for Assessment of Existing Platforms

PLATFORM H

A.5.2.2 Ultimate Strength Analysis Procedure

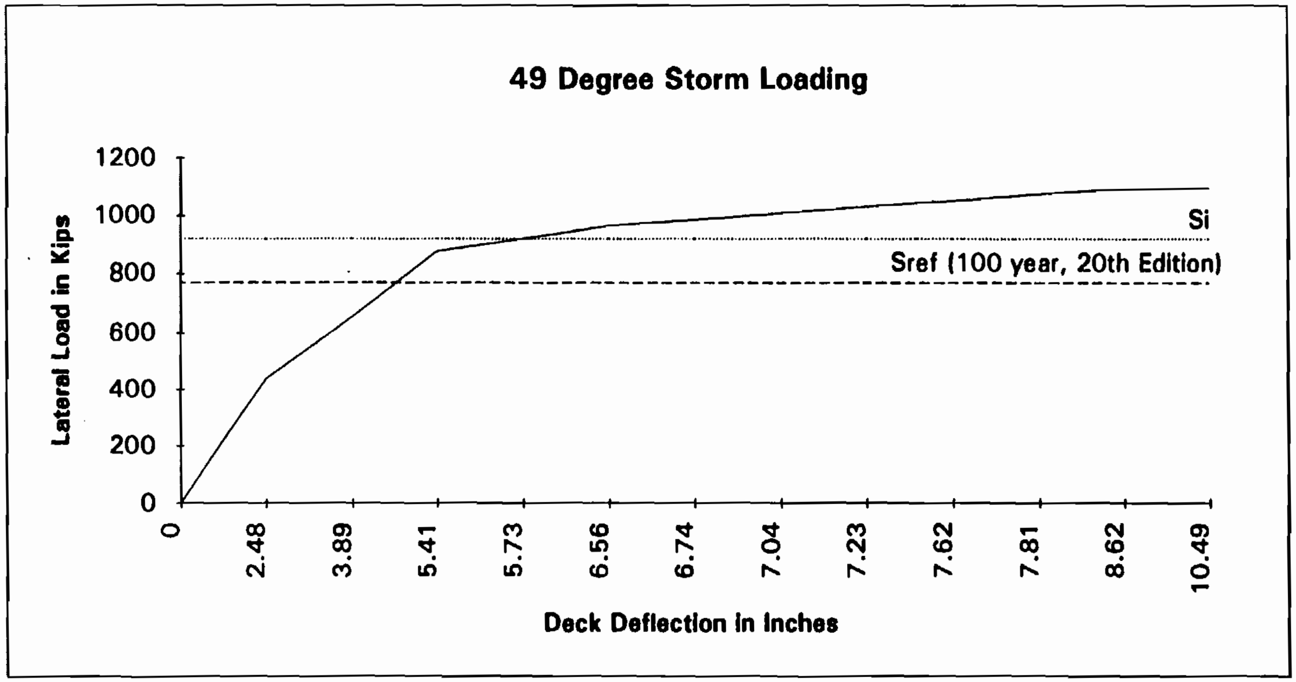
The ultimate strength analysis was performed as a series of incremental linear analyses. Lateral load on the structure was incrementally increased and as members or joints exceeded their buckling or yield strengths they were replaced by their residual capacities. This procedure was repeated until the deck deflection or the number of failed members increased with little increase in loading.

Platform H



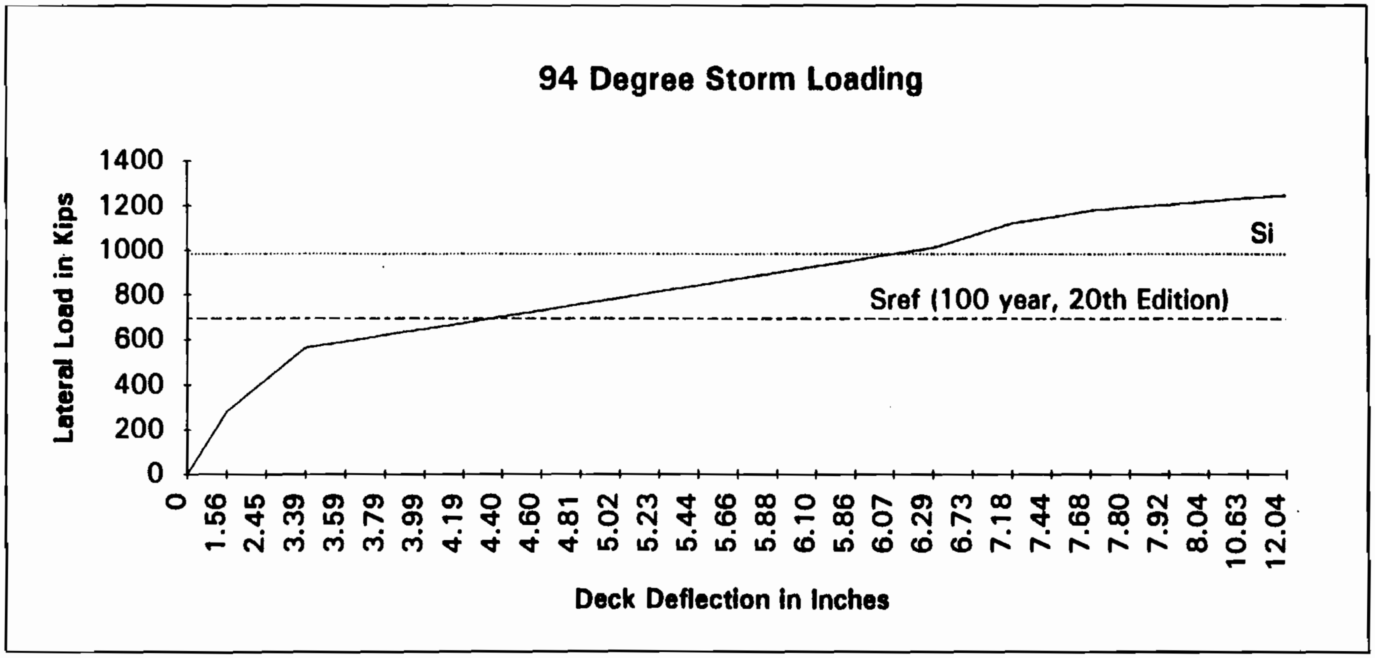
Load Level at which First Component Reaches I.R. of 1.0 (Si)	1116 Kips
Reference Level Load (Sref)	1125 Kips
Ultimate Capacity	1584 Kips
Reserve Strength Ratio (RSR)	1.41
Platform Failure Mode	Jacket

Platform H



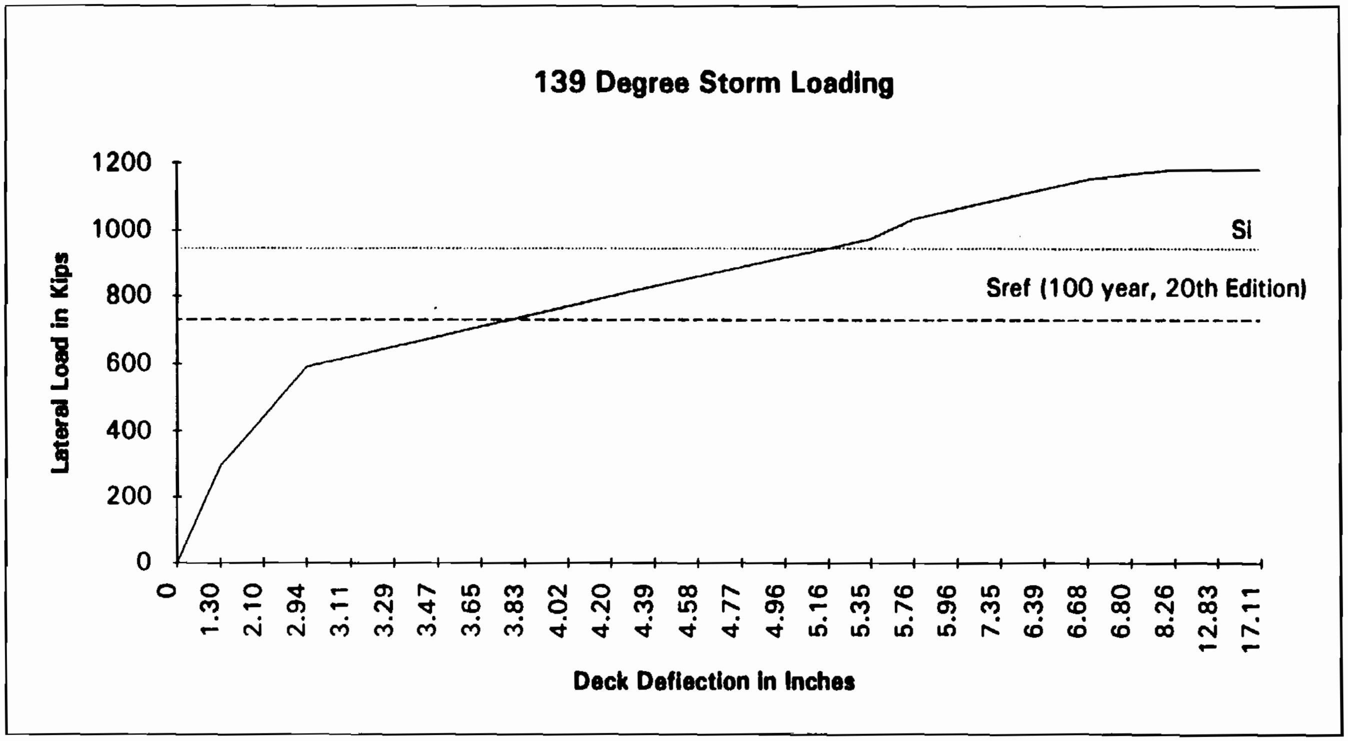
Load Level at which First Component Reaches I.R. of 1.0 (Si)	920.37 Kips
Reference Level Load (Sref)	770.86 Kips
Ultimate Capacity	1096 Kips
Reserve Strength Ratio (RSR)	1.42
Platform Failure Mode	Jacket

Platform H



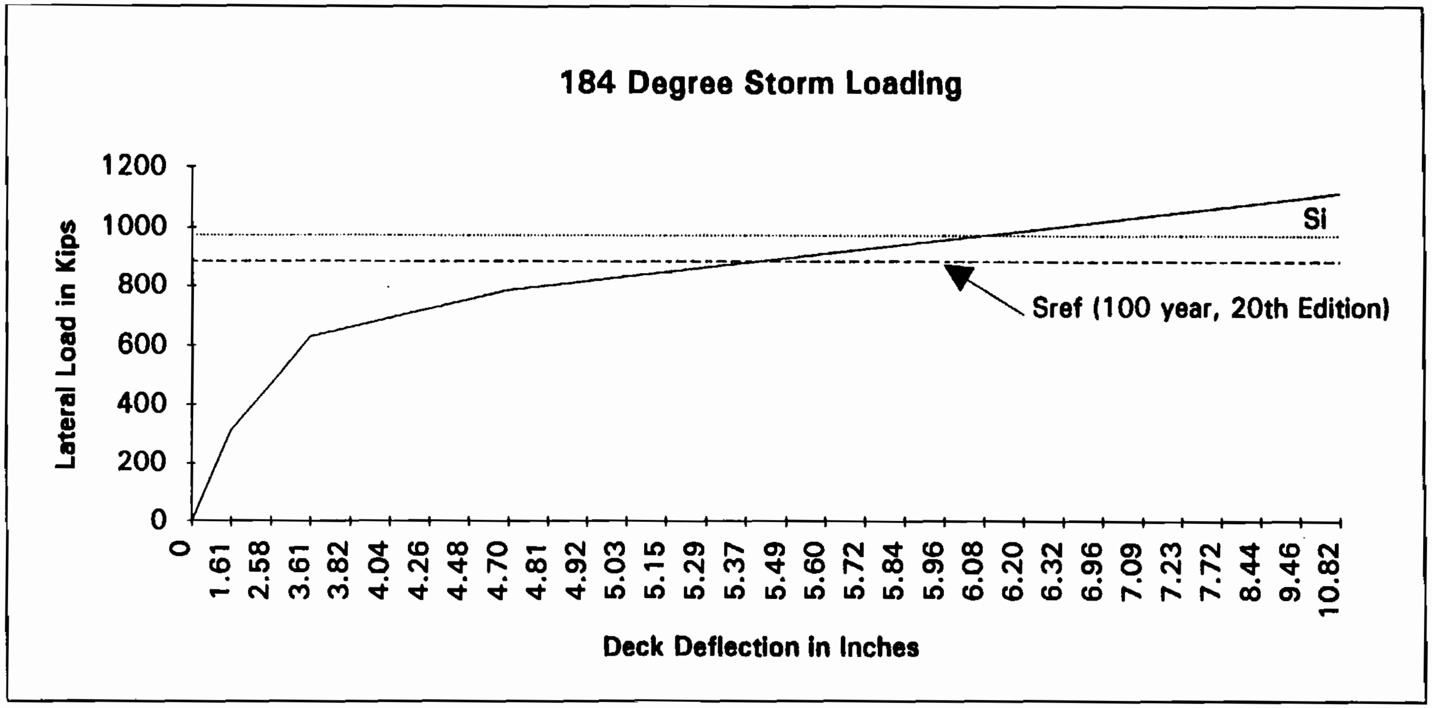
Load Level at which First Component Reaches I.R. of 1.0 (Si)	983.44 Kips
Reference Level Load (Sref)	693.23 Kips
Ultimate Capacity	1250.38 Kips
Reserve Strength Ratio (RSR)	1.80
Platform Failure Mode	Jacket

Platform H



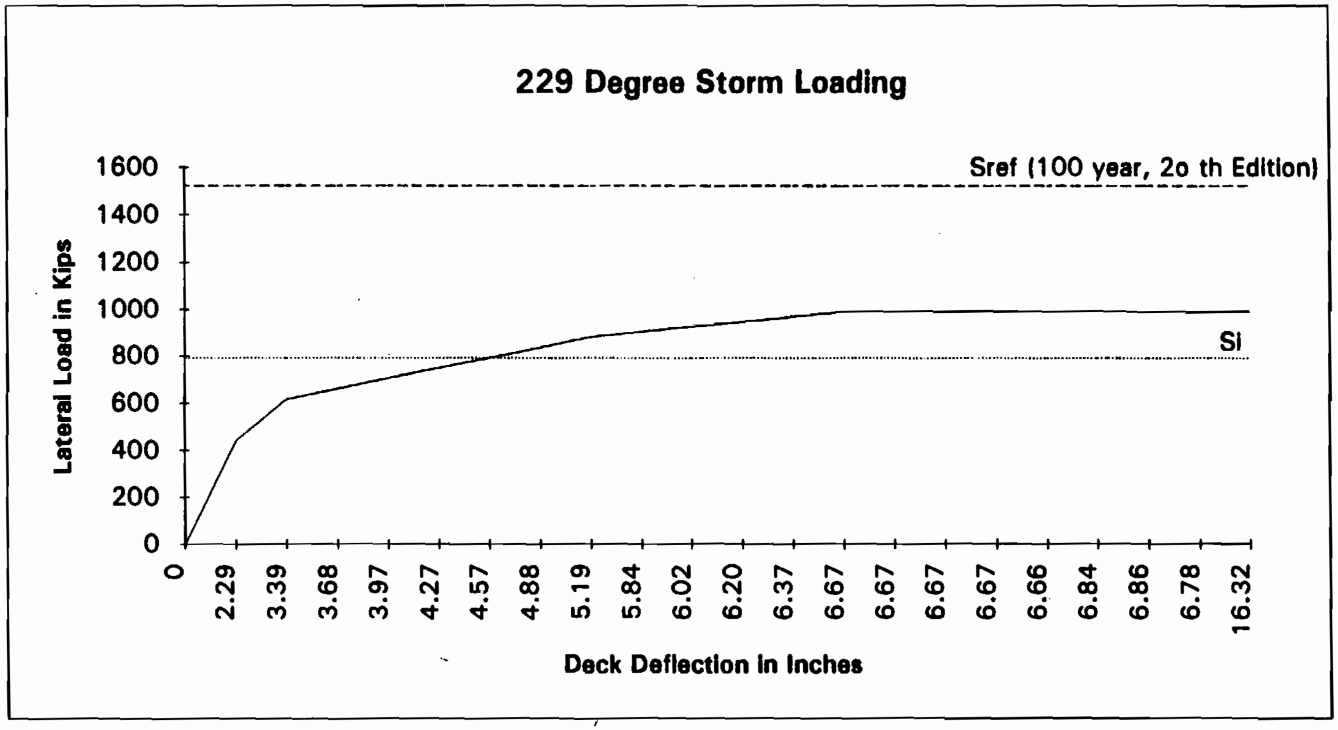
Load Level at which First Component Reaches I.R. of 1.0 (Si)	946.81 Kips
Reference Level Load (Sref)	731.44 Kips
Ultimate Capacity	1183.52 Kips
Reserve Strength Ratio (RSR)	1.62
Platform Failure Mode	Pile

Platform H



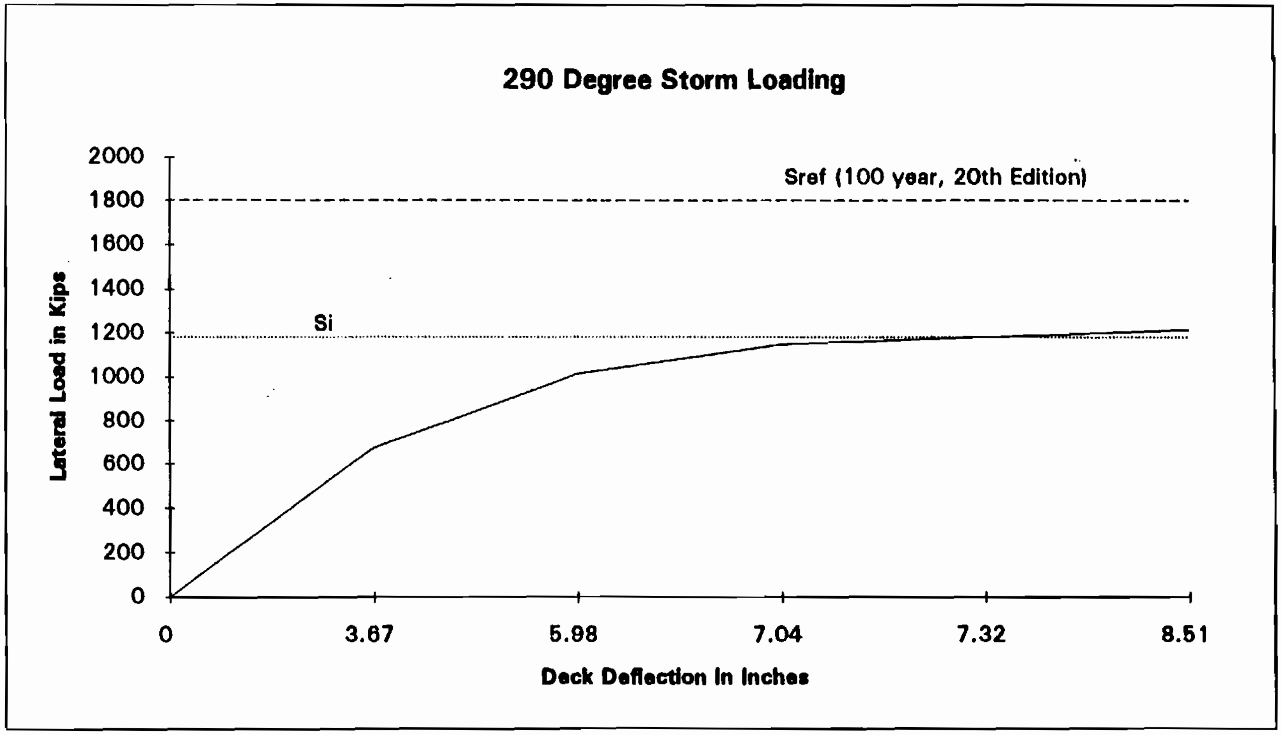
Load Level at which First Component Reaches I.R. of 1.0 (Si)	974.75 Kips
Reference Level Load (Sref)	885.13 Kips
Ultimate Capacity	1116.2 Kips
Reserve Strength Ratio (RSR)	1.26
Platform Failure Mode	Jacket

Platform H



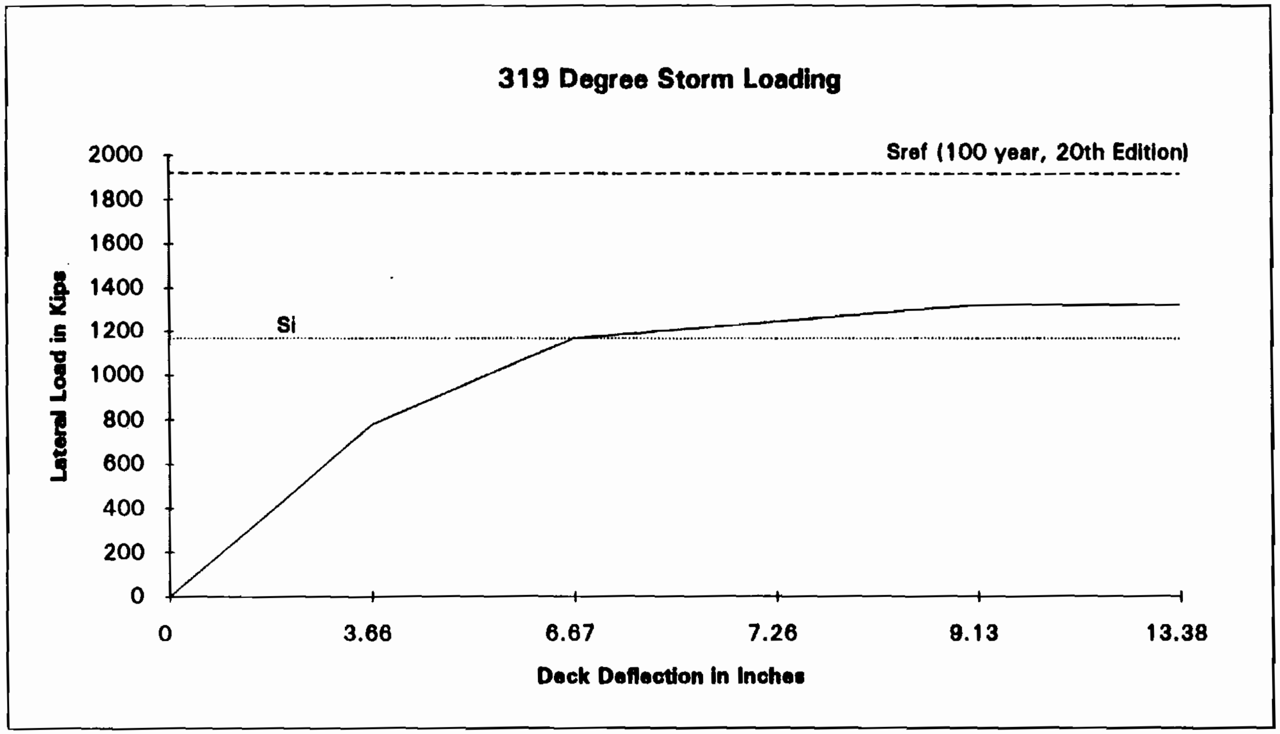
Load Level at which First Component Reaches I.R. of 1.0 (SI)	792.98 Kips
Reference Level Load (Sref)	1521.27 Kips
Ultimate Capacity	991 Kips
Reserve Strength Ratio (RSR)	0.65
Platform Failure Mode	Jacket

Platform H



Load Level at which First Component Reaches I.R. of 1.0 (Si)	1181.9 Kips
Reference Level Load (Sref)	1801.83 Kips
Ultimate Capacity	1215.67 Kips
Reserve Strength Ratio (RSR)	0.67
Platform Failure Mode	Pile

Platform H



Load Level at which First Component Reaches I.R. of 1.0 (Si)	1168 Kips
Reference Level Load (Sref)	1916 Kips
Ultimate Capacity	1324 Kips
Reserve Strength Ratio (RSR)	0.69
Platform Failure Mode	Pile

Trial Application of The
Draft API RP 2A Guidelines for
Assessment of Existing Platforms

PLATFORM H

A.6 Consideration of Mitigation

The reduction of topside loading or platform elements that contribute to lateral load generation is not feasible. Therefore, possible remedial actions to enable the structure to pass the assessment would include addition of jacket bracing members, grouting the piles or installation of a bracing structure. Further analysis of these measures or other mitigating methods has not been undertaken for the trial application.

A.7 Summary

Platform H has had several load generating components added since its reinstallation in 1989. These modifications have resulted in more than a 10% increase in loading from its design basis. The platform is manned, but evacuated during forecasted storms and poses an insignificant environment impact.

Application of the Design Level Analysis metocean criteria indicated that several members would be overstressed for this loading. Therefore, the assessment proceeded to an Ultimate Strength Analysis.

An incremental linear analysis procedure was employed for the Ultimate Strength Analysis. This analysis indicated that the platform had a Reserve Strength Ratio less 1.0 for the three most extreme loading directions. Although not undertaken for the trial application, the application of a non-linear ultimate strength analysis may indicate higher RSR values. Also, the addition of jacket bracing and/or grouting may enable the structure to pass the Design Level Check.

PLATFORM "I"

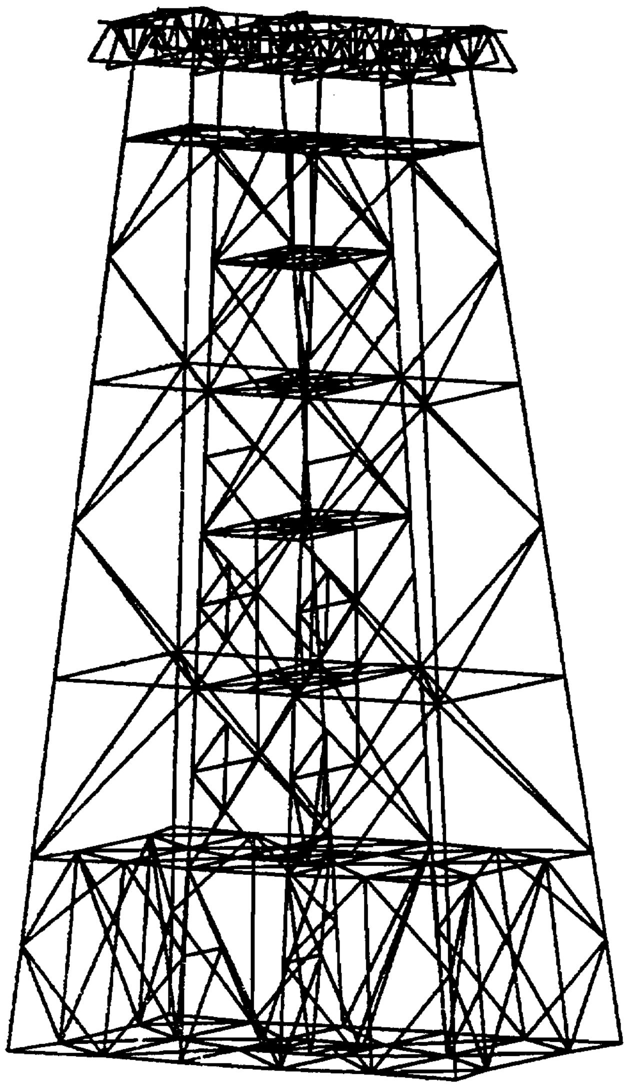
TRIAL APPLICATION OF THE DRAFT API RP 2A GUIDELINES FOR ASSESSMENT OF EXISTING PLATFORMS

For the trial application of the draft API guidelines, ' analyzed SP62A, a Gulf-of-Mexico jacket structure of 1967 vintage in 340 feet of water. The structure passed both the design-level and ultimate-capacity analysis criteria as prescribed in the draft Chapter 17 of the API RP 2A.

1.0 PLATFORM INFORMATION

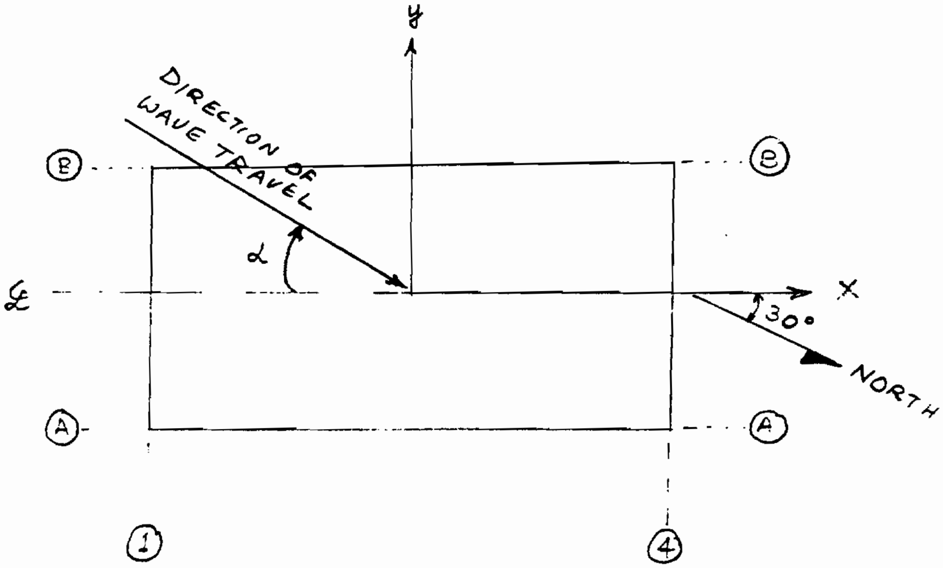
1. Name: SP62A
2. Block: South Pass, Block 62, in the Gulf of Mexico
3. Operator:
4. Function: Manned Drilling and Production (currently production only)
5. Year designed: 1966
6. Year installed: 1967
7. Water depth: 340 ft.
8. No. of legs: 8
9. Foundation: 8 ungrouted main and 8 grouted skirt piles
10. Conductors: 18
11. Deck clearance: 44 ft above mean water level
12. Deck payload: 7433 kips
13. Last inspection: Under water inspection in 1990
14. Structural damage: None except pitting due to corrosion
15. Soil properties: Compiled from boring logs of adjacent sites

Figures 1.1 through 1.16 show the structural framing plan, orientation of the platform, and the pile makeup. Member stress ratios (interaction ratios) greater than 0.85 have been indicated on the figures.



SOUTH PASS 62-A

Figure 1.0



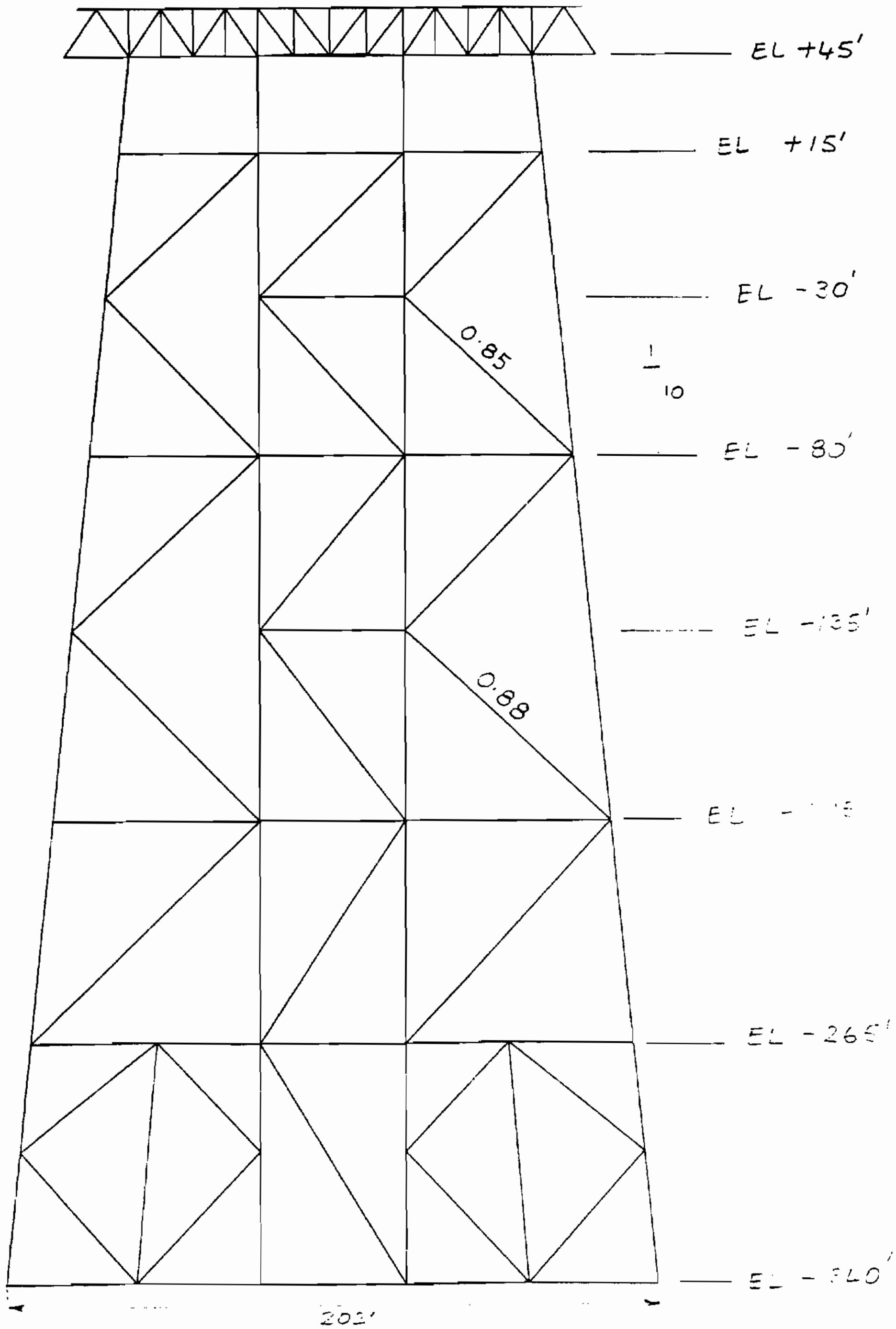
ORIENTATION OF SP62A

α = direction of wave travel
in structure co-ordinate system

θ = wave direction towards clockwise
from North, API RP2A convention

$$\boxed{\theta = \alpha - 30^\circ}$$

Figure 1.2



ROW A

① ②

Figure 1.3

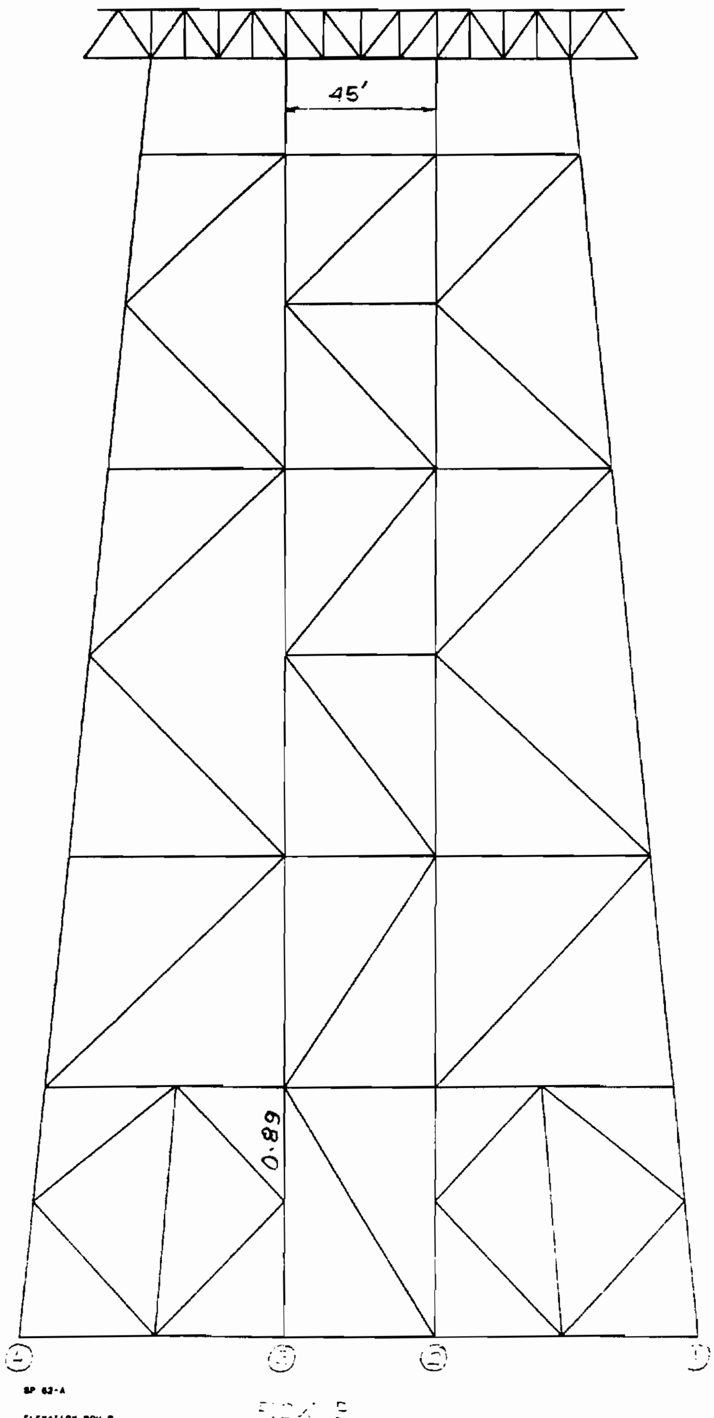


Figure 1.4

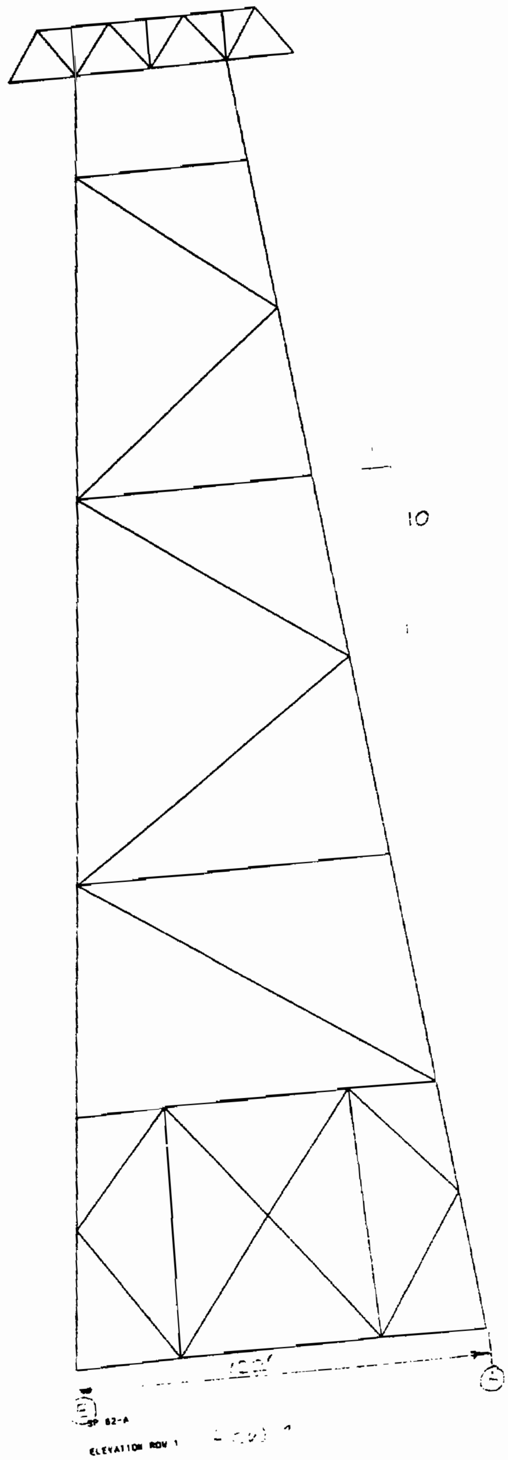
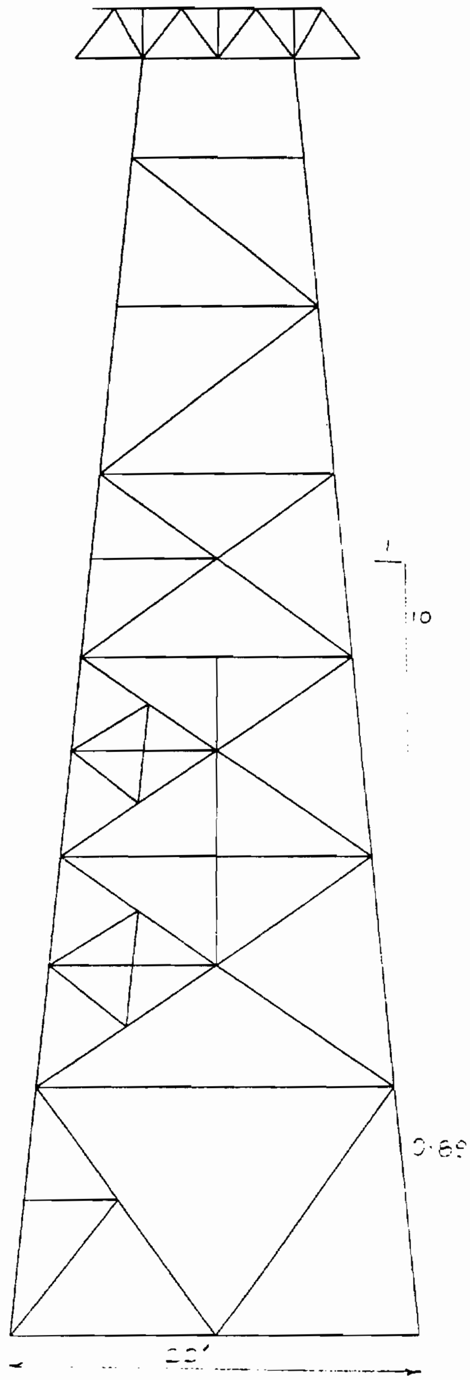


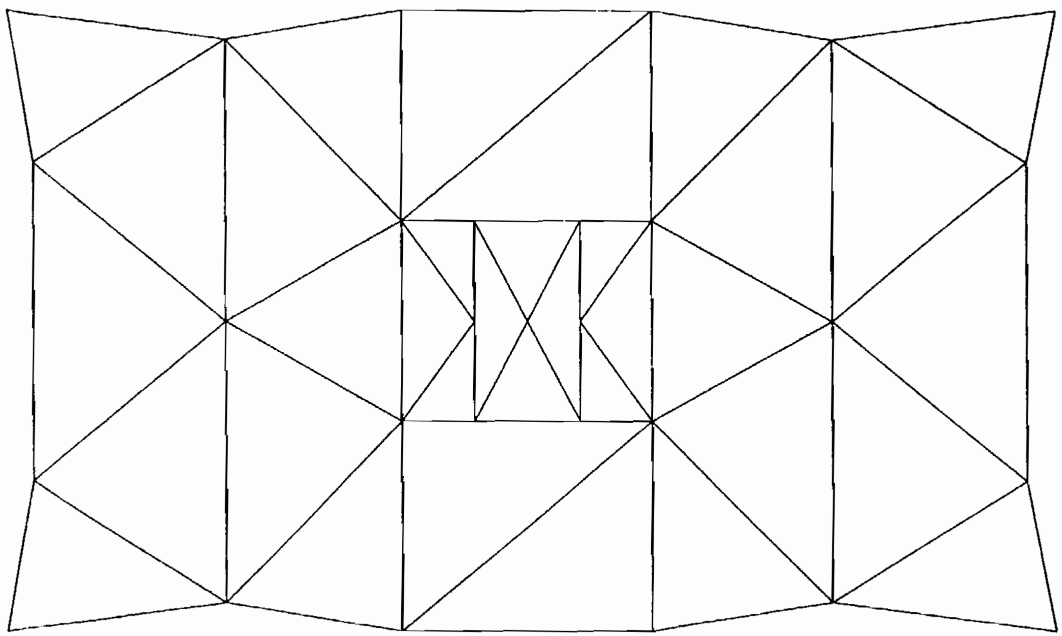
Figure 1.5



SP 62-A

ELEVATION ROW 3

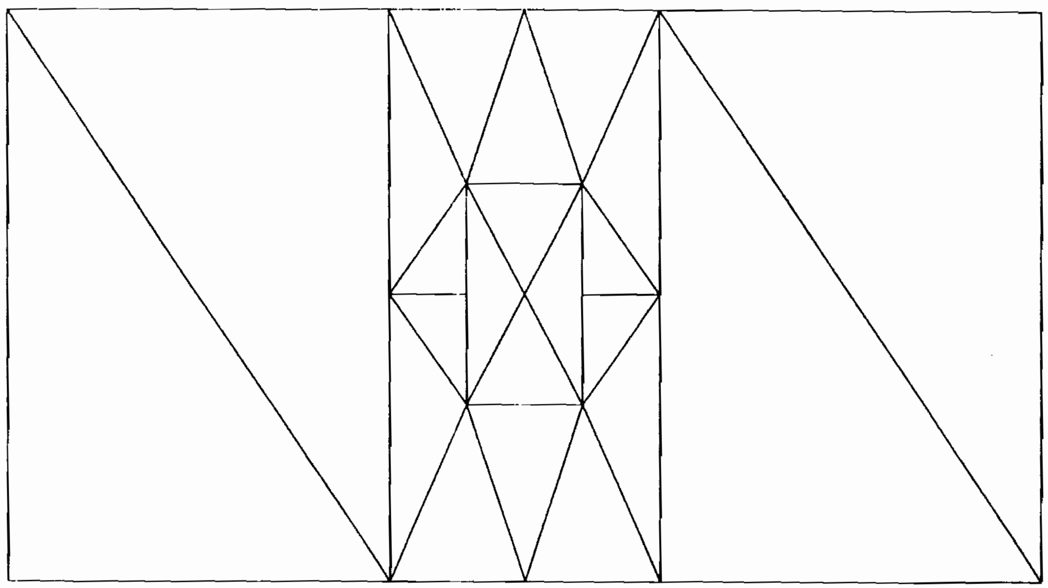
Figure 17



PLAN @ EL. -265'

PLAN AT ELEV. -265'

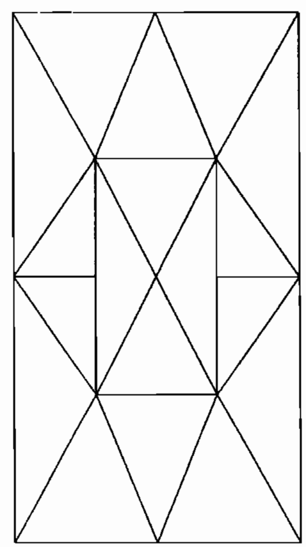
Figure 1.10



SP 82-A
PLAN @ EL. -195'

PLAN AT ELEV. -195'

Figure 1.11



SP 82-A

PLAN @ EL. -135'

PLAN AT ELEV. -135'

Figure 1.12

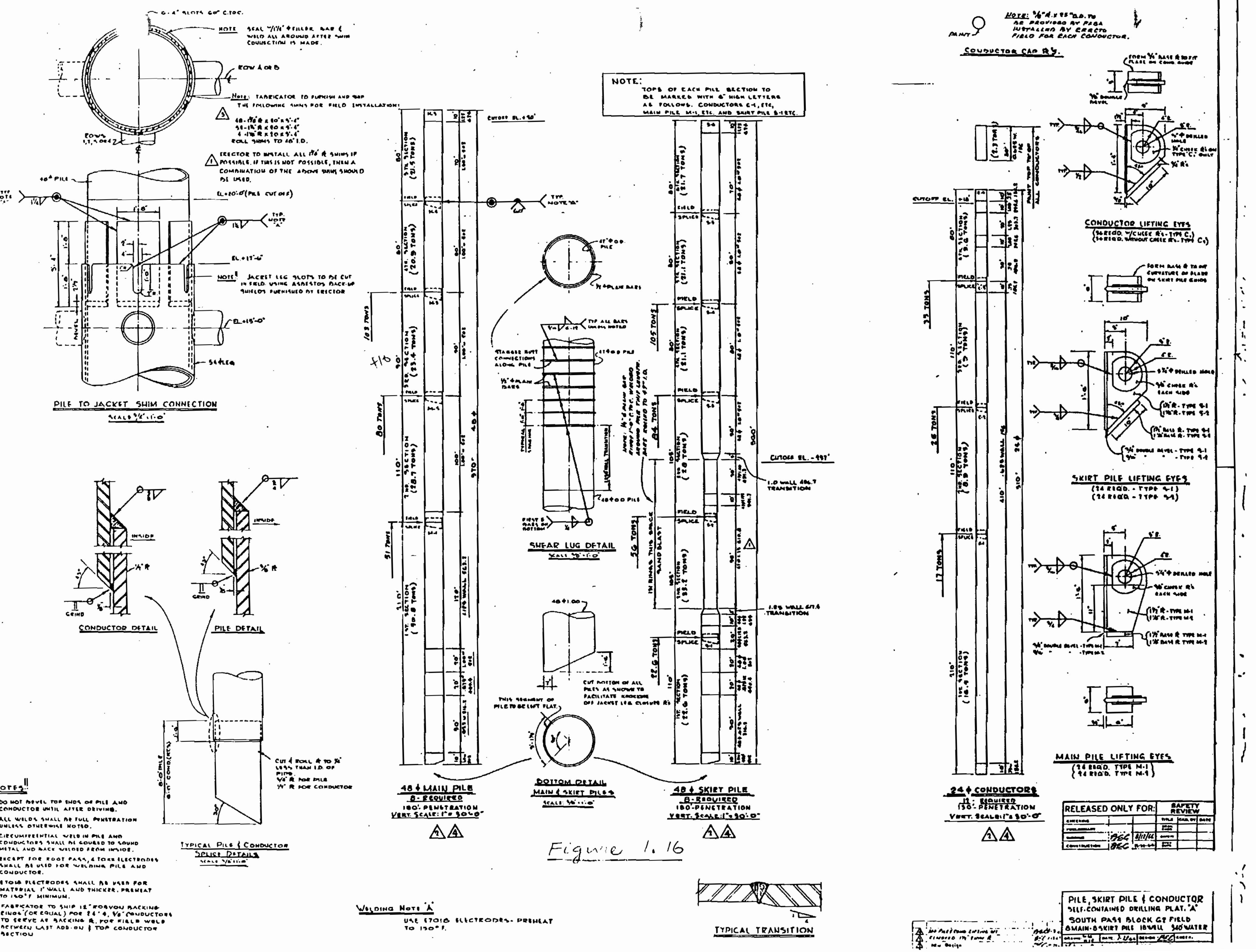

NOTE: TOPS OF EACH PILE SECTION TO BE MARKED WITH 6" HIGH LETTERS AS FOLLOWS. CONDUCTORS C-1, ETC, MAIN PILE M-1, ETC. AND SKIRT PILE S-1 ETC.
PILE TO JACKET SHIM CONNECTION
CONDUCTOR DETAIL
PILE DETAIL
TYPICAL PILE & CONDUCTOR SPLICE DETAILS
48" ϕ MAIN PILE
8-REQUIRED
180'-PENETRATION
VERT. SCALE: 1" = 30'-0"
SHEAR LUG DETAIL
BOTTOM DETAIL MAIN & SKIRT PILES
48" ϕ SKIRT PILE
8-REQUIRED
180'-PENETRATION
VERT. SCALE: 1" = 30'-0"
24" ϕ CONDUCTORS
11-REQUIRED
150'-PENETRATION
VERT. SCALE: 1" = 30'-0"
CONDUCTOR CAP R's.
CONDUCTOR LIFTING EYES
SKIRT PILE LIFTING EYES
MAIN PILE LIFTING EYES
TYPICAL TRANSITION
WELDING NOTE "A"
USE E7018 ELECTRODES - PREHEAT TO 150° F.
RELEASED ONLY FOR: SAFETY REVIEW
PILE, SKIRT PILE & CONDUCTOR
SELF-CONTAINED DRILLING PLAT. "A"
SOUTH PASS BLOCK 62 FIELD

Figure 1.16

PART A: PLATFORM ASSESSMENT

A.1 Platform Selection (Section 17.2)

Heavy marine growth (increase in lateral load) and corrosion observed during the underwater inspection of SP62A may be considered triggers for assessment. SP62A was selected for this study also because it is representative of a number of aging structures operated by Shell Offshore, Inc.

A.2 Condition Assessment (Section 17.4)

Underwater inspection revealed general corrosion and pitting. No other damage to structural members were reported. To model the loss of material due to corrosion, thickness of members were reduced by 1/32" from mean water line to -135 ft. elevation and by 1/64" below -135 ft. elevation.

The inspection revealed heavy marine growth. A large part of the marine growth was reported as soft. The hard portion of the reported marine growth was taken into account for hydrodynamic load calculation.

A.3 Exposure Category (Section 17.4)

The platform was categorized as a "manned evacuated" platform with "insignificant environmental impact".

A.4 Design Basis Checks (Section 17.5 and 17.6)

The platform was not designed to the 9th edition of RP2A (1977) and hence the design basis check is not applicable.

A.5 Analysis Checks (Section 17.5 and 17.6)

The structure passed both the design-level and ultimate-capacity checks. Design-level analysis was carried out using SESAS, an in-house structural analysis program. The ultimate capacity analysis was carried out using CAP, a nonlinear structural analysis program by PMB.

A.5.1 Metocean Criteria

Sudden hurricane loading criteria as prescribed in the draft Chapter 17 of API RP2A were used. Details of the criteria are described separately under sections on design-level and ultimate capacity analyses.

A.5.2 Screening

No screening analysis was carried out for this structure.

A.5.3 Design Level Analysis

A.5.3.1 Metocean Criteria (Sections 17.5 and 17.6)

a.	Wave height:	47.5 ft.
b.	Storm tide:	2.5 ft.
c.	Deck height (min. reqd.) :	36.6 ft.
d.	Wave and current direction:	Omni
e.	Current speed:	1.2 kts
f.	Wave period:	11.3 sec.
g.	Wind speed (1-hr @10m):	55 kts

The 20th edition RP2A recipe was used for calculating environmental loading. Some of the parameters used in the calculation are listed below. The marine growth profile used in the analysis was compiled from the under-water inspection report and the RP2A guideline. Effect of conductor shielding was neglected.

• Apparent wave period:	11.6 sec.
• Wave kinematics factor:	0.88
• Current blockage factor:	0.70 (end on)
	0.85 (diagonal)
	0.80 (broadside)
• Marine growth:	2.4" (+1 to -20 ft. elev.)
	1.5" (-20 to -150 ft. elev)
	1.2" (-150 ft to -340 ft. elev.)
• Drag coefficient	0.65 (smooth)
	1.05 (rough)
• Inertia coefficient	1.60 (smooth)
	1.20 (rough)
• Wave theory:	Stokes Vth order

A.5.3.1 Lateral Load Level

Load combinations analyzed included structural dead weight, equipment loads, and environmental loadings from the following directions.

Wind, wave, and current approach angle (degrees)		Base Shear	OTM
Structure co-ordinate system (see Fig. 1.2)	API convention (towards, clockwise from North)	(kips)	(ft-kips)
0	330	2641	671,660
10	340	2809	722,944
30	0	2782	740,898
70	40	2877	846,925
90	60	2866	874,675
135	105	2815	896,140
150	120	2802	895,155
170	140	2817	898,389
180	150	2639	849,402
190	160	2807	890,833
210	180	2777	872,492
225	195	2789	864,375
250	220	2868	857,651
270	240	2858	827,336
290	260	2879	800,564
305	275	2835	761,007
320	290	2807	729,591
330	300	2803	718,006
342	312	2807	712,005

A.5.3.2 Members with Utilization Ratio > 0.85

Under the action of the above load combinations, none of the jacket members or piles was found to be overstressed. Only three members had stress ratios over 0.85; these stress ratios are shown against the respective members in Figures 1.3, 1.4, and 1.7. Representative joints checked were found to be adequate. For joint checks the criteria specified in Section 17.7.2c of the draft guidelines was used.

A.5.3.2 Foundation

The make up of piles and conductors and their penetrations is as shown in Figure 1.16. All the main and skirt piles have 48" diameter and 180 ft. penetration. The soil profile used in the analysis was compiled from Geotechnical Investigations AM-3 and AM-3-N (Report Nos. 66-292-1 & 5, McClelland Engineers). The maximum main pile axial load is 1918 kips and that for the skirt pile is 2718 kips. According to Geotechnical Investigation AM-3 (Report No. 66-292-1, McClelland Engineers), the axial capacity of 48" diameter piles at 180 ft. penetration is 7600 kips. Hence, it was concluded that adequate axial capacity of the foundation exists.

A.5.4 Ultimate Strength Analysis

Push-over analyses have been performed to calculate the ultimate strength of the platform using the non-linear program "CAP".

A.5.4.1 Metocean Criteria for Ultimate Strength Analysis

The platform is classified as "insignificant environmental impact/manned-evacuated". The following metocean criteria was used per Section 17:

a. MGL water depth = 340 feet

b. Tide = 2.5 feet

c. Wave height varies by direction as follows:

Approach Angle (deg)	Wave Ht (feet)
0.0	61.5
45.0	58.4
90.0	52.3
135.0	43.1
180.0	43.1
225.0	43.1
270.0	46.1
315.0	55.4

d. Wave period = 12.5 second for all waves

e. 3rd order Stream Function wave theory was used.

f. Current was applied to the structure per Section 17. Current velocity varies by direction. Surface current for each direction is as follows:

Approach Angle (deg)	Current Velocity (fps)
0.0	2.13
45.0	2.45
90.0	2.06
135.0	1.81
180.0	1.49
225.0	1.81
270.0	1.82
315.0	2.33

Note: Currents include appropriate blockage factors and directionality corrections.

g. Wind Velocity = 70 knots

h. Marine growth

Based upon marine growth survey performed on this structure, the following marine growth thickness was used:

(+)1' el.	to	(-)20' el.	=	2.4"
(-)20' el.	to	(-)150' el.	=	1.5"
(-)150' el.	to	(-)340' el.	=	1.2"

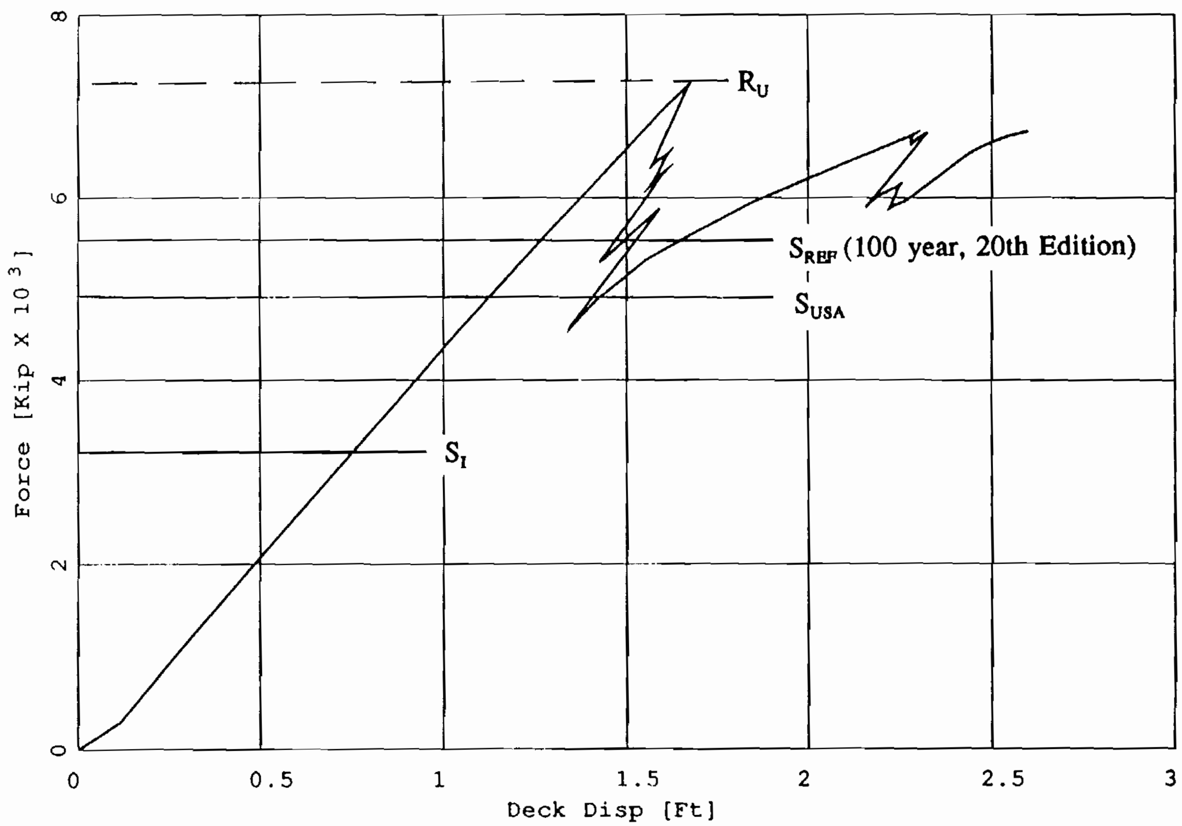
i. Wave kinematics factor = 0.88

j. Drag and Inertia Coefficients

Smooth	$C_D = 0.65$	$C_M = 1.6$
Rough	$C_D = 1.05$	$C_M = 1.2$

A.5.4.2 Analysis Results

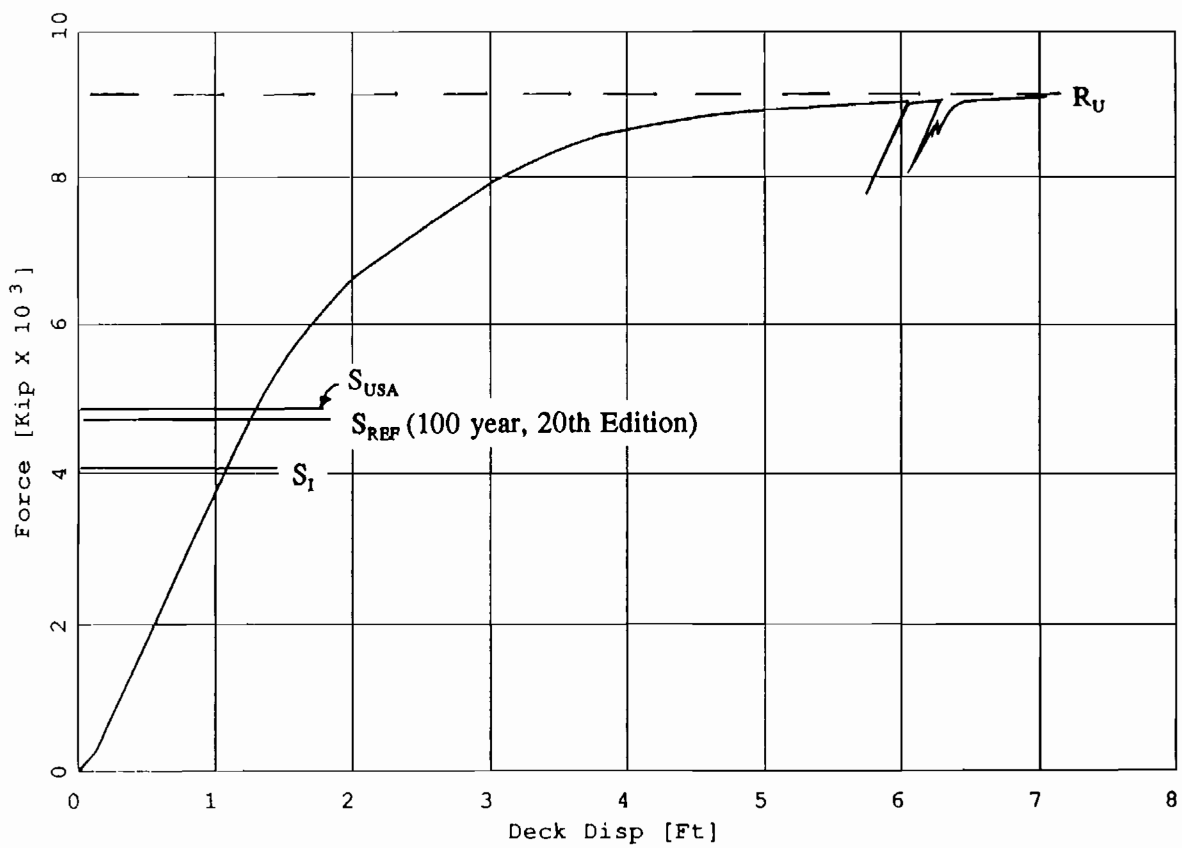
The results of ultimate strength analysis are summarized in the attached Figures and Tables. The collapse mechanism of the platform in each direction is depicted in Figures 4, 5, and 6.



ULTIMATE STRENGTH ANALYSIS RESULTS

1.	Load level at which first component reaches I.R. of 1.0 (S_I)	3260 kips
2.	100 year, 20th Edition Reference Level Load (S_{REF})	5567 kips
3.	Sudden Hurricane Ultimate Strength Analysis Load (S_{USA})	4929 kips
4.	Ultimate Capacity (R_U)	7238 kips
5.	Reserve Strength Ratio (RSR)	1.30
6.	Platform Failure Mode	Jacket

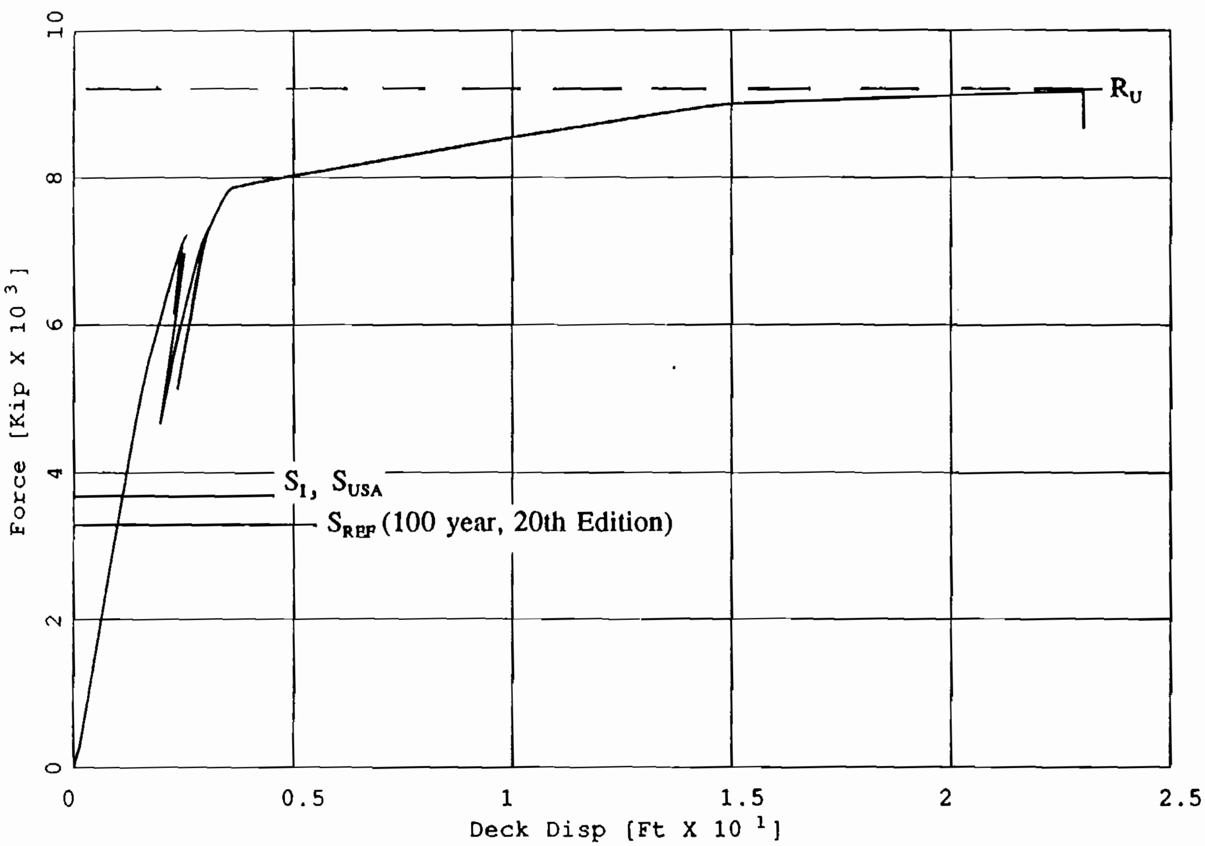
Figure A.5.4.1: ULTIMATE STRENGTH ANALYSIS - LATERAL LOAD vs. DECK DISPLACEMENT ENDON (0 DEG) DIRECTION



ULTIMATE STRENGTH ANALYSIS RESULTS

1.	Load level at which first component reaches I.R. of 1.0 (S_I)	4071 kips
2.	100 year, 20th Edition Reference Level Load (S_{REF})	4738 kips
3.	Sudden Hurricane Ultimate Strength Analysis Load (S_{USA})	4857 kips
4.	Ultimate Capacity (R_U)	9100 kips
5.	Reserve Strength Ratio (RSR)	1.92
6.	Platform Failure Mode	Pile

Figure A.5.4.2: ULTIMATE STRENGTH ANALYSIS - LATERAL LOAD vs. DECK DISPLACEMENT DIAGONAL (45 DEG) DIRECTION

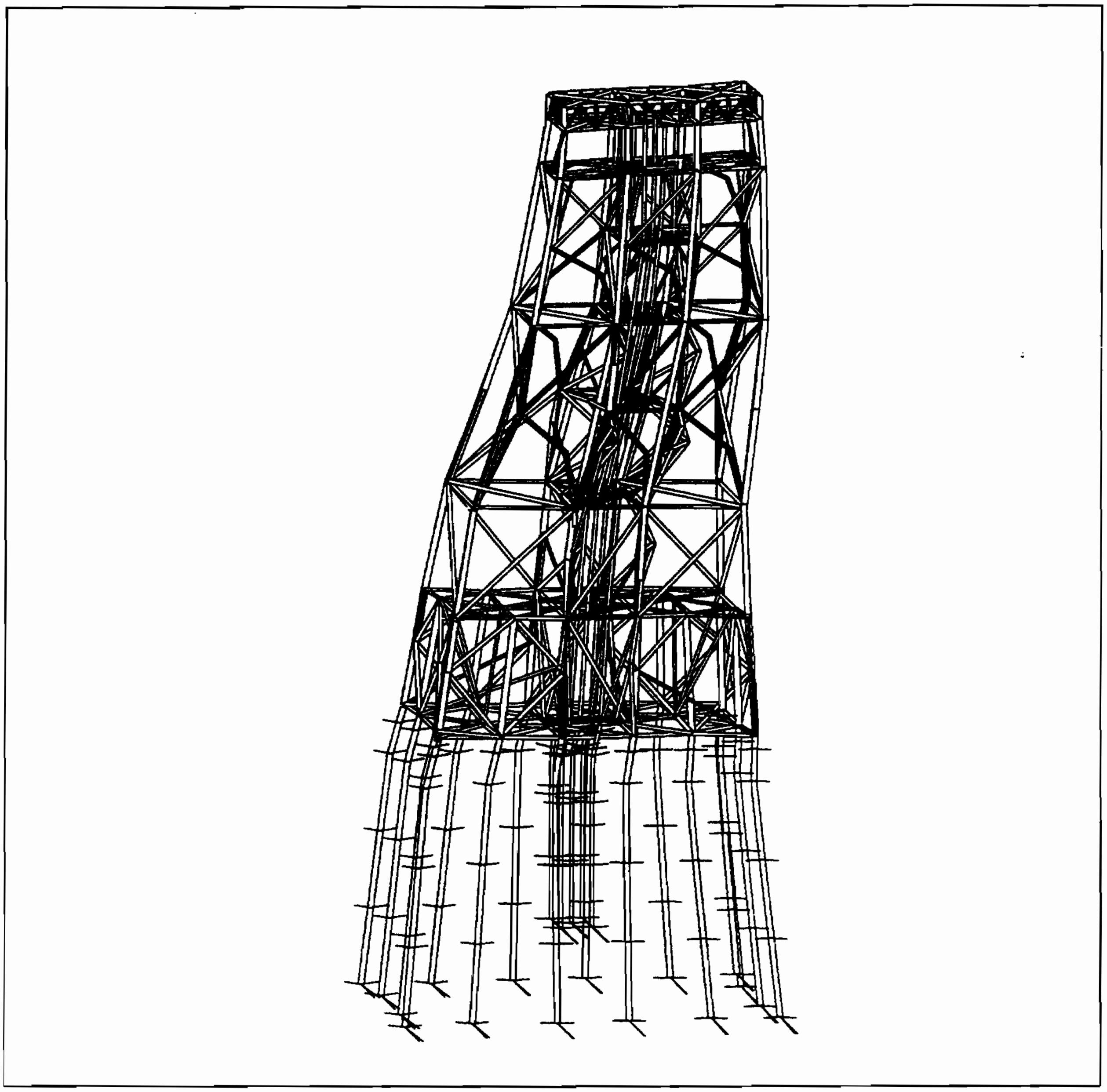


ULTIMATE STRENGTH ANALYSIS RESULTS

1.	Load level at which first component reaches I.R. of 1.0 (S_I)	3623 **kips**
2.	100 year, 20th Edition Reference Level Load (S_{REF})	3307 **kips**
3.	Sudden Hurricane Ultimate Strength Analysis Load (S_{USA})	3657 **kips**
4.	Ultimate Capacity (R_U)	9140 **kips**
5.	Reserve Strength Ratio (RSR)	2.76
6.	Platform Failure Mode	**Pile**

Figure A.5.4.3: ULTIMATE STRENGTH ANALYSIS - LATERAL LOAD vs. DECK DISPLACEMENT BROADSIDE (90 DEG) DIRECTION

Project: platform Model: hkim Version: 5 Wed Jul 27 08:56:20 1994 *



CAP

Figure A.5.4.4: COLLAPSE MECHANISM - ENDON

Inelastic Events Legend

	Elastic		Strut Buckling
	Strut Residual		Strut Reloading
	Plastic Strut/NLTruss		Beam Clmn Initial Yield
	Beam Clmn Fully Plastic		Fracture

Project: platform Model: hkim Version: 6 Wed Jul 27 09:03:03 1994 *

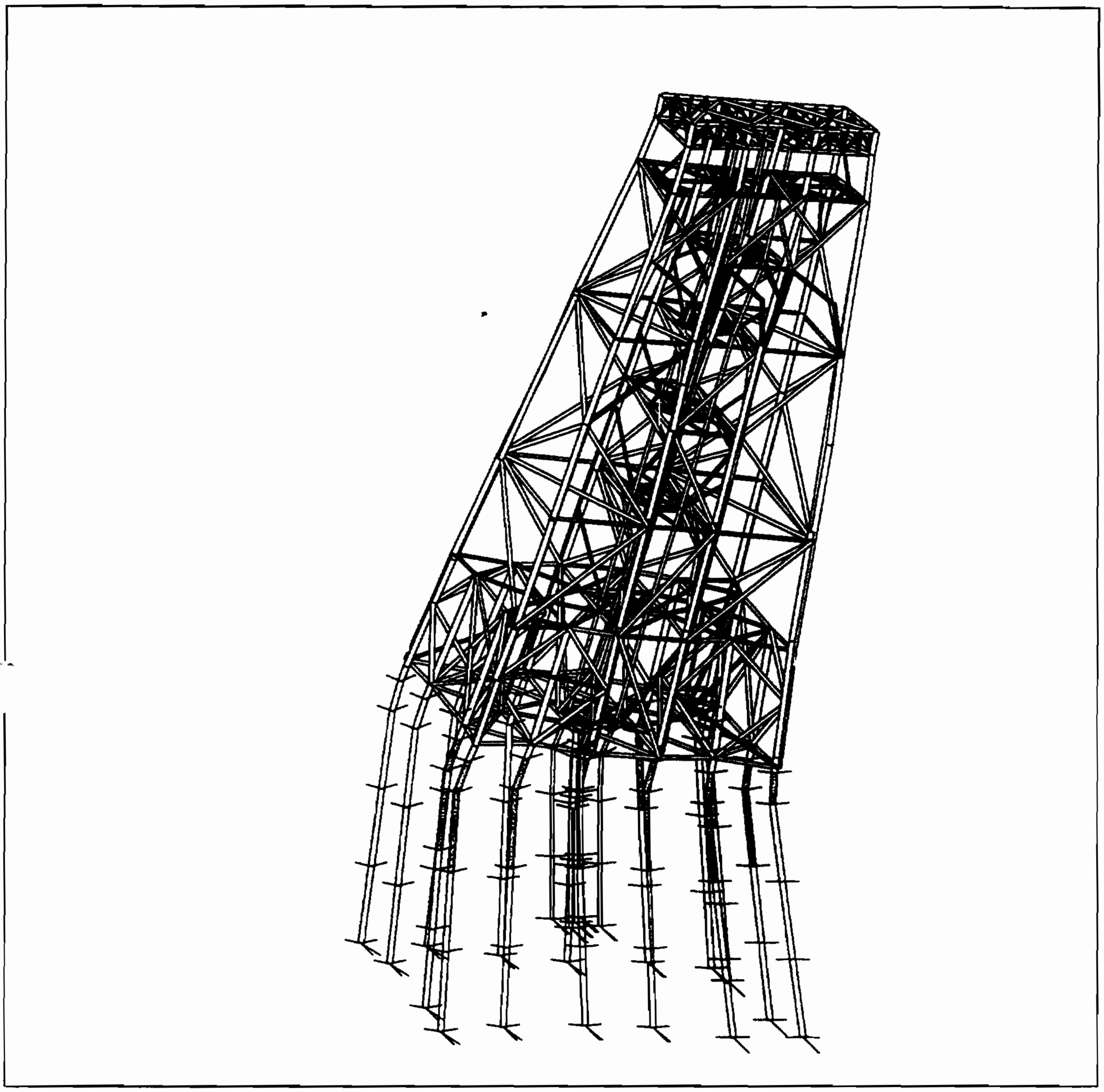
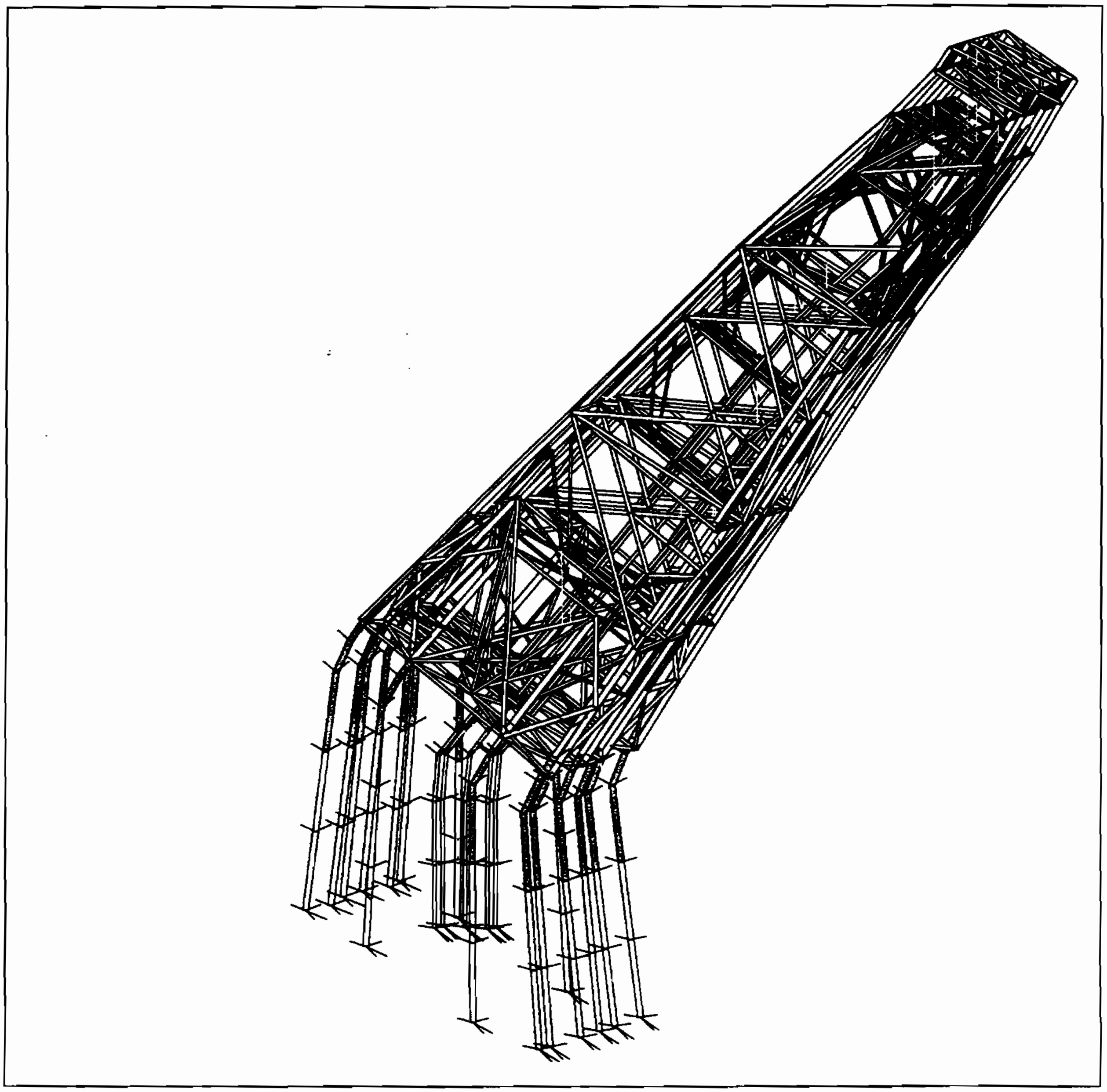


Figure A.5.4.5: COLLAPSE MECHANISM - DIAGONAL

Inelastic Events Legend

	Elastic		Strut Buckling
	Strut Residual		Strut Reloading
	Plastic Strut/NLTruss		Beam Clmn Initial Yield
	Beam Clmn Fully Plastic		Fracture

Project: platform Model: hkim Version: 7   Wed Jul 27 09:26:52 1994 *



CAP

Figure A.5.4.6: COLLAPSE MECHANISM-BROADSIDE

Inelastic Events Legend

	Elastic		Strut Buckling
	Strut Residual		Strut Reloading
	Plastic Strut/NLTruss		Beam Clmn Initial Yield
	Beam Clmn Fully Plastic		Fracture

Table A.5.4.1

ULTIMATE STRENGTH ANALYSIS
LATERAL LOAD vs. DECK DISPLACEMENT
ENDON (0 DEG) DIRECTION

(Load level at which first component reaches I.R. of 1.0 = 3260 kips)

Load Step	Deck Disp (feet)	Lateral load (kips)
0	0.000	0.000
1	0.115	287.628
2	0.413	1674.615
3	0.710	3042.408
4	1.005	4375.228
5	1.298	5669.838
6	1.591	6928.174
7	1.605	6985.452
8	1.615	7020.893
9	1.631	7083.240
10	1.659	7190.594
11	1.666	7215.041
12	1.669	7225.863
13	1.671	7230.551
14	1.673	7237.938
15	1.669	7210.443
16	1.609	6698.163
17	1.584	6476.988
18	1.572	6380.752
19	1.571	6370.854
20	1.569	6352.897
21	1.588	6416.562
22	1.597	6438.893
23	1.611	6477.823
24	1.613	6481.709
25	1.615	6490.626
26	1.610	6461.395
27	1.592	6319.133
28	1.587	6284.346
29	1.579	6228.762
30	1.575	6205.183
31	1.568	6163.548
32	1.566	6145.994
33	1.584	6220.829
34	1.590	6238.781
35	1.600	6271.152
36	1.604	6285.159
37	1.606	6289.915
38	1.601	6269.345
39	1.522	5827.145
40	1.461	5487.124
41	1.435	5339.430
42	1.432	5323.893
43	1.431	5317.012
44	1.429	5305.042
45	1.425	5280.173
46	1.430	5305.302
47	1.434	5315.950
48	1.436	5315.630
49	1.439	5322.024
50	1.441	5335.886
51	1.458	5394.206
52	1.468	5429.596
53	1.484	5492.607
54	1.513	5601.759
55	1.562	5787.749
56	1.571	5815.166
57	1.585	5860.387
58	1.589	5870.414
59	1.584	5859.217
60	1.589	5869.690
61	1.584	5875.461
62	1.589	5868.464

Load Step	Deck Disp (feet)	Lateral load (kips)
63	1.585	5846.640
64	1.514	5472.752
65	1.393	4829.510
66	1.380	4759.739
67	1.357	4639.040
68	1.347	4586.924
69	1.344	4555.854
70	1.419	4873.394
71	1.551	5308.282
72	1.568	5348.421
73	1.584	5385.423
74	1.592	5401.533
75	1.604	5429.365
76	1.609	5441.390
77	1.618	5462.262
78	1.634	5495.752
79	1.644	5517.894
80	1.648	5527.428
81	1.656	5544.030
82	1.659	5551.218
83	1.664	5563.696
84	1.667	5568.997
85	1.671	5578.313
86	1.673	5582.206
87	1.676	5588.718
88	1.680	5598.935
89	1.682	5603.313
90	1.686	5610.360
91	1.690	5619.638
92	1.693	5626.491
93	1.698	5636.634
94	1.700	5641.105
95	1.703	5647.991
96	1.708	5657.565
97	1.709	5660.643
98	1.712	5666.143
99	1.716	5674.670
100	1.720	5684.422
101	1.722	5688.872
102	1.726	5695.653
103	1.730	5705.362
104	1.732	5709.808
105	1.735	5716.871
106	1.740	5726.479
107	1.744	5736.521
108	1.746	5740.875
109	1.750	5747.519
110	1.754	5757.111
111	1.756	5761.534
112	1.760	5768.896
113	1.764	5778.190
114	1.769	5788.055
115	1.771	5792.055
116	1.775	5801.867
117	1.780	5811.481
118	1.784	5820.901
119	1.789	5830.763
120	1.793	5840.525
121	1.798	5850.128
122	1.802	5859.540
123	1.807	5868.661
124	1.811	5878.150
125	1.816	5888.294

Load Step	Deck Disp (feet)	Lateral load (kips)
126	1.818	5892.773
127	1.821	5899.714
128	1.826	5909.051
129	1.830	5918.779
130	1.835	5927.835
131	1.839	5936.289
132	1.843	5944.765
133	1.848	5952.681
134	1.852	5960.321
135	1.857	5967.514
136	1.861	5975.878
137	1.863	5979.727
138	1.868	5987.673
139	1.872	5994.874
140	1.877	6003.148
141	1.879	6007.137
142	1.882	6012.686
143	1.887	6019.997
144	1.891	6028.132
145	1.896	6036.096
146	1.900	6043.505
147	1.905	6051.204
148	1.910	6059.311
149	1.914	6067.845
150	1.919	6075.316
151	1.923	6082.940
152	1.928	6090.610
153	1.932	6098.002
154	1.937	6104.158
155	1.941	6110.754
156	1.945	6118.356
157	1.950	6126.768
158	1.952	6130.703
159	1.956	6137.650
160	1.957	6140.625
161	1.960	6145.304
162	1.964	6153.219
163	1.969	6160.902
164	1.973	6168.193
165	1.978	6175.614
166	1.982	6182.341
167	1.987	6189.355
168	1.991	6197.144
169	1.996	6205.725
170	2.000	6214.401
171	2.005	6222.337
172	2.009	6230.067
173	2.014	6237.035
174	2.018	6244.159
175	2.023	6250.489
176	2.027	6256.986
177	2.032	6264.143
178	2.036	6272.437
179	2.041	6281.619
180	2.043	6285.589
181	2.046	6291.751
182	2.051	6299.831
183	2.055	6307.926
184	2.060	6315.107
185	2.064	6322.372
186	2.069	6329.091
187	2.073	6336.224
188	2.078	6343.554

Load Step	Deck Disp (feet)	Lateral load (kips)
189	2.082	6352.346
190	2.087	6361.008
191	2.092	6368.817
192	2.096	6376.394
193	2.101	6383.997
194	2.105	6391.323
195	2.110	6398.979
196	2.114	6405.959
197	2.119	6413.677
198	2.123	6422.613
199	2.125	6425.977
200	2.130	6433.640
201	2.134	6441.122
202	2.139	6448.895
203	2.143	6456.673
204	2.148	6464.074
205	2.153	6471.689
206	2.157	6479.269
207	2.162	6486.722
208	2.166	6493.833
209	2.171	6500.907
210	2.175	6508.016
211	2.180	6515.707
212	2.184	6523.927
213	2.189	6532.381
214	2.193	6539.514
215	2.198	6546.424
216	2.202	6554.183
217	2.207	6561.972
218	2.211	6568.945
219	2.216	6576.479
220	2.220	6584.320
221	2.225	6592.149
222	2.229	6600.295
223	2.234	6608.279
224	2.239	6615.504
225	2.243	6622.928
226	2.248	6629.754
227	2.252	6636.296
228	2.256	6642.974
229	2.261	6650.007
230	2.265	6658.007
231	2.270	6665.808
232	2.275	6673.869
233	2.279	6682.122
234	2.284	6689.984
235	2.280	6676.959
236	2.279	6643.755
237	2.275	6599.782
238	2.277	6584.803
239	2.296	6653.593
240	2.305	6676.536
241	2.309	6684.452
242	2.311	6687.749
243	2.313	6693.704
244	2.318	6702.096
245	2.323	6710.492
246	2.319	6698.865
247	2.252	6371.762
248	2.224	6233.483
249	2.174	5993.963
250	2.168	5968.111
251	2.159	5923.253

Load Step	Deck Disp (feet)	Lateral load (kips)
252	2.155	5903.737
253	2.153	5895.288
54	2.157	5908.146
55	2.163	5930.334
56	2.173	5968.765
257	2.192	6031.595
258	2.206	6057.480
259	2.230	6108.102
260	2.241	6129.771
261	2.245	6139.158
262	2.247	6143.219
263	2.245	6127.921
264	2.242	6098.320
265	2.236	6047.105
266	2.226	5959.020
267	2.222	5920.820
268	2.220	5904.508
269	2.219	5897.445
270	2.218	5885.262
271	2.240	5928.282
272	2.250	5945.103
273	2.255	5951.919
274	2.262	5964.236
275	2.273	5998.780
276	2.293	6058.070
277	2.327	6160.704
278	2.386	6338.125
279	2.412	6414.511
280	2.423	6448.094
281	2.442	6500.030
282	2.453	6523.232
283	2.472	6562.849
284	2.492	6596.780
285	2.525	6648.516
286	2.533	6658.365
287	2.536	6663.111
288	2.542	6671.338
289	2.545	6674.857
290	2.550	6680.365
291	2.569	6699.974
292	2.572	6702.108
293	2.575	6705.812
294	2.582	6712.242
295	2.585	6715.007
296	2.589	6719.805
297	2.591	6721.870
298	2.595	6725.470
299	2.597	6727.003
300	2.599	6729.697

Table A.5.4.1a

ULTIMATE STRENGTH ANALYSIS
FAILED ELEMENTS & FAILURE MODE
ENDON (0 DEG) DIRECTION

Member Name	Load Step	Element Type	Worst Event Description
10hz-12	79	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-13	173	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-14	209	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-15	193	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-16	184	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-17	224	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-18	184	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-40	191	Beam Column	Beam Clmn Initial Yield (0,1)
10hz-5	71	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-6	112	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-7	118	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-8	55	Beam Column	Beam Clmn Initial Yield (1,1)
10hz-9	77	Beam Column	Beam Clmn Initial Yield (1,1)
3hz1-11	285	Beam Column	Beam Clmn Initial Yield (0,1)
3hz1-6	284	Beam Column	Beam Clmn Initial Yield (1,0)
5hz2-27	196	Beam Column	Beam Clmn Initial Yield (1,0)
5hz2-30	240	Beam Column	Beam Clmn Initial Yield (0,1)
9hz-38	285	Beam Column	Beam Clmn Initial Yield (1,0)
9hz-41	237	Beam Column	Beam Clmn Initial Yield (1,1)
legA1-3	285	Beam Column	Beam Clmn Initial Yield (0,1)
legA1-6	283	Beam Column	Beam Clmn Initial Yield (1,0)
legB3-15	231	Beam Column	Beam Clmn Initial Yield (0,1)
legB4-10	284	Beam Column	Beam Clmn Initial Yield (0,1)
legB4-13	221	Beam Column	Beam Clmn Initial Yield (0,1)
legB4-9	232	Beam Column	Beam Clmn Initial Yield (1,0)
vd2-28	262	Strut	Strut Buckling
vd2-31	245	Strut	Strut Buckling
vd2-32	134	Strut	Strut Buckling
vd2-34	281	Strut	Plastic Strut
vd2-35	234	Strut	Strut Buckling
vd2-36	281	Strut	Plastic Strut
vd3-37	55	Strut	Plastic Strut
vd3-38	283	Strut	Plastic Strut
vd3-39	290	Strut	Plastic Strut
vd3-40	37	Strut	Strut Buckling
vd3-41	10	Strut	Strut Buckling
vd3-42	25	Strut	Strut Buckling
vd4-43	58	Strut	Strut Buckling
vd4-44	7	Strut	Strut Buckling
vd4-45	14	Strut	Strut Buckling

Table A.5.4.2

ULTIMATE STRENGTH ANALYSIS
LATERAL LOAD vs. DECK DISPLACEMENT
DIAGONAL (45 DEG) DIRECTION

(Load level at which first component reaches I.R. of 1.0 = 4071 kips)

Load Step	Deck Disp (feet)	Lateral load (kips)
0	0.000	0.000
1	0.131	282.833
2	0.469	1653.561
3	0.807	3010.188
4	1.145	4318.964
5	1.218	4588.310
6	1.344	5035.513
7	1.441	5335.909
8	1.526	5578.633
9	1.591	5744.074
10	1.629	5831.384
11	1.694	5984.017
12	1.807	6227.459
13	1.887	6396.740
14	1.922	6467.902
15	1.957	6538.051
16	1.972	6567.612
17	1.991	6598.621
18	2.004	6618.161
19	2.027	6653.070
20	2.066	6713.310
21	2.136	6806.268
22	2.182	6866.080
23	2.261	6970.270
24	2.397	7150.317
25	2.635	7456.908
26	2.981	7892.170
27	2.998	7912.321
28	3.027	7943.817
29	3.049	7965.502
30	3.088	8003.747
31	3.155	8068.089
32	3.272	8180.332
33	3.473	8353.716
34	3.588	8438.623
35	3.789	8576.410
36	3.817	8587.615
37	3.832	8593.352
38	3.858	8603.410
39	3.904	8620.812
40	3.983	8650.949
41	4.119	8703.102
42	4.355	8781.304
43	4.414	8800.181
44	4.440	8808.410
45	4.484	8822.665
46	4.503	8828.838
47	4.536	8839.519
48	4.594	8856.203
49	4.612	8859.471
50	4.643	8865.608
51	4.697	8876.250
52	4.720	8880.749
53	4.761	8888.727
54	4.778	8892.094
55	4.786	8893.549
56	4.799	8896.135
57	4.821	8900.616
58	4.831	8902.509
59	4.835	8903.326
60	4.843	8904.779
61	4.855	8906.559
62	4.878	8909.511

Load Step	Deck Disp (feet)	Lateral load (kips)
63	4.887	8910.448
64	4.904	8912.603
65	4.933	8916.321
66	4.982	8922.764
67	5.004	8925.477
68	5.041	8930.304
69	5.058	8932.337
70	5.086	8935.955
71	5.098	8937.478
72	5.119	8940.190
73	5.155	8944.891
74	5.171	8946.868
75	5.198	8950.391
76	5.210	8951.872
77	5.231	8954.516
78	5.266	8959.091
79	5.281	8961.016
80	5.308	8964.445
81	5.320	8965.882
82	5.340	8968.450
83	5.374	8972.899
84	5.389	8974.771
85	5.415	8978.107
86	5.460	8983.887
87	5.479	8986.313
88	5.488	8987.363
89	5.502	8989.237
90	5.509	8990.024
91	5.520	8991.258
92	5.539	8993.436
93	5.571	8997.213
94	5.628	9003.751
95	5.653	9006.433
96	5.695	9011.331
97	5.714	9013.329
98	5.746	9016.995
99	5.760	9018.501
100	5.784	9021.245
101	5.825	9026.001
102	5.843	9027.953
103	5.874	9031.518
104	5.928	9037.693
105	5.951	9040.223
106	5.992	9044.848
107	6.009	9046.742
108	6.040	9050.211
109	6.053	9051.629
110	6.051	9051.424
111	6.046	9024.676
112	6.048	9034.000
113	6.052	9050.076
114	6.046	9021.291
115	6.034	8971.426
116	6.013	8885.067
117	5.977	8735.622
118	5.914	8476.651
119	5.806	8028.118
120	5.760	7833.904
121	5.755	7812.862
122	5.746	7776.433
123	5.742	7760.683
124	5.749	7786.970
125	5.760	7832.336

Load Step	Deck Disp (feet)	Lateral load (kips)
126	5.779	7910.910
127	5.813	8046.932
128	5.872	8282.597
129	5.973	8690.776
130	5.984	8734.964
131	5.989	8753.910
132	5.994	8774.853
133	5.999	8795.884
134	6.004	8817.018
135	6.007	8825.909
136	6.012	8846.909
137	6.014	8856.074
138	6.018	8871.759
139	6.023	8892.811
140	6.026	8901.971
141	6.030	8916.577
142	6.035	8936.512
143	6.040	8956.725
144	6.042	8965.523
145	6.046	8977.235
146	6.052	8993.908
147	6.057	9006.553
148	6.059	9009.686
149	6.063	9014.215
150	6.070	9018.754
151	6.073	9020.616
152	6.078	9023.251
153	6.080	9024.333
154	6.084	9026.285
155	6.086	9027.091
156	6.089	9027.664
157	6.094	9028.563
158	6.103	9030.405
159	6.107	9031.055
160	6.114	9032.400
161	6.125	9034.723
162	6.130	9035.730
163	6.139	9037.456
164	6.143	9038.199
165	6.149	9038.952
166	6.161	9040.437
167	6.180	9043.020
168	6.189	9044.112
169	6.204	9046.043
170	6.230	9049.387
171	6.241	9050.798
172	6.260	9053.302
173	6.268	9054.360
174	6.283	9056.235
175	6.281	9056.019
176	6.276	9028.811
177	6.278	9038.084
178	6.282	9054.118
179	6.284	9059.061
180	6.278	9055.624
181	6.258	8952.089
182	6.283	9056.612
183	6.281	9048.175
184	6.195	8667.006
185	6.191	8649.587
186	6.104	8271.299
187	6.095	8230.393
188	6.091	8212.736

Load Step	Deck Disp (feet)	Lateral load (kips)
189	6.084	8182.073
190	6.081	8168.908
191	6.098	8213.982
192	6.100	8218.861
193	6.103	8226.082
194	6.182	8502.709
195	6.216	8621.843
196	6.231	8671.027
197	6.240	8695.914
198	6.244	8707.726
199	6.242	8701.948
200	6.247	8709.269
201	6.243	8708.105
202	6.245	8700.410
203	6.242	8696.966
204	6.244	8684.353
205	6.242	8666.175
206	6.247	8686.236
207	6.249	8701.715
208	6.253	8693.198
209	6.252	8703.970
210	6.257	8724.200
211	6.257	8708.880
212	6.261	8714.088
213	6.260	8697.229
214	6.260	8683.845
215	6.261	8673.534
216	6.263	8665.624
217	6.265	8659.648
218	6.267	8655.251
219	6.270	8652.146
220	6.267	8630.159
221	6.264	8609.341
222	6.267	8603.461
223	6.264	8584.228
224	6.267	8604.118
225	6.288	8692.852
226	6.313	8791.994
227	6.320	8815.549
228	6.322	8825.709
229	6.328	8844.447
230	6.333	8860.974
231	6.335	8868.288
232	6.339	8880.562
233	6.344	8895.499
234	6.349	8910.696
235	6.352	8914.469
236	6.356	8922.763
237	6.357	8925.776
238	6.360	8931.290
239	6.365	8940.597
240	6.370	8950.371
241	6.373	8954.511
242	6.377	8961.854
243	6.378	8964.942
244	6.381	8970.419
245	6.386	8979.152
246	6.391	8983.666
247	6.400	8994.680
248	6.404	8999.280
249	6.406	9001.242
250	6.409	9004.779
251	6.414	9010.715

Load Step	Deck Disp (feet)	Lateral load (kips)
252	6.419	9016.975
253	6.421	9018.964
254	6.425	9023.194
255	6.426	9024.952
256	6.429	9028.085
257	6.434	9033.063
258	6.455	9042.841
259	6.464	9047.600
260	6.468	9047.434
261	6.476	9048.737
262	6.488	9051.052
263	6.493	9052.052
264	6.502	9053.780
265	6.518	9056.778
266	6.525	9058.081
267	6.537	9059.249
268	6.558	9061.525
269	6.594	9065.491
270	6.610	9067.024
271	6.616	9067.706
272	6.628	9068.964
273	6.648	9071.144
274	6.657	9072.026
275	6.672	9073.660
276	6.699	9076.491
277	6.710	9077.636
278	6.730	9079.754
279	6.739	9080.614
280	6.753	9082.200
281	6.779	9084.951
282	6.790	9086.054
283	6.809	9088.105
284	6.818	9088.937
285	6.832	9090.475
286	6.857	9093.141
287	6.868	9094.220
288	6.887	9096.219
289	6.895	9097.027
290	6.909	9098.525
291	6.933	9101.122
292	6.944	9102.171
293	6.962	9104.114
294	6.970	9104.900
295	6.984	9106.358
296	7.007	9108.883
297	7.018	9109.900
298	7.035	9111.791
299	7.043	9112.555
300	7.056	9113.971

Table A.5.4.2a

ULTIMATE STRENGTH ANALYSIS
FAILED ELEMENTS & FAILURE MODE
DIAGONAL (45 DEG) DIRECTION

Member Name	Load Step	Element Type	Worst Event Description
6hz1-1	33	Beam Column	Beam Clmn Initial Yield (1,1)
6hz1-10	29	Beam Column	Beam Clmn Initial Yield (1,1)
9hz-17	234	Beam Column	Beam Clmn Initial Yield (0,1)
9hz-34	218	Beam Column	Beam Clmn Initial Yield (1,0)
9hz-41	212	Beam Column	Beam Clmn Initial Yield (0,1)
legA2-12	251	Beam Column	Beam Clmn Initial Yield (0,1)
legA3-13	223	Beam Column	Beam Clmn Initial Yield (0,1)
legA3-5	296	Beam Column	Beam Clmn Initial Yield (1,0)
mmpile-250	295	Beam Column	Beam Clmn Initial Yield (1,0)
mmpile-251	69	Beam Column	Beam Clmn Initial Yield (1,0)
mmpile-252	45	Beam Column	Beam Clmn Initial Yield (1,0)
mmpile-253	86	Beam Column	Beam Clmn Initial Yield (1,0)
mmpile-258	297	Beam Column	Beam Clmn Initial Yield (0,1)
mmpile-259	80	Beam Column	Beam Clmn Initial Yield (0,1)
mmpile-260	53	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-292	94	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-293	99	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-296	271	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-297	291	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-298	42	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-299	36	Beam Column	Beam Clmn Initial Yield (0,1)
mspile-300	86	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-301	85	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-304	166	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-305	278	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-306	40	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-307	34	Beam Column	Beam Clmn Initial Yield (1,0)
msrow2-4	147	Strut	Strut Buckling
msrow3-6	198	Strut	Strut Buckling
vd2-31	109	Strut	Strut Buckling
vd2-32	174	Strut	Strut Buckling
vd2-35	146	Strut	Strut Buckling
vd3-37	269	Strut	Plastic Strut

Table A.5.4.3

ULTIMATE STRENGTH ANALYSIS
LATERAL LOAD vs. DECK DISPLACEMENT
BROADSIDE (90 DEG) DIRECTION

(Load level at which first component reaches I.R. of 1.0 = 3623 kips)

Load Step	Deck Disp (feet)	Lateral load (kips)
0	0.000	0.000
1	0.145	279.717
2	0.527	1630.091
3	0.907	2965.303
4	1.286	4250.318
5	1.360	4485.456
6	1.433	4713.741
7	1.559	5100.145
8	1.668	5372.840
9	1.725	5507.700
10	1.774	5618.889
11	1.817	5714.654
12	1.890	5880.853
13	2.018	6161.143
14	2.146	6435.554
15	2.273	6698.346
16	2.384	6920.669
17	2.480	7097.587
18	2.484	7102.154
19	2.489	7111.851
20	2.492	7114.136
21	2.486	7105.327
22	2.492	7112.657
23	2.401	6684.176
24	2.362	6495.278
25	2.294	6170.087
26	2.293	6161.212
27	2.291	6141.551
28	2.387	6453.751
29	2.553	6983.852
30	2.550	6974.884
31	2.546	6958.083
32	2.539	6929.769
33	2.527	6880.733
34	2.506	6795.795
35	2.469	6648.682
36	2.406	6393.940
37	2.296	5952.603
38	2.106	5188.092
39	2.024	4857.014
40	1.989	4713.631
41	1.985	4698.125
42	1.983	4691.406
43	1.980	4679.765
44	1.977	4657.379
45	2.403	5874.363
46	2.718	6629.035
47	2.769	6748.383
48	2.859	6953.211
49	2.898	7041.259
50	2.943	7126.553
51	2.975	7169.792
52	3.028	7240.563
53	3.022	7188.207
54	3.020	7222.361
55	3.023	7192.025
56	2.592	5820.812
57	2.484	5478.449
58	2.438	5330.053
59	2.417	5265.943
60	2.382	5154.713
61	2.379	5142.777
62	2.372	5121.920

Load Step	Deck Disp (feet)	Lateral load (kips)
63	2.382	5154.125
64	2.398	5209.479
65	2.427	5305.355
66	2.477	5471.422
67	2.564	5759.342
68	2.710	6235.703
69	2.822	6587.159
70	2.933	6910.066
71	2.980	7049.567
72	3.067	7268.437
73	3.121	7345.615
74	3.148	7380.219
75	3.197	7442.500
76	3.281	7546.238
77	3.365	7648.404
78	3.374	7659.591
79	3.390	7678.877
80	3.397	7686.420
81	3.409	7700.203
82	3.414	7706.035
83	3.417	7708.555
84	3.421	7713.029
85	3.427	7720.779
86	3.439	7734.198
87	3.444	7739.648
88	3.451	7746.389
89	3.461	7758.408
90	3.466	7763.183
91	3.468	7765.297
92	3.472	7769.059
93	3.478	7775.204
94	3.484	7780.647
95	3.490	7786.373
96	3.493	7788.785
97	3.498	7793.069
98	3.506	7800.383
99	3.509	7803.489
100	3.515	7807.706
101	3.518	7809.697
102	3.523	7813.194
103	3.525	7814.679
104	3.528	7817.300
105	3.534	7821.606
106	3.540	7825.954
107	3.551	7833.646
108	3.556	7836.595
109	3.562	7838.981
110	3.573	7843.818
111	3.592	7852.102
112	3.600	7855.675
113	3.603	7857.030
114	3.609	7859.396
115	3.612	7860.422
116	3.616	7862.196
117	3.624	7865.272
118	3.628	7865.577
119	3.634	7866.385
120	3.644	7867.696
121	3.661	7869.960
122	3.692	7873.882
123	3.745	7880.679
124	3.837	7892.467
125	3.876	7897.950

Load Step	Deck Disp (feet)	Lateral load (kips)
126	3.945	7907.066
127	4.064	7921.298
128	4.116	7928.333
129	4.138	7931.394
130	4.148	7932.721
131	4.165	7934.665
132	4.194	7938.217
133	4.244	7943.542
134	4.331	7953.258
135	4.483	7970.103
136	4.744	7999.349
137	4.751	8001.499
138	4.757	8001.035
139	4.763	8002.705
140	4.863	8013.247
141	4.928	8018.676
142	5.041	8029.392
143	5.237	8048.116
144	5.575	8080.653
145	5.973	8121.986
146	6.371	8161.652
147	6.769	8201.494
148	7.167	8241.467
149	7.566	8281.615
150	7.964	8321.986
151	8.362	8362.611
152	8.761	8403.482
153	9.159	8444.550
154	9.557	8485.855
155	9.955	8527.438
156	10.353	8566.057
157	10.751	8602.439
158	11.149	8639.170
159	11.547	8675.987
160	11.945	8713.041
161	12.343	8750.110
162	12.740	8787.256
163	13.138	8824.622
164	13.536	8862.295
165	13.934	8899.559
166	14.033	8908.805
167	14.155	8918.294
168	14.366	8939.780
169	14.457	8947.283
170	14.569	8954.497
171	14.681	8961.754
172	14.793	8968.993
173	14.905	8975.784
174	15.017	8979.824
175	15.115	8982.208
176	15.199	8984.155
177	15.272	8985.837
178	15.335	8987.288
179	15.389	8988.856
180	15.444	8990.178
181	15.499	8991.480
182	15.553	8992.674
183	15.608	8994.173
184	15.663	8995.387
185	15.717	8996.694
186	15.772	8997.991
187	15.827	8999.293
188	15.881	9000.592

Load Step	Deck Disp (feet)	Lateral load (kips)
189	15.936	9001.876
190	15.991	9003.175
191	16.045	9004.479
192	16.100	9005.776
193	16.155	9007.070
194	16.209	9008.359
195	16.264	9009.661
196	16.319	9010.955
197	16.373	9012.248
198	16.428	9013.531
199	16.483	9014.830
200	16.537	9016.121
201	16.592	9017.406
202	16.647	9018.687
203	16.701	9019.982
204	16.756	9021.269
205	16.811	9022.555
206	16.865	9023.834
207	16.920	9025.130
208	16.975	9026.415
209	17.029	9027.699
210	17.084	9028.975
211	17.139	9030.273
212	17.193	9031.561
213	17.248	9032.839
214	17.303	9034.122
215	17.357	9035.415
216	17.412	9037.228
217	17.459	9037.654
218	17.507	9039.833
219	17.554	9039.802
220	17.601	9040.865
221	17.649	9041.930
222	17.696	9042.991
223	17.744	9044.058
224	17.791	9045.118
225	17.838	9046.182
226	17.886	9047.243
227	17.933	9048.306
228	17.980	9049.362
229	18.028	9050.425
230	18.075	9051.483
231	18.122	9052.545
232	18.170	9053.600
233	18.217	9054.666
234	18.264	9055.718
235	18.312	9056.779
236	18.359	9057.828
237	18.407	9058.894
238	18.454	9059.947
239	18.501	9061.008
240	18.549	9062.062
241	18.596	9063.122
242	18.643	9064.176
243	18.691	9065.236
244	18.738	9066.290
245	18.785	9067.352
246	18.833	9068.394
247	18.880	9069.456
248	18.928	9070.482
249	18.975	9071.508
250	19.022	9072.505
251	19.070	9073.512

Load Step	Deck Disp (feet)	Lateral load (kips)
252	19.117	9074.516
253	19.164	9075.502
254	19.176	9075.254
255	19.197	9075.532
256	19.232	9076.283
257	19.294	9077.587
258	19.400	9079.820
259	19.584	9083.676
260	19.904	9090.353
261	20.004	9090.890
262	20.176	9095.476
263	20.251	9097.051
264	20.381	9099.911
265	20.605	9103.867
266	20.993	9111.973
267	21.392	9119.717
268	21.790	9126.761
269	22.189	9134.527
270	22.587	9141.017
271	22.986	9147.879
272	22.992	8932.491
273	22.998	9148.438
274	23.001	9126.494
275	23.002	8641.854

Table A.5.4.3a

ULTIMATE STRENGTH ANALYSIS
FAILED ELEMENTS & FAILURE MODE
BROADSIDE (90 DEG) DIRECTION

Member Name	Load Step	Element Type	Worst Event Description
1hz2-17	72	Beam Column	Beam Clmn Initial Yield (0,2)
1hz2-18	52	Beam Column	Beam Clmn Initial Yield (1,1)
6hz1-10	49	Beam Column	Beam Clmn Initial Yield (1,1)
6hz1-7	152	Beam Column	Beam Clmn Initial Yield (1,1)
9hz-10	77	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-11	80	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-12	74	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-13	73	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-15	89	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-16	77	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-17	72	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-34	49	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-35	70	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-36	70	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-37	46	Beam Column	Beam Clmn Initial Yield (1,2)
9hz-38	70	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-39	71	Beam Column	Beam Clmn Initial Yield (0,2)
9hz-40	70	Beam Column	Beam Clmn Initial Yield (2,0)
9hz-41	49	Beam Column	Beam Clmn Initial Yield (1,2)
9hz-9	72	Beam Column	Beam Clmn Initial Yield (2,0)
deckleg-1	165	Beam Column	Beam Clmn Initial Yield (0,1)
legA1-108	75	Beam Column	Beam Clmn Initial Yield (2,1)
legA1-2	147	Beam Column	Beam Clmn Initial Yield (0,1)
legA2-114	70	Beam Column	Beam Clmn Initial Yield (0,2)
legA2-12	70	Beam Column	Beam Clmn Initial Yield (0,2)
legA2-4	70	Beam Column	Beam Clmn Initial Yield (2,0)
legA2-9	72	Beam Column	Beam Clmn Initial Yield (0,1)
legA3-10	72	Beam Column	Beam Clmn Initial Yield (0,2)
legA3-115	70	Beam Column	Beam Clmn Initial Yield (0,2)
legA3-13	70	Beam Column	Beam Clmn Initial Yield (1,2)
legA3-5	70	Beam Column	Beam Clmn Initial Yield (2,0)
legB2-110	52	Beam Column	Beam Clmn Initial Yield (0,2)
legB2-13	70	Beam Column	Beam Clmn Initial Yield (0,2)
legB2-14	112	Beam Column	Beam Clmn Initial Yield (0,1)
legB2-8	73	Beam Column	Beam Clmn Initial Yield (2,0)
legB3-111	48	Beam Column	Beam Clmn Initial Yield (0,2)
legB3-14	52	Beam Column	Beam Clmn Initial Yield (0,2)
legB3-15	117	Beam Column	Beam Clmn Initial Yield (0,1)
legB3-9	71	Beam Column	Beam Clmn Initial Yield (2,0)
legB4-113	46	Beam Column	Beam Clmn Initial Yield (0,2)
legB4-13	95	Beam Column	Beam Clmn Initial Yield (0,1)
legB4-9	48	Beam Column	Beam Clmn Initial Yield (1,2)
mmpile-242	164	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-243	163	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-244	163	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-245	168	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-246	233	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-247	210	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-248	213	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-249	218	Beam Column	Beam Clmn Initial Yield (1,1)
mmpile-250	148	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-251	148	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-252	148	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-253	147	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-254	163	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-255	177	Beam Column	Beam Clmn Initial Yield (2,2)
mmpile-256	173	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-257	162	Beam Column	Beam Clmn Initial Yield (2,1)
mmpile-258	149	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-259	149	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-260	149	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-261	148	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-262	164	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-263	184	Beam Column	Beam Clmn Initial Yield (0,2)

mmpile-264	178	Beam Column	Beam Clmn Initial Yield (0,2)
mmpile-265	164	Beam Column	Beam Clmn Initial Yield (0,2)
mpile-180	162	Beam Column	Beam Clmn Initial Yield (0,1)
mpile-52	164	Beam Column	Beam Clmn Initial Yield (0,1)
mpile-54	245	Beam Column	Beam Clmn Initial Yield (1,0)
mpile-55	266	Beam Column	Beam Clmn Initial Yield (1,0)
mpile-58	267	Beam Column	Beam Clmn Initial Yield (1,0)
mpile-59	227	Beam Column	Beam Clmn Initial Yield (1,0)
mspile-292	146	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-293	152	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-294	152	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-295	167	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-296	166	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-297	151	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-298	151	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-299	146	Beam Column	Beam Clmn Initial Yield (0,2)
mspile-300	152	Beam Column	Beam Clmn Initial Yield (2,0)
mspile-301	145	Beam Column	Beam Clmn Initial Yield (2,1)
mspile-302	151	Beam Column	Beam Clmn Initial Yield (2,1)
mspile-303	164	Beam Column	Beam Clmn Fully Plastic (2,2)
mspile-304	164	Beam Column	Beam Clmn Fully Plastic (2,2)
mspile-305	150	Beam Column	Beam Clmn Initial Yield (2,1)
mspile-306	151	Beam Column	Beam Clmn Initial Yield (2,0)
mspile-307	145	Beam Column	Beam Clmn Initial Yield (2,1)
mspile-308	158	Beam Column	Beam Clmn Initial Yield (1,1)
mspile-309	159	Beam Column	Beam Clmn Initial Yield (1,1)
mspile-311	262	Beam Column	Beam Clmn Initial Yield (1,1)
mspile-312	164	Beam Column	Beam Clmn Fully Plastic (2,2)
mspile-313	163	Beam Column	Beam Clmn Fully Plastic (2,2)
mspile-314	262	Beam Column	Beam Clmn Initial Yield (1,1)
msrow2-4	20	Strut	Strut Buckling
msrow3-5	164	Strut	Plastic Strut
msrow3-6	19	Strut	Strut Buckling
msrow4-2	29	Strut	Strut Buckling
vd8-5	52	Strut	Strut Buckling

PLATFORM "J"

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2.0 INTRODUCTION

At the request of performed an analysis of the structural capacity of the South Pelto 11-F Platform. The objective of this study was to assess the current capacity of the platform and provide recommendations on the potential of extending its operational life.

2.1 Platform History

A general view of the jacket and topsides facilities is shown in Figure 1.1.1 The platform was originally erected at the Ship Shoal 54-B field. It was subsequently salvaged in January of 1957 and extended by 14 feet at Morgan City to match the 37 feet water depth at South Pelto 11-F field. The installation was completed that same year, and the jacket has been at its current location for over 36 years.

2.2 General Data

The platform is located at Latitude N28 55 49 and Longitude W90 42 43 in the Gulf of Mexico. It was originally a tender drilling platform. The jacket is actually composed of three separate jackets each with 12 legs, for a total of 36 legs with a 20-inch diameter. The piles are 18 inches in diameter and range in penetration from 132 to 141 feet. There are also three conductors with diameters ranging from 24 to 26 inches.

The topsides consists of a single level deck at elevation +31 feet with overall dimensions of 52 by 90 feet, along with a 30 by 30 feet heliport. Although there is still a substantial amount of equipment on the deck, it probably weighs less than when the platform was being used for drilling.

3.0 PLATFORM MODELS

In modeling the behavior of the platform, two types of analyses were performed. The first was a series of linear elastic analyses using the DAMS computer program. For the second set of analyses, a non-linear analysis program, KARMA, was used to assess the ultimate capacity of the platform. A brief description of the models and a summary of the fundamental assumptions are provided in the sections that follow.

3.1 DAMS Model

The first model was developed using the DAMS computer program. Since this program performs a linear elastic analysis, the main objective was to simulate the behavior of the platform at lower load levels where the member stresses are well below yield.

Figures 3.1.1 through 3.1.4 illustrate various views of the DAMS model. All the jacket and deck members were modelled as per the as-built drawings with the following revisions.

Although the as-built drawings indicated a water depth of 37 feet at the site, the latest inspection report (Ref. 3.1.1) showed a water depth of only 32 feet. Since it could not be confirmed which of the two was correct, the more conservative 37-foot water depth was used in the model. This issue needs to be resolved in the next underwater inspection.

To account for the degradation due to corrosion, the wall thickness of all the jacket members at or below the waterline level was reduced by 10%. No reduction was taken in the nominal wall thickness of the piles as they are generally protected by the legs from the corrosive action of the seawater. The trapped water in the leg/pile annulus and inside the pile is believed to be much less corrosive due to limited access to oxygen. The issue of the amount of corrosion that has taken place needs further validation through measurements in future inspections.

A yield stress of 39.6 ksi, which represents a 20% increase over the nominal yield stress of 33 ksi, was used in the analyses. This difference between the nominal and average yield stress is typical for the lower strength steels. Similar differences, however, are not observed for the higher grade (i.e. 50 ksi) steels which are also used in offshore platforms. Since the objective of this analysis was to obtain a "best estimate" of the platform response and capacity with as little bias as possible, estimated average values where used for all parameters.

Because the structure is highly redundant, the local damage of a few members was not viewed as critical to the ultimate capacity of the platform, and hence this damage was not incorporated into the model. However, as the number of damaged members increases, their cumulative effect could become significant and should be considered in future assessments.

All major appurtenances were modelled to capture their associated wave load but not their stiffness. These include: boat landings, barge bumpers, risers, and conductors. Although the conductors could have a non-trivial contribution to the lateral resistance in a typical new design, that is not the case for this platform. In a new design, the ratio of the number of conductors to the number of legs is much higher. For South Pelto there are only three conductors compared to 36 legs. This, coupled with the potential gaps between the conductor guides and the conductors, support the practice of not modelling them as load-resisting members.

The environmental loads, wave and current, are described in Section 4.0. The loads on the jacket were computed internally by DAMS. However, since the waves overtop the deck even for waves with a recurrence interval as low as 25 years, these additional loads on both the deck members and equipment were computed separately as described in Section 4.0 and applied as lateral joint loads.

The foundation was modelled using non-linear springs to represent the lateral and axial behavior of the piles in the soil. DAMS has the capability of analyzing the platform and its interaction with the non-linear soil foundation. The program condenses the jacket stiffness matrix and applied external load vector to the interface joints (i.e. the piles at the mudline elevation) , and then iterates between the condensed jacket stiffness and foundation springs until the residual loads fall within the prescribed convergence criteria. The details of the soil properties and generation of the non-linear soil springs are described in Section 5.0.

3.2 KARMA Model

To assess the ultimate capacity of the platform, a non-linear analysis model using the KARMA program was developed. Figure 3.2.1 through 3.2.3 provide various views of this model.

This model has the same fundamental assumptions of the DAMS model described in the previous sections. These include: the increase in yield stress over the nominal value, the reduction in the member wall thickness for corrosion allowance, and other miscellaneous assumptions.

The most important difference from the DAMS model is the use of non-linear elements to model the structural members. KARMA has the capability to model both the material and geometric non-linearities which will occur at the higher load levels. The element types were selected based on the expected behavior of each member at its ultimate load level. An example is the use of strut elements to model the vertical diagonals. Although these members carry some bending at lower load levels, their expected yielding/hinging at the member ends will cause them to act as struts as they approach the ultimate load.

The following is a listing of the element types along with a brief description of their behavior. Also listed are the structural members that were used to model in the KARMA analyses. More detailed descriptions on the formulation and behavior of each element are available in references 3.2.1 and 3.2.2.

WAVL: non-structural general wave load element

Conductors

Miscellaneous cellar deck members

Walkways and stairs

Bumper rub strips

Boat landings

LANB: Three-dimensional, large displacement, inelastic beam-column element with distributed plasticity and failure options

- Jacket legs
- Piles

SHER: Three-dimensional shear transfer element

- Shim plates between piles and jacket legs

PSAS: Three-dimensional nonlinear pile-soil system analysis element

- Soil springs

BMEC: Three-dimensional inelastic beam-column element with p-delta effects and failure options

- Jacket horizontals

NTRS: Three-dimensional, large displacement non-linear truss element, with failure options

- Connections between jacket spacer frames

ISTR: Three-dimensional, large displacement, post-buckling Gidwani strut element with stiffness degradation, strength deterioration and failure options

- Selected braces above and below waterline

LBEM: Three-dimensional linear beam element

- Deck beams

In performing the ultimate load analysis, the dead loads are applied first. Like DAMS, KARMA internally computes a reference set of wave/current loads on the jacket for the 100-year conditions. The associated wave impact loads on the deck were input as nodal loads. The environmental loads were applied incrementally to determine the non-linear platform behavior up to failure. As with any non-linear analysis, the magnitude of the load increments is an important parameter in achieving convergence to properly determine the ultimate load. Based on previous experience, a total of about 1000 load steps were specified to achieve the ultimate load level in each of the three directions that were analyzed.

3.3 Model Calibration

Since the jacket and deck portions of the KARMA model are a direct translation of the corresponding portions of the DAMS model, only a general check of this portion of the model is required. However, this is not the case for the foundation portion of the model. DAMS uses a finite difference approach with the P-Y, T-Z, and Q-Z curves to model the lateral, skin-friction, and end-bearing response of the soil, respectively. On the other hand, KARMA requires a specific number of pile and soil spring elements at prescribed spacings. Therefore, to ensure that the foundation response in both was equivalent, the response of a single pile was modelled in each of the two programs and compared. As illustrated in Figure 3.3.1, the load vs. pile head displacement was found to be very similar for both models.

4.0 ENVIRONMENTAL LOADS

4.1 Oceanographic Criteria

The mean water depth at South Pelto 11-F is 37 feet. Due to this shallow depth the extreme wave heights will be limited by depth-induced wave breaking. Therefore, the wave conditions corresponding to various annual exceedance probabilities (return periods) will be directly a function of the storm surge values associated with those annual exceedance probabilities.

The storm surge values for South Pelto 11-F have been obtained by taking GUMSHOE hindcast results (Ref. 4.1.1) for a number of grid points in this area of the Gulf of Mexico for the return period of interest and obtaining a linear fit between storm surge and water depth. In all cases the astronomical tide phase is assumed to be such that the tidal level is at the mean water level, i.e., 0.8 feet above mean low water. This is slightly less conservative than the assumption of high tide that is used in new designs, but it is a better estimate of the expected value of the astronomical tide at the time of peak storm surge. The Storm Water Depth (SWD) is the sum of the mean low water depth, the astronomical tide and the storm surge.

The depth-limited breaking wave height is found from the following equation:

$$H/gT^2 = 0.02711 \tanh [28.77 \, d/gT^2]$$

where

H = wave height (feet)

T = wave period (taken as 13 seconds in these calculations)

g = gravitational acceleration (32.17 feet/sec^2)

d = storm water depth (feet)

The currents associated with the peak storm surge and wave height values are obtained following the procedure prescribed in the 20th edition of API RP2A (Ref. 4.1.2). For this location in the Gulf of Mexico, the direction that is approximately parallel to the bottom contours is towards 260°, i.e., 10° south of due west. Therefore, the vector component of the current associated with the wave loads for the three main platform headings is obtained by multiplying the currents associated with various return periods by the appropriate cosine factors:

Diagonal	cos (17.4°)
Broadside	cos (62.4°)
End - on	cos (27.6°)

In addition, there is a current blockage factor that is applied to further reduce the effective current for each platform heading to account for the overall effect exerted on the current by the platform:

Diagonal	0.85
Broadside	0.80
End-on	0.70

For each effective current and directional wave height the Doppler-shifted, or apparent, wave period (T_{app}) is calculated using the RP2A procedure (section 2.3.1b.1).

The wave crest elevation is calculated using the 10th-order Stream function wave theory representation of a wave of the specified height, associated apparent period, and storm water depth. The 10% "profile factor" normally applied to the crest elevation in Mobil's design practice for new platforms was not used for two reasons: 1) in requalification pushover analyses it is desirable to remove all unwarranted conservatism in criteria, and 2) for depth-limited breaking waves, this 10% factor results in an extremely high ratio of wave crest elevation to wave height that describes a wave which is very unstable.

The oceanographic criteria as a function of return period are summarized in Table 4.1.1.

Table 4.1.1

Oceanographic Criteria for South Pelto 11-F

SOUTH PELTO 11-F OCEANOGRAPHIC CONDITIONS						
Oceanographic Parameter	**Return Period in Years**					
	25	**50**	**100**	**250**	**500**	**1000**
SWD (feet)	41.4	42.4	43.4	44.7	45.7	46.7
H (feet)	31.8	32.5	33.3	34.2	35.0	35.7
T_{app} - BS (sec)	13.6	13.7	13.9	14.0	14.1	14.2
T_{app} -Diag (sec)	13.7	13.9	14.0	14.2	14.3	14.4
T_{app} - EO(sec)	13.3	13.4	13.4	13.5	13.5	13.6
Current - BS (feet/s)	1.8	2.0	2.5	3.0	3.3	3.7
Current - Diag (feet/s)	2.1	2.5	2.9	3.5	3.9	4.3
Current - EO (feet/s)	0.85	1.0	1.2	1.4	1.6	1.8
Crest (above MLW) BS (feet)	31.0	32.6	34.2	36.2	37.9	39.4
Crest (above MLW) Diag (feet)	31.0	32.6	34.3	36.3	38.0	39.5
Crest (above MLW) BS (feet)	30.9	32.5	34.1	36.1	37.7	39.2

Note: The historical hurricane direction distribution at the site is 5% in the Broadside (BS) direction, 50% in the End-On (EO) direction, and 45% in the Diagonal (Diag) direction.

The South Pelto 11-F platform experienced fairly strong oceanographic conditions during Hurricane Andrew in August 1992. Table 4.1.2 summarizes the hourly conditions at the platform site during the peak of the storm, as taken from the Oceanweather, Inc. hindcast of Hurricane Andrew (Ref. 4.2.1).

Table 4.1.2

Hourly Hindcast Conditions at South Pelto 11-F During Hurricane Andrew

Tide	Surge	SWD	H	T_{Hmax}	T_{App}	Wave Dir	Pltfrm dir	Inline Current	Crest above SWD	Crest above MLW	Wave Load on Deck
feet	feet	feet	feet	sec	sec			ft/sec	feet	feet	kips
0.40	1.84	39.2	30.0	12.2	14.2	257.6	Diagonal	5.8	25.4	27.6	30
0.55	3.00	40.6	31.2	13.2	15.6	287.9	Diagonal	6.6	26.7	30.3	260
0.70	4.39	42.1	32.2	12.2	14.0	317.4	End-on	5.5	27.0	32.1	510
0.95	5.27	43.2	32.9	11.8	12.8	339.1	End-on	3.1	27.1	33.3	520
1.20	5.40	43.6	34.0	11.9	12.0	350.8	Diagonal	0.4	27.7	34.3	500

Particularly important is the second line of the table where the inline current is equal to 6.6 feet per second. At this time the waves were also fairly high (31.2 feet) so the resulting load is expected be quite large.

4.2 Deck Loads Due to Wave Impact

The deck elevation is very low by today's standards, since the South Pelto 11-F platform was designed in the 1950s. In fact, present estimates indicate that during the extreme oceanographic conditions, the waves will overtop the bottom of the deck steel resulting in horizontal impact loads on the deck. Estimates of these loads have been made for the various return period directional criteria that was summarized in Table 4.1.1. These loads were computed using the procedures described in reference 4.2.2 and are summarized in the following table.

Table 4.2.1

Wave Impact Loads on Deck (kips)

Wave Direction	RETURN PERIOD (years)					
	25	50	100	250	500	1000
Broadside	210	360	525	740	940	1120
Diagonal	300	420	550	750	950	1130
End-on	320	380	450	560	700	830

The loads were calculated using the following equation:

$$F_{Deck} = 1/2\ \rho\ C_d A_p \left[KF{*}u_w + V_{IL} \right]^2$$

where

	ρ	=	density of water
	C_d	=	drag coefficient (1.5 for diagonal, 2.0 for end-on and broadside)
	A_p	=	projected area of the deck inundated with water
	KF	=	wave kinematics factor, 0.88 for hurricanes
	u_w	=	wave-induced water particle velocity at wave crest
	V_{IL}	=	in-line component of current.

The wave-induced water particle velocities are obtained from 10th-order Stream function wave theory. Results for deck force from this equation are multiplied by a factor of 0.91 to account for an apparent bias in this equation for large waves as found in analysis of model test data.

5.0 GEOTECHNICAL CRITERIA

5.1 Background

The South Pelto 11-F (PL11F) platform is supported by 36 piles 18 inches in diameter with a penetration of 132 to 141 feet. Available geotechnical data was researched to develop soil spring criteria for these piles for the pushover study. No site specific soil borings were available at the platform site, located at Louisiana coordinates X = 2,198,794 and Y = 96,386 (according to PLATDAT). However, there were the following three borings in the general vicinity of the platform:

Table 5.1.1

Soil Boring Locations

SOIL BORING	X (LA Coord., ft.)	Y (LA Coord. ft.)	DISTANCE FROM PL11F PLATFORM (ft.)
McClelland Engineers Block 10 South Pelto Area: "B" Platform, 1976 (Ref. 5.1)	2,194,942	102,807	7,490
McClelland Engineers Block 10 Well No. 2: "C" Platform, 1978 (Ref 5.2)	2,201,067	102,119	6,170
Law Engineering Block 10 S. Pelto: "D" Platform Location, 1979 (Ref. 5.3)	2,188,378 (approx.)	101,906 (approx.)	11,790

The field facility map showing the relative locations of the above platform boring locations to the PL11F location is provided in Figure 5.1.1.

5.2 Soil Profile Estimation

Review of Driving Records All three of the nearby borings had similar shear strength profiles down to 150 feet as shown on Figure 5.1.2. The two closest boreholes, "B" and "C," show a significant sand stratum starting at 160 to 166 feet below the mudline. Pile and conductor driving records were checked to ascertain: 1) if this sand stratum was penetrated by the piles at PL11F, and 2) whether anomalous sand layers may exist at the site. Driving records can provide a qualitative assessment of the presence of sand or stiff layers through increased blowcounts (assuming the hammer is operating properly).

A typical blowcount record at PL11F is provided in Figure 5.1.3. This and other driving records showed a gradual increase in driving resistance with depth which is consistent with the assumed normally consolidated clay profile for the full pile penetration. No zones of increased driving resistance were found, so significant sand layers are considered unlikely.

Shear Strength Profile Estimation Superimposed on Figure 5.1.2 is an adjusted interpreted shear strength profile based on the 1979 McClelland regional soil atlas (Ref. 5.4). The McClelland shear strength profile is based on trends established from various borings across the shelf. The reliability of the data is a function of the borehole density and proximity to the site. Local geologic variabilities such as sand lenses from buried channels can cause inconsistencies, but the pile driving records generally supported the generalized profile as mentioned above.

According to the McClelland report, the shear profile in the vicinity of PL11F was based on four borings within about a block of the site (see Figure 5.1.4). Two of the four local borings were those performed by McClelland for MOEPSI (Refs. 5.1 and 5.2).

From the late 1960s to the early 1980s, the standard offshore practice was to perform unconfined compression tests on 2.25-inch wireline driven samples. Since the 1980s, industry practice and the API recommended sampling procedure changed to Unconsolidated Undrained (UU) triaxial tests on pushed 3.0-inch samples as the standard. Industry practice also tends to weigh the results of minivane tests when developing a recommended design shear strength profile.

There can be considerable variation in shear strength profiles based on sampling technique, in situ and laboratory tests. All three of the nearby soil boring shear strength profiles (see Table 5.1.1) were based on 2.25-inch driven samples. The shear strength profile from the McClelland regional atlas has been adjusted for sample disturbance and increases the 2.25-inch driven sample values to equivalent unconfined compression test strengths on high quality 3.0-inch pushed samples. Figure 5.1.5 shows the difference in shear strength for pushed versus driven samples at Eugene Island, an area of similar geology to South Pelto.

A comparison of unconfined compression strengths and UU triaxial strengths (the current industry standard) was not included in the report by McClelland. Other studies have shown that UU strengths tend to have less scatter and are slightly higher than unconfined compression strengths, but the magnitude of a modification factor for test type is less clear than the modification factor for sample method and size.

Comparison of the interpreted shear strength profile to shear strength based on the borings at PL10 "B," "C," and "D" is shown on Figure 5.1.2. The ratio of the shear strength at the two closest borings (PL10 "B" and "C") is 1.37 compared to the recommended modification factors of 1.5 by McClelland and 1.56 by Emrich (see Figure 5.1.5). As a result, it was considered appropriate to select the McClelland adjusted and interpreted profile as the most likely at PL11F.

5.3 Axial Pile Behavior

The program APILE (Ref.5.5) was used to develop pile axial load deflection curves, both T-Z (skin friction) and Q-Z (end bearing). APILE does not accept a constantly increasing shear strength profile so the design shear strength was broken into the following step function:

Table 5.3.1

Shear Strength Profile

DEPTH BELOW MUDLINE (ft)	SHEAR STRENGTH, psf (psi)
0 to 20	300 (2.08)
20 to 50	500 (3.47)
50 to 100	750 (5.21)
100 to 150	1,150 (7.99)

The APILE output that gives the T-Z and Q-Z curves used in the pushover analyses is provided in Appendix A. The method of Vijayvergiya (Ref. 5.7) was selected and assumptions used in generating the curves are cited in the output. Note that the T-Z curve noted for 20 feet applies to the entire interval of 0 to 20 feet, the curve for 50 feet applies to the interval from 20 to 50 feet and so on. The load deflection curve for the entire pile is given at the end of the output.

5.4 Lateral Pile

The program LPILE (Ref. 5.6) was used to develop the lateral load deflection curves. Distribution of strength parameters with depth came from the adjusted interpreted shear strength profile shown in Figure 5.1.2. P-Y curves used in the pushover analyses are provided in the LPILE output in Appendix B. Matlock's soft clay criteria (Ref. 5.8) was adopted and assumptions used to generate the P-Y curves are noted in the output.

6.0 STRUCTURAL ANALYSES

The following sections describe the results of both the linear elastic analysis using the DAMS program along with the ultimate capacity evaluation that was performed using the KARMA program.

6.1 Linear Elastic Analysis

The linear elastic analysis of the platform was performed to evaluate its behavior at lower load levels. These analyses focus on the behavior of the platform at the element level in that the output consists mainly of the interaction ratios for each member. These ratios give an indication of the level of utilization in each member as compared to the nominal allowable stresses.

The environmental conditions corresponding to the 25-year return period were analyzed, since this corresponds to the original design criteria for this platform. This was the industry practice up to the mid-1960s, when, following hurricane Hilda and Betsy, the design practice was changed to the 100-year environmental conditions. The platform was analyzed in the three directions illustrated in Figure 6.1.1. To estimate if any members had yielded, the basic allowable stresses were increased by a factor of 1.7. This accounts for the difference between the elastic and plastic section modulus and the increase in the allowables to full yield stress. With this factor, interactions greater than one would indicate full yielding or buckling of members.

For the 25-year environmental loads, no members yielded. The most highly stressed members were the pile sections at the mudline elevation. As discussed in the next section, this was consistent with the trend found at the higher load levels in the static pushover analysis.

A series of runs were also performed to determine the environmental loads corresponding to recurrence intervals ranging from 25 to 1000 years. A summary of these loads is provided on Table 6.1.1 and Figures 6.1.2 through 6.1.4. Note that the difference between the loads for the 100-year event and the 1000-year event is only 44%. This is because the wave height at these shallow water depths is governed by breaking waves. Of the total environmental load, the portion from the wave impacts on the deck ranges from 16% for the 25-year recurrence interval up to 44% for the 1000-year loads. This occurs because with the higher waves, a larger portion of the deck is impacted, thus dramatically increasing this component of the total load (e.g. by over 500% in the broadside direction when comparing the 25- to the 1000-year loads). However, the associated increase in the jacket loads is much smaller (35%). It is also clear that, although removal of some deck equipment would reduce these loads, it would not have a significant impact on the total load.

Since South Pelto 11-F was in the general path of Hurricane Andrew, the loads associated with the estimated wave and current conditions during the hurricane were computed and are summarized in Table 6.2.2:

Table 6.2.2
Hurricane Andrew Environmental Loads

WAVE DIRECTION	JACKET LOAD (kips)	DECK LOAD (kips)	TOTAL (kips)
Diagonal (287.9°)	2145	260	2405
Diagonal (350.8°)	990	500	1490
End-On (317.4°)	1396	510	1906
End-On (339.1°)	1027	520	1547

Note that the resulting loads are about 20% and 40% higher than the 100-design loads (Table 6.1.1) in the diagonal and end-on directions, respectively. The loadings imposed on the South Pelto 11-F Platform during Hurricane Andrew correspond to a 200-year event.

6.2 Static Pushover Analysis

This analysis focuses on the overall response of the platform at the system level. Yielding, and even failure of local members is allowed, with the objective to estimate the ultimate load that the platform can resist. This analysis accounts for any ductility and redundancy in the structure which will allow loads to redistribute after the failure of a given member. The issue of redundancy is particularly important for some of these older structures with many more legs and framing members than a new design. In these cases, yielding or failure of one member is even less significant to the overall ultimate capacity of the platform.

To determine the ultimate capacity of the platform, static pushover analyses were performed in the same three directions as the linear elastic analysis. The deck displacement vs. base shear of the structure is illustrated in Figures 6.2.1 through 6.2.3. The non-linear behavior at low load levels is due entirely to the non-linear response of the soil, as modelled by the P-Y, T-Z, and Q-Z curves. The analysis indicates that the first yielding of the structure is at levels corresponding to about the 100-year loads. Although a few localized jacket members yield at this load level, the most critical yielding is of the pile sections at the mudline elevation. This is consistent with the high interaction ratios that were found in the DAMS analysis.

As the load is increased, there is additional yielding of selected jacket members. In addition, many of the piles now hinge at the mudline elevation. This leads to the increased non-linearity in the response of the structure. Finally, near the ultimate load levels the piles yield and form a second hinge at lower elevations below the mudline. At these loads, the behavior of the platform is highly non-linear. Once the two hinges exist in the piles, the platform behavior becomes highly non-linear as reflected in the flattening of the load-displacement curves (Figs. 6.2.1-6.2.3). The formation of the two hinges in the piles then creates a failure mechanism, with the structure unable to withstand any additional loads.

TABLE 6.1.1
SOUTH PELTO 11-F
SUMMARY OF ENVIRONMENTAL LOADS

RETURN PERIOD (yrs.)	WAVE/CURRENT LOAD (kips)								
	BROADSIDE			END-ON			DIAGONAL		
	DECK	JACKET	TOTAL	DECK	JACKET	TOTAL	DECK	JACKET	TOTAL
25	210	1125	1335	320	805	1125	300	1331	1631
50	360	1168	1528	380	849	1229	420	1424	1844
100	525	1268	1793	450	899	1349	550	1513	2063
250	740	1367	2107	560	960	1520	750	1646	2396
500	940	1436	2376	700	1014	1714	950	1743	2693
1000	1120	1521	2641	830	1066	1896	1130	1840	2970

RETURN PERIOD (yrs.)	NORMALIZED WAVE/CURRENT LOAD								
	BROADSIDE			END-ON			DIAGONAL		
	DECK	JACKET	TOTAL	DECK	JACKET	TOTAL	DECK	JACKET	TOTAL
25	16%	84%	100%	28%	72%	100%	18%	82%	100%
50	24%	76%	100%	31%	69%	100%	23%	77%	100%
100	29%	71%	100%	33%	67%	100%	27%	73%	100%
250	35%	65%	100%	37%	63%	100%	31%	69%	100%
500	40%	60%	100%	41%	59%	100%	35%	65%	100%
1000	42%	58%	100%	44%	56%	100%	38%	62%	100%

6.3 Reserve Strength

A common measure for assessing the ultimate capacity of an offshore platform is the Reserve Strength Ratio (RSR) which is defined as:

$$RSR = \frac{R_u}{F_d}$$

where:

R_u - ultimate platform resistance (expressed as base shear in kips)

F_d - design environmental load (100-year load in kips)

The RSR can be viewed as an estimated safety factor against platform collapse relative to the design environmental event.

Table 6.3.1 provides a summary of environmental loads, ultimate capacities, and RSR values in the three directions that were analyzed.

TABLE 6.3.1

Summary of Results

DIRECTION	ULTIMATE RESISTANCE (kips) (1)	100-year LOAD (kips) (2)	RSR (1)/(2)
Broadside	3568	1793	1.99
End-On	2083	1349	1.54
Diagonal	3007	2063	1.46

The diagonal direction governs since it has the lowest RSR. Also note that by comparing the ultimate resistance with the environmental loads in Table 6.1.1, the ultimate resistance of the platform corresponds to the 1000-year environmental loads.

Hurricane Andrew environmental loads (Table 6.2.2) are greater than the 100-year event, but less than the ultimate capacity of the platform. So the fact that the South Pelto 11-F platform did not fail during Hurricane Andrew is consistent with the analyses. The results (Figure 6.2.3) indicate that at these load levels there should have been yielding of selected members. In the next underwater inspection program, it is recommended that an inspection be performed of the welds at the members that are predicted to have yielded first.

Unfortunately, some of the members in this category are the pile sections below the mudline. Since these sections are not be accessible, it will not be possible to confirm if they yielded. However, yielding of one or two piles is not detrimental to the overall capacity of the platform. Furthermore, it is anticipated that in some cases, yielding of selected members predicted in the analyses may not be confirmed in the inspections. A recently completed study (Ref. 7.2) indicates that current analysis procedures are conservative and have a positive bias of about 15% to 20%. For these reasons, comparison of observed results with the predictions will be useful to calibrate the analyses procedures.

7.0 FAILURE PROBABILITY

As part of the overall assessment, annual probabilities of failure were computed for the platform. These values are useful in economic evaluations, such as cost benefit analyses, as well as performing relative comparisons with other structures that have been previously analyzed.

The procedures utilized in computing the probabilities of failure are described in detail in Ref. 7.1. For the loading, base shears corresponding to wave/current loads with recurrence intervals ranging from 25 to 1000 years were developed as previously described. For the resistance, the ultimate capacities expressed in terms of base shears obtained from the static pushover analyses were utilized, assuming that these represent mean values of the ultimate capacity in each of the three directions analyzed. The only additional information required is the breakdown of the distribution of sea states into the three directions.

A plot of the base shear as a function of the log of the return period indicates a strong linear relationship for the environmental loads (Figure 7.1). Note that this same trend was observed in all three directions. Based on this, it can be assumed that the right-hand tail of the wave loading (x) is exponentially distributed with the parameters a and b.

$$F(x) = 1 - \exp\left(-\frac{(x-a)}{b}\right)$$

Based on this linear relationship coupled with the assumption that the resistance will follow a normal distribution, the annual probability of failure (PFA) can be computed using the following closed form equation:

$$PFA = \exp\left(-\frac{m-a}{b} + \frac{s^2}{2*b^2}\right)$$

where:

m - the mean push-over load

s - coefficient of variation

a,b - the parameters defining the tail of the loading distribution.

For the resistance distribution a Coefficient of Variation (COV) of 9% was assumed, based on a similar assessment for the Hurricane Andrew JIP (Ref. 7.2).

The above formulation was utilized to compute annual probabilities of failure for the base case scenario, with two sensitivity cases corresponding to 15% and 20% positive bias on the ultimate strength. Details of the computations are given in Tables 7.1 through 7.3. The sensitivity cases were selected based on the Hurricane Andrew JIP findings (Ref. 7.2) regarding a conservatism in this range in the current ultimate strength calculation algorithms. Although this was only a preliminary finding, it was based on the analytical results of the ultimate strength of various platforms that were subjected to significant environmental loads during Hurricane Andrew. The analytical results were compared to the actual response of the structures during this hurricane and

TABLE 7.1
Failure Probability Analysis for South Pelto 11-F Platform

Case 1: Base Case Scenario

WAVE LOADS:

Return Period		Load (kips)		
(years)	(Ln)	Broadside	End-On	Diagonal
25	3.22	1335	1125	1631
50	3.91	1528	1229	1844
100	4.61	1793	1349	2063
250	5.52	2107	1520	2396
500	6.21	2376	1714	2693
1000	6.91	2641	1896	2970

COMPUTATION OF PROBABILITY OF FAILURE:

	Period	Ln(Period)	Broadside	End-On	Diagonal
Point 1:	25	3.22	1335	1125	1631
Point 2:	1000	6.91	2641	1896	2970
Slope (b):			354.04	209.01	362.98
Intercept (a):			195.40	452.23	462.60
Mean Resistance (m):			3568	2083	3007
Sig. Res. (s=0.09xm)			321	187	271
Partial Probability of Failure:			1.10E-04	6.11E-04	1.19E-03
Sea State Breakdown			5%	50%	45%

Annual Failure Probability - PFA: 8.48E-04

NOTES:
(1) The Mean Resistance (m) is the ultimate strength in each direction.
(2) The overall resistance distribution is assumed to be normally distributed with coefficient of variation of 9%.

TABLE 7.2
Failure Probability Analysis for South Pelto 11-F Platform

Case 2: 15% Positive Bias on Ultimate Strength

WAVE LOADS:

Return Period		Load (kips)		
(years)	(Ln)	Broadside	End-On	Diagonal
25	3.22	1335	1125	1631
50	3.91	1528	1229	1844
100	4.61	1793	1349	2063
250	5.52	2107	1520	2396
500	6.21	2376	1714	2693
1000	6.91	2641	1896	2970

COMPUTATION OF PROBABILITY OF FAILURE:

	Period	Ln(Period)	Broadside	End-On	Diagonal
Point 1:	25	3.22	1335	1125	1631
Point 2:	1000	6.91	2641	1896	2970
Slope (b):			354.04	209.01	362.98
Intercept (a):			195.40	452.23	462.60
Mean Resistance (m):			4103	2395	3458
Sig. Res. (s=0.09xm)			369	216	311
Partial Probability of Failure:			2.77E-05	1.56E-04	3.76E-04
Sea State Breakdown			5%	50%	45%

Annual Failure Probability - PFA: 2.49E-04

NOTES:
(1) The Mean Resistance (m) is the ultimate strength in each direction.
(2) The overall resistance distribution is assumed to be normally distributed with coefficient of variation of 9%.

TABLE 7.3
Failure Probability Analysis for South Pelto 11-F Platform

Case 3: 20% Positive Bias on Ultimate Strength

WAVE LOADS:

Return Period		Load (kips)		
(years)	(Ln)	Broadside	End-On	Diagonal
25	3.22	1335	1125	1631
50	3.91	1528	1229	1844
100	4.61	1793	1349	2063
250	5.52	2107	1520	2396
500	6.21	2376	1714	2693
1000	6.91	2641	1896	2970

COMPUTATION OF PROBABILITY OF FAILURE:

	Period	Ln(Period)	Broadside	End-On	Diagonal
Point 1:	25	3.22	1335	1125	1631
Point 2:	1000	6.91	2641	1896	2970
Slope (b):			354.04	209.01	362.98
Intercept (a):			195.40	452.23	462.60
Mean Resistance (m):			4282	2500	3608
Sig. Res. (s=0.09xm)			385	225	325
Partial Probability of Failure:			1.76E-05	9.92E-05	2.57E-04
Sea State Breakdown			5%	50%	45%

Annual Failure Probability - PFA: 1.66E-04

NOTES:
(1) The Mean Resistance (m) is the ultimate strength in each direction.
(2) The overall resistance distribution is assumed to be normally distributed with coefficient of variation of 9%.

the extent of predicted to observed damage along with the survivability of the structures. Based on these analysis, the following results were obtained:

Scenario	PFA
1. Base Case	8.48×10^{-4}
2. 15% Positive Bias	2.49×10^{-4}
3. 20% Positive Bias	1.66×10^{-4}

In summary, for the base case scenario, the South Pelto 11-F platform is expected to have an annual probability of failure of about 10^{-3}. This is about the same as was found for the Vermilion 46-A platform (Ref. 3.2.1); not surprising, since the two platforms are of a similar design and age. For a new design, the probability of failure is expected to be about an order of magnitude lower, 10^{-4} (Ref. 3.2.1).

8.0 PLATFORM ASSESSMENT

API has recently proposed a DRAFT version of the proposed criteria for the assessment of existing platforms (Ref. 8.1). The general flowchart describing the proposed assessment procedure is given in Figure 8.1.

The API procedure focuses on two variables for defining the recommended loads for the assessment. The first is the life exposure consequences defined in three categories: 1) manned, non-evacuated, 2) manned, evacuated, and 3) unmanned. The second variable is the potential impact of a failure on the environment. For this there are two categories: 1) significant environmental impact and 2) insignificant environmental impact. Based on the classification of the structure, a corresponding assessment process and associated environmental criteria are specified. The more significant the consequence of failure, the more severe the environmental criteria that are used in the assessment. Thus, the most severe criteria would be for a manned, non-evacuated, significant environmental impact platform. On the other end of the spectrum would be an unmanned, insignificant environmental impact platform.

It is important to note that the API procedures are intended to address only life safety exposure and potential environmental impact, but not economic risk. The assessment and determination of the acceptable level of economic risk is left to the discretion of the operator.

If based on its current operation and facilities (the South Pelto 11-F platform is classified as an unmanned, insignificant environmental impact structure), then the ultimate strength analysis should demonstrate that the structure can survive the loads associated with the following environmental conditions:

Wave Height:	26 feet
Wave Period:	11.5 sec
Storm Tide:	3 feet
Current Speed:	1.0 kts
Wind Speed:	50 kts

These environmental conditions are significantly less (wave height, current speed) than the 1000-year events which correspond to the ultimate resistance of the platform. Therefore, it can be concluded by inspection that the platform in its current condition satisfies the proposed API criteria.

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4.2.2 *Vermilion 46A Deck Loads,"* Technical Memorandum, File 426-25, October 5, 1993.

5.1 McClelland Engineers, *"Soil and Foundation Investigation Block 10, South Pelto Area,"* ("B" Platform Location), Report to Oil Corp. New Orleans, January 1976.

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7.1 , *"Failure Probabilities Associated with A Push-Over Analysis for Vermilion 46-A,"* Technical Memorandum, File 054-86, July 1, 1993.

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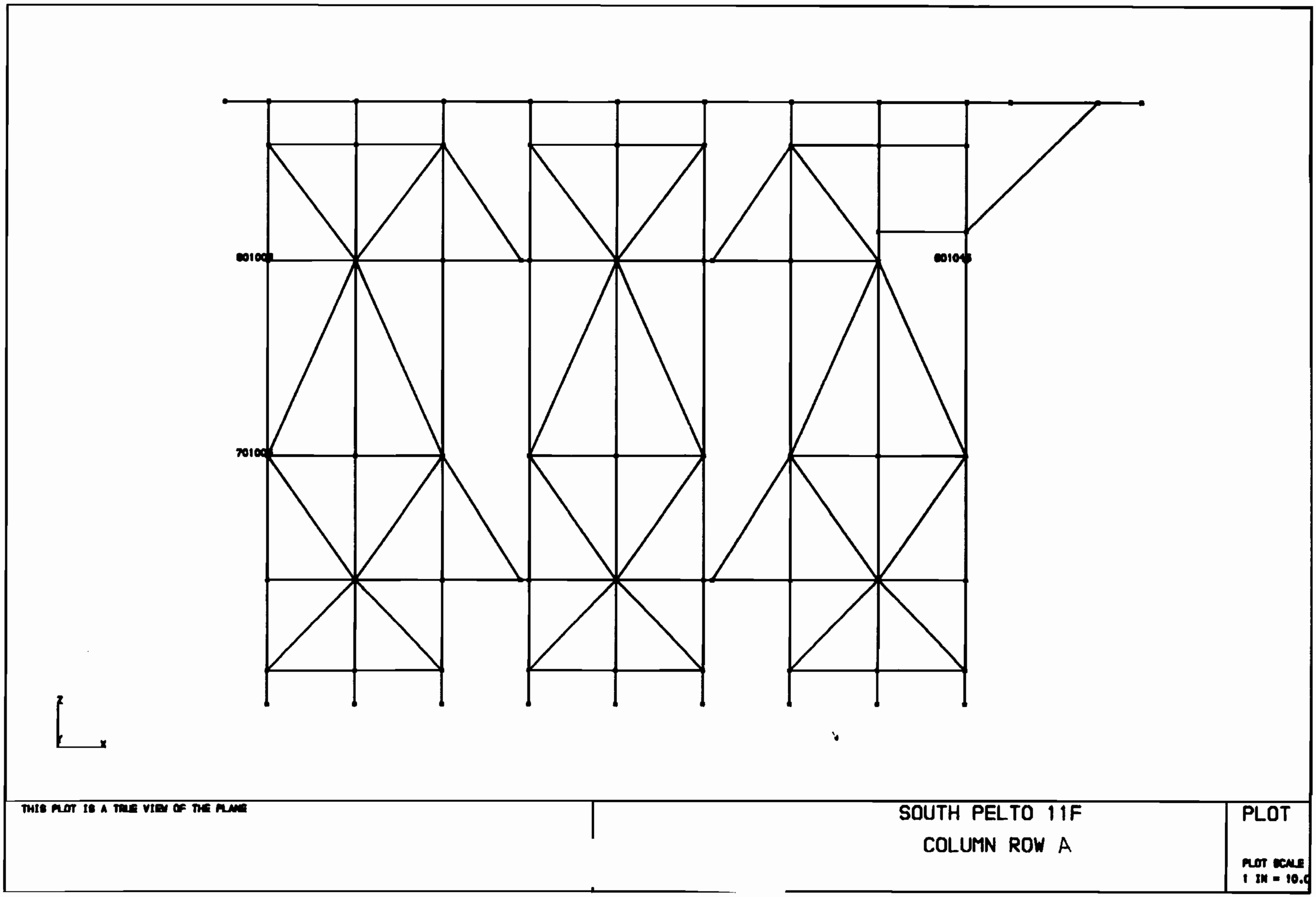


Figure 3.1.1 - Dams Model - Column Row A

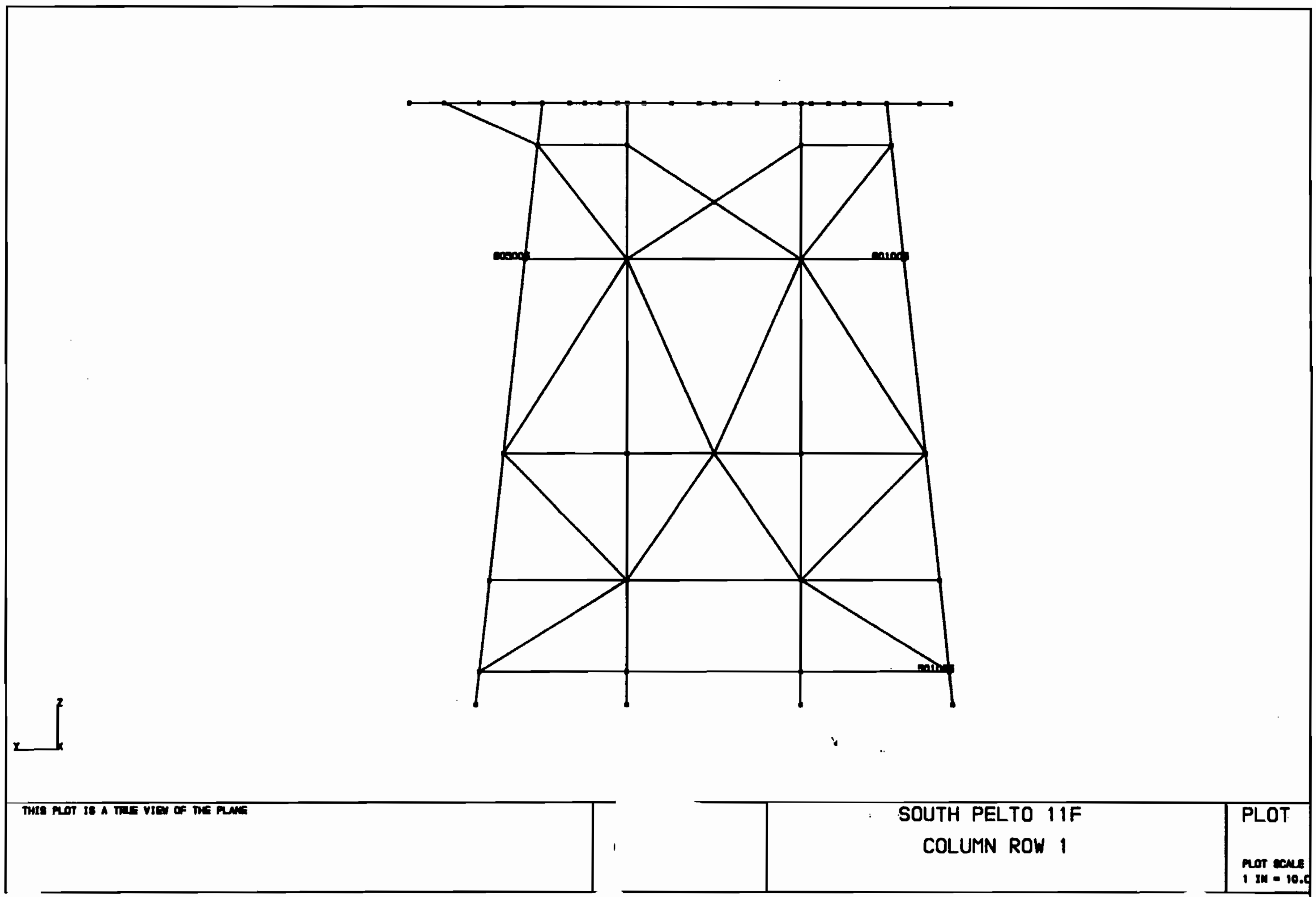
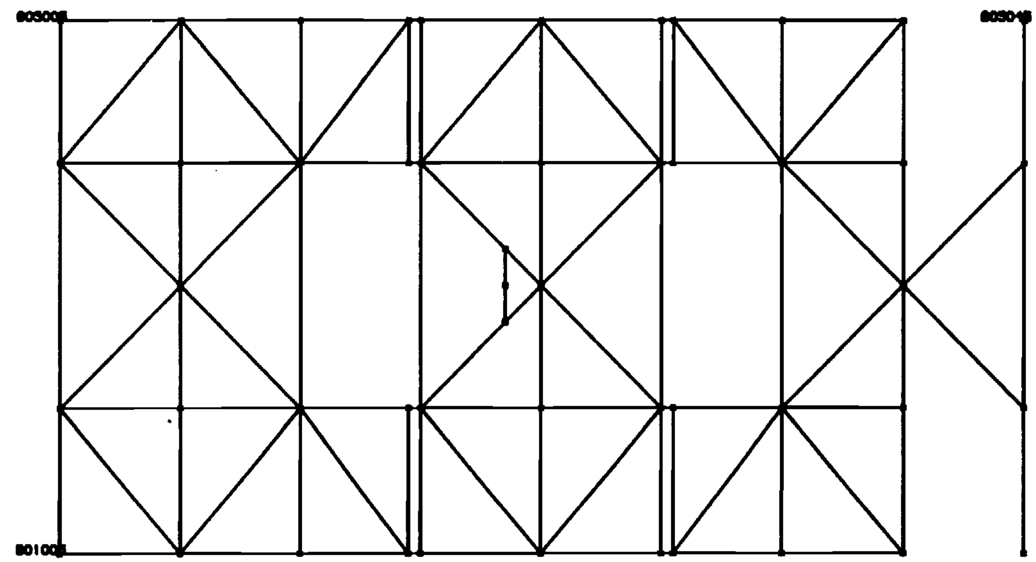


Figure 3.1.2 - Dams Model - Column Row 1



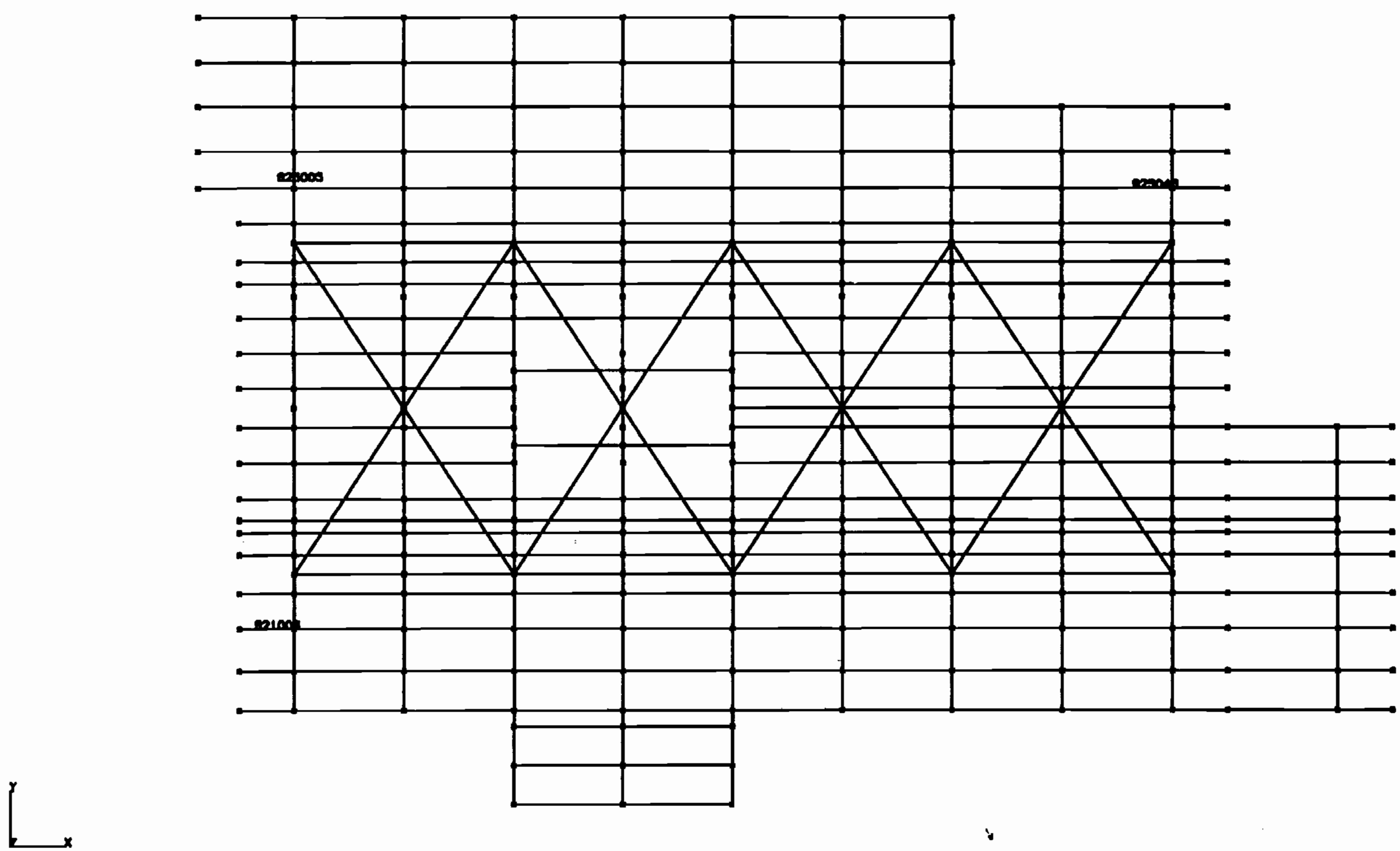
THIS PLOT IS A TRUE VIEW OF THE PLANE

SOUTH PELTO 11F
TYPICAL HORIZONTAL PLAN

PLOT

PLOT SCALE
1 IN = 10.0

s Model - Typical Horizontal Plan



THIS PLOT IS A TRUE VIEW OF THE PLANE

SOUTH PELTO 11F
DECK PLAN VIEW

PLOT

PLOT SCALE
1 IN = 10.0

Demo Model - Deck Plan View

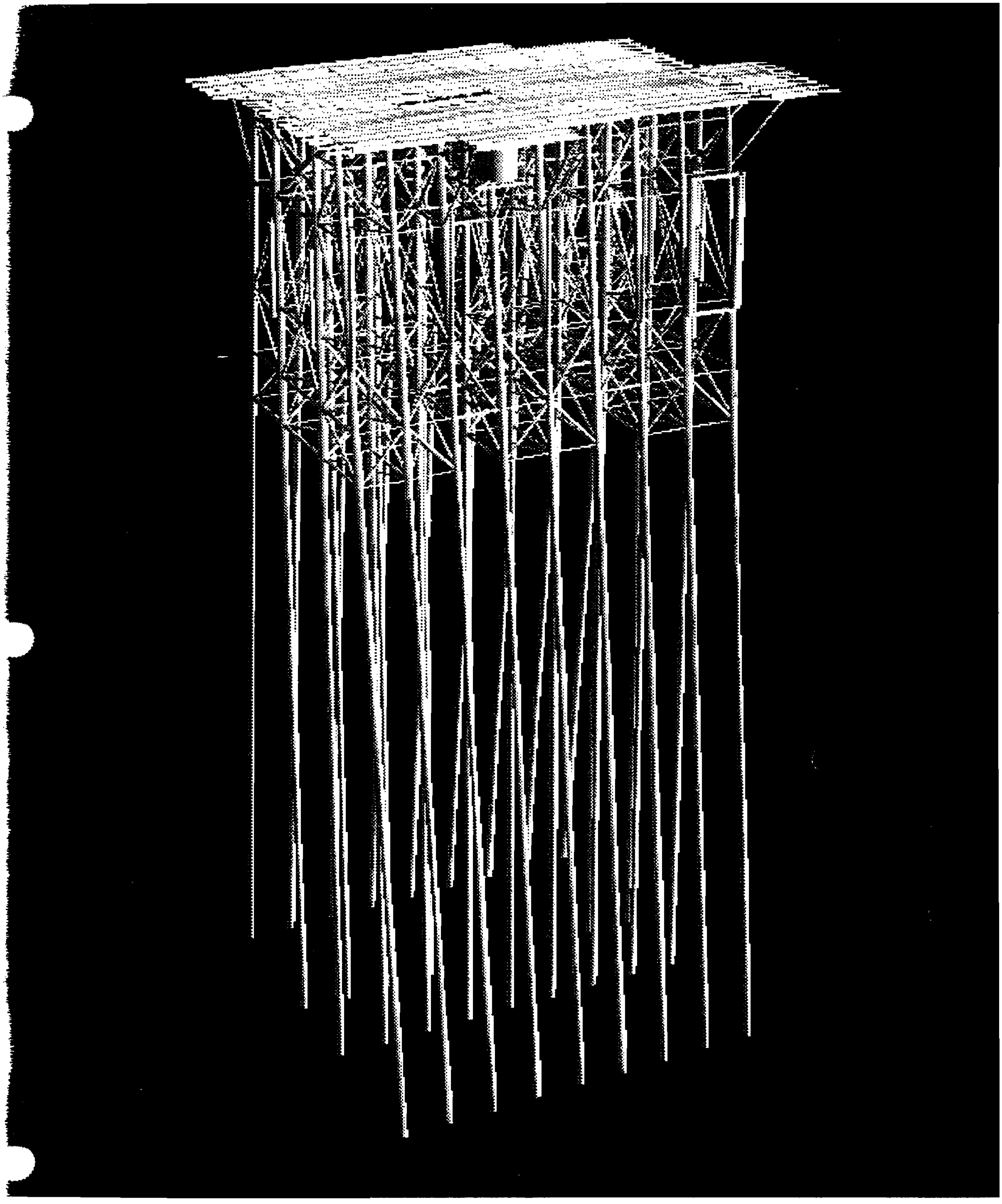
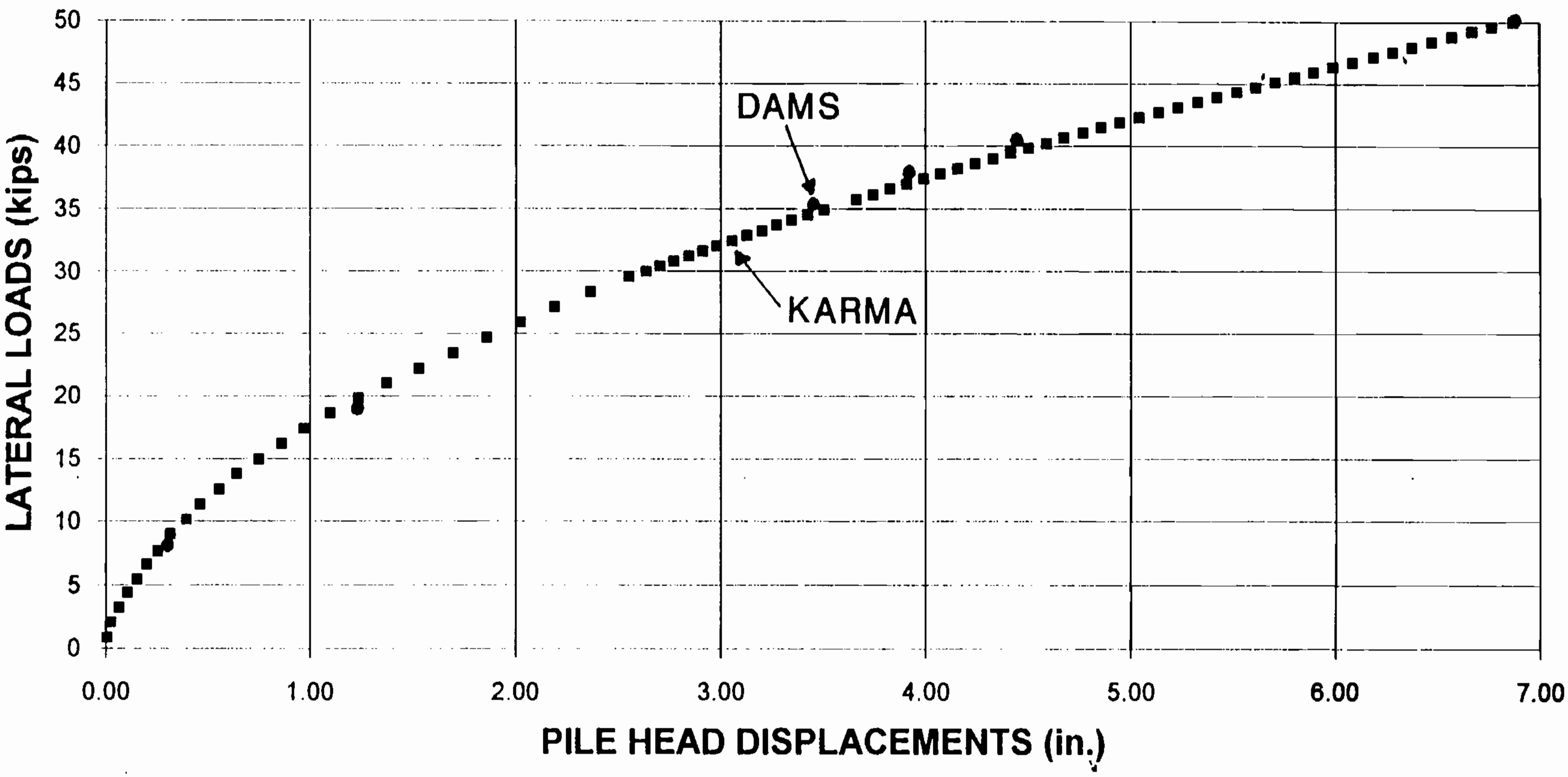


Figure 3.2.1 - Karma Model - Overall View

FIGURE 3.3.1

SOUTH PELTO 11-F
DAMS vs. KARMA PILE HEAD RESPONSE



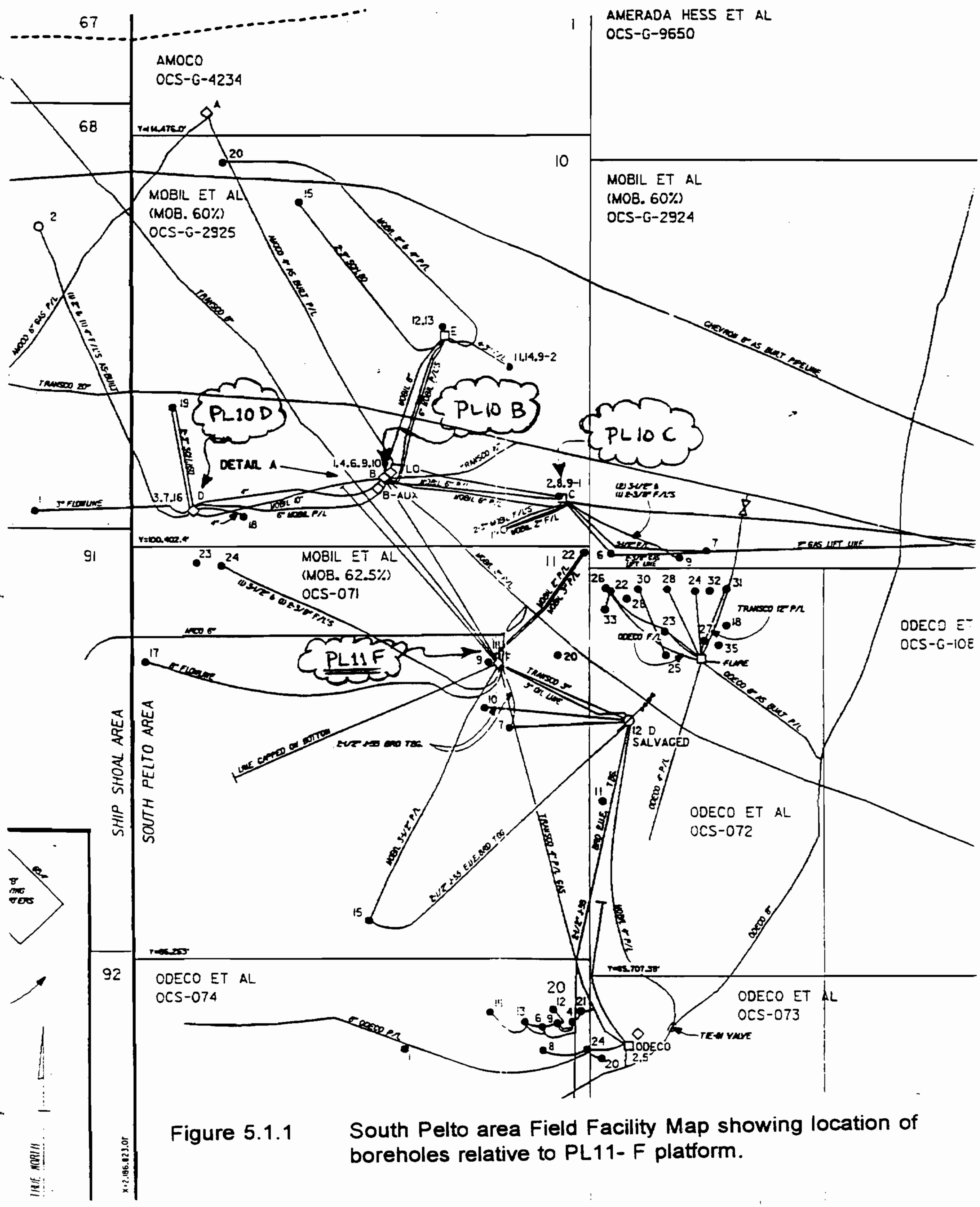


Figure 5.1.1 South Pelto area Field Facility Map showing location of boreholes relative to PL11- F platform.

ENGINEERS CALCULATION SHEET

FOR DJP

JOB OR AUTH. NO.

LOCATION

PAGE

SUBJECT South Pelto 11F (PL 11F)

DATE 15 JULY 93

BY RWE

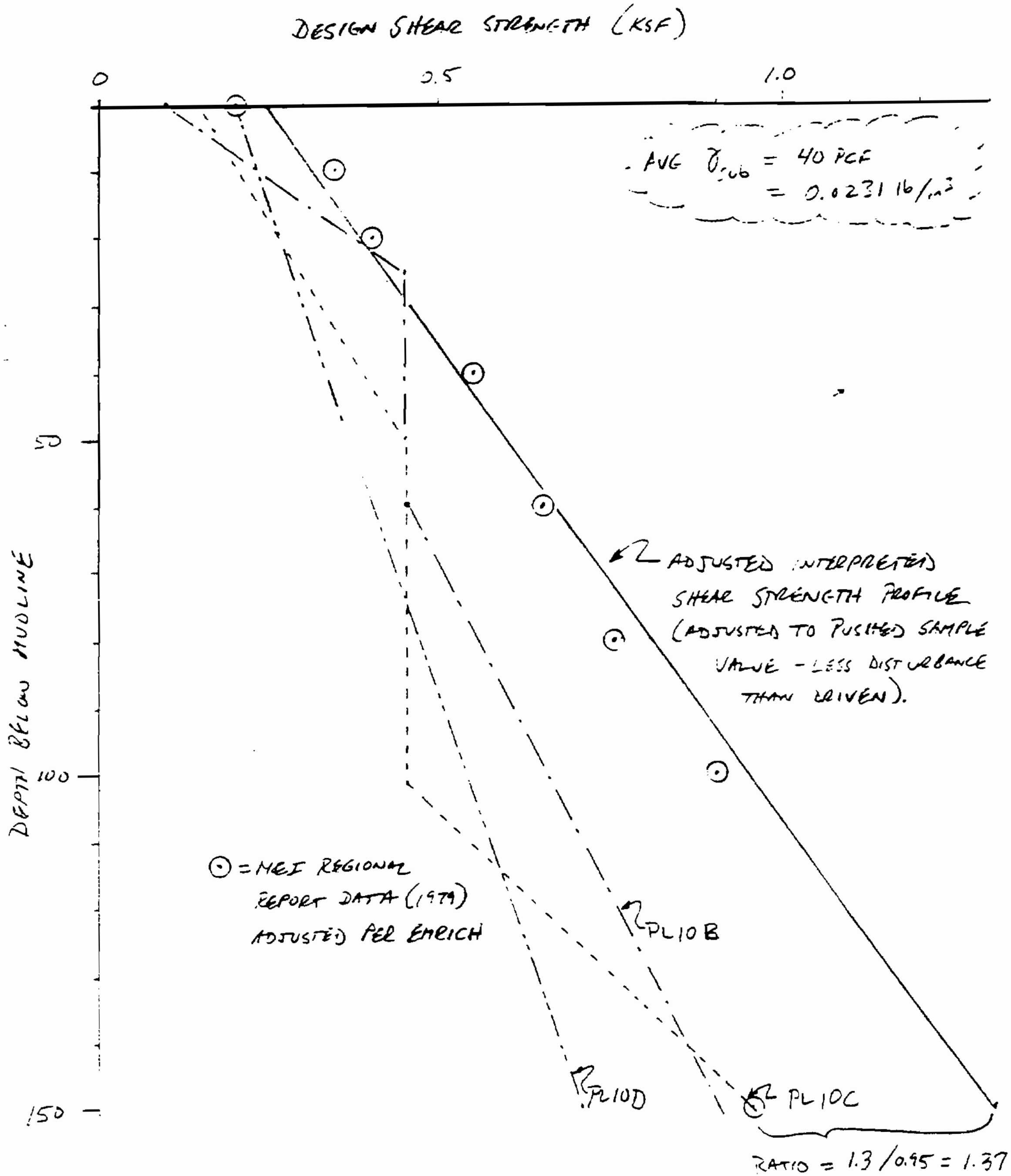


Figure 5.1.2 Comparison of shear strength profiles from PL10 "B", "C" and "D" locations. Superimposed is an interpreted shear strength profile that has been increased to account for sample disturbance. Increased disturbance and lower strengths resulted in these site investigations because smaller 2.25 inch diameter samplers were used (Refs. 5.1 to 5.3).

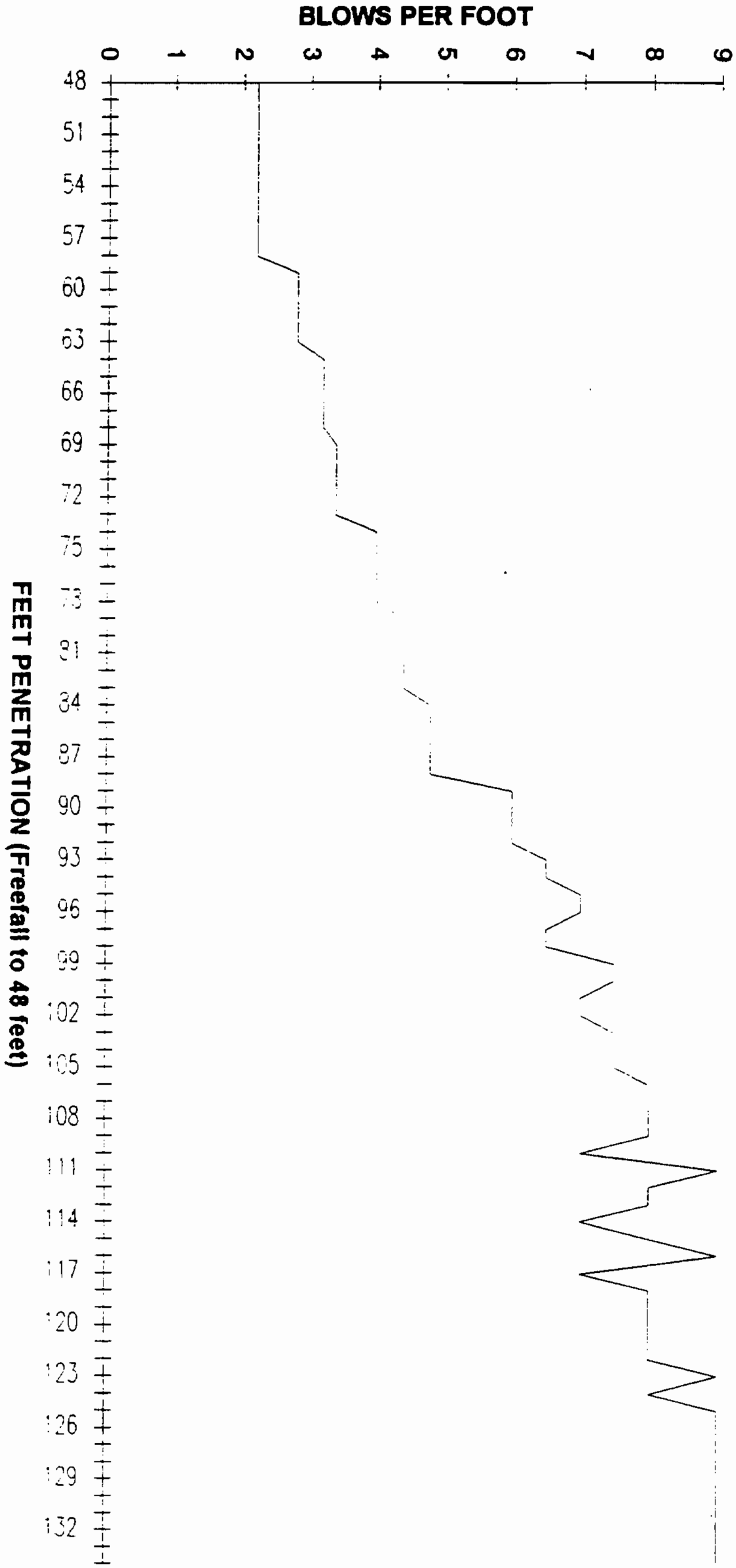


Figure 5.1.3 BLOW COUNT RECORD FOR PELTO 11 - F (PILE B - 6)

PILE OD = 18in

HAMMER = VULCAN TYPE

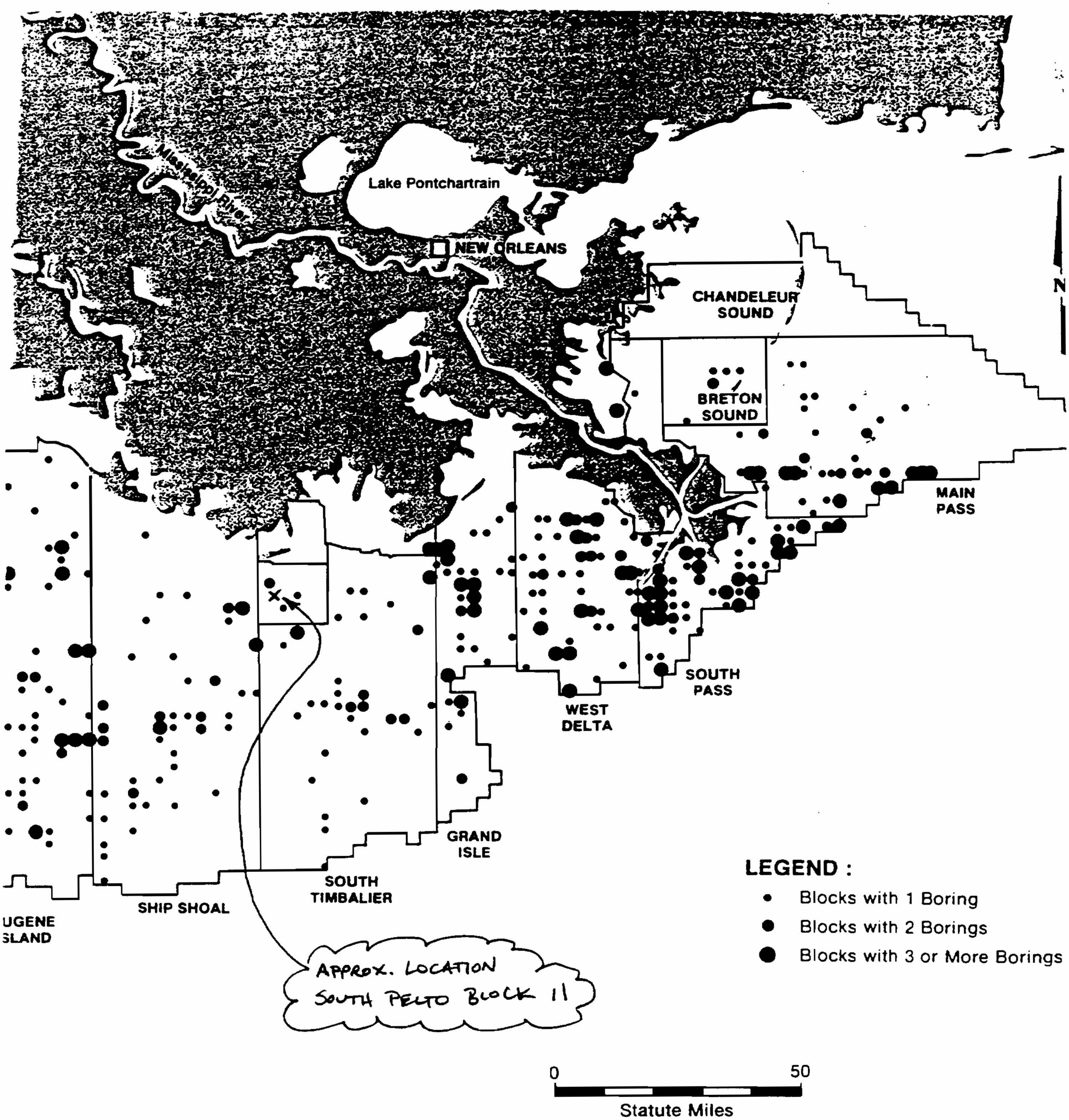
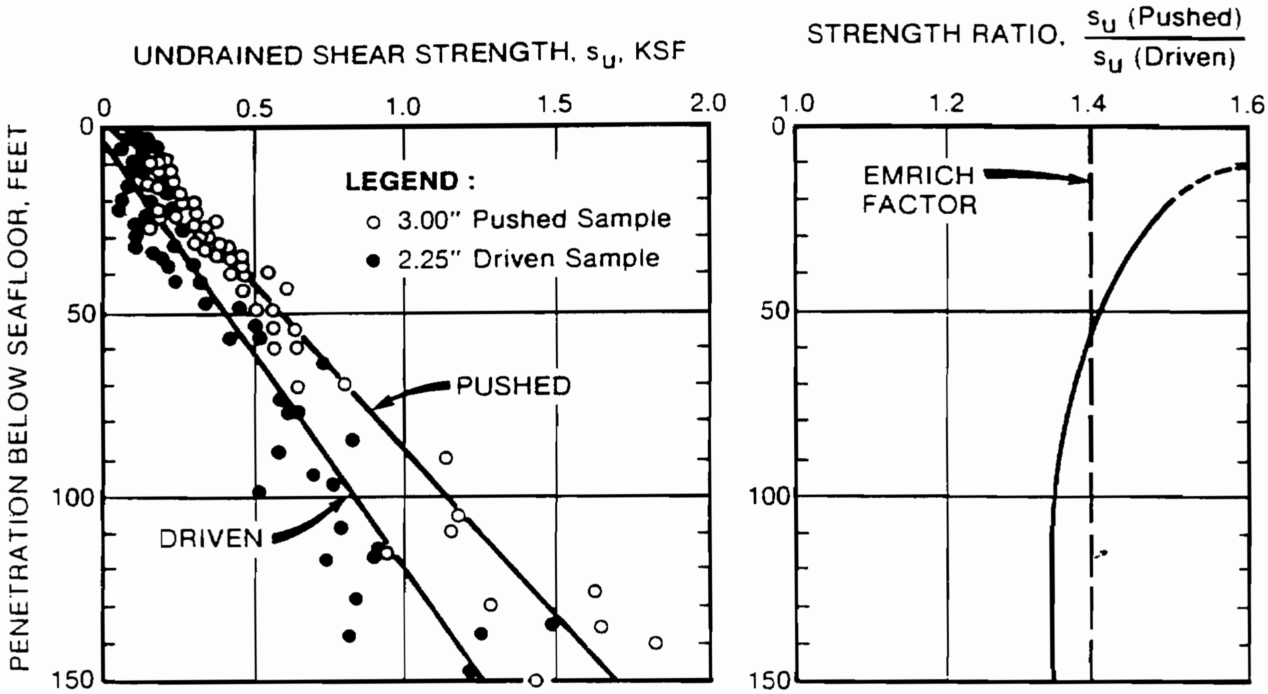
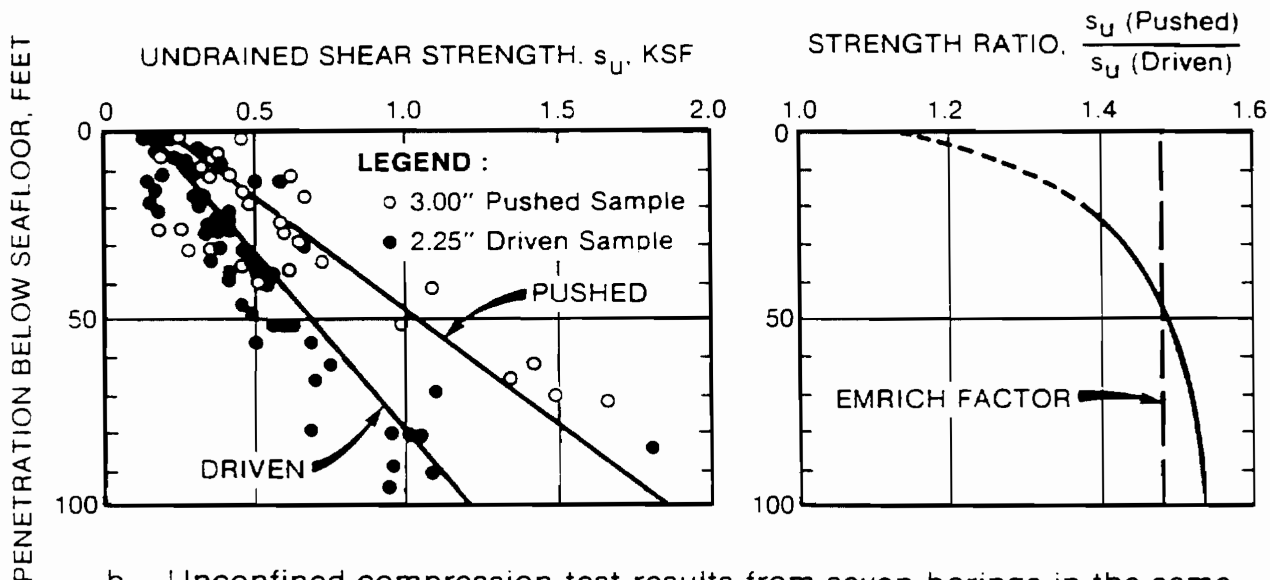


Figure 5.1.4 McClelland database used to develop adjusted shear strength profile shown in Figure 5.1.2 (from Ref. 5.4)



a. Miniature vane test results from five borings in the same block in West Delta Area



b. Unconfined compression test results from seven borings in the same block in Eugene Island Area

Figure 5.1.5 Comparison of shear strength from pushed versus driven samples at a location in the Eugene Island area (from Ref. 5.4).

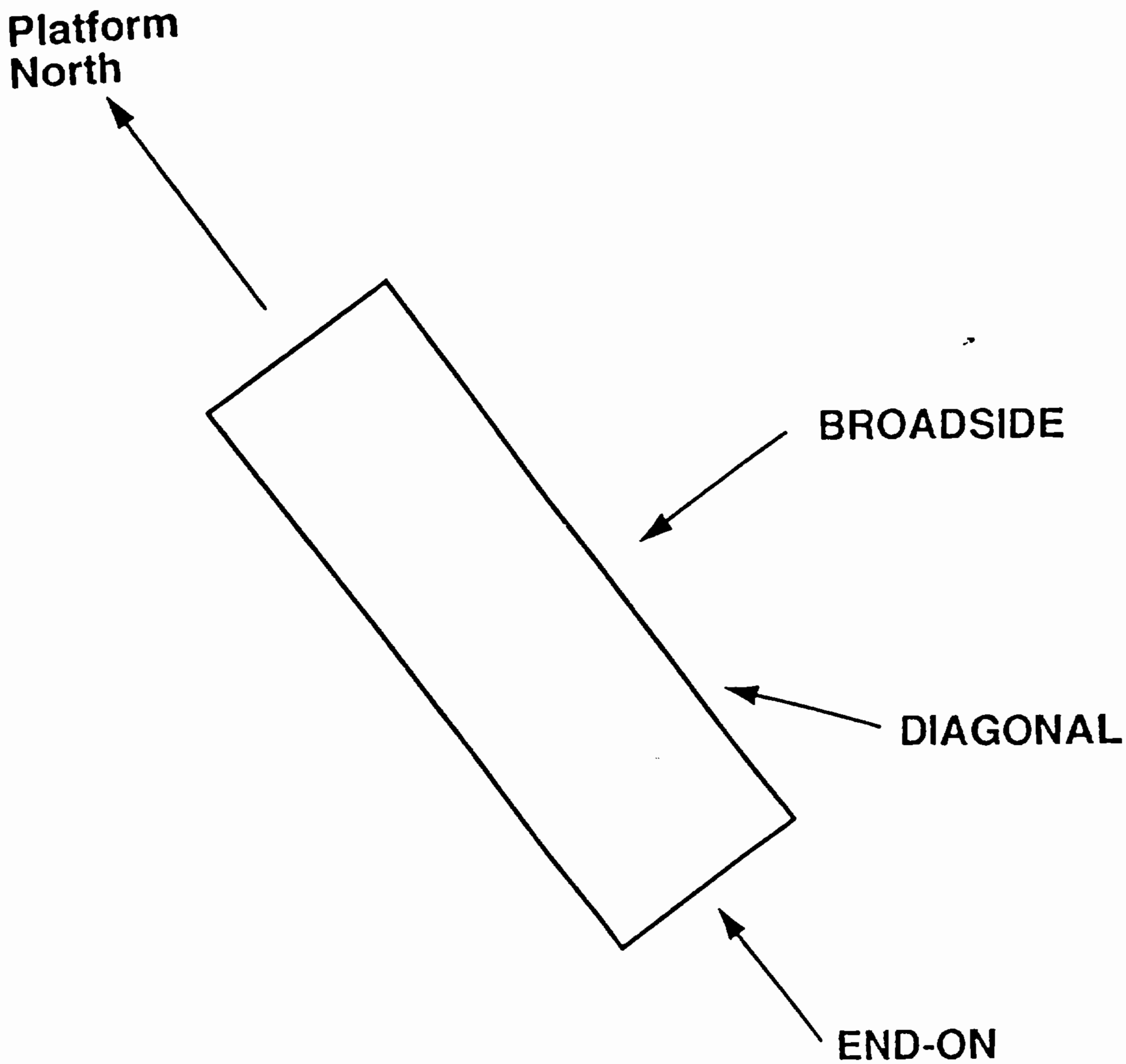


Figure 6.1.1 - Environmental Load Directions

SOUTH PELTO 11-F
SUMMARY OF ENVIRONMENTAL LOADS
BROADSIDE DIRECTION

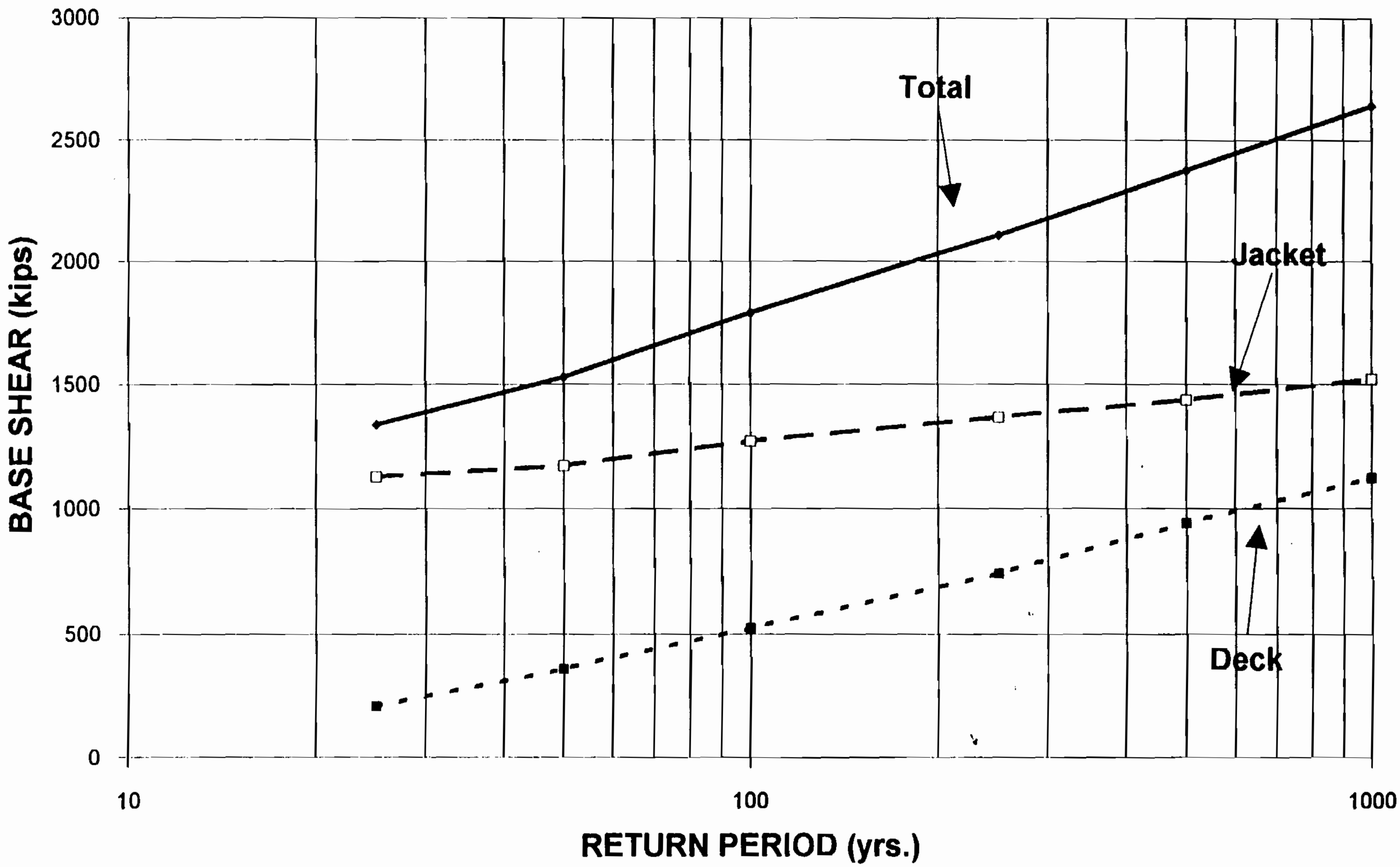
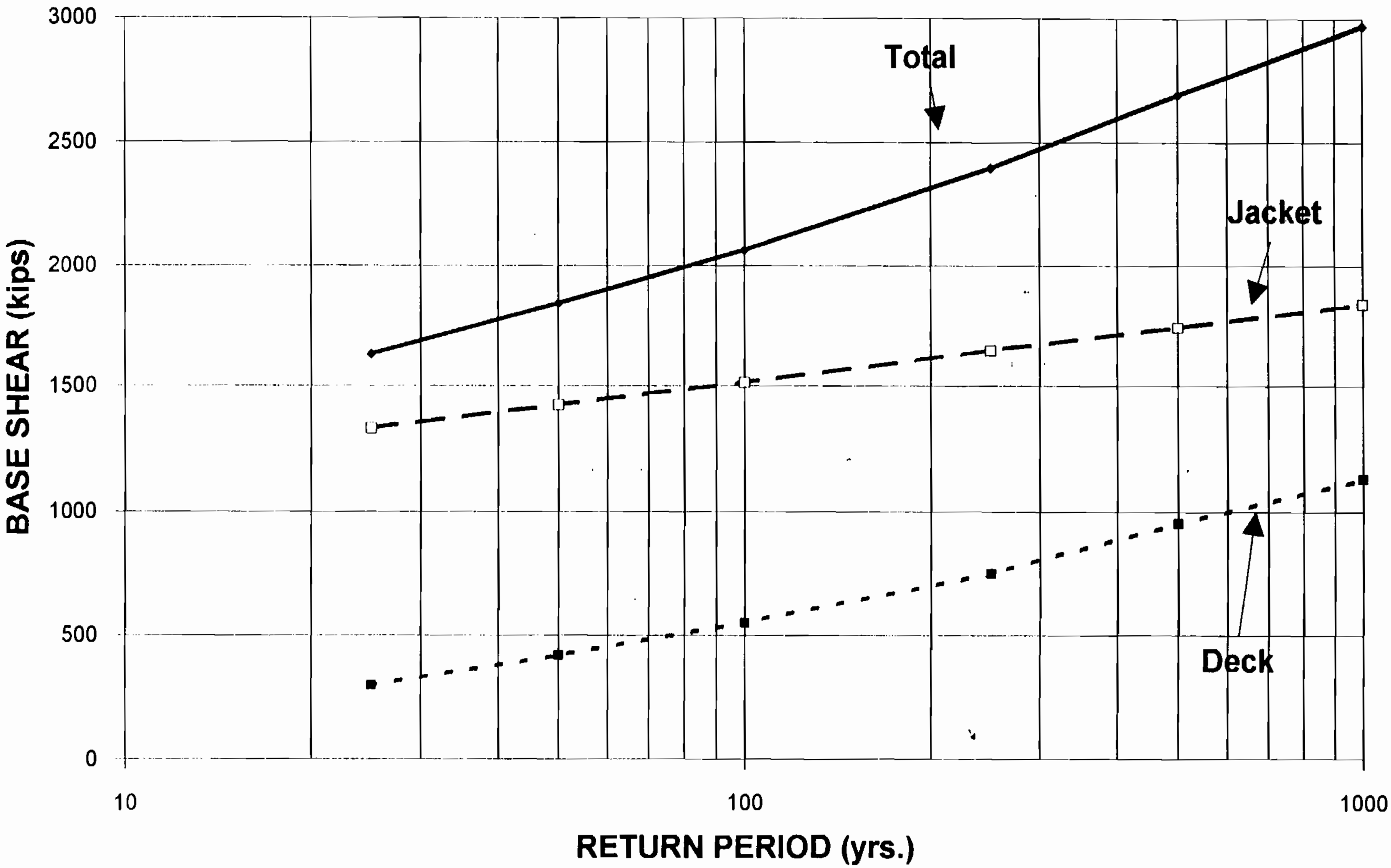


Figure 6.1.2

SOUTH PELTO 11-F
SUMMARY OF ENVIRONMENTAL LOADS
DIAGONAL DIRECTION



Fi) 6.1.4

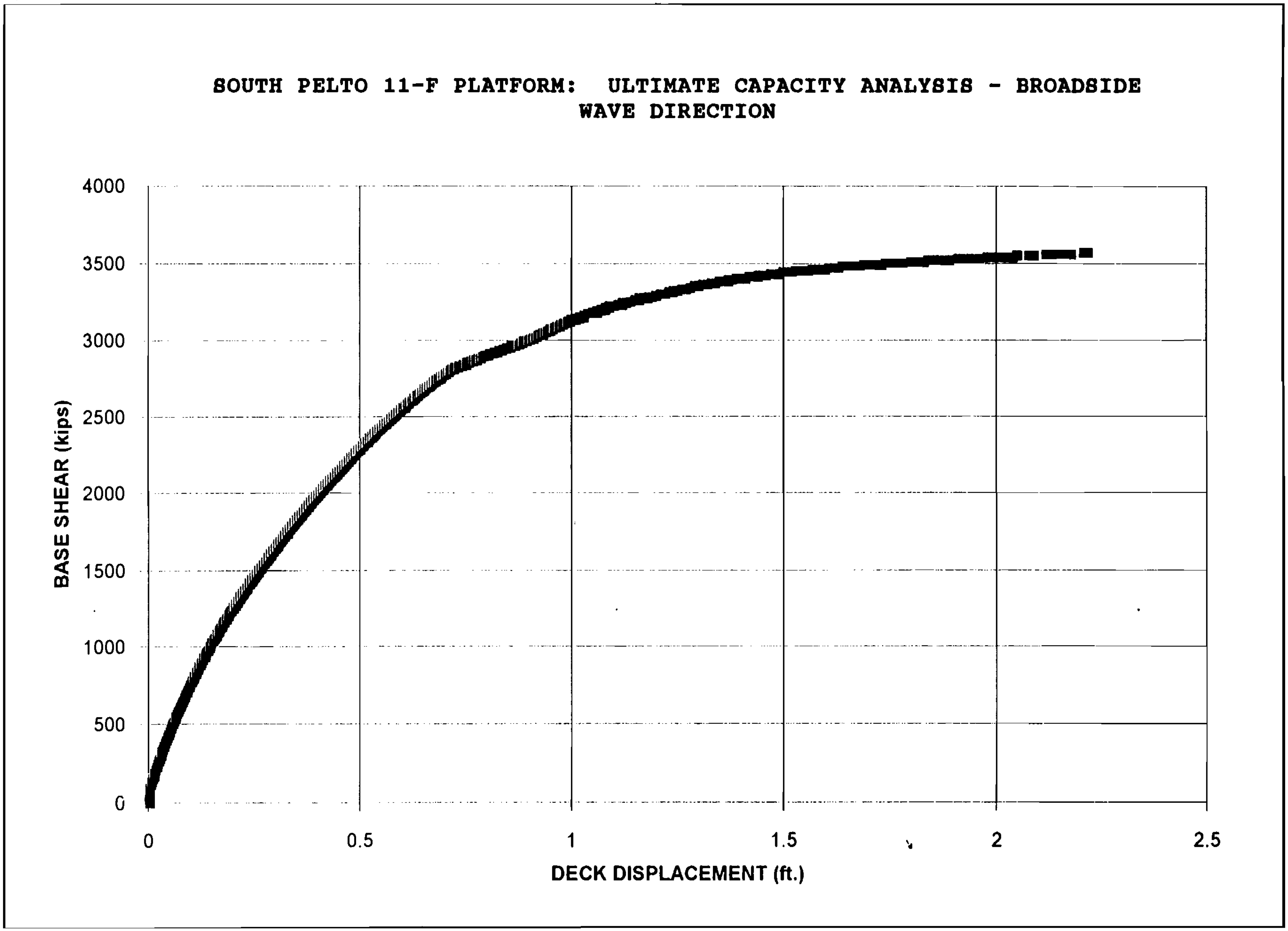

SOUTH PELTO 11-F PLATFORM: ULTIMATE CAPACITY ANALYSIS - BROADSIDE WAVE DIRECTION
BASE SHEAR (kips)
4000
3500
3000
2500
2000
1500
1000
500
0
0
0.5
1
1.5
2
2.5
DECK DISPLACEMENT (ft.)

Figure 6.2.1

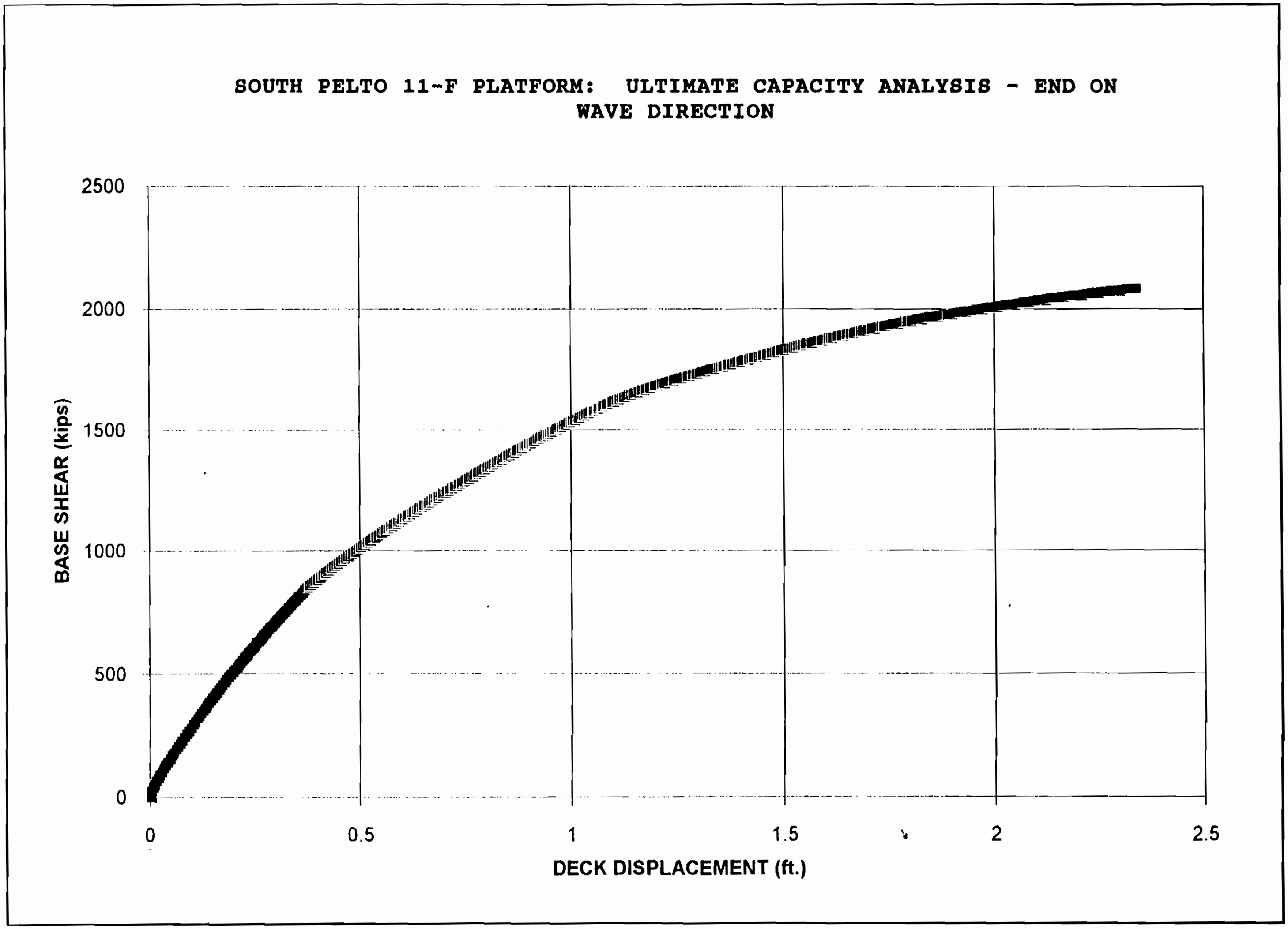

SOUTH PELTO 11-F PLATFORM: ULTIMATE CAPACITY ANALYSIS - END ON WAVE DIRECTION
BASE SHEAR (kips)
2500
2000
1500
1000
500
0
0
0.5
1
1.5
2
2.5
DECK DISPLACEMENT (ft.)

Figure :.2

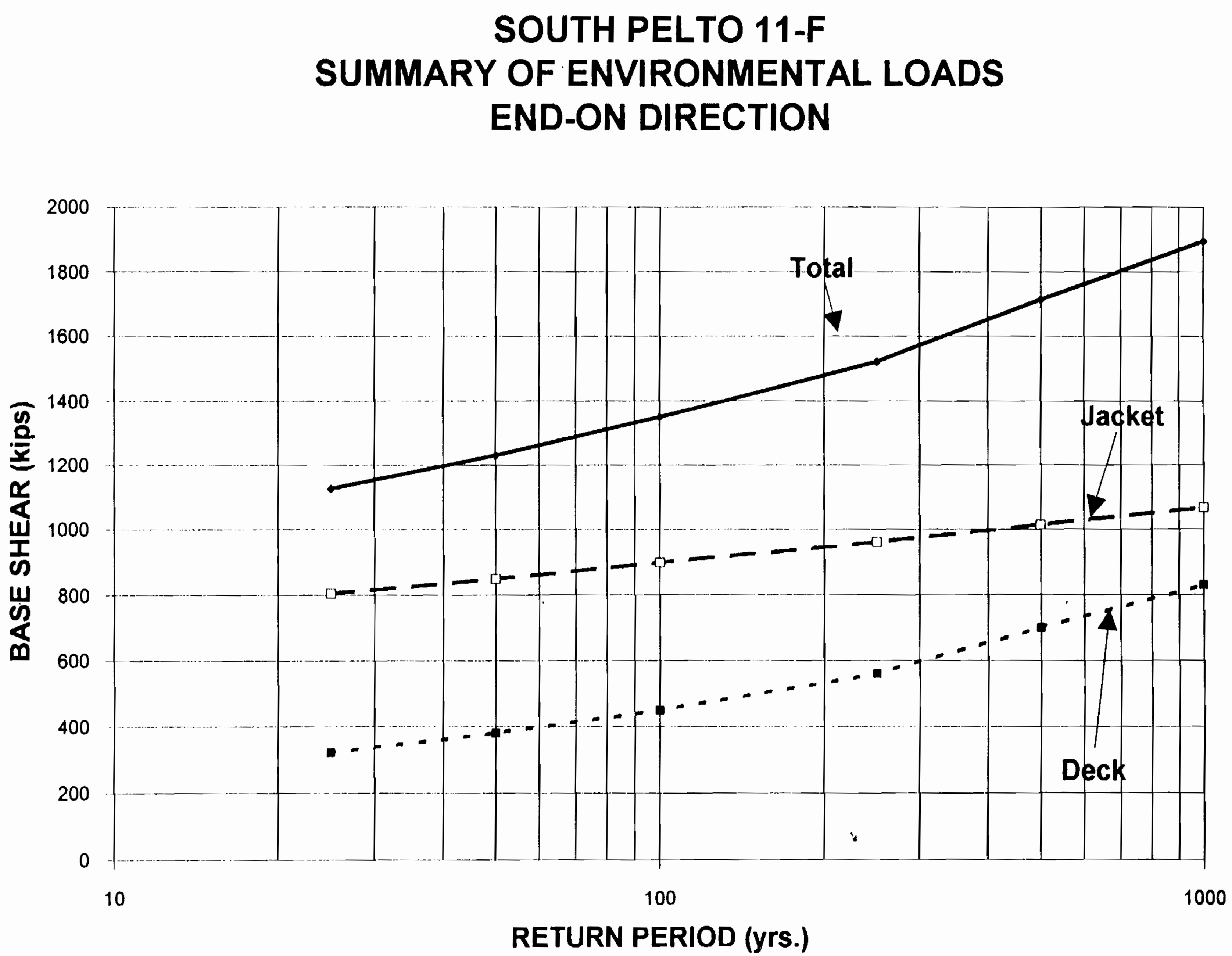

SOUTH PELTO 11-F
SUMMARY OF ENVIRONMENTAL LOADS
END-ON DIRECTION
BASE SHEAR (kips)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
10
100
1000
RETURN PERIOD (yrs.)
Total
Jacket
Deck

Figure 6.1.3

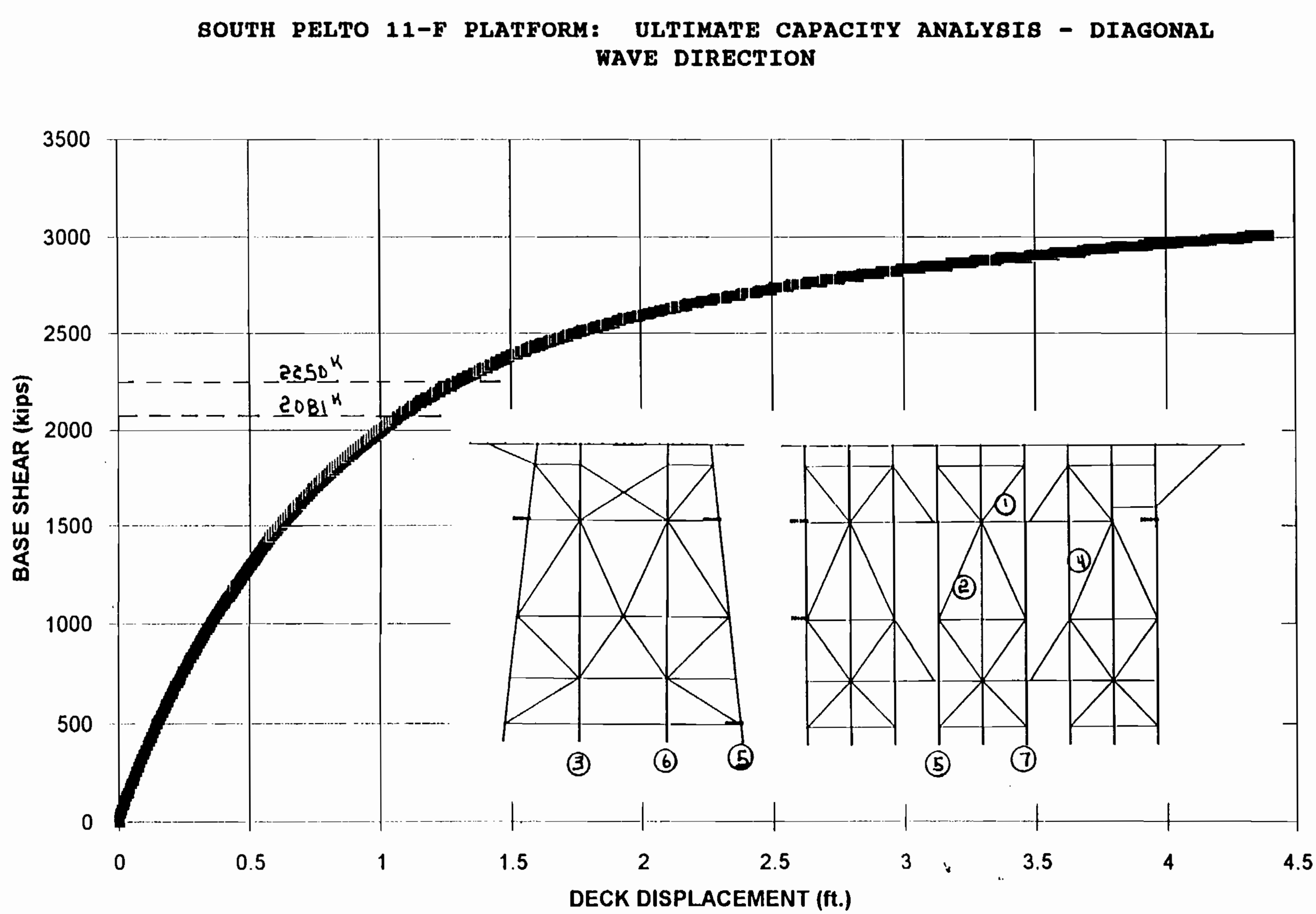
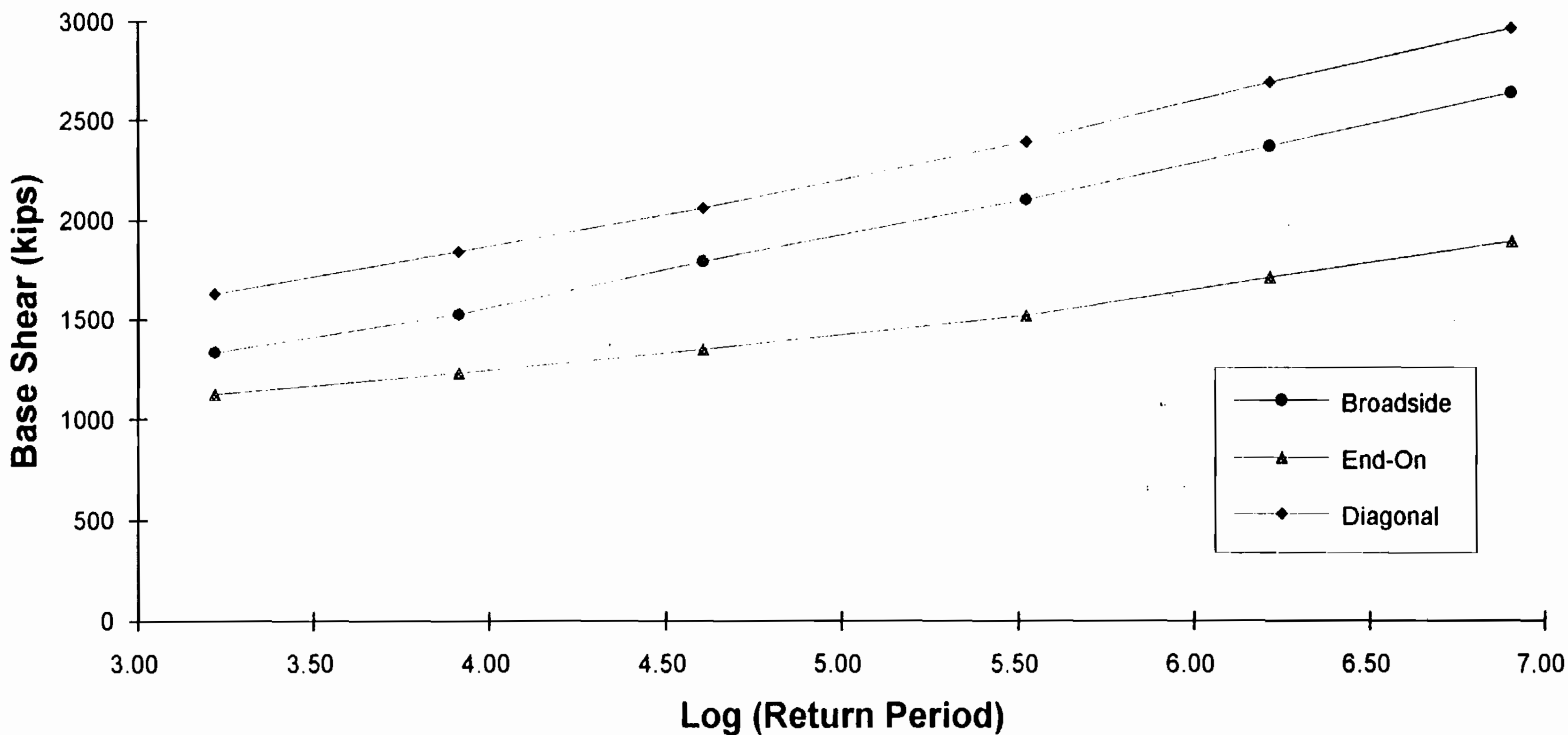

SOUTH PELTO 11-F PLATFORM: ULTIMATE CAPACITY ANALYSIS - DIAGONAL WAVE DIRECTION
BASE SHEAR (kips)
3500
3000
2500
2000
1500
1000
500
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
DECK DISPLACEMENT (ft.)
2250K
2081K
1
2
3
4
5
6
7

Fig 6.2.3

Figure 7.1

SOUTH PELTO 11 - F
BASE SHEAR vs. RETURN PERIOD



PLATFORM "K"

TRIAL APPLICATION OF THE DRAFT API RP 2A-WSD PROCEDURE FOR ASSESSMENT OF EXISTING PLATFORMS

TRIAL DOCUMENT FOR PLATFORM K

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INTRODUCTION

This document summarizes the results of a trial application of the draft API RP 2A-WSD Section 17 procedure for assessment of existing platforms. This trial application covers the assessment of an existing 8-leg platform installed in 1965 in a water depth of 160 feet in the Gulf of Mexico. The format and content of this trial document is in accordance with instructions provided by PMB Engineering, Inc. who is conducting the Joint Industry Project for the Minerals Management Service (MMS) and Participants. This trial application is based on the following:

1. *API RP 2A-WSD 20th Edition, Draft Section 17, Assessment of Existing Platforms,* dated April 29, 1994.

2. *API RP 2A-WSD Draft Section 17.0 - Balloted Changes*, Letter from K. A. Digre to API Task Group 92-5 dated June 29, 1994.

3. *API RP 2A-WSD, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design,* Twentieth Edition, July 1, 1993.

1.0 PLATFORM INFORMATION

This section provides the platform information that was compiled to conduct the platform assessment. The type and format of the information generally follows the outline provided in C17.4.1 of the draft Section 17.

a) General Information

- Platform Location: Gulf of Mexico, West Delta Area

- Water Depth: 160 ft.

- Platform Type: 8-leg jacket structure (refer to Figures 1 and 2)
  - Single and K-brace configuration used for jacket
  - Top of jacket plan dimensions: 40'-0" x 96'-0"
  - Bottom of jacket plan dimensions: 77'-8 3/8" x 154'-0"
  - 2-level trussed deck

- Original Platform Function: Drilling/production

- Current Platform Function: Satellite well structure (future drilling with cantilevered jack-up rig)

- Originally designed for 10 well conductors (nine 20" OD and one 48" OD)

- Currently supporting 11 well conductors (one 20" OD conductor was added after initial installation)
- Platform Orientation: N 2.8° E (refer to Figure 3)
- Manning: Currently operated unmanned (satellite structure to host platform in field)

b) Original Design

- Date of Design: 1964 (assumed from date on construction drawings)
- Design Documentation: The original design calculations and documentation for this platform were not available.
- Design Code: Unknown (assumed consistent with contemporary practices)
- Environmental Design Criteria: Unknown. Based on design documentation for a similar platform installed one year earlier in the same area, the criteria for this platform is believed to be based on a 25-yr return period with the following parameters:

Wave height, Hmax = 58 ft.	Storm tide = 5.1 ft.
Wave period = 14.5 sec.	Drag coefficient, Cd = 1.0 above MWL
Crest height = 39 ft.	Drag coefficient, Cd = 1.25 below MWL

- Material Specifications: Unavailable (for platform assessment, all material assumed to be ASTM A-36 quality)
- Operational Criteria - Design Deck Loading
  - Main deck designed to support a conventional platform drilling rig
  - Cellar deck equipment area designed for live load of 600 PSF (as per drawings)
  - Cellar deck well bay area designed for live load of 400 PSF (as per drawings)
- Soil Data: The soil boring report used in the original platform design was not available. The soil data used for this assessment is discussed under Section A.2, "Condition Assessment".
- Design Cellar Deck Height (to bottom of deck steel): 44 feet above MLW (as per drawings)
- Main Piles: 8 main piles, 30-inch diameter to penetration of 182 feet (ungrouted).
- Skirt Piles: 10 skirt piles, 30-inch diameter to penetration of 182 feet (grouted).

- Conductors (original design): Nine 20-inch diameter to penetration of 182 feet and one 48-inch diameter (48-inch diameter to 80 ft penetration installed over a pre-drilled 30-inch conductor).

- Jacket joints: Procedures used for design of the jacket joints are not known. Heavier cans (34-1/2"OD x 0.750" WT) were used for most jacket leg joints. The nominal size of the jacket legs was 34"OD x 0.500" WT. In addition, all jacket leg joints were detailed with 3/4" plate gussets. The material used for the jacket leg joint cans is assumed to be ASTM A-36.

- Appurtenances (original design - based on drawings):
  - Fendering system consisting of two 30-ft bumper panels on west side that functioned as barge bumpers and boat landing
  - Four risers (8-5/8" to 10-3/4" OD)
  - Three 6-5/8" OD downcomers on east side
  - Two stairs to Elev.(+)10'-0"

- Corrosion Protection: Cathodic protection system using zinc anodes

c) <u>Construction</u>

- Fabrication Contractor: McDermott Fabricators

- Installation Contractor: Williams-McWilliams

- Date of Installation: July, 1965

- Construction Drawings: A near complete set of construction drawings were available for the deck, jacket, piles, and conductors. All main and secondary structural members were shown on the available drawings. Some "as-built" information was shown. Sketches showing the structural framing, member sizes, and dimensions for the jacket, piles, and deck are presented in Appendix A.

- Fabrication, Welding, and Construction Specifications: Unavailable (assumed consistent with contemporary practices)

- Pile Driving Records: Portions of the pile installation records were available (approx. 20%). Piles were installed using a jet/drive procedure.

- Jacket Installation: Launched

d) Platform History

- Environmental Loading History: This platform has experienced two significant hurricanes without any known structural damage:
  1) Hurricane Betsy (1965) - hindcast wave height of 62 feet at this site based on GUMSHOE study
  2) Hurricane Andrew (1992) - hindcast wave height of 53.3 feet at this site based on an Oceanweather study using the GUMSHOE hindcast model

- Operational Loading History: This platform has supported a conventional platform drilling rig for drilling the initial wells. These drilling loads occurred when the cellar deck facilities loads were greater than the current loading. The primary difference between the initial facilities loads and the current loads is replacement of two 500-bbl storage tanks with two 90-bbl tanks. Based on review of the platform file records and inspection surveys, this platform has not experienced any boat collisions or other significant accidental loadings.

- Repairs: From review of the platform file records and the underwater and topside inspections, no evidence was found to indicate that this platform has experienced any significant structural damage which required repairs.

- Modifications from Original Construction Drawings:
  1) The original fendering system consisting of two 30-ft landings on the west side was replaced with one 30-ft boat landing, two 30" OD barge bumpers, and a 24" OD vertical fender member
  2) An area of approximately 10 ft x 15 ft was removed from the NW corner of the main deck framing plan
  3) The main deck chord and the 20" OD diagonal knee-brace to the chord were removed on deck truss Row D, outboard on west side
  4) A grated pig trap landing (approximately 8 ft x 15 ft) was added to the north end of the cellar deck
  5) Framing was added to the south end of the deck for support of escape capsules

e) Present Condition

- Deck Levels - Present Size and Elevation
  - Main deck: nominal dimensions of 66 ft x 114 ft. (approx. 7348 ft^2); Main deck top of steel at Elev.(+)65'-0 7/16".
  - Cellar deck: nominal dimensions of 40 ft x 98 ft with an extension of 11 ft x 26 ft on the west side and an extension of 15 ft x 56 ft on the east side (approx. 5046 ft^2) Cellar deck top of steel at Elev.(+)47'-6 1/16".
  - Grated subcellar deck in well bay for support of a sump tank: nominal dimensions of 15 ft x 32'-8" (approx 490 ft^2) ; Subcellar deck top of steel at Elev.(+)40'-0 1/16".

- Existing Loading and Equipment Arrangement: The deck currently supports minimum production facilities, since the production flows by pipeline to a host platform for treating. The current gravity loading on the platform is significantly less than the design live loading. Figures 4 and 5 show the current main and cellar deck equipment layouts.

- Field Measured Deck Clearance Elevation: The deck height was measured during the topside inspection on January 24, 1994. The measurements showed that the height of the bottom of steel of the lowest cellar deck truss chords is 44 feet above water line. The height of the bottom of steel of the 15 ft x 32.7 ft subcellar deck is 39.3 feet above water line. These measured deck heights agree with the specified drawing dimensions from the initial installation.

- Production and Storage Inventory: Production from the platform currently flows through a 10-3/4" OD pipeline to a host platform in the field for treating. No production is stored on the platform. The only hydrocarbon storage on the deck is a 90 bbl (approx) tank for diesel storage.

- Appurtenances (present condition - refer to Figure 3):
  - One 30-ft boat landing on west side
  - Two 30" OD barge bumpers on west side
  - One 24" OD vertical "fender" member on west side
  - One 10-3/4" OD riser on north side
  - One 4" OD riser on east side
  - Two 6-5/8" OD downcomers to Elev.(+)10 ft on east side
  - One 10-3/4" OD downcomer to Elev.(+)10 ft on east side
  - Two stairs to Elev.(+)10'-0"

- Existing Well Conductors: The platform currently supports eleven well conductors (ten 20" OD and one 48" OD). Two of the 20" OD conductors and the 48" OD conductor have been plugged and abandoned (conductors are still in place).

- Recent Above Water Survey (Level I): A Level I survey of the topside was conducted on January 24, 1994. The scope and results of this survey are discussed under Section A.2, "Condition Assessment".

- Recent Underwater Platform Survey: An underwater inspection of this platform was performed in August, 1991. The scope and results of this survey are discussed under Section A.2, "Condition Assessment".

PART A: PLATFORM ASSESSMENT

A.1 Platform Selection (Platform Assessment Initiators)

This platform was selected for assessment due to the proposed addition of up to eight new 20-inch diameter well conductors. Addition of these conductors will result in increased environmental loading on the structure (assessment initiator). Therefore, the conditions of Section 17.2.3 require that the platform undergo the assessment process. The assessment process will determine if the increased wave loading from the conductor additions meets the definition of "significant" as stated in Section 17.2.6.

A.2 Condition Assessment

a) **Topside Survey.** A Level I survey of the topside was conducted on January 24, 1994. This survey focused on 1) reviewing the general condition of the deck and top of jacket, 2) inspection for any corrosion and/or mechanical damage to structural members, 3) confirming that the deck and top of jacket are in accordance with the structural drawings and documenting any differences, 4) identifying the type, size, and location of all platform appurtenances, and 5) documenting the current facilities layout. The overall conclusion from this survey is that the above-water areas of this platform have been adequately maintained and are generally in good condition. Much of the top of jacket and the deck had been recently re-painted. Except for two tubulars and several small floor beams in the deck, no significant corrosion pitting was identified and all members were in place and intact. No bent or dented members were identified. Severe localized corrosion (small area) was noted on a 12" OD and a 14" OD tubular in one of the deck trusses (these areas will be repaired). Several small 6" and 10" deck floor beams in the well bay had significant localized corrosion in the flanges (these will be repaired). In performing the topside survey, several deviations were noted between the existing structure and the original construction drawings. These deviations or modifications are listed under "Platform History" in the "Platform Information" section.

b) **Underwater Survey.** An underwater inspection of this platform was performed by divers in August, 1991. This inspection included 1) visual inspection of members and joints (Level II for overall jacket and Level III for selected joints), 2) flooded member detection at framing Elev.(-)26 ft. and above, and 3) magnetic particle inspection of 5 nodes (Level IV for selected joints). The inspection concluded that the underwater portion of this platform was in good condition and no damage or deterioration of platform members or joints was noted. Recent cathodic potential surveys show that the underwater portions of this platform are adequately protected against corrosion.

c) **Damage/Repairs.** From review of the platform file records and the underwater and topside inspections, no evidence was found to indicate that this platform has experienced any significant structural damage which required repairs.

d) **Soil Data.** The soil boring report used in the original platform design was not available. There is a possibility that this platform design was based on a soil boring taken for a previous platform design in the field (approx. 3.5 miles away). The soil data used for the present assessment was based on a boring taken at the site in 1988 for placement of a jack-up drilling rig. The "jack-up rig" boring, taken to a penetration of 140 feet, showed clay from the mudline to 92-ft penetration and sand from 92 to 140-ft penetration (sand characteristics were not defined in the boring report). Since the penetration of the platform piles is 182 feet, sand characteristics to 140 feet and soil type and characteristics between 140 feet and 182 feet penetration were inferred from other borings in the area and from portions of the pile installation records. The assumed soil profile used for this platform assessment is shown in Figure 6.

A.3 Categorization

For the assessment process, this platform has been categorized as "Insignificant environmental impact/Unmanned" or "Minimum Consequence". Reasons for this categorization are as follows:

- Platform is operated unmanned (this platform is a satellite well structure to a host platform in the field).
- All wells on the platform are equipped with subsurface safety valves which are periodically tested.
- There are no major pipelines to or from the platform (only a small in-field 10-3/4" OD pipeline to the host platform).
- There is no significant hydrocarbon storage on the platform deck.

A.4 Design Basis Check

The "Design Basis Check" does not apply to this platform, since it was not designed to the 9th Edition of API-RP2A (1977), or later (refer to Sections 17.5.2 and 17.6.2a.3 of Draft Section 17 of API-RP2A). This platform was installed in 1965.

A.5 Analysis Checks - Metocean Loading

A.5.1 Deck Height Check

As stated, this platform in 160 ft. water depth in the Gulf of Mexico is categorized as "Insignificant environmental impact/Unmanned" or "Minimum Consequence". The deck height criteria based on sudden hurricanes therefore applies (Figure 17.6.2-3b of draft Section 17). From Figure 17.6.2-3b, this platform should have a deck height of at least 36.4 feet above MLLW. Based on measurements from a recent topside survey of this platform, the height of the bottom of steel of the lowest cellar deck truss chord is 44 feet above water line. The height of the bottom of steel of the 15 ft x 32.7 ft subcellar deck is 39.3 ft above water line. Therefore, this platform passes the deck height check.

A.5.2 Section 17 Metocean Loading Criteria

Since this platform is categorized as "Insignificant environmental impact/Unmanned" or "Minimum Consequence", the metocean loading criteria is in accordance with Section 17.6.2a.4c. Here, the winter storm population applies and the wave height criteria are omni-directional. The metocean criteria for the Design Level Analysis and the Ultimate Strength Analysis were taken from Table 17.6.2-1, Figure 17.6.2-5a, and Figure 17.6.2-5b in the draft Section 17. The criteria obtained from Section 17 for these analyses are presented in Table 1.

A.5.3 Design Level Analysis

a) Significance of Increased Loading

The first step of the Design Level Analysis was calculation of wave (metocean) loading on the platform. The wave load calculations considered both the current or "as-is" condition and the condition with addition of eight new 20" OD well conductors. Results from the wave load calculations showed that addition of the new conductors increases the overall wave load on the platform by approximately 8 percent. Therefore, this loading increase is not defined as significant (refer to Section 17.2.6) and the platform passes assessment according to the alternative procedure defined in Section 17.5.2.3.

b) Structural Steel Design (Member Strength)

The Design Level Analysis was performed in accordance with the procedures outlined in Sections 17.5.2.3 and 17.7.2. The in-place analytical model of the platform included all primary structural members in the jacket and deck, the initial well conductors, wave load modeling of the existing jacket appurtenances, and the wave load effects of eight new well conductors. The foundation model considered nonlinear pile-soil-structure interaction using the soil profile shown in Figure 6. Gravity loads applied to the structure were as shown in Table 2. Wave, current, and wind loads were applied to the structure from eight equally spaced approach directions using the Design Level Analysis criteria presented in Table 1. The maximum base shears and overturning moments on the platform due to wave, wind, and current are shown in Table 3 for the end-on, broadside, and diagonal approach directions.

Results of the member stress analysis showed that all structural members are adequate for the Design Level Analysis metocean loading (all stress unity checks were less than

1.0). Three members, shown in Figure 7, had unity check greater than 0.85. This platform, therefore, passes the Design Level Analysis for member strength.

c) Connections (Joint Strength)

The strength of the non-gusseted jacket joints was analyzed in accordance with Section 4 of API RP 2A. The gusseted joints were analyzed using an in-house procedure that accounts for the presence of the gussets. ASTM A-36 material was assumed for the jacket joints. Joint strength unity checks were calculated based on the actual loads derived from the global analysis of the platform for the Design Level metocean loading. Results showed that a number of the jacket joints were stressed beyond allowables (joint strength unity checks greater than 1.0). The platform, therefore, fails the assessment based on inadequate jacket joint strength. The joints with strength unity checks greater than 1.0 are shown in Figures 8 through 11.

Although a number of jacket joints failed the Design Level Analysis strength check, the acceptability of the joints could be demonstrated through documentation of prior hurricane exposure along with Level III/IV underwater surveys of the critical joints. This is discussed under paragraph f below.

d) Fatigue

In accordance with Section 17.1.2d, a new detailed fatigue analysis of the platform was not performed. A prior underwater inspection of this platform did not show that fatigue damage on any of the jacket joints has occurred. A new Level III/IV survey will be conducted on selected jacket joints prior to installation of the new well conductors. Any fatigue damage identified in this new survey will be assessed and/or repaired. In selecting joints for inspection in the new underwater survey, the fatigue-sensitivity of the jacket joints were prioritized based on 1) joint stress level from the joint strength analysis and 2) calculated joint stress concentration factors.

e) Foundation Capacity

Pile axial loads from the eight storm directions analyzed, as well as from a gravity load only case, were compared to the pile capacity in both tension and compression. Pile capacities were based on the soil profile shown in Figure 6 with full end-bearing in sand assumed. On this basis, all piles have acceptable safety factors in accordance with Section 6.3.4 of API RP 2A.

However, there is some uncertainty in the pile axial capacities because of the unconfirmed soil properties and because of the jet and drive procedure used for installation of the piles in 1965. While some "healing" of the disturbed soil is postulated to occur with time, the amount of degradation and subsequent healing is difficult to quantify. Therefore, conclusive verification of foundation strength will be based on prior exposure of the platform to severe storms (refer to next section).

f) Prior Exposure

This platform has experienced two significant hurricanes without any known structural damage, Hurricane Betsy in 1965 and Hurricane Andrew in 1992. Using the GUMSHOE model, the hindcast maximum wave heights at this platform site were 62 ft. and 53.3 ft for Betsy and Andrew, respectively. Since these wave heights exceed the "Ultimate Strength" wave height of 46.5 ft for this "Minimum Consequence" platform in a water depth of 160 ft, assessment based on prior exposure could be considered.

Hurricane Andrew experience was used to verify the acceptability of the platform joints and the foundation for one direction of loading (i.e. the hindcast Andrew wave direction at this site). Analyses of joint strength and foundation capacity were then performed to demonstrate the acceptability of the platform for other loading directions. Actual joint capacities back-calculated from the Andrew loading were compared to joint loads for the Section 17 Ultimate Strength analysis conditions for other approach directions and were found to exceed them, thus demonstrating acceptable joint performance. Note that this approach is contingent upon a joint inspection to confirm acceptable joint behavior during Andrew, which will be performed in conjunction with the survey for fatigue damage described above.

Ultimate strength analyses of the foundation's global capacity were used in a similar fashion to determine the soil strength demonstrated by the Andrew loading. Minimum global foundation capacities for other approach directions were then calculated which account for the unsymmetric foundation arrangement. These calculated capacities compare favorably with platform loads per Section 17 Ultimate Strength criteria, demonstrating the acceptability of the foundation.

A.5.4 Ultimate Strength Analysis

a) Analysis Procedure

A global inelastic static push-over analysis was performed to assess the ultimate strength of the platform. This analysis was performed in accordance with the general guidelines in Section 17.7.3. The computer program KARMA (INTRA) was employed in the push-over analysis. KARMA is a three-dimensional, inelastic, nonlinear, static and dynamic finite element analysis program for offshore structures subjected to environmental loads.

Modeling techniques used successfully in the past were adopted in this analysis. Specifically, the following idealizations were employed:

Deck elements. Linear beam elements (LBEM) were used to model the deck. The LBEM element is a linear elastic beam-column element capable of resisting axial, flexural, and twist forces. While the material response is linear, geometric nonlinearities were incorporated to account for P-delta effects. These effects were accounted for in all the elements considered in the analysis. Since the deck was not impacted by the environmental loads considered in this analysis (adequate deck height), it was considered that the deck material response would likely be linear. Thus, the LBEM elements were employed to save computational time.

Braces in N-S jacket elevations. Marshall B-strut elements (STRT) were used to model the braces in the N-S (longitudinal) jacket elevations. The STRT element is a phenomenological post-buckling element which can be arbitrarily oriented in space. It is capable of resisting axial loads only. Sherman's equation was used to define the buckling response. The behavior of these braces show the greatest tendency toward solely axial response and these braces are likely to fail by axial buckling, if at all. Thus, STRT elements were employed.

Braces in E-W jacket elevations. Nonlinear beam elements (BEMC) were used to model the braces in the E-W (transverse) jacket elevations. The BEMC element is an inelastic phenomenological beam-column element in which the nonlinear material behavior is characterized by trilinear axial force-displacement, moment-rotation, and torque-twist curves. An elliptical surface was chosen to govern the interaction between axial force, moment, and torque. Plastic hinges are assumed to form at the element ends. These braces experience significant moments and are likely to fail under combined bending and axial force. The axial capacity and response is governed by Sherman's equation rather than cross-sectional yield, by the post-buckling shedding of load is not modeled.

Pile-leg connections. Compatibility between the main piles and the jacket legs was provided by shear transfer (SHER) elements. These elements transfer shear loads only between the pile and the leg while allowing axial slippage.

Piles, legs, and all other members. Nonlinear beam elements (BEMC) were used.

Foundation modeling. Self-aligning near field elements (SANE) were used to model the foundation. SANE elements can either model lateral and axial soil-pile interaction or pile tip bearing and suction response. The former is the P-Y/T-Z mode and the latter is the tip mode. The P-Y/T-Z mode consists of three orthogonal nonlinear elastic non-hysteretic springs. The tip mode consists of one such spring. The P-Y/T-Z springs are symmetric with positive and negative deformation while the tip springs are not.

Mean values of material yield stress were assumed for the BEMC and STRT elements. Best estimates of the cyclic soil-pile foundation behavior were also adopted.

Joint flexibility and nonlinear behavior was not modeled in this analysis. This is in keeping with traditional practice and greatly reduces the analysis effort. In assessing platform reserve strength, traditional practice usually assumes the joints were designed to API RP 2A requirements and, for an overload condition, failure of the jacket braces will occur prior to joint failure. However, for this older platform (1965), linear analysis results showed that a number of joints were overstressed, indicating that some joints may fail prior to member failure. In lieu of more detailed nonlinear analyses, an evaluation of the effect of joint capacity on platform performance for acceptance to Section 17 criteria will be based on prior exposure to severe storms.

b) Analytical Results

Reserve Strength Ratio (RSR) Based on API RP 2A-WSD, 20th Edition Loading

Figures 14 through 22 present the load-deflection curves and displaced shapes at failure for the 9 wave directions analyzed (the wave approach directions and the API 20th Ed. wave criteria are presented in Figure 12 and Table 1). The load-deflection curves are expressed in terms of the component of base shear in the direction of the wave and the component of deck displacement at node 314 in the direction of the wave. The location of node 314 is defined in Figure 13 and is centrally located on the deck. Figure 13 depicts the KARMA model analyzed. Tables 4 through 12 provide additional information regarding the push-over analysis results for the various wave approach directions.

For all wave directions, the failure mode is characterized by nonlinear soil response followed by double-hinge pile failure. Since KARMA has no procedure for navigating a peak and convergence is difficult near the peak, the analyses were stopped at incipient failure. At this point, most if not all of the skirt piles had failed, along with 4 to 7 of the main piles. The remaining main piles were near failure and an uncontrolled mechanism would follow their failure.

For many of the wave directions, a number of horizontal members became nonlinear early, but their behavior did not influence the platform failure mode. Likewise, near incipient failure, some of the braces in the E-W jacket elevations exhibited nonlinearity but did not noticeably influence the failure.

The capacity of the platform in all directions is within the range of 5100 to 5600 kips of base shear. Thus, the capacity is relatively independent of wave direction. This may be attributable to the pile-foundation failure mode which is not overly influenced by asymmetries in the platform. Because of the relatively constant capacity with wave direction, the RSR is primarily determined by the amount of wave loading in a given direction.

The lowest RSR is 0.94 in the 90 degree wave direction, which is the direction with the greatest base shear load. The RSR is also less than 1.0 for the 290 degree wave, while the 225 and 315 degree waves produced RSRs near 1.0. The RSRs were greater than 1.0 for the remaining wave approach directions.

Ultimate Strength According to API RP 2A Section 17 Loading Criteria

The maximum base shear on the platform calculated with the Section 17 Ultimate Strength metocean criteria (Table 1) was 3150 kips. Platform base shear capacities demonstrated by the KARMA nonlinear push-over analyses are considerably greater than this loading (5100 kips minimum). Therefore, the KARMA analyses confirm the acceptability of the members and the foundation (with undegraded soil strength) previously demonstrated by the Section 17 Design Level Analysis. Acceptable joint capacities, as well as foundation capacity with sensitivity to the soil parameters considered, are as demonstrated by prior hurricane exposure as noted in paragraph A.5.3(f) above. The platform, therefore, passes the Section 17 Ultimate Strength Analysis.

A.6 Consideration of Mitigations

The platform passes assessment based on the alternative design level analysis discussed in Section 17.5.2.3 (the increased loading on the platform was not significant according to the definition in Section 17.2.6). However, review of the platform information (Condition Assessment) and the Design Level Analysis identified two concerns regarding the platform structural adequacy for resisting extreme environmental loads. First, analytical results show that the allowable loadings in a number of jacket joints are less than the Design Level Analysis loading. Secondly, the ultimate axial capacity of the piles is unknown due to the unconfirmed soil profile and the jet/drive procedure employed for pile installation. Due to these concerns, several mitigation measures will be specified for this platform to reduce the effects of the proposed conductor additions and to ensure adequacy of the jacket joints:

1) **Wave Loading.** Three of the existing well conductors have been plugged and abandoned (two 20" OD and one 48" OD). To reduce the effects of adding eight new 20" OD conductors along with the 20" OD conductor added in the 1980's, the three plugged and abandoned well conductors will be removed. This results in a net addition of approximately five new conductors when considering additional wave load on the platform beyond the original design basis loading (i.e. an increase of approximately 5% in overall wave loading on the platform). Consideration will be given to local load effects in locating the new conductors inside the jacket framing.

2) **Gravity Loading.** Due to the uncertainties regarding axial pile capacity, it will be specified that a conventional drilling rig will not be placed on the platform deck in the future. All future drilling operations at this platform will be performed using a jack-up drilling rig.

3) **Joint Inspections.** Underwater inspection of selected jacket joints will be performed to ensure joint integrity prior to installation of the eight new conductors. Selection of the joints for inspection will be based on 1) the magnitude of overstressing identified in the joint strength analyses, 2) fatigue sensitivity, and 3) criticality to overall platform integrity. The joint inspections will consist of a Level III survey in accordance with API-RP2A with several joints subjected to a Level IV survey (i.e. NDT). All future underwater inspections of this platform will include Level III/IV surveys of selected joints.

A.7 Summary Note - Part A

This trial application of the draft API RP 2A-WSD Section 17 procedure for assessment of existing platforms covers the assessment of an 8-leg platform installed in 1965 in a water depth of 160 feet in the Gulf of Mexico. This platform is currently operated unmanned and functions as a satellite well structure to a host platform in the field. The following summarizes the results/findings for each component of the assessment process:

SCREENING. The platform did not pass screening, since 1) it did not pass the Design Basis Check (Design Basis Check was not applicable) and 2) an assessment initiator existed. The assessment process, therefore, required at least a Design Level Analysis.

Platform Selection. This platform was selected for assessment due to the proposed addition of eight new well conductors (i.e. the assessment initiator was the increased environmental loading on the platform due to the conductor additions).

Categorization. This platform was categorized as "Insignificant environmental impact/Unmanned" or "Minimum Consequence".

Condition Assessment. Recent topside and underwater surveys of the platform showed that the platform was generally in good condition and had been adequately maintained. Construction drawings of the platform were available; however, the original design criteria and documentation were unavailable. Some assumptions were made regarding soil data since the original soil boring report for the site was unavailable (i.e. some uncertainties regarding ultimate pile capacity).

Design Basis Check. The Design Basis Check did not apply to this platform, since it was designed prior to the 9th Edition of API RP 2A.

ANALYSIS CHECKS - METOCEAN LOADING

Deck Height Check. The platform passed the deck height check for the defined exposure category of "Minimum Consequence". Therefore, an Ultimate Strength Analysis is not required if the platform passes the Design Level Analysis.

Design Level Analysis.

The first step of the Design Level Analysis was calculation of wave load on the platform in its current condition and with the addition of the eight new conductors. Results showed that the new conductors increased the wave loading on the platform by approximately 8 percent. This increase in loading is not considered to be significant according to Section 17.2.6; therefore, the platform passes assessment at this point and no further analytical work is required.

Continuation of the Design Level Analysis showed that platform member strength and foundation capacity (based on undegraded soil) passed assessment. However, the platform jacket failed the Design Level Analysis based on inadequate joint strength. Further analyses of prior hurricane exposure (hindcast wave heights at the site) in conjunction with underwater inspections demonstrated the acceptability of the jacket joints. These analyses of prior exposure also alleviated the uncertainties regarding pile capacity and demonstrated the adequacy of the foundation.

Ultimate Strength Analysis.

An inelastic static push-over analysis showed that the platform has a minimum reserve strength ratio of 0.94 based on the metocean criteria in API RP 2A-WSD, 20th Edition. The mode of failure at ultimate loading was shearing of the pile foundation. The minimum ultimate base shear loading was 5100 kips, while the Section 17 Ultimate Strength metocean loading was 3150 kips. The platform, therefore, passes the Section 17 Ultimate Strength Analysis.

MITIGATIONS

Although the platform passes assessment, concerns remain regarding the ultimate pile capacity and the adequacy of the jacket joints. Therefore, several mitigation measures will be taken to reduce the effects of the proposed conductor additions: 1) three plugged and abandoned well conductors will be removed prior to installation of the eight new conductors, 2) a conventional drilling rig will not be placed on the platform in the future (will utilize jack-up drilling), and 3) Level III/IV underwater surveys of selected jacket joints will be performed to ensure joint integrity.

PART B: REVIEW AND FEEDBACK TO THE API TG 92-5

The following questions and comments were developed during the trial application of the draft API RP 2A-WSD Section 17:

1. As seen in this trial application, a platform can pass assessment when the jacket joints would be shown to be inadequate in a Design Level Analysis. In this case, the platform passes assessment based on the definition of "significant increased loading" (refer to Sections 17.2.6 and 17.5.2.3). Wave load calculations, the first step in the Design Level Analysis, showed that the increased loading due to conductor additions to the platform was less than 10% or not significant. Therefore, the platform passes assessment at this point. However, if the Design Level Analysis is carried further, results would show that the strength of a number of jacket joints is inadequate and the platform would then fail assessment. Consideration should be given to adding text to address this inconsistency.

2. The assessment process flowchart (Figure 17.5.2) does not reflect a check to determine if platform damage or increased platform loading is significant according to Section 17.2.6. Some analytical work is necessary to determine if the damage or increased loading is significant. The analytical work may show the damage or increased loading to be insignificant and, if no other initiators exist, the platform passes assessment. This process for an alternative design level analysis is discussed in Section 17.5.2.3.

3. Sections 17.2.3 and 17.2.5 with Section 17.2.6 indirectly state that platform damage or increased loading would not be assessment initiators if the cumulative damage or cumulative changes from the design premise were not significant (i.e. less than 10% decrease in capacity or less than 10% increase in loading). It is assumed that the wording in these sections applies to all platforms, regardless of exposure category. However, wording in Sections 17.5.2.3 and 17.5.2.4 implies that the "not significant" definition only applies to "minimum consequence" platforms. This should be clarified.

4. Section 17.5.2.3 states that "an acceptable alternative to satisfying the design level analysis requirement is to demonstrate that the damage or increased loading is not significant relative to the as-built condition, as defined in Section 17.2.6. This would involve ***design level analysis of both the existing and as-built structures***." If a <u>full</u> design level analysis is required for both the existing and as-built structures, then what is the incentive for pursuing this alternative approach? A design level analysis of only the existing or current structure would determine is the structure passes assessment or not. If a design level analysis is performed on the existing structure, then it appears that the design level analysis results for the original or as-built structure would be irrelevant. It is possible that the author of this section was considering wave load increases as relates to the definition of "significant". Here, a design level <u>wave loading</u> analysis on the existing and as-built structures would determine if the loading increase due to platform changes was significant (a full design level <u>stress</u> analysis for both conditions is not

required if the loading increase is not significant). For clarity, wording in this section should be revised to better describe the intent of the alternative approach.

5. Comment 4 above regarding the alternative approach also applies to the ultimate strength analysis in Section 17.5.2.4. An ultimate strength analysis of only the existing or current structure would determine is the structure passes assessment or not. If an ultimate strength analysis is performed on the existing structure, then it appears that the ultimate strength analysis results for the original or as-built structure would be irrelevant. For clarity, wording in this section should be revised to better describe the intent of the alternative approach.

6. For clarity, it is recommended that the two sentences prior to Section 17.5.2.4 be revised to read as follows (note blank line after first sentence):

   "Significant damage or change in design premise is defined in Section 17.2.6.

   For platforms that have significant damage, have an inadequate deck height for their category (Ref. Figures 17.6.2-2b, 3b, 5b), and/or have experienced significant changes from their design premise, the following applies:"

7. From the wording under "Design Basis Check" in Section 17.5.2 and the wording in the heading for Section 17.6.2a.3, it appears that a platform can only pass assessment by Design Basis Check if it was designed to API RP 2A, 9th Edition (1977) or later. It is possible that a platform designed prior to 1977 could have been designed to a hydrodynamic loading that meets the reference level forces in the 9th Edition. Could this platform pass assessment by the Design Basis Check? This should be clarified. Further comment: It appears that the design basis check concerns only the magnitude of wave loading used for design of the platform. Are there any other design criteria or design procedure issues that should be addressed?

8. It is likely that many older structures with adequate deck heights could pass the Design Level Analysis for member strength and foundation capacity, but fail assessment based on inadequate jacket joint strength. Wording should be added to Section 17.7.2c to state that adequate joint strength can be demonstrated through Level III and/or Level IV inspection of critical joints in conjunction with documentation of prior hurricane exposure.

9. In Section 17.2.6, the 10% threshold for defining a "significant load increase" will likely be interpreted as a 10% increase in overall loading on the platform (i.e. the interpretation would be based on global loading with no consideration of local effects). Wording should be added to this section to state that additional loading of less than 10% should be considered significant if the additional loading induces failure of local elements that would, in turn, lead to overall failure of the platform.

10. In Section 17.5.3.4, the wording "The platforms have been surveyed" should be changed to "The platforms have been surveyed to at least Level II as defined in Section 14.3.2".

11. The last sentence under "Lateral Soil Resistance Modeling" in Section C17.7.3c.3.g implies that lateral pile displacements greater than 10% of the pile diameter should only be considered for ultimate capacity analysis. This further implies that lateral pile displacement in elastic design of foundations be limited to 10% of the pile diameter. The wording here may be contested by many platform designers, since this "10% rule" for lateral displacement in the design of pile foundations has not typically been followed. Consideration should be given to revising the wording in this section.

12. In the second sentence of the third paragraph of Section 17.6.2a.4a, change the words "of this recommended practice" to "from Section 2.3.4" (change in two places in the sentence). This change will add clarity to the sentence and avoid misinterpretations.

Miscellaneous comments (editorial changes, typographical errors, etc.):

13. In Section 17.2.6, change the word "and" to "and/or".

14. In Section 17.3.1c, insert the word "is" after the word "platform".

15. In Section 17.4.1, the title of the paper "An Integrated Approach for Underwater Survey and Damage Assessment of Offshore Platforms" should be italicized.

16. In the first paragraph of Section 17.5.2, change "environ- mental" to "environmental".

17. In the first paragraph of Section 17.5.3, use a capital "S" for the word "section" (i.e. Section 17.3).

18. The headings for Sections 17.5.3.4, 17.5.3.5, 17.5.3.6, 17.5.4.3, 17.5.4.4, and 17.5.4.5 should be in bold type, similar to the headings in Section 17.5.2.

19. In Section 17.5.3.4, delete the blank line after the first line of text.

20. In Section 17.6.1, use a capital "S" for the word "section" (i.e. Section 17.3).

21. In Section 17.6.2a.1, should the words "directional spreading" be replaced with the words "wave kinematics"?

22. In the last sentence of the third paragraph of Section 17.6.2a.4.b, should the words "directional spreading" be replaced with the words "wave kinematics"?

23. In Section 17.6.2b.1, should the words "directional spreading" be replaced with the words "wave kinematics"?

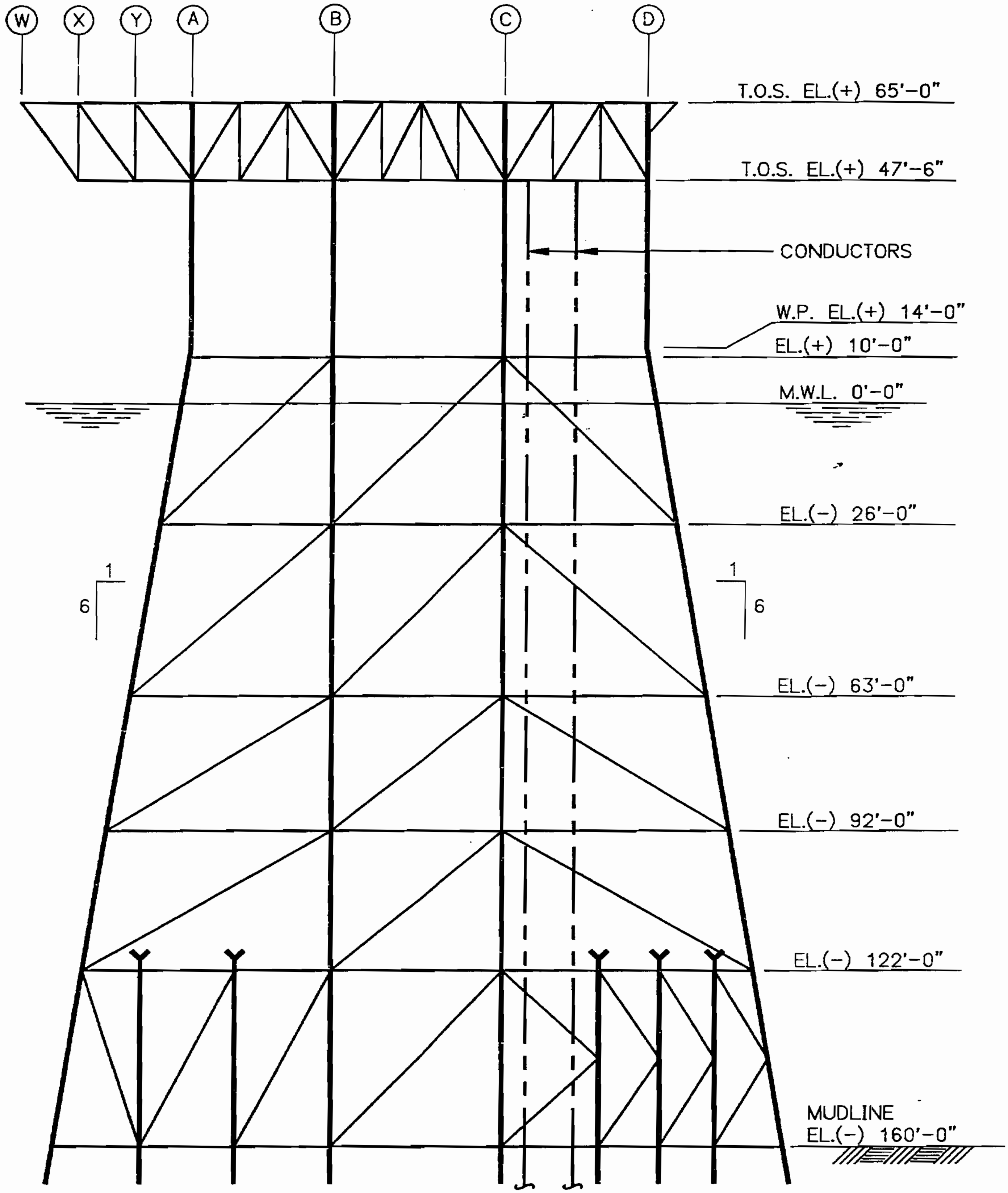
24. The word "actual" in the title of reference 5 under "REFERENCES" should be capitalized (i.e. Actual).

25. In Section C17.2.4, change the words "Platform installed in deeper water than design for" to "Platform installed in deeper water than the design depth".

26. Change the heading for Section C17.5.3 from "Assessment for Seismic Assessment" to "Assessment for Seismic Loading".

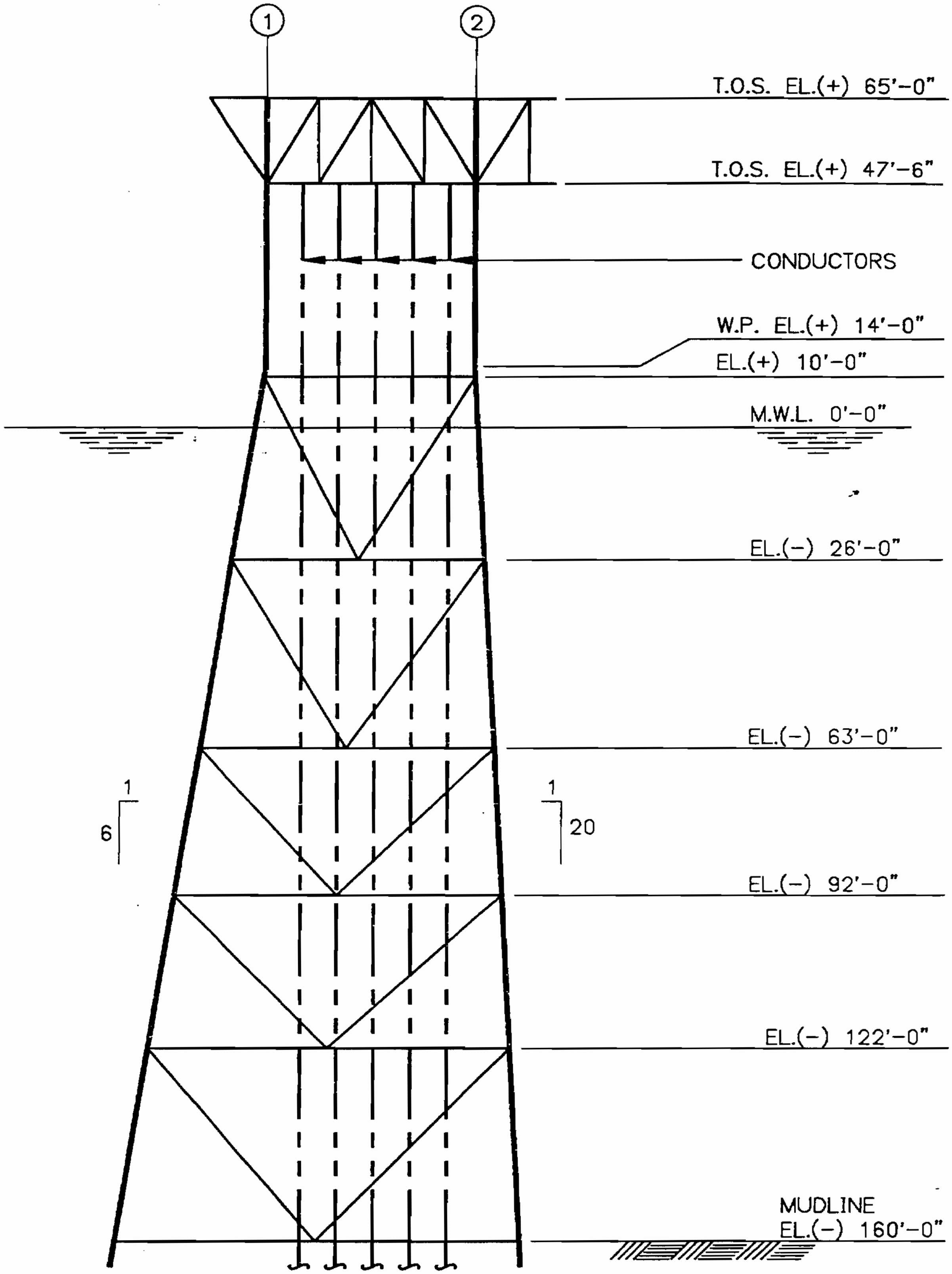
27. In Section C17.7.3c.3.d, change "load-defotmation" to "load-deformation".

Figures



EAST ELEVATION

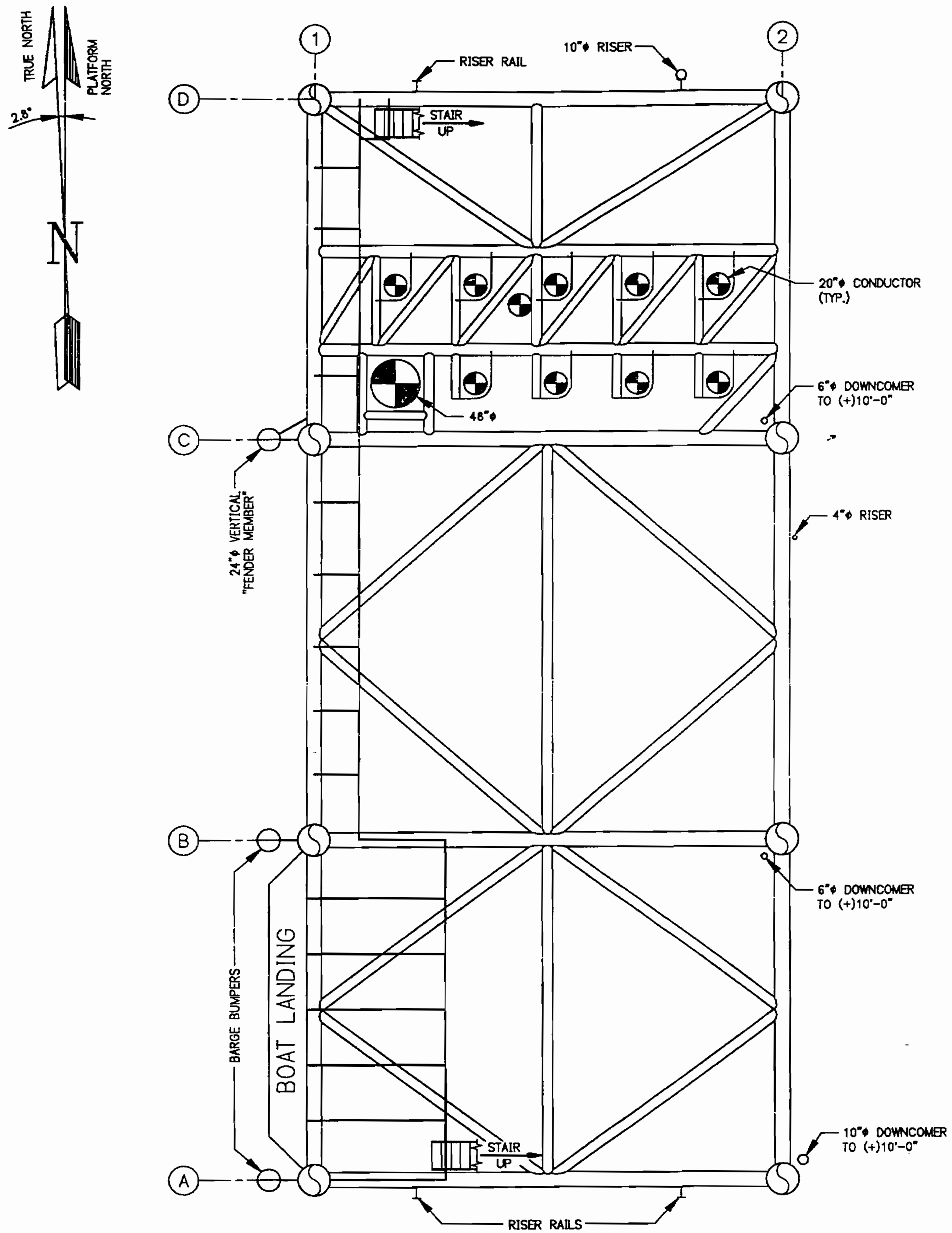
Figure 1



SOUTH ELEVATION

Figure 2

P\SOUTHEL 1=360 JLT 7/20/94

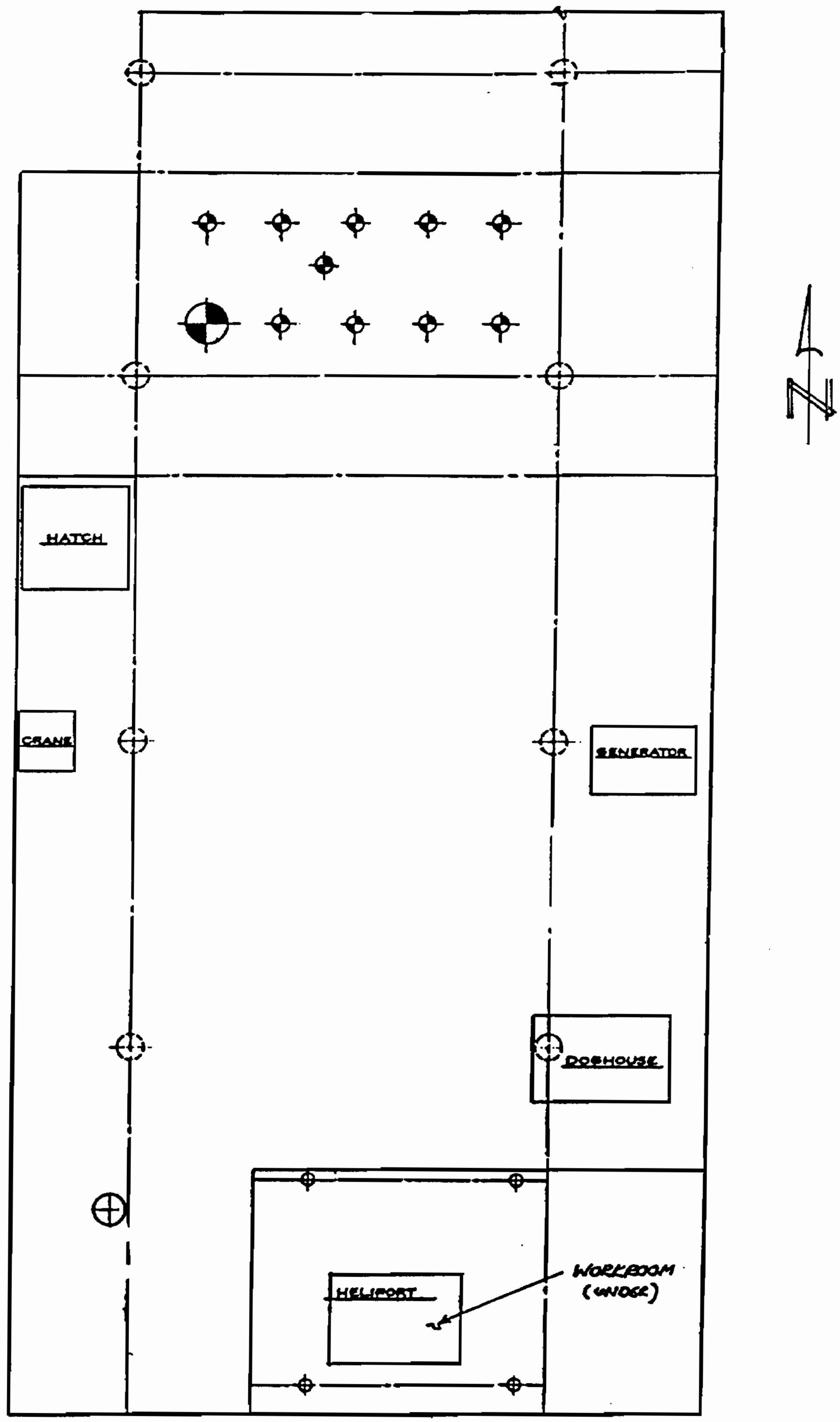


PLATFORM ORIENTATION/EXISTING APPURTENANCE LAYOUT

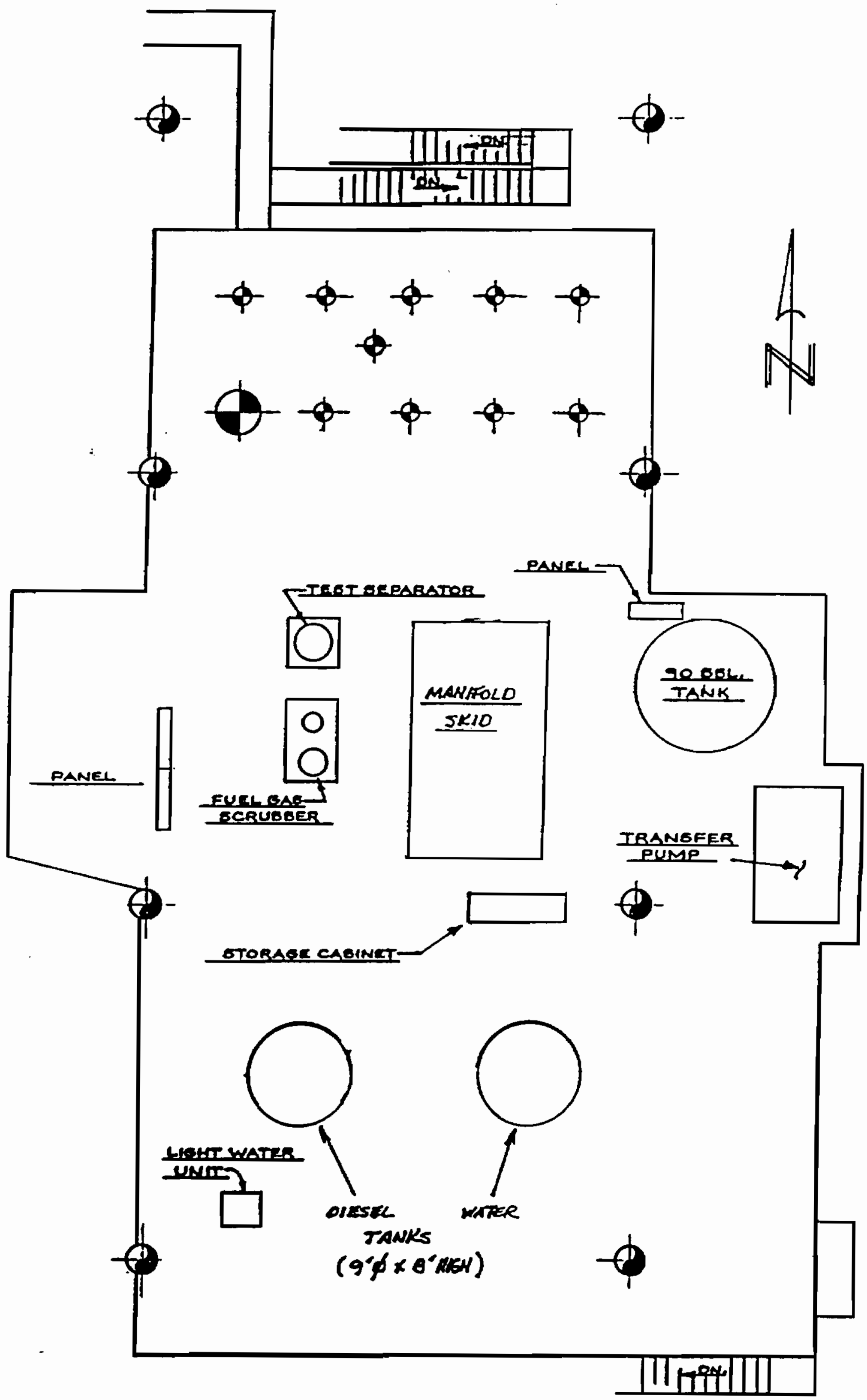
(1" = 12"-0")

Figure 3

\UP\F3 1=360 Rev 7/25/94

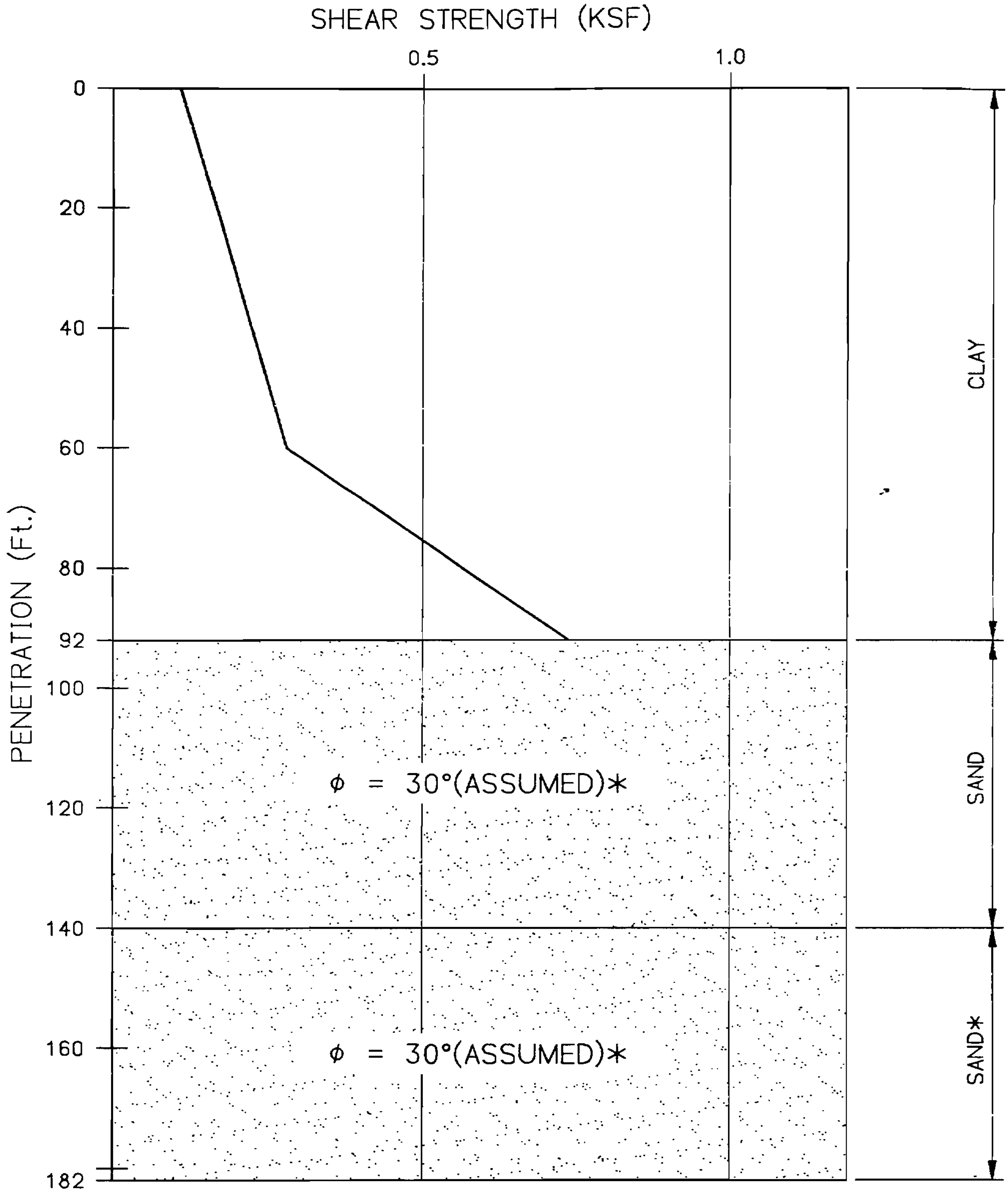


Main Deck Facilities Layout - Existing
Figure 4



<u>Cellar Deck Facilities Layout - Existing</u>

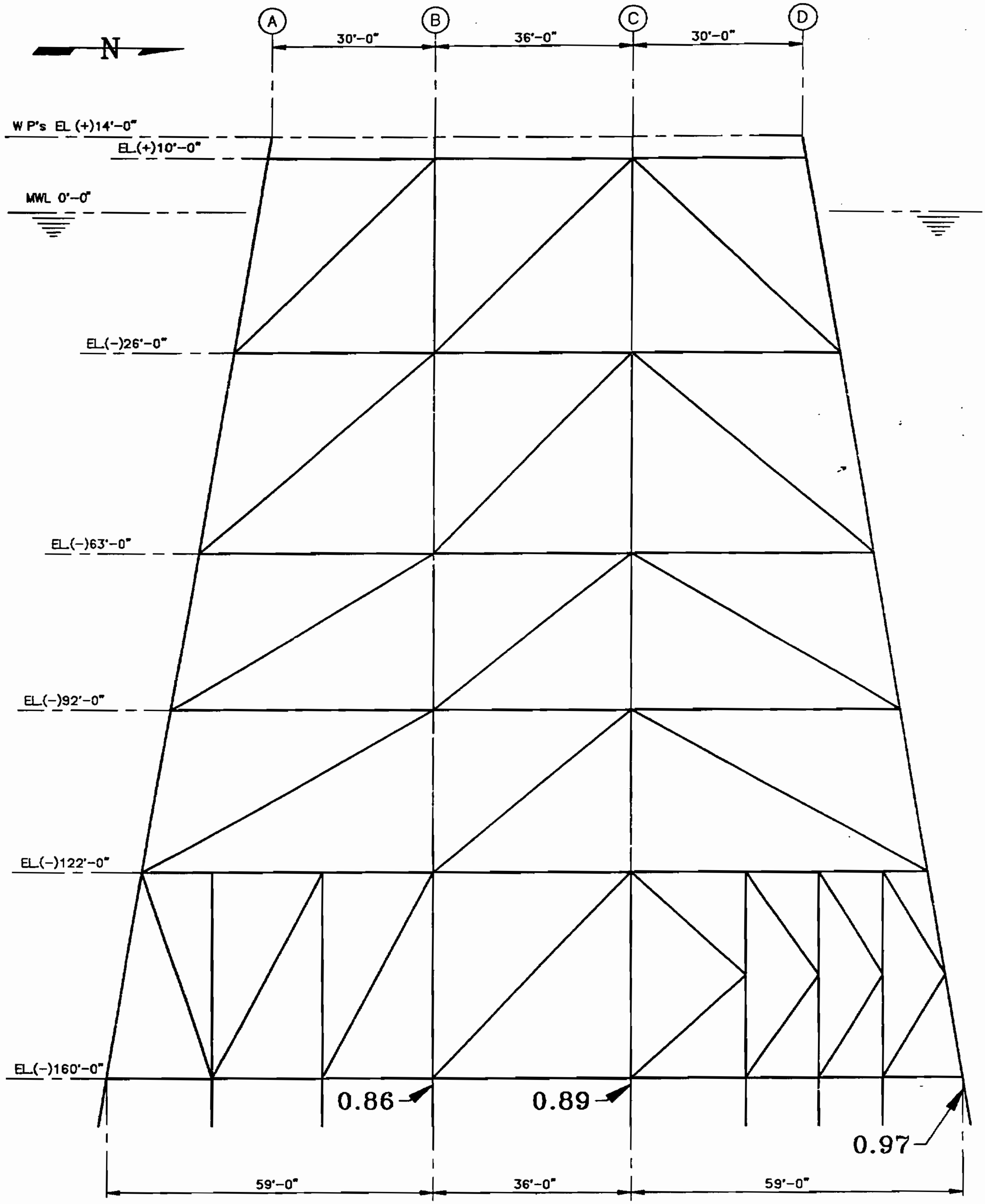
Figure 5



* – A SOIL BORING AT THIS SITE WAS TAKEN IN 1988 TO A PENETRATION OF 140 FT. SAND WAS CONFIRMED FROM 92 FT. TO 140 FT. PENETRATION. SAND CHARACTERISTICS TO 140 FT., AND SOIL TYPE AND CHARACTERISTICS FROM 140 FT. TO PILE PENETRATION OF 182 FT. WERE INFERRED FROM OTHER BORINGS IN THE AREA AND FROM THE PILE INSTALLATION RECORDS.

SOIL PROFILE

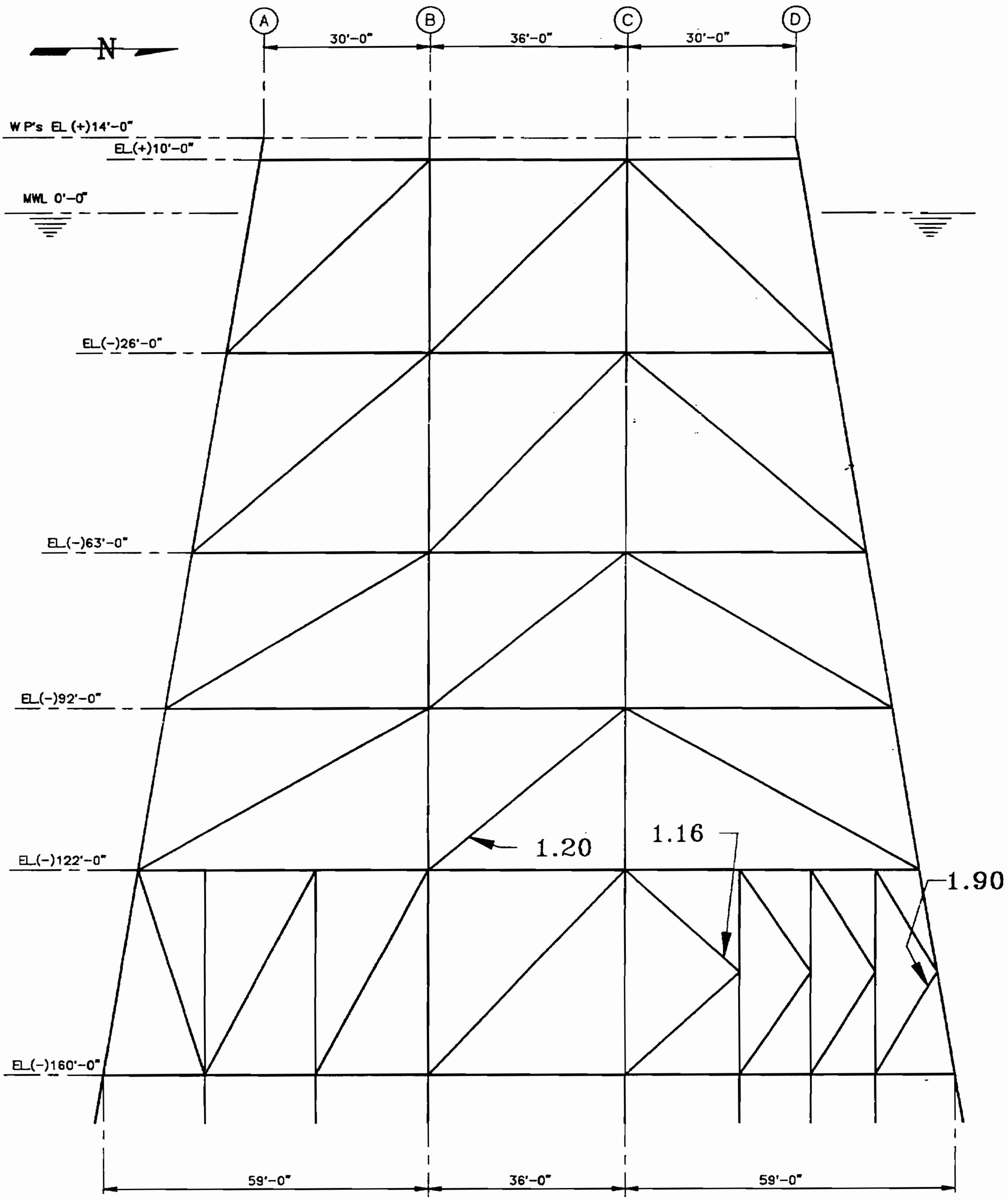
Figure 6



MEMBER STRENGTH UNITY CHECKS > 0.85
DESIGN LEVEL ANALYSIS

ELEVATION @ COLUMN ROW "2"
(1" =25'-0")

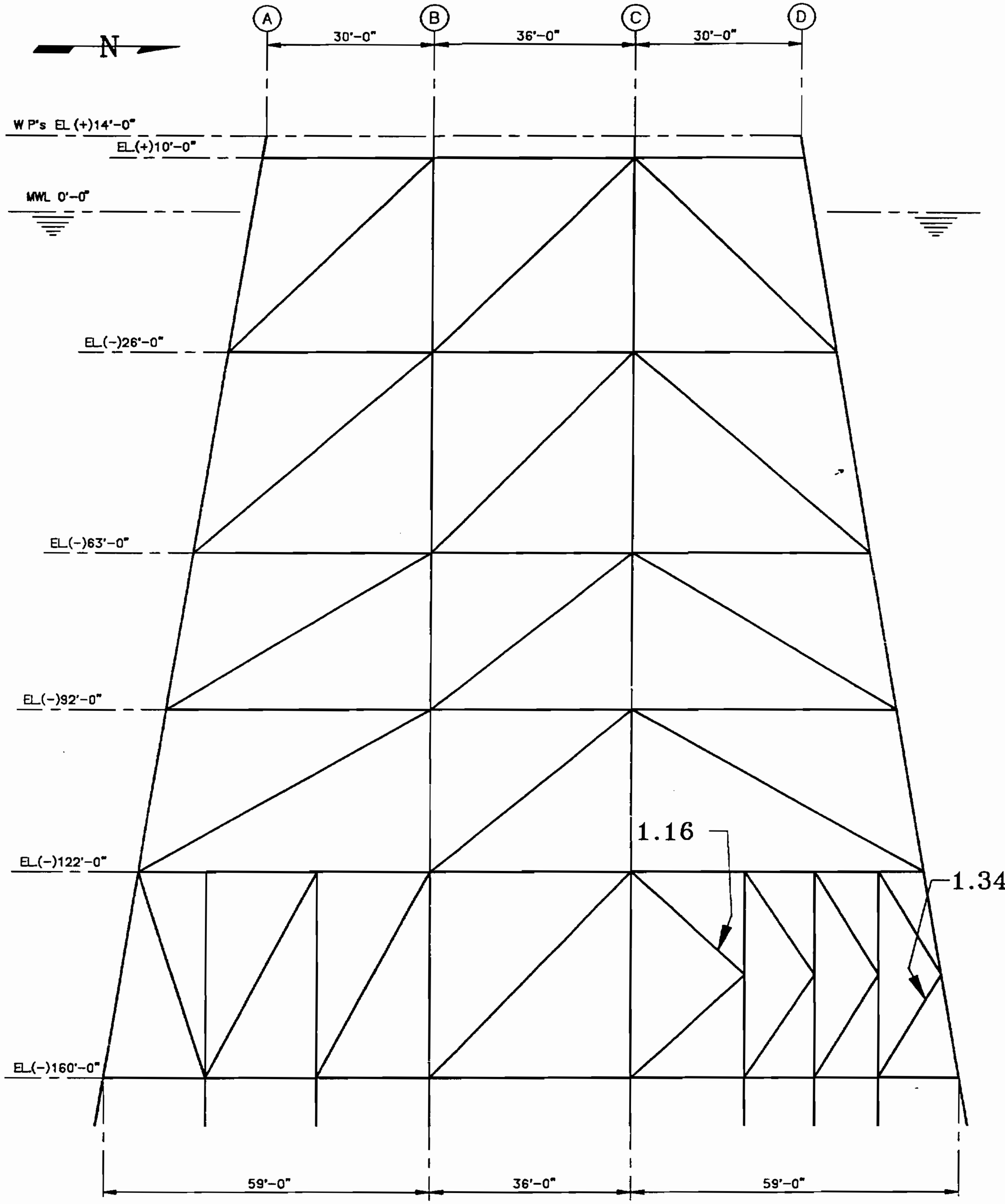
Figure 7



JOINT STRENGTH UNITY CHECKS > 1.0
DESIGN LEVEL ANALYSIS

ELEVATION @ COLUMN ROW "1"
(1" =25"-0")

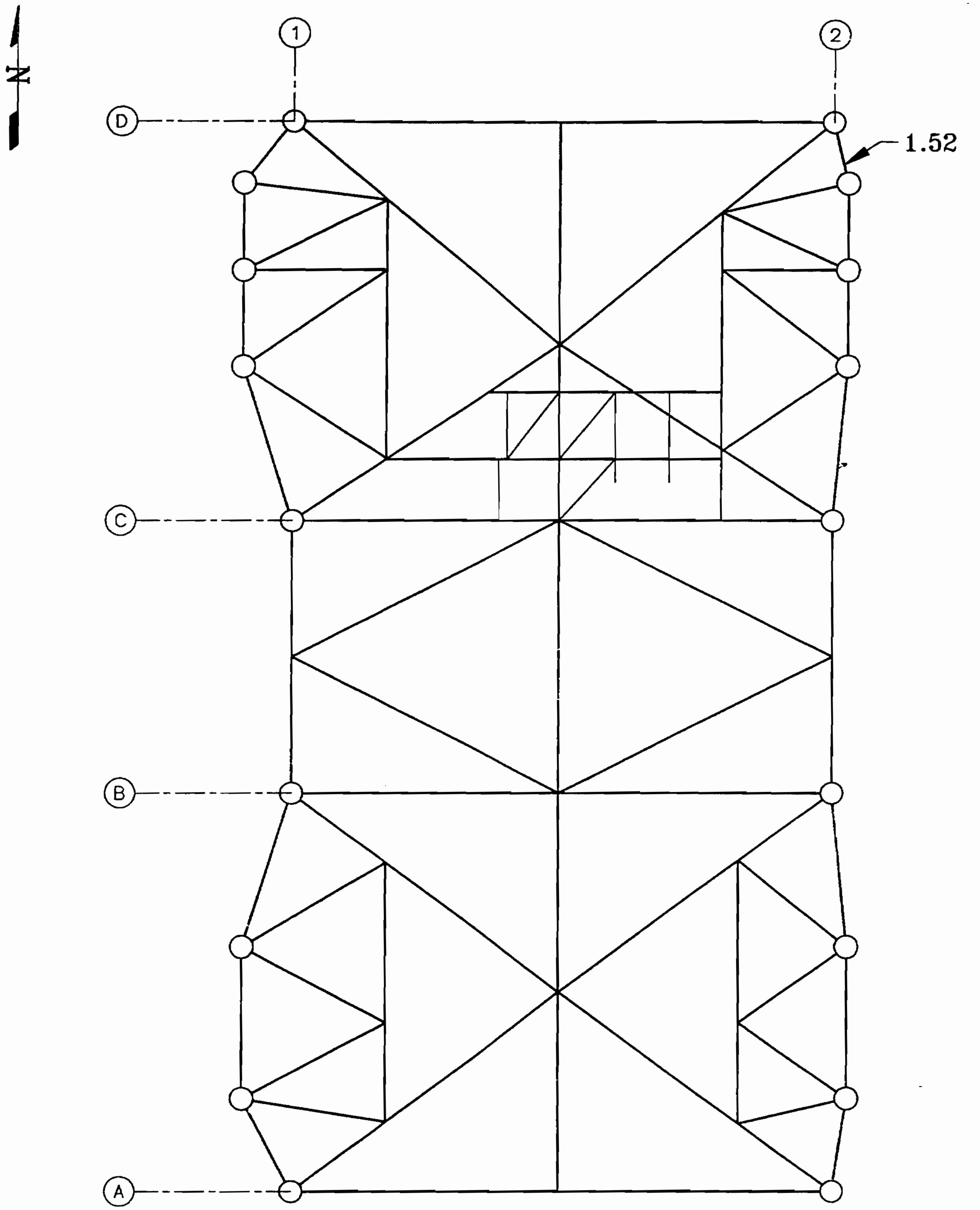
Figure 8



JOINT STRENGTH UNITY CHECKS > 1.0
DESIGN LEVEL ANALYSIS

ELEVATION @ COLUMN ROW "2"
(1" =25"-0")

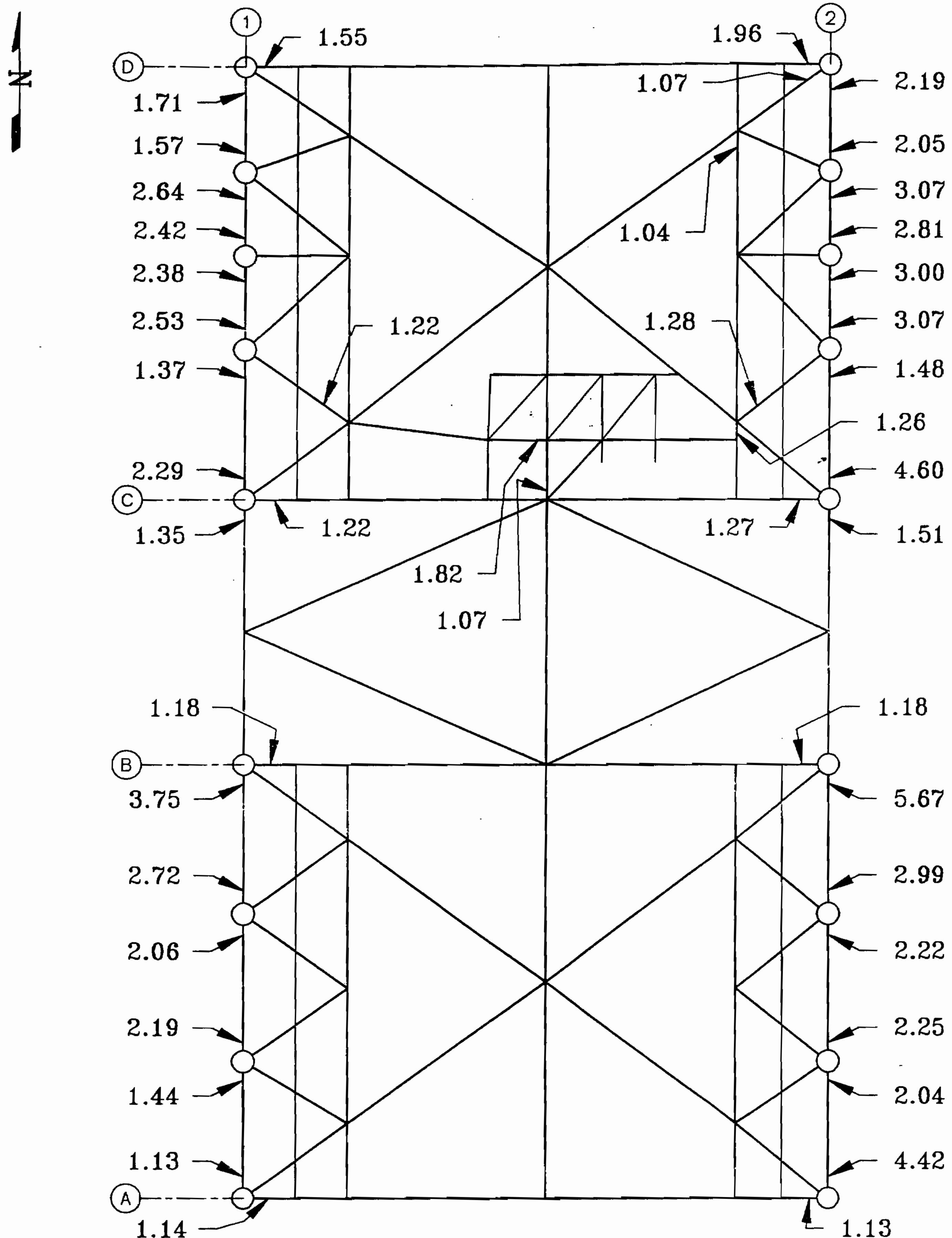
Figure 9



JOINT STRENGTH UNITY CHECKS > 1.0
DESIGN LEVEL ANALYSIS

PLAN @ EL.(–)122'-0"
(1" =18'-0")

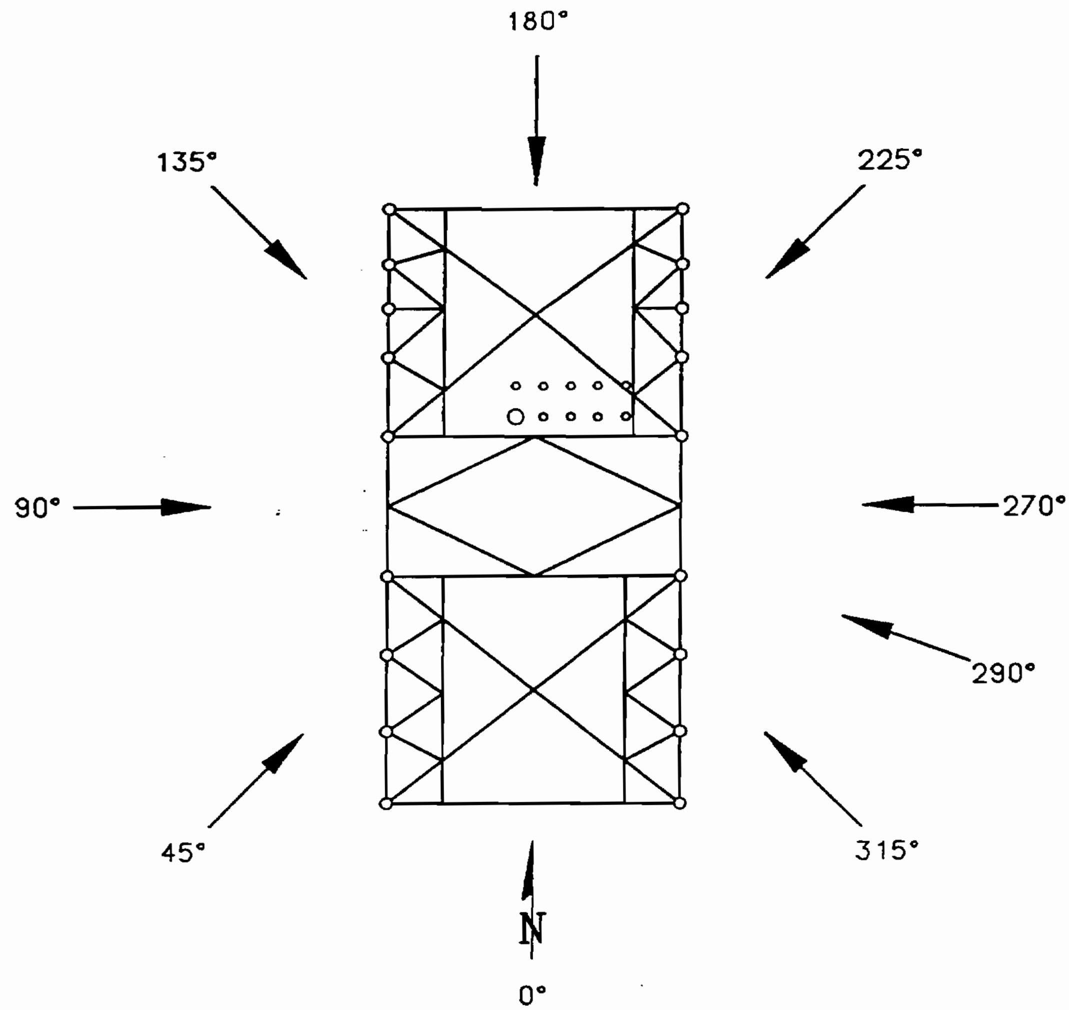
Figure 10



JOINT STRENGTH UNITY CHECKS > 1.0
DESIGN LEVEL ANALYSIS

PLAN @ EL.(–)160'–0"
(1" =18"–0")

Figure 11



Wave Approach Direction (deg.)	0	45	90	135	180	225	270	290	315
Directional Factor	0.85	0.70	0.70	0.70	0.75	0.90	1.00	1.00	0.95
Wave Height, Hmax (ft)	53.6	44.2	44.2	44.2	47.3	56.8	63.1	63.1	59.9
Current Velocity (kts)	0.2	0.2	0.2	0.2	0.2	2.09	2.09	2.09	2.09
Current Direction (deg.)	0	45	90	135	180	280	280	280	280

Wave Height and Current for Ultimate Strength Analysis
RSR Calculations

(Based on API RP 2A, 20th Ed.; refer to Table 1 for other metocean parameters)

Figure 12

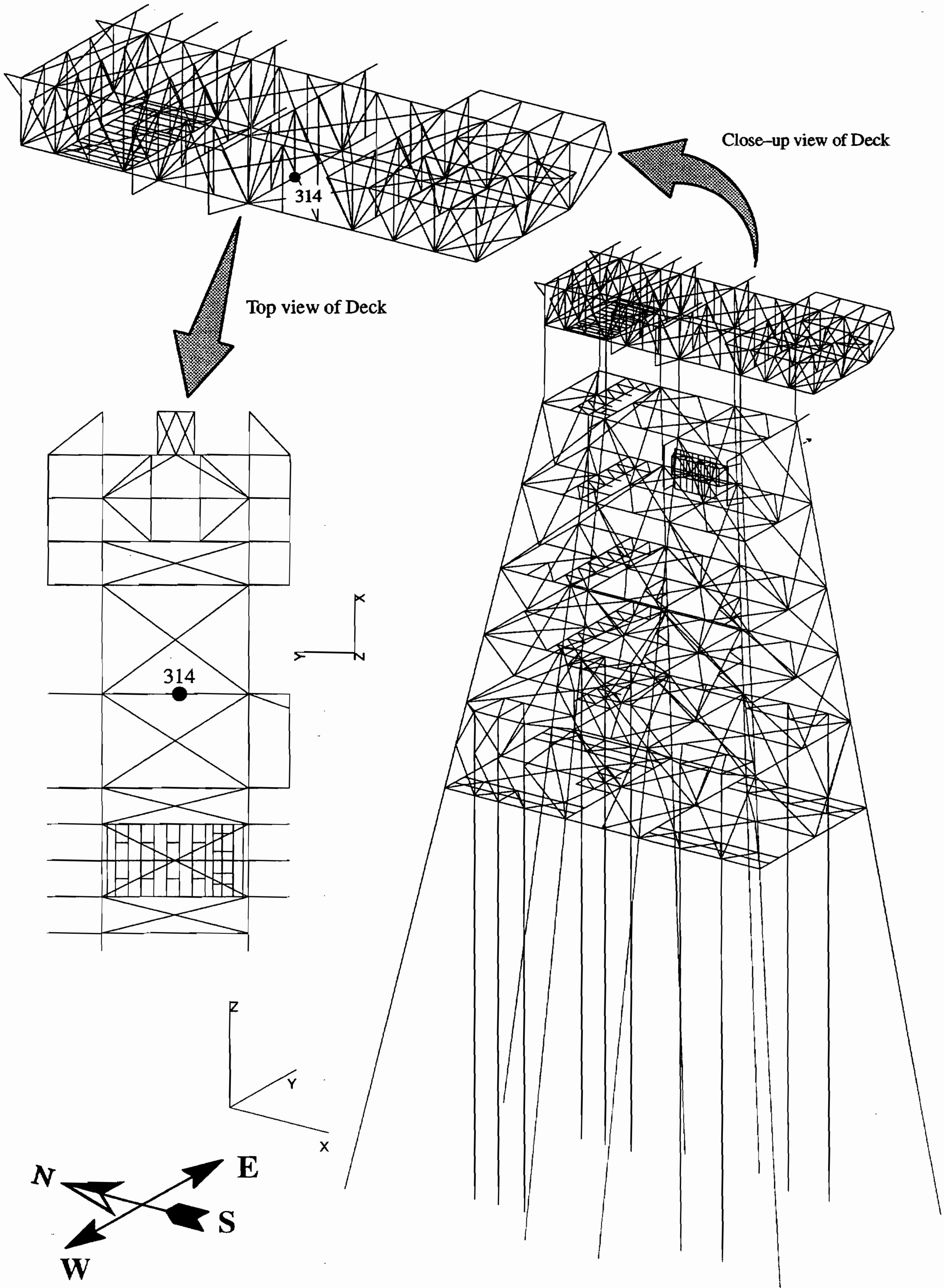
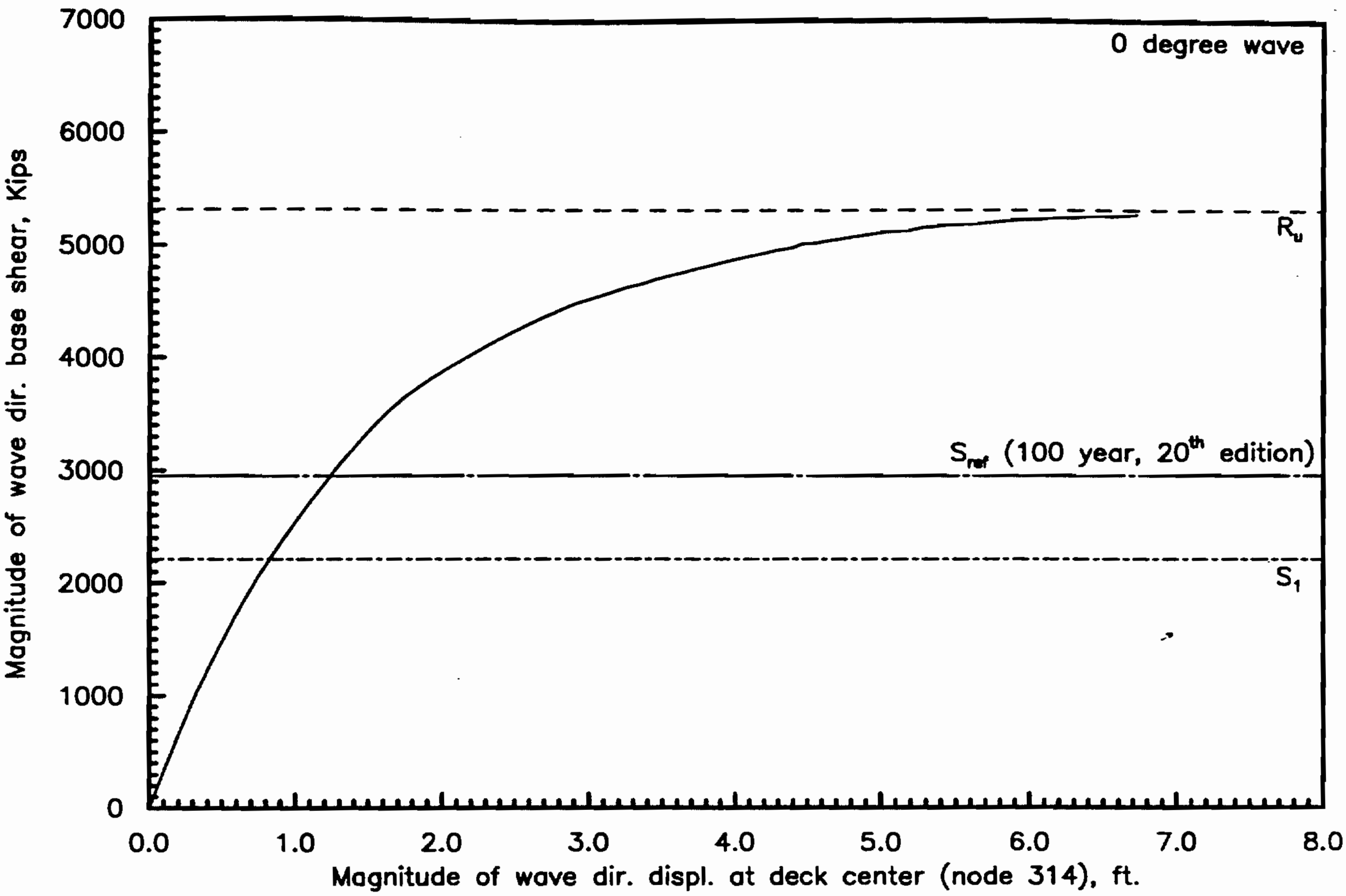
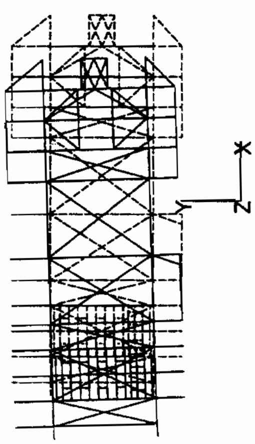
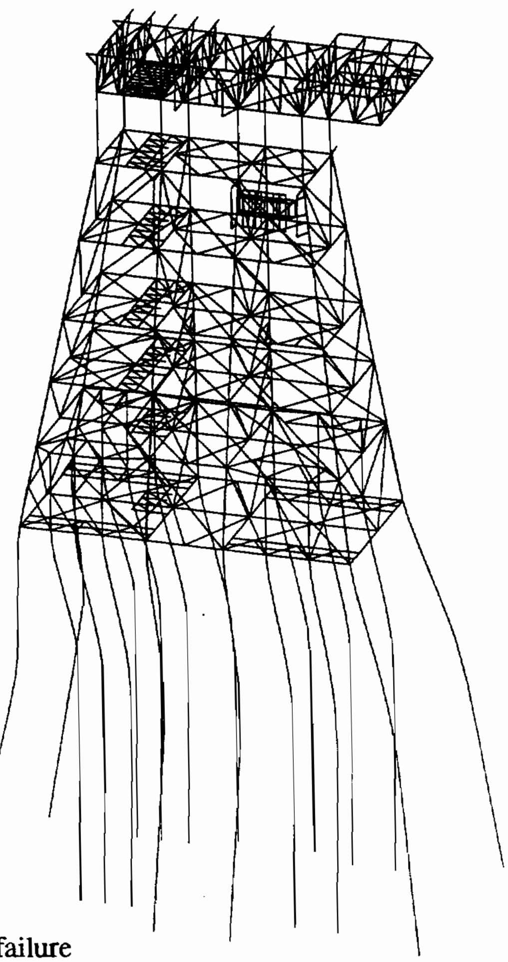


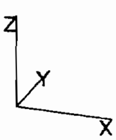
Fig. 13, Platform KARMA model





Deck displacement top view





Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips: <u>2213.3</u>

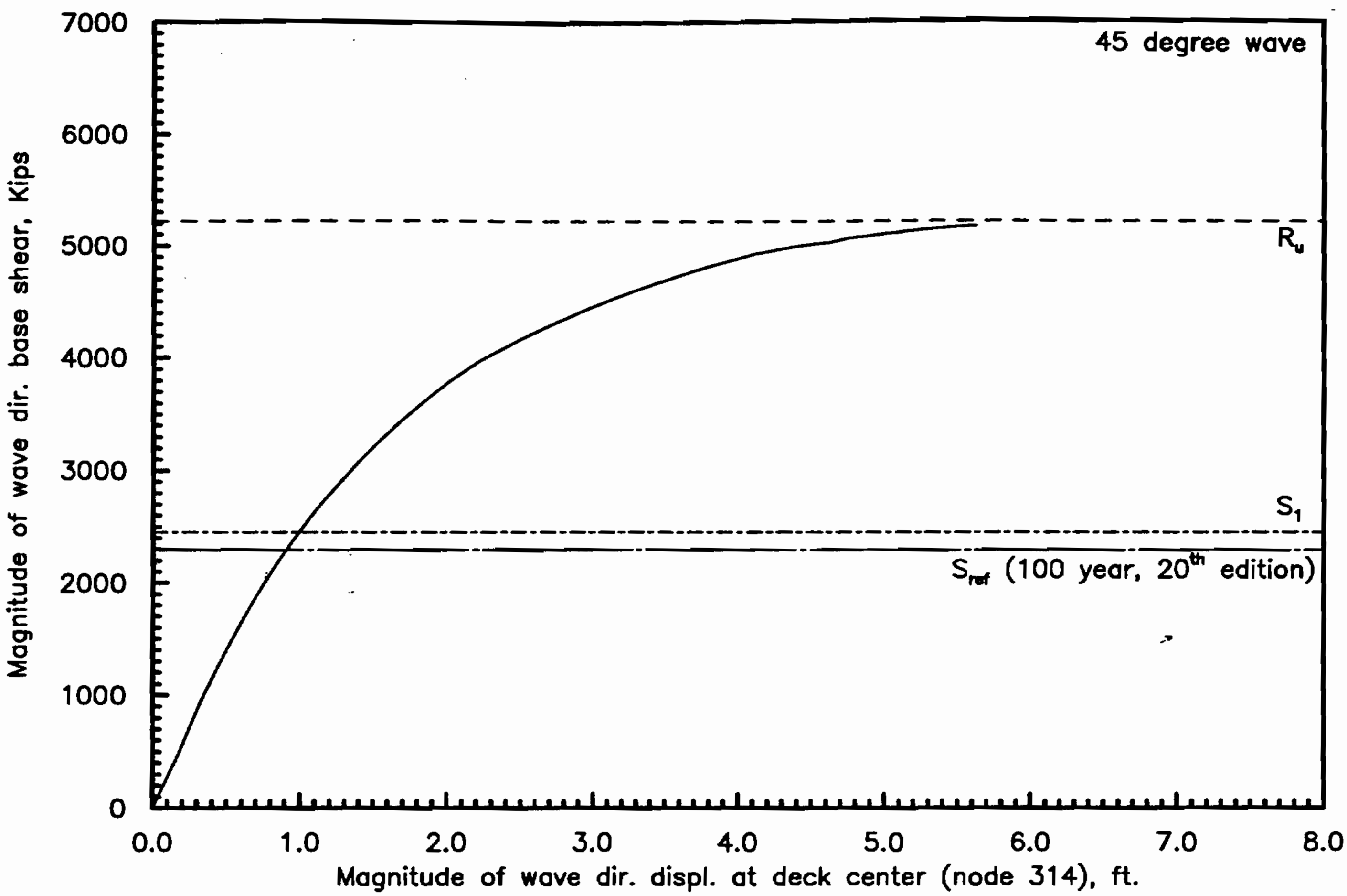
Reference Level Load (S_{ref}), Kips: <u>2956.0</u>

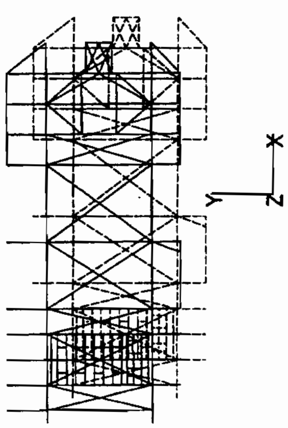
Ultimate Capacity (R_u), Kips: <u>5315.0</u>

Reserve Strength Ratio (RSR): <u>1.80</u>

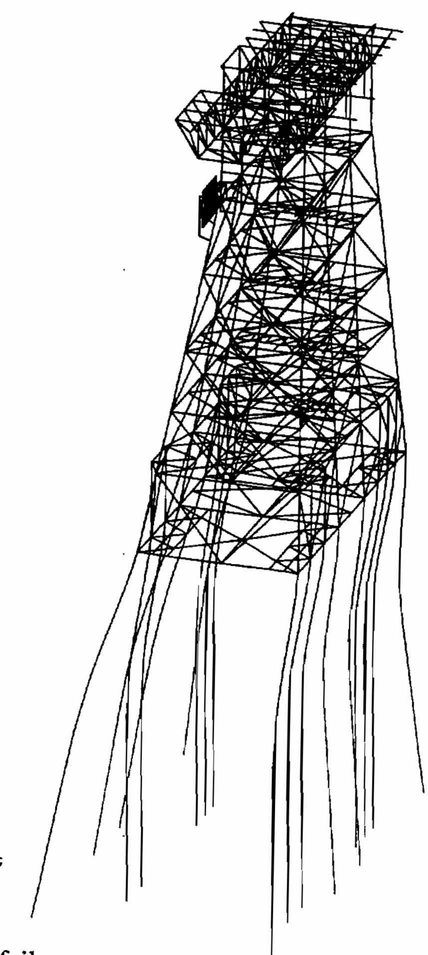
Platform Failure Mode: Foundation failure followed by failure of the piles.

Fig. 14, Load deflection response for 0 degree wave





Deck displacement
top view

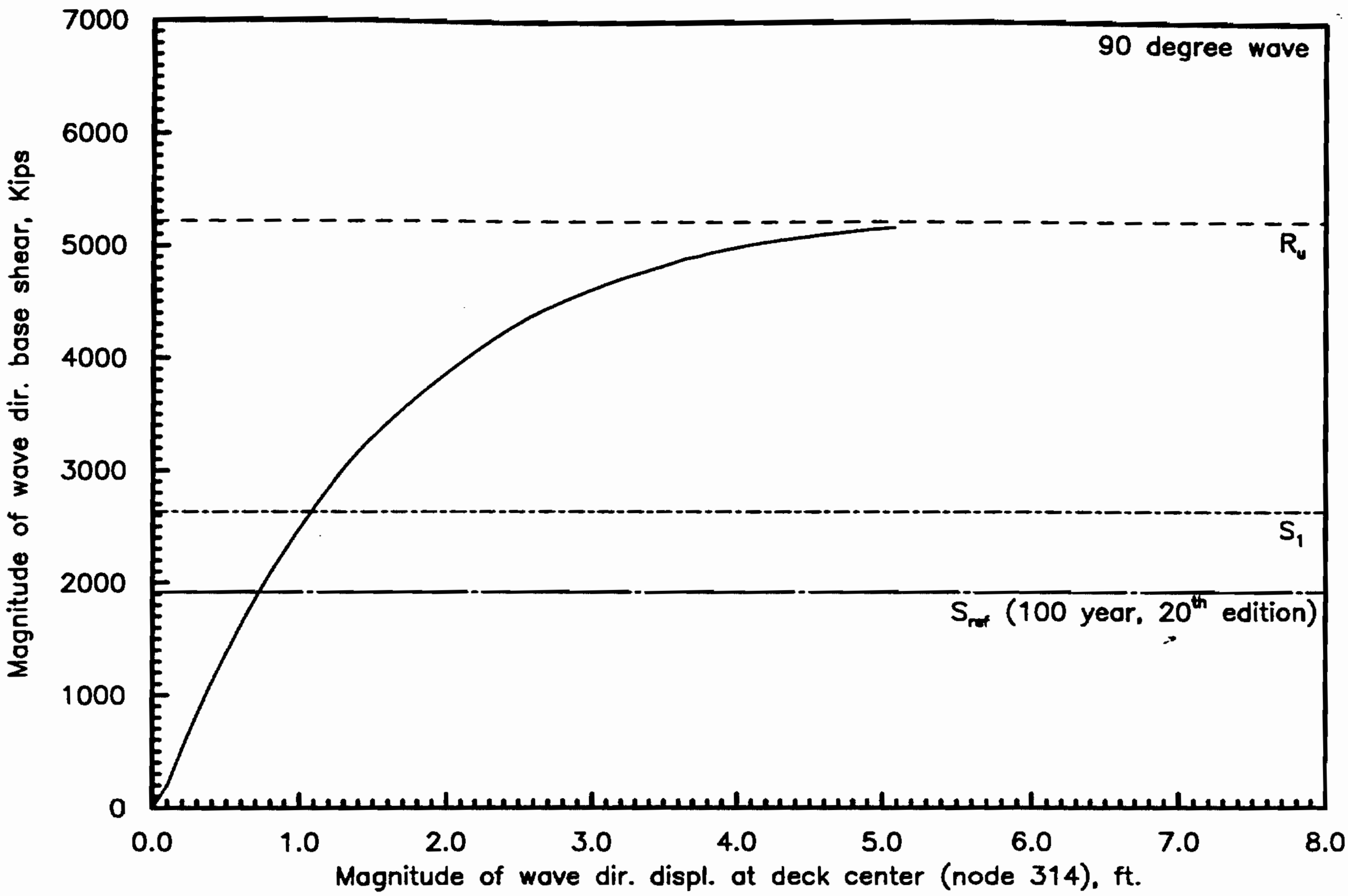


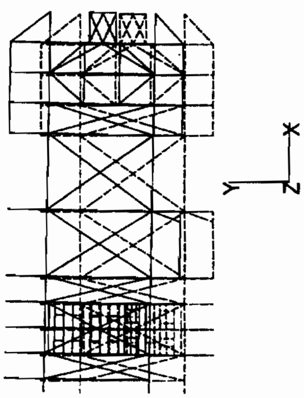
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2457.2
Reference Level Load (S_{ref}), Kips:	2297.0
Ultimate Capacity (R_u), Kips:	5220.3
Reserve Strength Ratio (RSR):	2.27

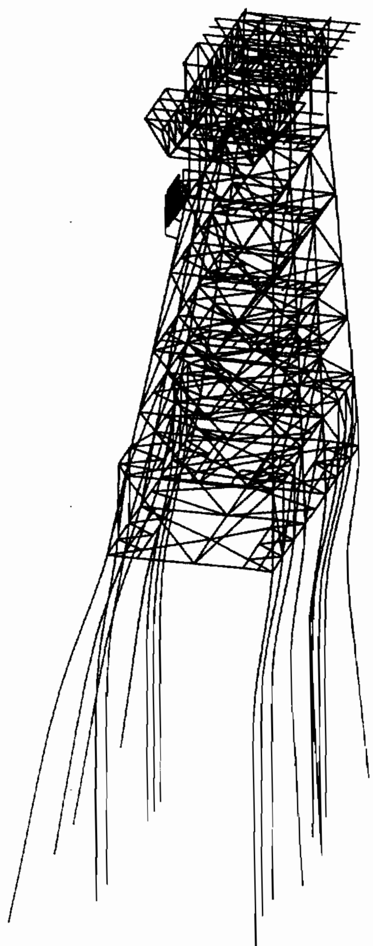
Platform Failure Mode: Foundation failure followed by failure of the piles.

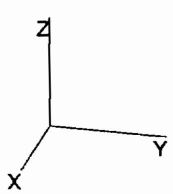
Fig. 15, Load deflection response for 45 degree wave





Deck displacement
top view



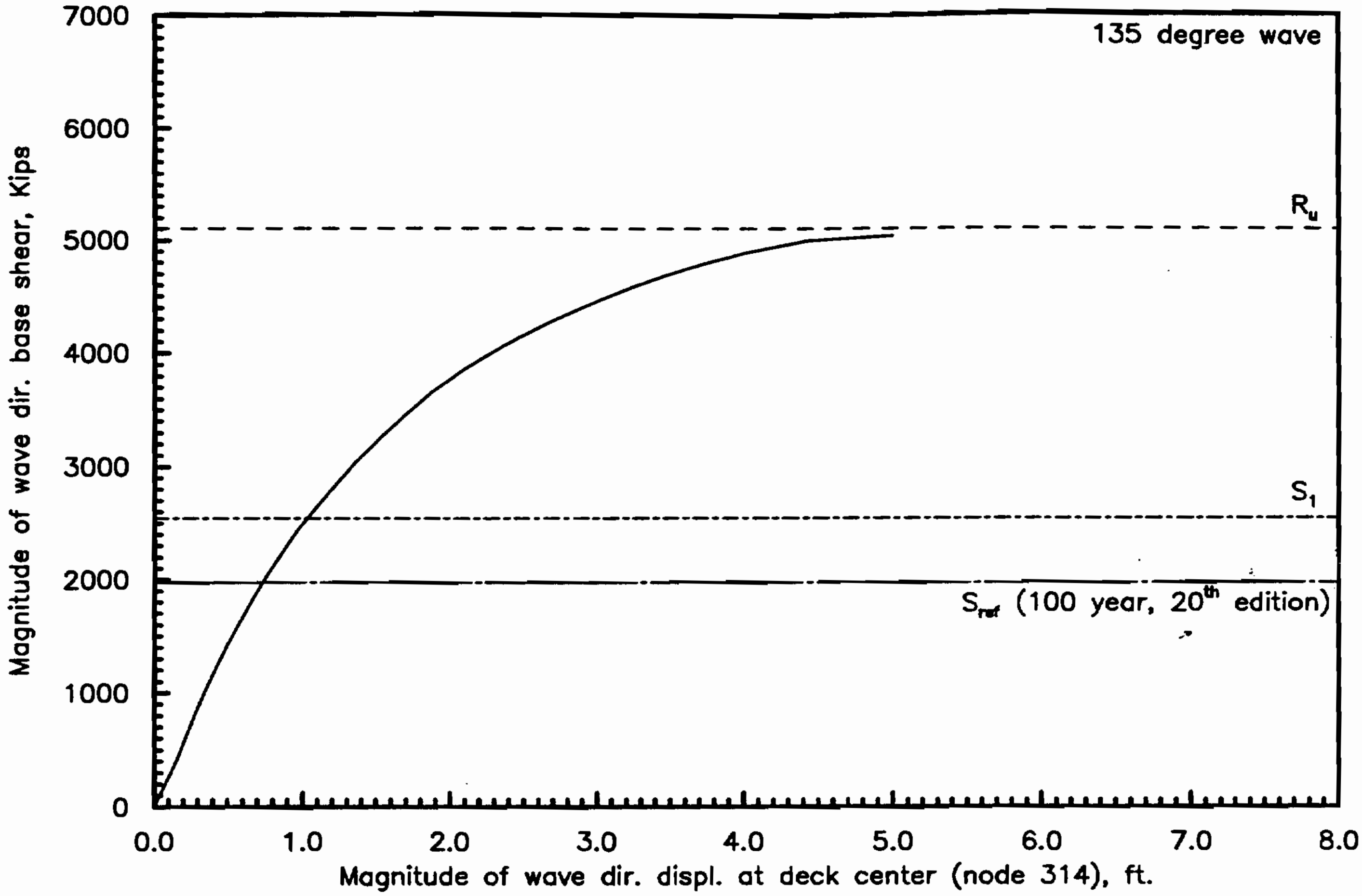


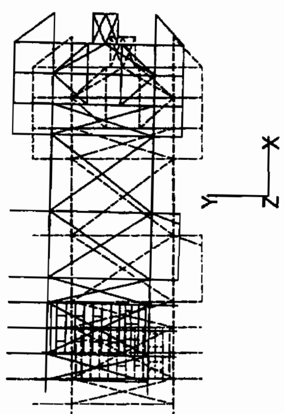
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2628.9
Reference Level Load (S_{ref}), Kips:	1920.0
Ultimate Capacity (R_u), Kips:	5215.1
Reserve Strength Ratio (RSR):	2.72

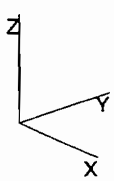
Platform Failure Mode:	Foundation failure followed by failure of the piles.

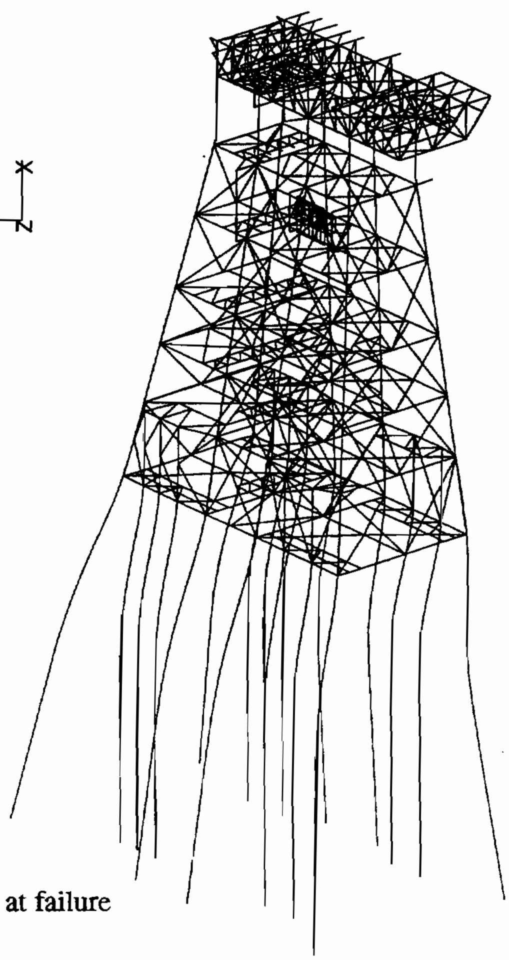
Fig. 16, Load deflection response for 90 degree wave





Deck displacement top view



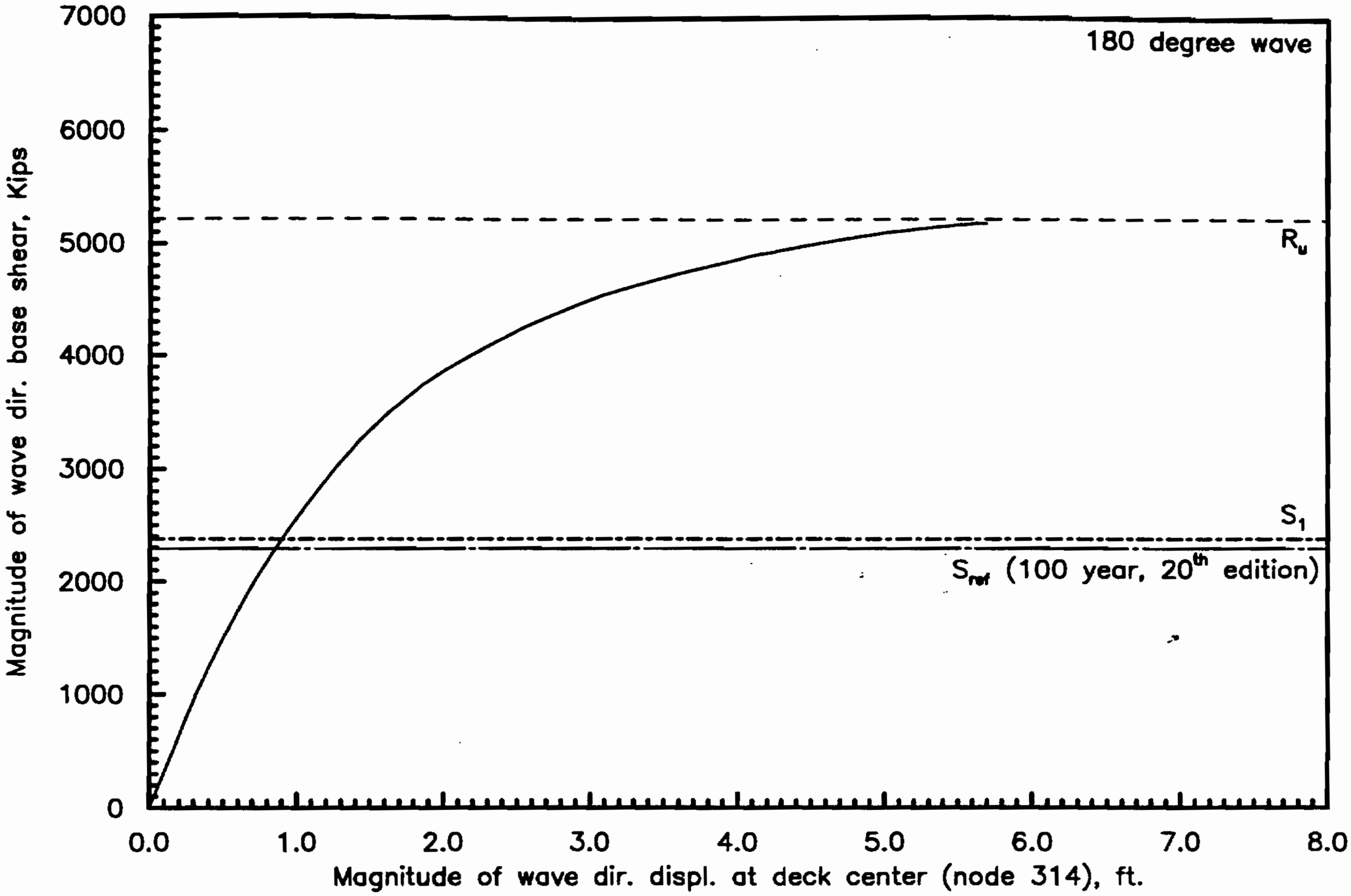


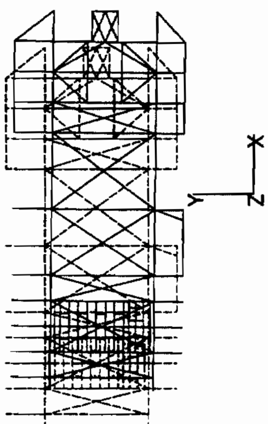
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2547.0
Reference Level Load (S_{ref}), Kips:	1977.0
Ultimate Capacity (R_u), Kips:	5104.7
Reserve Strength Ratio (RSR):	2.58

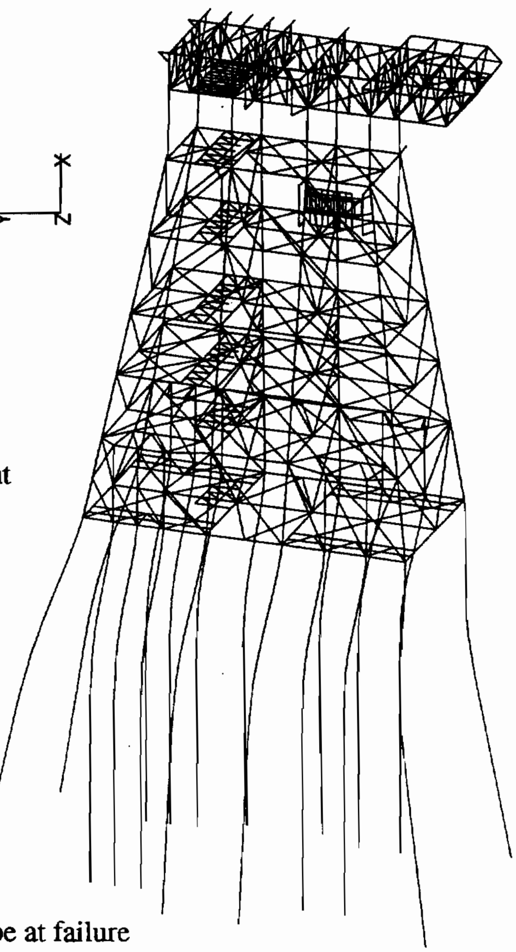
Platform Failure Mode: Foundation failure followed by failure of the piles.

Fig. 17, Load deflection response for 135 degree wave





Deck displacement top view

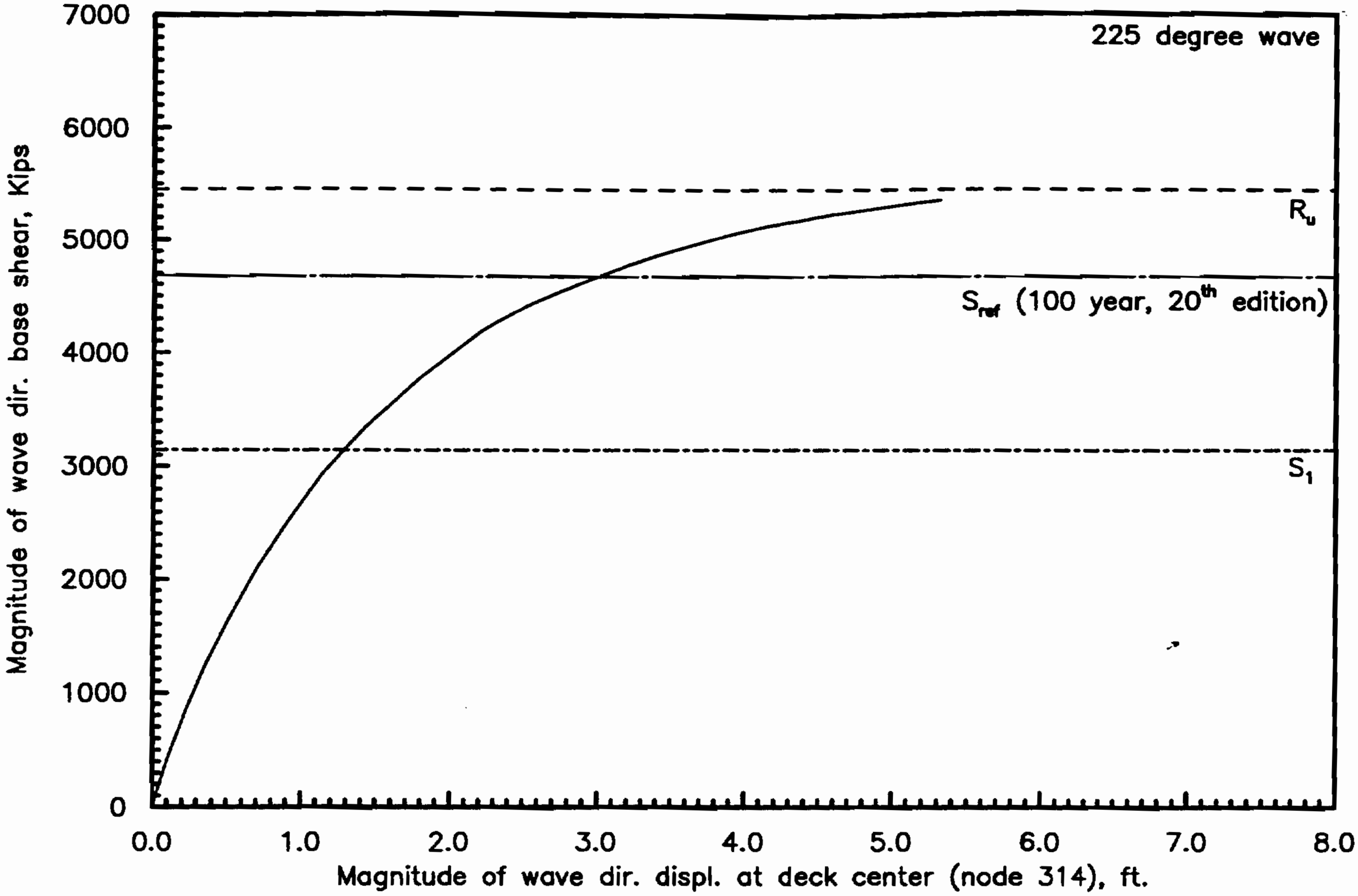


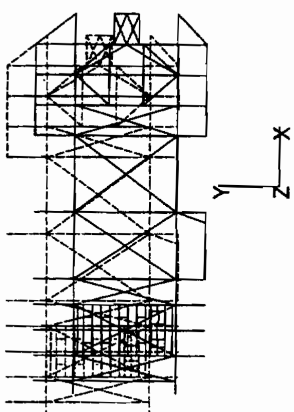
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	<u>2437.6</u>
Reference Level Load (S_{ref}), Kips:	<u>2352.0</u>
Ultimate Capacity (R_u), Kips:	<u>5345.6</u>
Reserve Strength Ratio (RSR):	<u>2.27</u>

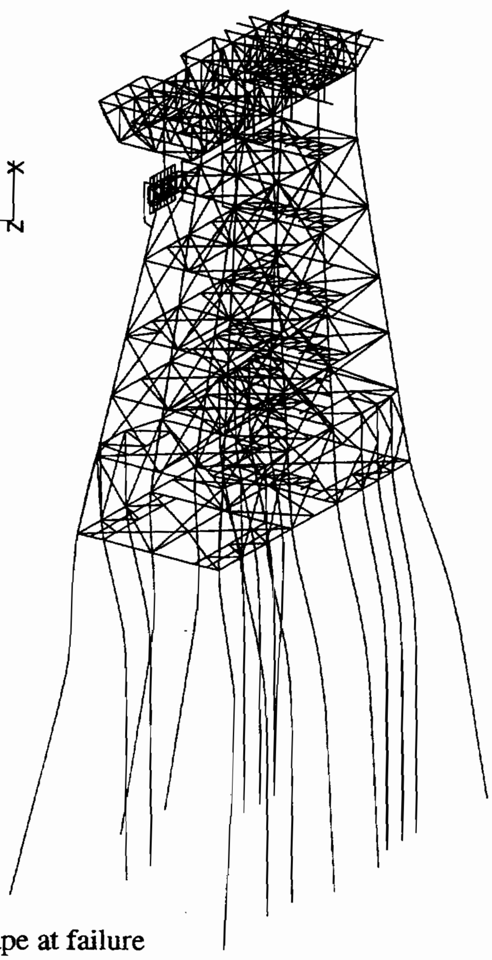
Platform Failure Mode: Foundation failure followed by failure of the piles.

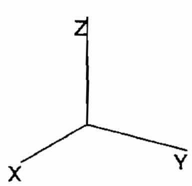
Fig. 18, Load deflection response for 180 degree wave





Deck displacement top view



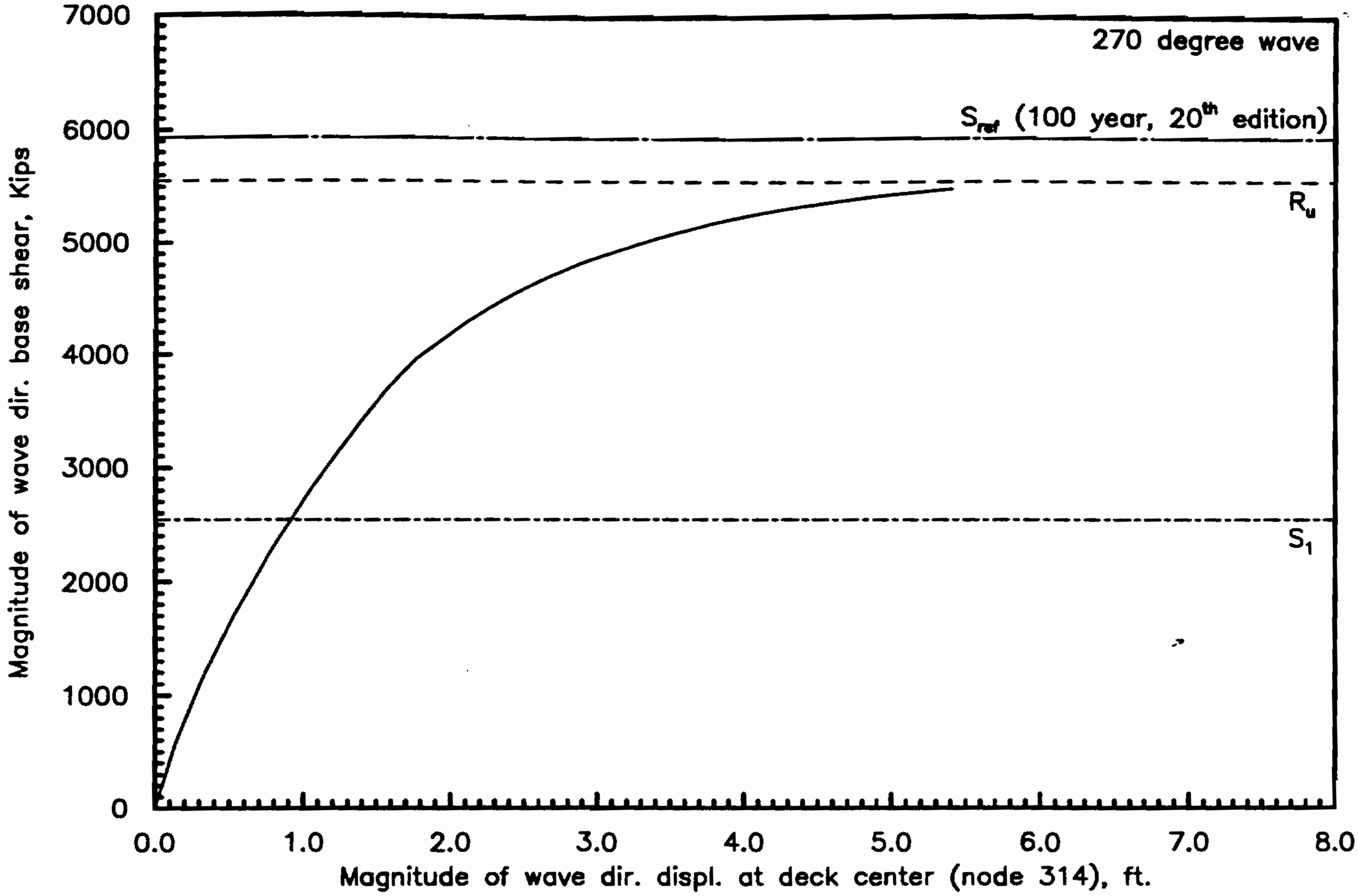


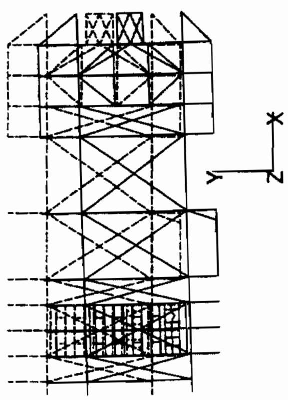
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	3163.4
Reference Level Load (S_{ref}), Kips:	4707.0
Ultimate Capacity (R_u), Kips:	5484.8
Reserve Strength Ratio (RSR):	1.17

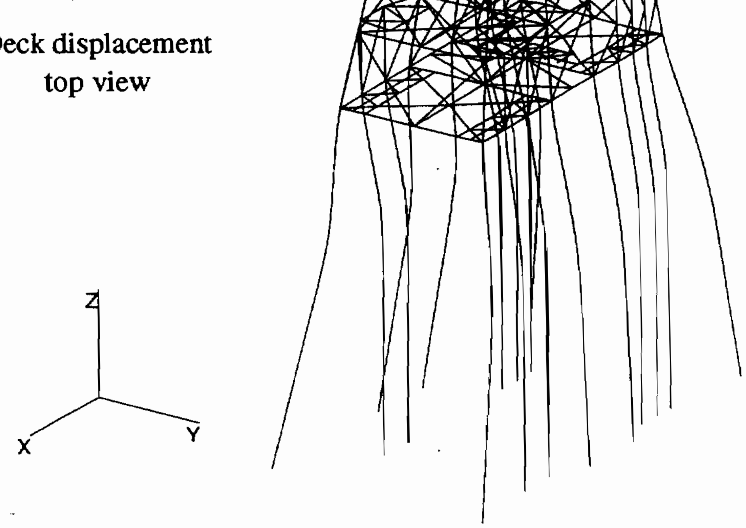
Platform Failure Mode: Foundation failure followed by failure of the piles.

Fig. 19, Load deflection response for 225 degree wave





Deck displacement top view

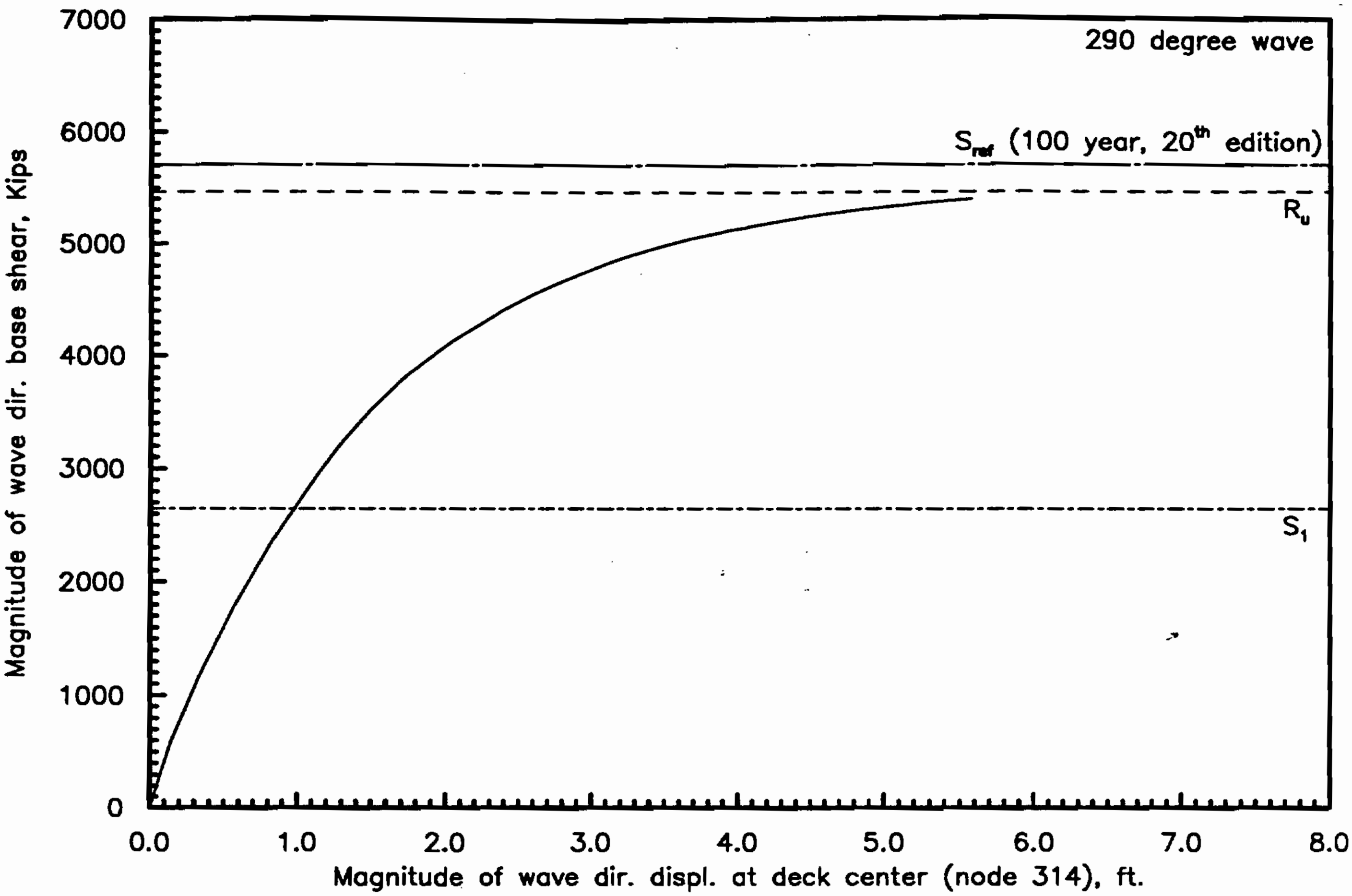


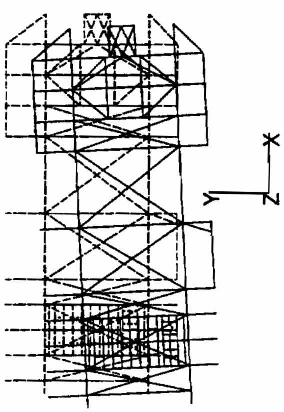
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2545.7
Reference Level Load (S_{ref}), Kips:	5932.0
Ultimate Capacity (R_u), Kips:	5551.9
Reserve Strength Ratio (RSR):	0.94

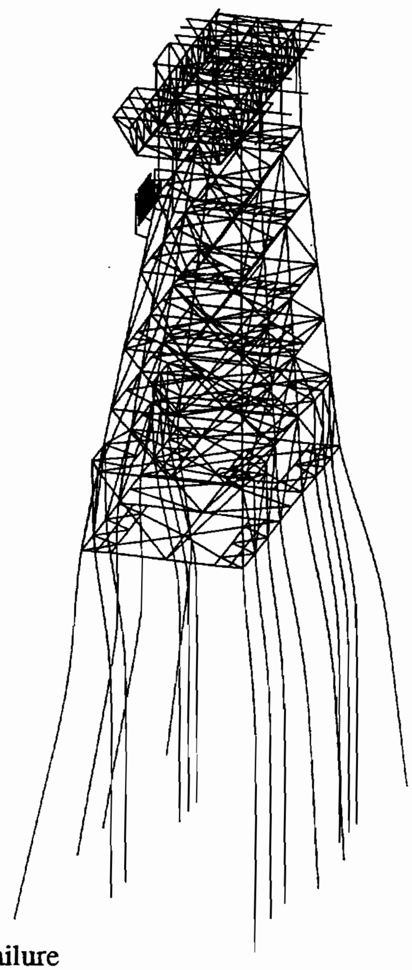
Platform Failure Mode: Foundation failure followed by failure of the piles.

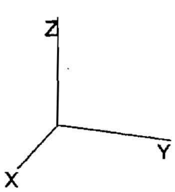
Fig. 20, Load deflection response for 270 degree wave





Deck displacement top view



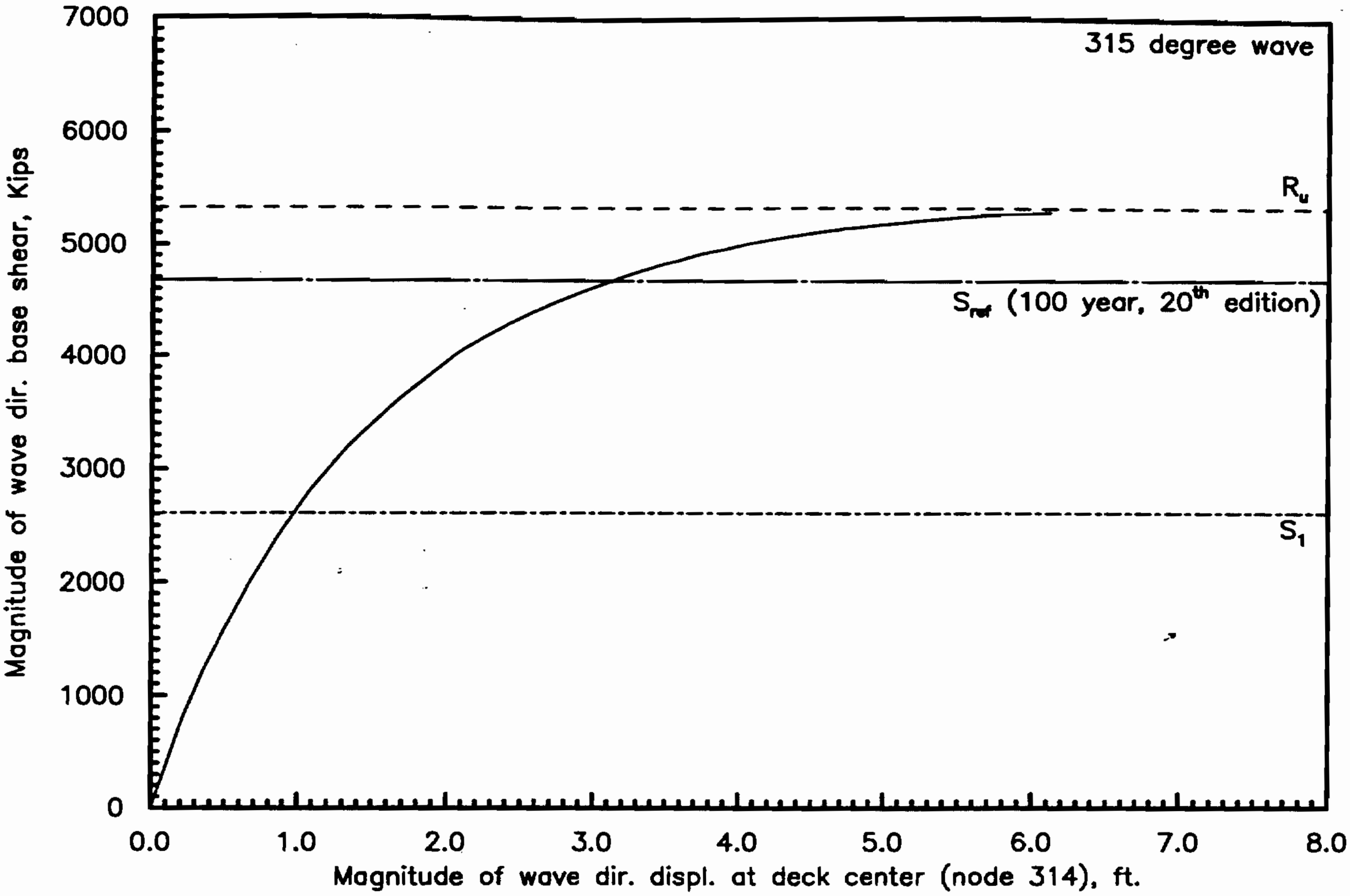


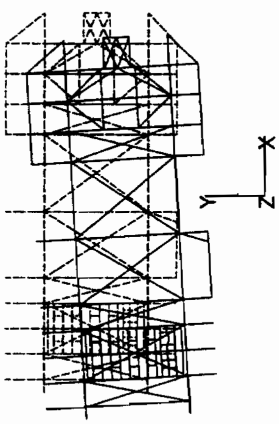
Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2654.8
Reference Level Load (S_{ref}), Kips:	5724.0
Ultimate Capacity (R_u), Kips:	5476.9
Reserve Strength Ratio (RSR):	0.96

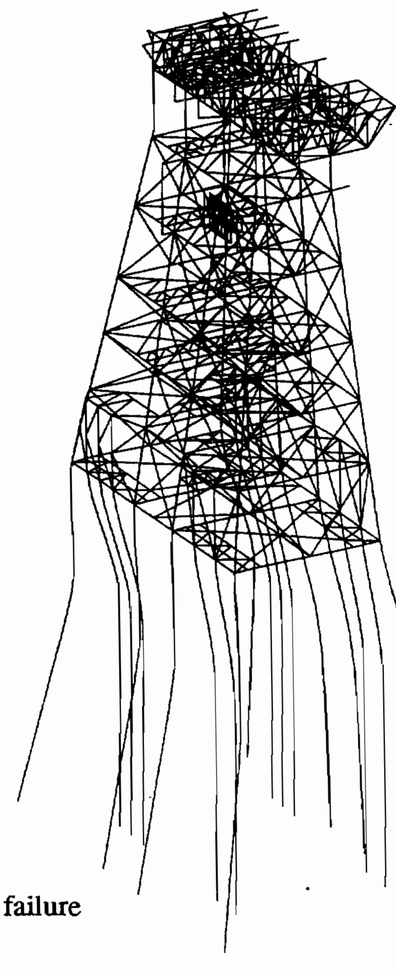
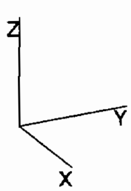
Platform Failure Mode: Foundation failure followed by failure of the piles.

Fig. 21, Load deflection response for 290 degree wave





Deck displacement top view

Displaced shape at failure

Load Level at which First Component Reaches I.R. of 1.0 (S_1), Kips:	2635.4
Reference Level Load (S_{ref}), Kips:	4721.0
Ultimate Capacity (R_u), Kips:	5380.8
Reserve Strength Ratio (RSR):	1.14

Platform Failure Mode: Foundation failure followed by failure of the piles.

Fig. 22, Load deflection response for 315 degree wave

Tables

	API RP 2A-WSD Draft Section 17 Criteria for "Insignificant Environmental Impact/Unmanned"		
Environmental Criteria	Design Level Analysis	Ultimate Strength Analysis	API RP 2A-WSD 20th Edition Criteria
Classification	Winter Storm	Winter Storm	Full Population Hurricanes
Wave Height (ft.)	37.5	46.5	44.2 to 63.1
Wave Period (sec.)	10.5	11.5	13.0
Current (kts)	0.9	1.0	2.09 (0.2 minimum)
Wave & Current Direction	Omni-Directional	Omni-Directional	Directional
Wave Kinematics Factor	1.0	1.0	0.88
Drag Coef., Cd (smooth)	0.65	0.65	0.65
Drag Coef., Cd (rough)	1.05	1.05	1.05
Inertia Coef., Cm (smooth)	1.6	1.6	1.6
Inertia Coef., Cm (rough)	1.2	1.2	1.2
Current Blockage Factor	0.7 to 0.85	0.7 to 0.85	0.7 to 0.85
Conductor Shielding Factor	0.9	0.9	0.9
Marine Growth Thickness (in.)	1.5	1.5	1.5
Storm Tide (ft.)	3.0	3.0	3.5
Wind Velocity, 1-hr (kts)	45	50	80
Minimum Deck Height (ft.)	N/A	36.4	51

Table 1. Metocean Loading Criteria, Gulf of Mexico, Water Depth = 160 ft.

Description	Weight (Kips)
Deck Deadload	1061
Jacket Deadload (including misc. appurtenances & piles below mud)	2343
Jacket Buoyancy	-1310
Deck Liveload	322
Piles Below Mudline	765
Total	3181

Table 2. Gravity Loading Summary

Wave Approach Direction	Shear (Kips)	Overturning Moment (Kip-Ft)
End-On	1880	223000
Diagonal	1970	226000
Broadside	2060	236000

Table 3. Wave, Wind, and Current Forces

(Design Level Criteria-- 8 Conductors Added)

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2213.3 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.7408	2043.0	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.539	3405.0	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	2.327	4120.0			
4	3.178	4660.0			
5	3.856	4836.0			
6	4.617	5041.0	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	5.353	5180.0			
8	5.820	5218.0			
9	6.293	5269.0			
10	6.723	5278.0	Peak capacity		

Table 4, Ultimate Strength Analysis Results for 0 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2454.7 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.9456	2366.0	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.397	3076.6	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	1.836	3608.4			
4	2.465	4141.5			
5	2.897	4389.0			
6	3.337	4635.1	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	3.892	4848.6			
8	4.501	5012.7			
9	5.002	5103.9			
10	5.621	5176.7	Peak capacity		

Table 5, Ultimate Strength Analysis Results for 45 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2628.8 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	1.049	2480.0	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.471	3274.0	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	2.002	3869.0			
4	2.441	4266.0			
5	2.945	4584.0			
6	3.404	4790.0	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	3.850	4951.0			
8	4.353	5065.0			
9	4.750	5127.0			
10	5.082	5169.0	Peak capacity		

Table 6, Ultimate Strength Analysis Results for 90 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2544.7 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.9678	2443.1	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.367	3053.3	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	1.877	3664.2			
4	2.639	4071.5			
5	2.856	4377.0			
6	3.456	4683.2	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	3.707	4782.2			
8	4.011	4885.4			
9	4.426	4997.1			
10	4.994	5035.3	Peak capacity		

Table 7, Ultimate Strength Analysis Results for 135 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: <u>2378.1</u> **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.8313	2253.0	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.426	5254.0	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	2.022	3880.0			
4	2.552	4256.0			
5	3.076	4538.0			
6	3.671	4758.0	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	4.150	4909.0			
8	4.631	5022.0			
9	5.154	5116.0			
10	5.696	5182.0	Peak capacity		

Table 8, Ultimate Strength Analysis Results for 180 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 3143.7 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	1.139	2933.8	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.783	3771.7	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	2.321	4275.2			
4	2.873	4608.9			
5	3.391	4859.2			
6	3.940	5062.9	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	4.454	5201.5			
8	4.908	5286.3			
9	5.152	5330.9			
10	5.320	5355.6	Peak capacity		

Table 9, Ultimate Strength Analysis Results for 225 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2544.8 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.771	2262.0	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.379	3393.0	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	1.762	3958.0			
4	2.328	4467.0			
5	2.847	4795.0			
6	3.381	5032.0	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	3.865	5203.0			
8	4.332	5321.0			
9	4.990	5434.0			
10	5.403	5485.0	Peak capacity		

Table 10, Ultimate Strength Analysis Results for 270 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2646.5 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.8206	2352.4	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.496	3528.6	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	1.727	3822.4			
4	2.481	4469.8			
5	3.097	4821.8			
6	3.461	4974.9	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	3.950	5126.7			
8	4.510	5252.0			
9	5.001	5328.4			
10	5.569	5398.8	Peak capacity		

Table 11, Ultimate Strength Analysis Results for 290 degree wave

Analysis case: 3–D model

Lateral Load level for first member with unity check = 1.0: 2608.1 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.8641	2407.7	Linear range	All failures combined bending and axial yielding, with bending only for piles and bending predominate for all members except jacket–to–skirt pile sleeve braces. These braces approximately equal in both. Axial yielding occurs at Sherman's buckling load with no post–peak load shedding.	All wave directions experience the same failure mode: pile–foundation failure with a double hinge in the piles. At peak capacity, nearly all skirt piles and 4–7 of the main piles have failed. Comments as to element failures general to all wave directions.
2	1.342	3210.3	Secondary hz. mbrs. fail. Piles just below mudline go nonlinear. Pile failure propagates to mudline, forms top hinge.		
3	2.059	4012..8			
4	2.541	4363.6			
5	3.050	4638.6			
6	3.516	4833.8	Piles at depth fail, form double hinge. Jacket leg–to–skirt pile sleeve braces begin to fail near peak load.		
7	4.084	5015.5			
8	4.729	5146.3			
9	5.532	5250.3			
10	6.121	5291.3	Peak capacity		

Table 12, Ultimate Strength Analysis Results for 315 degree wave

PLATFORM "L"

1.0 Platform Information

1.1 As-is Condition

1.1.1 Physical Features and Operational Information

Platform L is a 4 pile drilling and production platform installed in 1970 in East Cameron, Gulf of Mexico in a water depth of 160 ft. The platform supports 8 well conductors, all 20" diameter, located in the center of the platform. The conductors are supported at the cellar deck and at the horizontal levels of the jacket.

The deck structure has two levels, a cellar deck and a main or drilling deck. The top of steel elevation of the cellar deck is 49' above the MLLW and the top of the main deck, at 67 ft elevation. The gas production capacity is expected to to go up to 88 MMSCFD. The major equipment consist of gas processing equipment, a vent boom and a small living quarters. It is proposed to add new compressors to the platform. With the addition of new facilities the platform is expected to be manned full time.

The jacket is a two level X-braced structure. The piles are 36" in diameter, ungrouted and shimmed to the top of the jacket legs. There are two boat landings at west and south faces of the jacket and 4 barge bumpers. At present there are no risers.

1.1.2 Inspection Information

Inspection report available at present indicate that there are no damage to the members. Therefore, it is assumed that all members are in their as-designed condition.

1.1.3 Soil and Foundation Information

The soil boring log available indicate that the soil mostly consist of over-consolidated clays of varying strength. The shear strength profile is shown at the end of this section. The p-y, t-z and tb-z curves have been constructed based on this profile and API RP 2A, 20th edition recommendations.

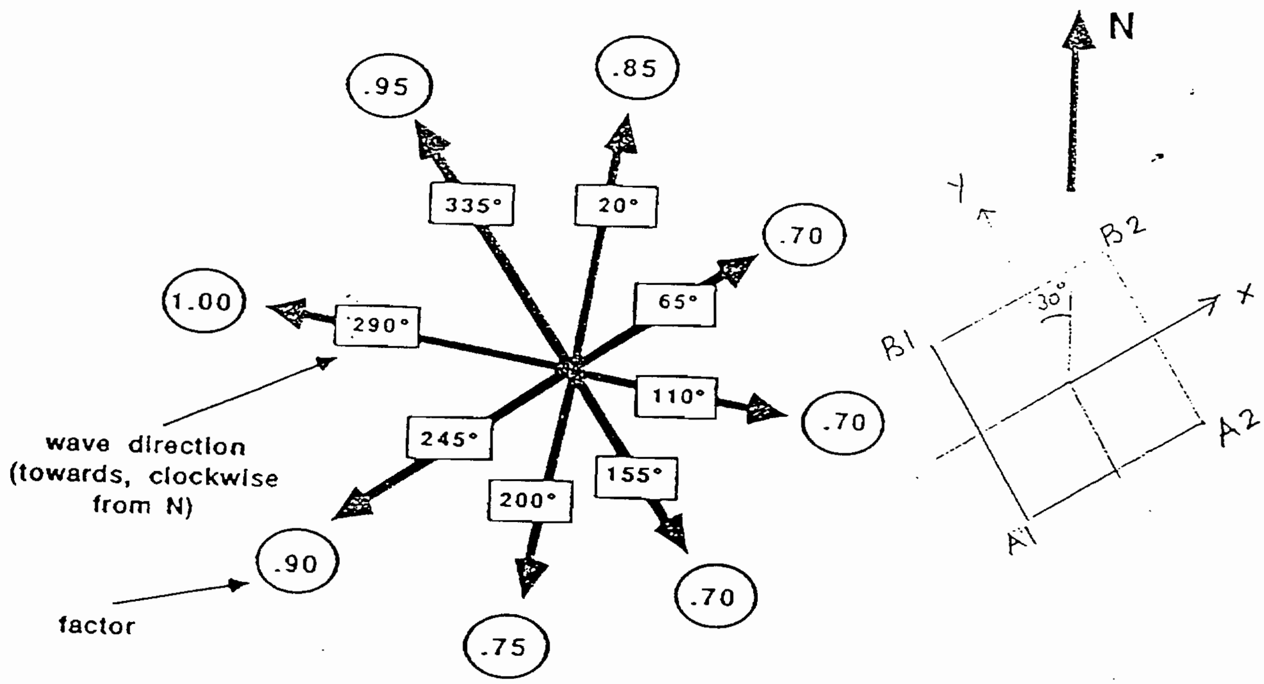
The original design penetration of the piles was 240 ft. However, during platform installation none of the piles had reached this penetration. Two piles had been jetted beyond the end of the pile tip. The actual penetrations are summarized below:

Pile	Actual Penetration	Remarks
Pile A-1	215 ft	Jetted
Pile A-2	227 ft	Driven
Pile B-1	222 ft	Driven
Pile B-2	190 ft	Jetted

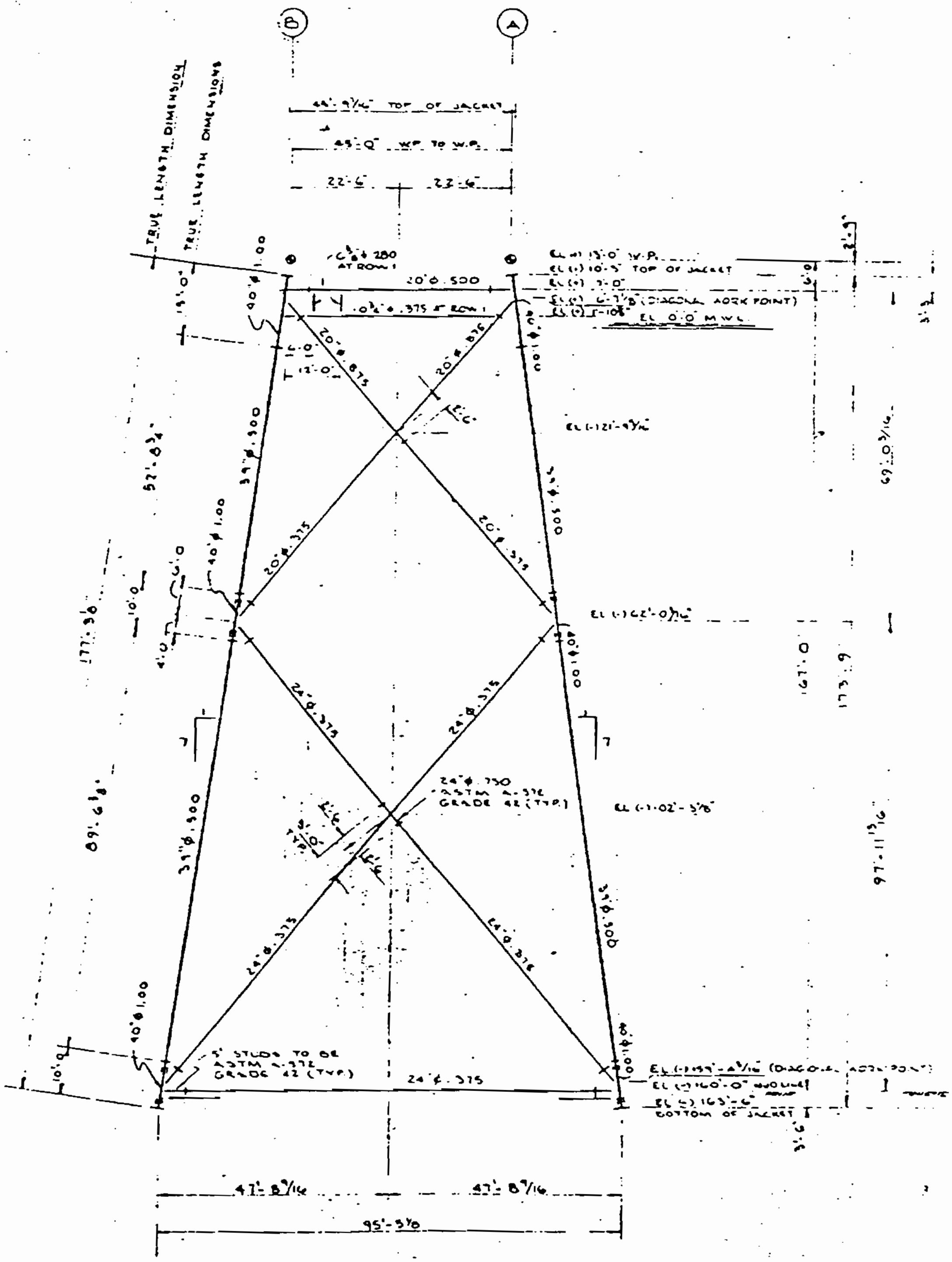
In the structural assessment, therefore, the actual conditions of the piles have been represented by assuming actual penetrations and neglecting end bearing for the two piles that were jetted.

1.2 Platform Sketches

The platform orientation, vertical elevations and horizontal framing plans are shown in the attached sketches. The pile makeup and details are also included.

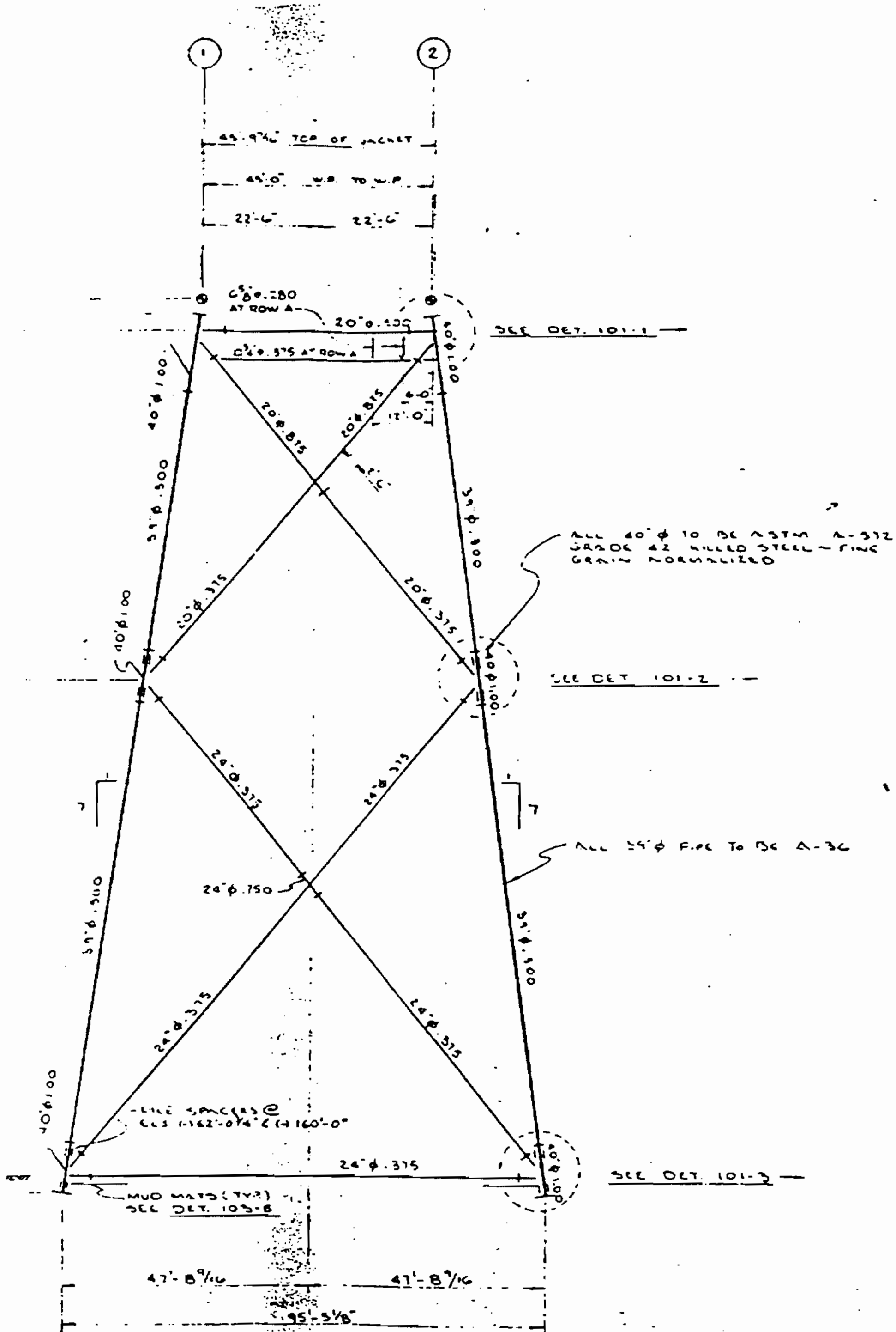


Platform Orientation and Wave Directions (Fig. 2.3.4-4 of API RP 2A)



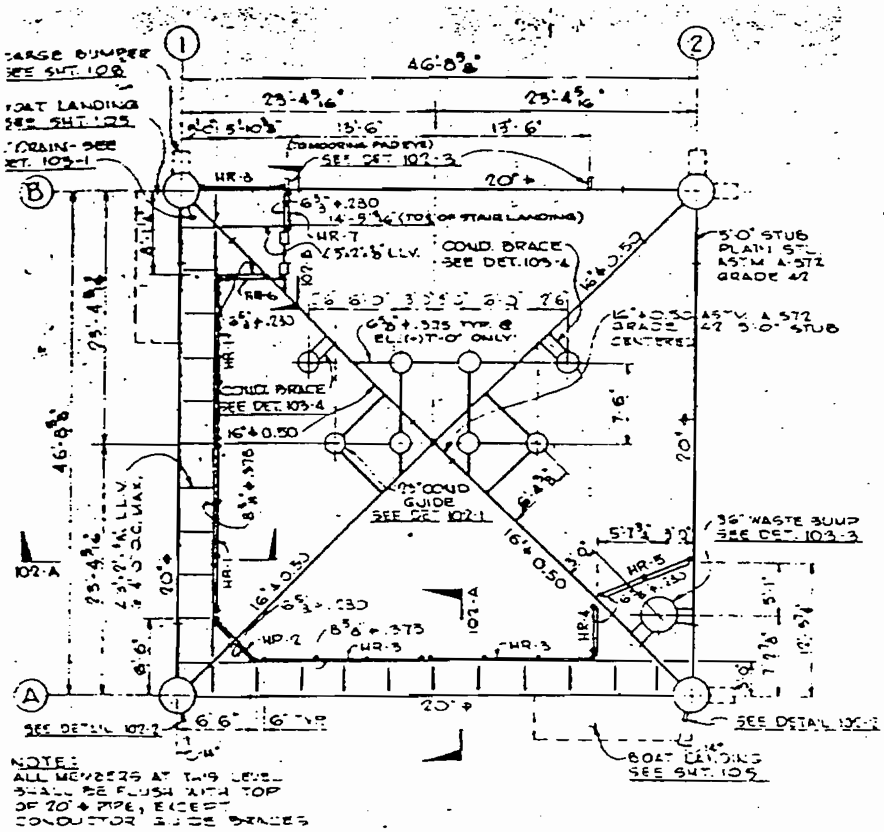
VERTICAL FRAMING AT COLUMN ROWS 1 & 2

1/16" = 1'-0"



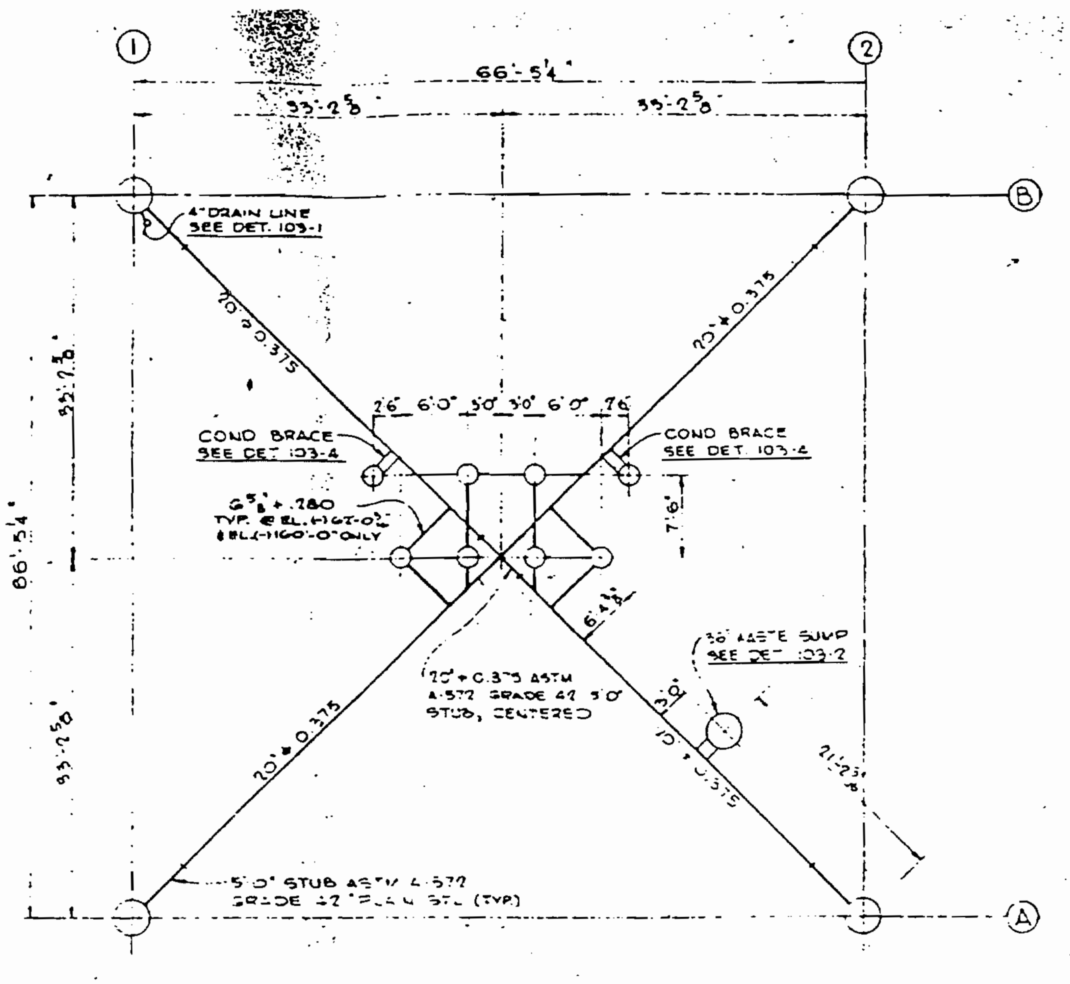
VERTICAL FRAMING AT COLUMN ROWS A&B

1/16" = 1'-0"

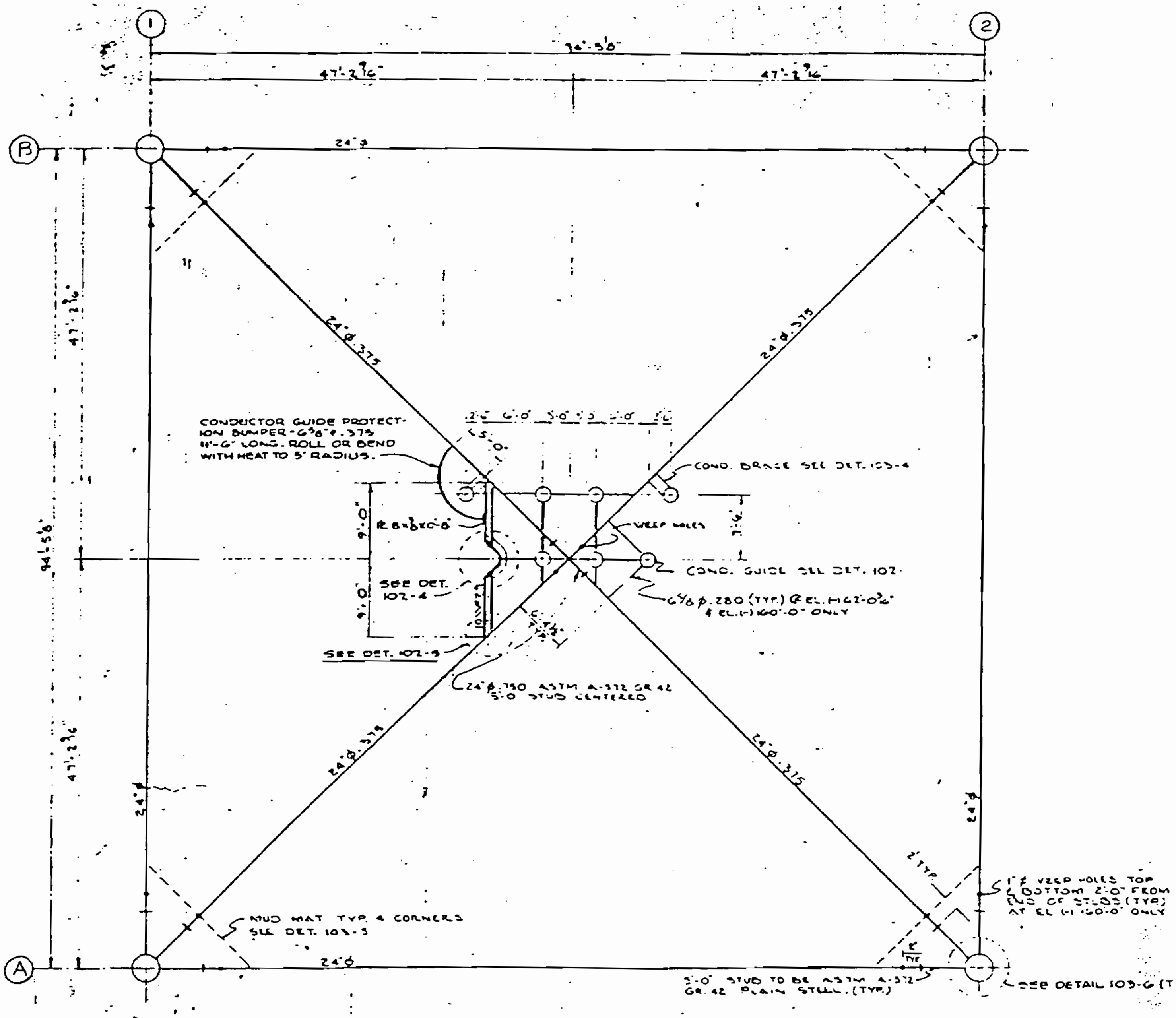


FRAMING PLAN AT EL (+) 7'-0"

1/8" = 1'-0"

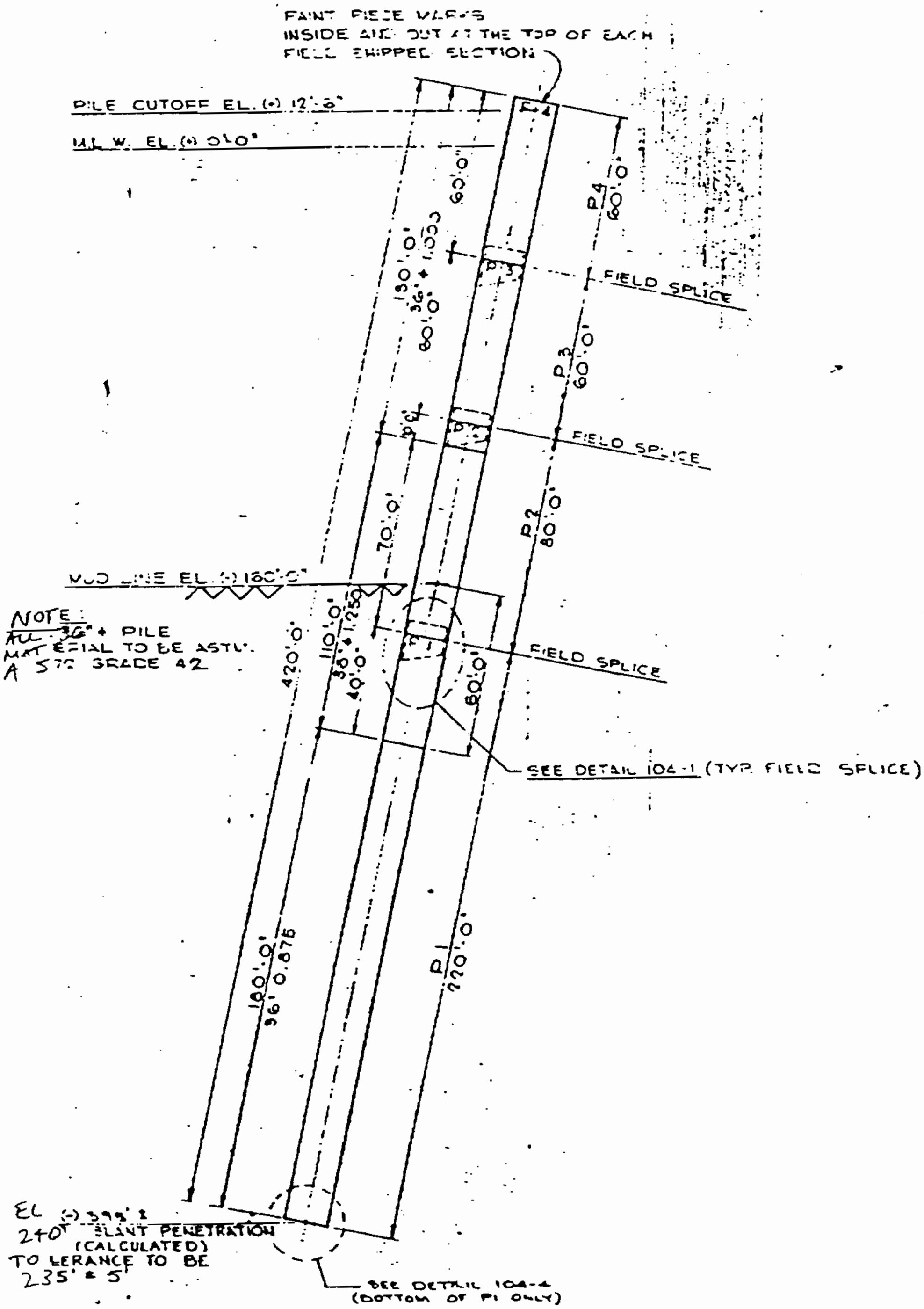


FRAMING PLAN AT EL. (-) 62'-0 3/16"

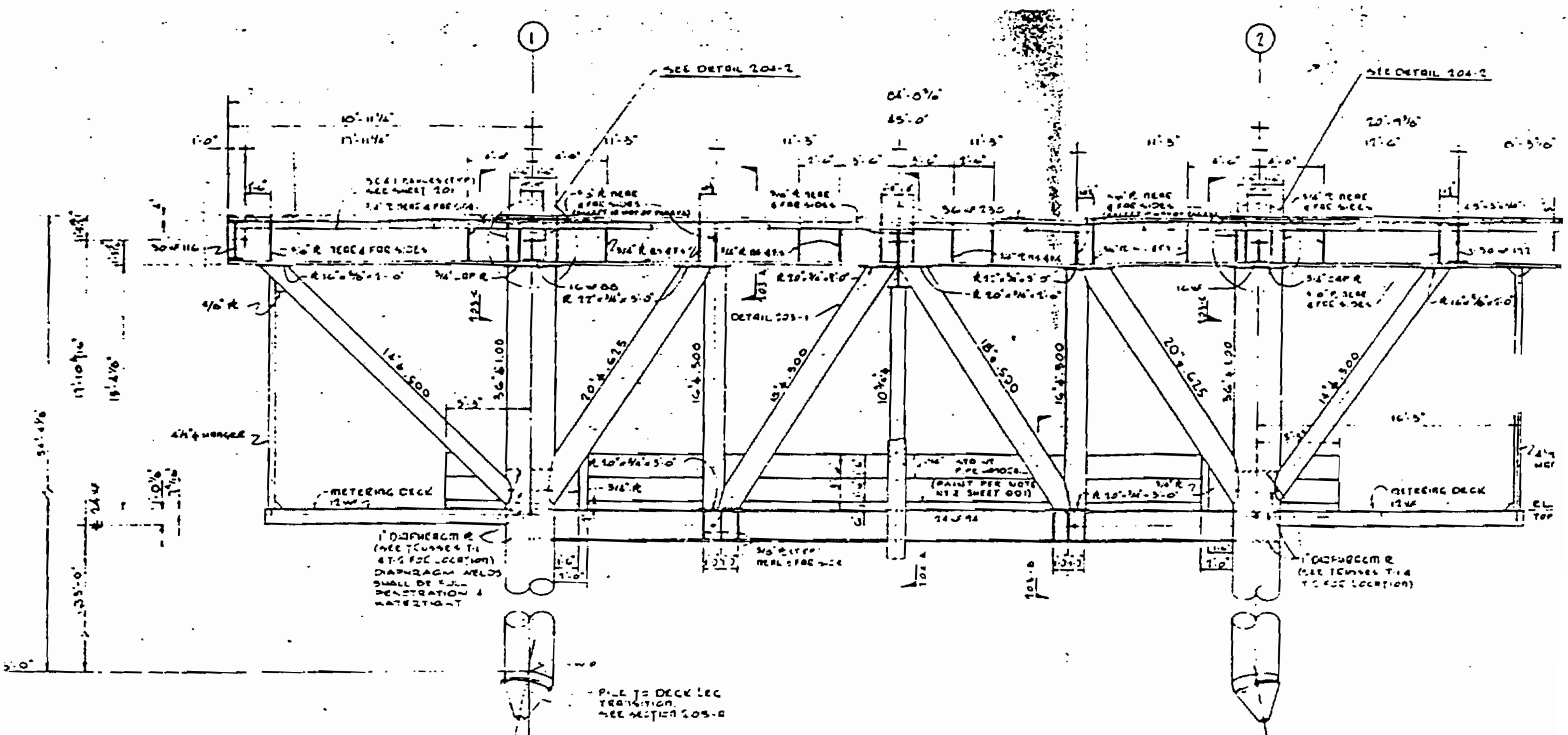


FRAMING PLAN AT ELEV. (-) 160'-0"

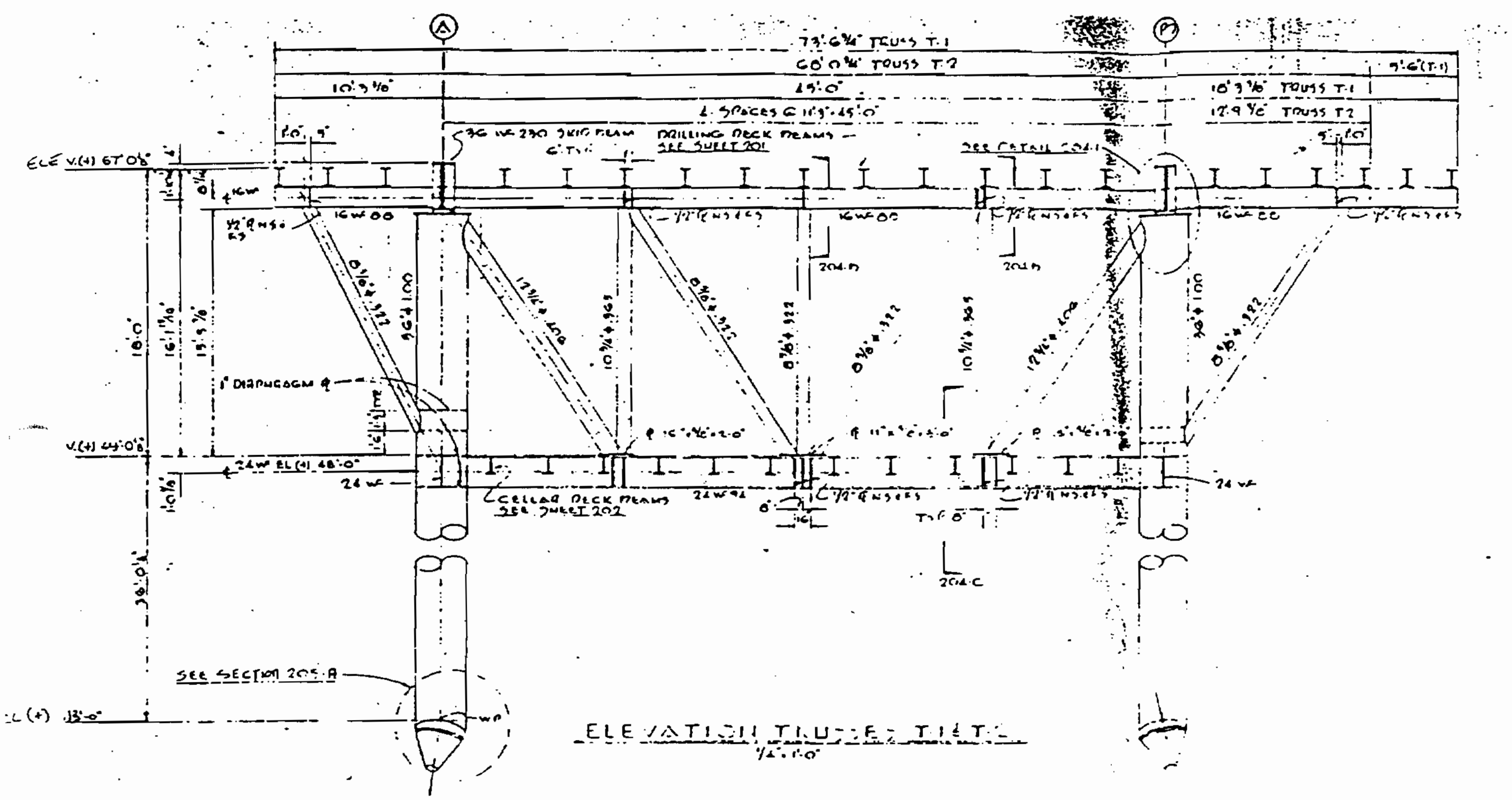
1/8" = 1'-0"



PILE DETAIL 4 REQ'D.
1" = 30'-0"



ELEVATION • TRUSSES T-A & T-B


73'-6¾" TRUSS T-1
60'-0¾" TRUSS T-2
10'-3 ⅜"
45'-0"
10'-3 ⅜" TRUSS T-1
12'-9 ⅜" TRUSS T-2
4 SPACES @ 11'-3" = 45'-0"
36 WF 230 SKID BEAM
DRILLING DECK BEAMS
SEE SHEET 201
SEE DETAIL
16 WF 88
204-B
1" DIAPHRAGM
24 WF
CELLAR DECK BEAMS
SEE SHEET 202
204-C
SEE SECTION 205-A
18'-0"
24 WF EL (+) 48'-0"
ELE V. (+) 67'-0"

PART A: Platform Assessment

A.1 Platform Selection

The platform requalification has been initiated by the proposed addition of compressor equipment on the deck and the plan to make it a fully manned platform.

A.2 Condition Assessment

The platform structural information has been obtained from detailed design drawings available. It is known that Level 1 and Level 2 underwater surveys were conducted. It has been concluded from the survey that there has been no damage to any member. The detailed survey reports were, however, not made available to us. The soil information has been assumed as mentioned in Section 1.1.3.

A.3 Categorization

Since the platform is located in the Gulf of Mexico, it has been categorized as a "Manned, Evacuated" from life safety considerations.

The details of the safety features of the production facilities as well as gas properties are unknown to us. In view of this, no detailed environmental assessment could be performed. Therefore, the exposure category for environmental impact consideration has been conservatively assumed as "Significant Environmental Impact".

A.4 Design Basis Checks

The platform was designed prior to 1977, also the original design details are not available. Furthermore, additional equipment are proposed to be installed on the platform. Therefore, design basis check was not relevant.

A.5 Analysis Checks

Initially design level check was performed on the platform. It was found that the platform did not pass the requirements. Therefore, ultimate strength analysis was also performed. In both the analyses, gravity loads and environmental loads including wind, wave and current have been considered. The details of the analysis are reported here.

A.5.1 Metocean Criteria

The Metocean criteria for the platform analysis has been obtained from Table 17.6.2-1 and Figure 17.6.2-2a of the API RP 2A - WSD, Section 17 (Draft).

The criteria are summarized below:

	Design Level Analysis	Ultimate Strength Analysis
Water depth	160 ft + 3 ft tide	160 ft + 3 ft tide
Wave height	55 ft	67.5 ft
Wave period	12.1 sec	13.5 sec
Wind	65 knots	85 knots
Current	1.6 knots	2.3 knots
Wave kinematic factor	0.88	0.88

Stream Function wave theory has been used for the wave kinematics. Current stretching and Doppler shift effects have been included in accordance with 20th edition.

For the ultimate level wave height, the wave crest level including tide is determined as 46 ft. The bottom of the cellar deck being at 47 ft, the deck height criteria is met. Therefore, there would be no wave loading on the deck for the ultimate condition.

The marine growth profile has been assumed as recommended in API RP 2A, 20th edition, a uniform thickness of 1.5 inch from 10 ft above mudline to 1 ft above MLLW.

The hydrodynamic coefficients assumed are:

	C_d	C_m
Smooth	0.65	1.60
Rough	1.05	1.20

Wind areas for the deck have been input for wind force calculations. Drag loads for boat landing and barge bunpers have been calculated based on equivalent block areas.

A.5.2 Screening

This is not relevant for the present platform.

A.5.3 Design Level Analysis

Since the platform is fairly symmetric from geometry and deck loading stand point, it has been analyzed for three wave approach directions. For the design level analysis, the 55 ft wave is omnidirectional. Therefore, two orthogonal and one diagonal direction have been analyzed. The selected load conditions analyzed are:

Load Combination 6 :	Gravity
Load Combination 7 :	Gravity + 90 degree wave
Load Combination 8 :	Gravity +135 degree wave
Load Combination 9 :	Gravity +180 degree wave

The platform has an on-bottom weight of 2041 kips including piles in the jacket legs and conductors up to the mudline. The weight break down is as follows:

Computed weight of platform in air	=	1256 kips
Input deck self weight	=	277 kips
Equipment weight in main deck	=	416 kips
Equipment weight in cellar deck	=	548 kips

For environmental loads, the maximum base shears are given below:

		Fx	Fy	Resultant
LC7:	90 degree wave:	0.	1460 k	1460 k
LC8:	135 degree wave:	-1011 k	1025 k	1440 k
LC9:	180 degree wave:	-1438 k	0.	1438 k

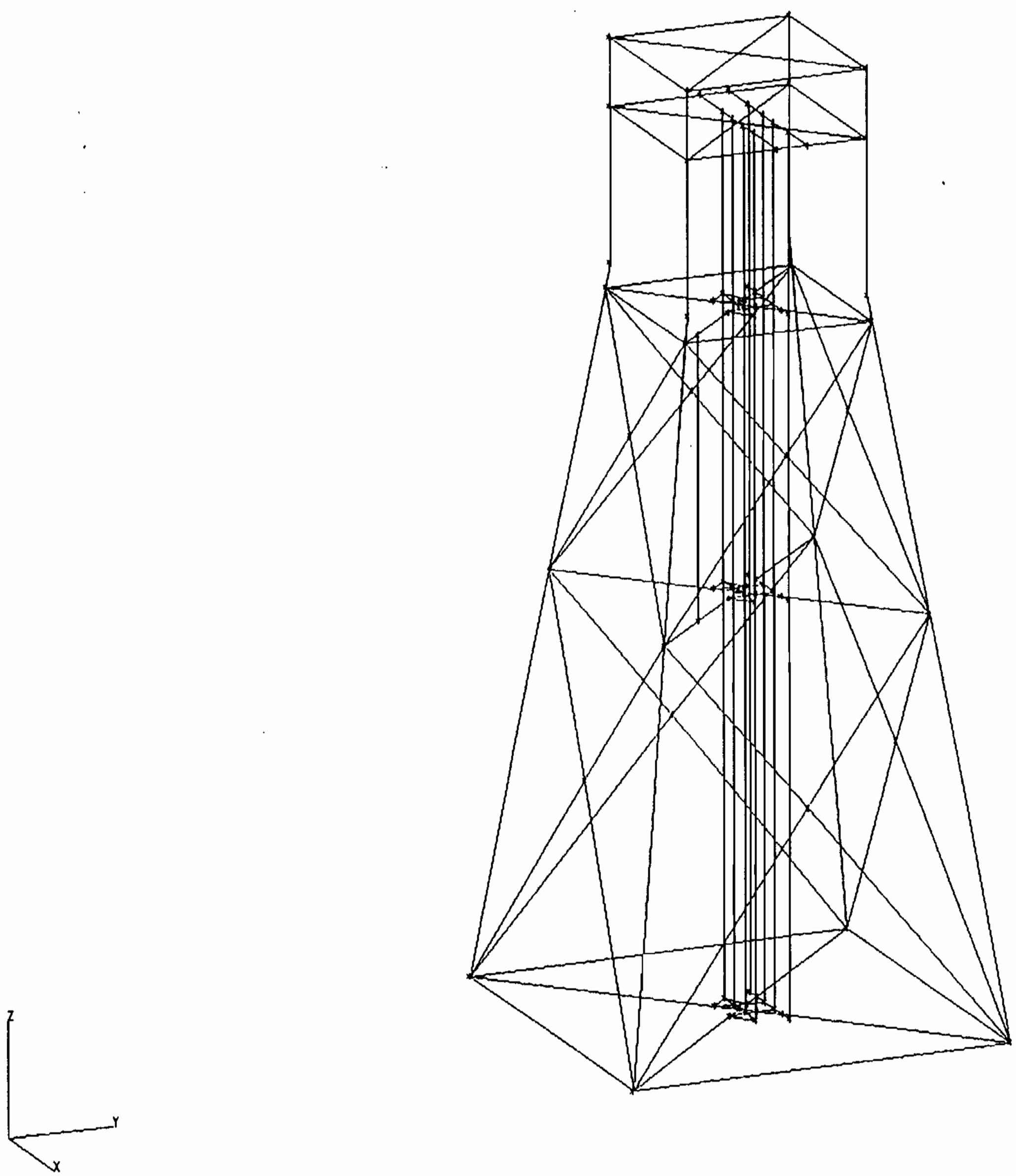
All the major structural elements have been modelled. The piles have been modelled as ungrouted and conductors below the mudline have been modelled as piles. Conductors have been assumed to have only lateral supports from the jacket. K-factors have been assumed as recommended in API RP 2A. Allowable stresses have been increased by 33% as per API RP 2A. Non-linear soil-structure interaction effects have been included.

The results of the linear analysis are summarized in plots at the end of this section. The analysis shows that some of the major diagonal members in the vertical framing of the jacket have failed to pass the screening test with a maximum unity ratio of 1.29. Therefore, an ultimate strength analysis is necessary.

PLATFORM L - TRIAL BASIS ANALYSIS

3DVIEW

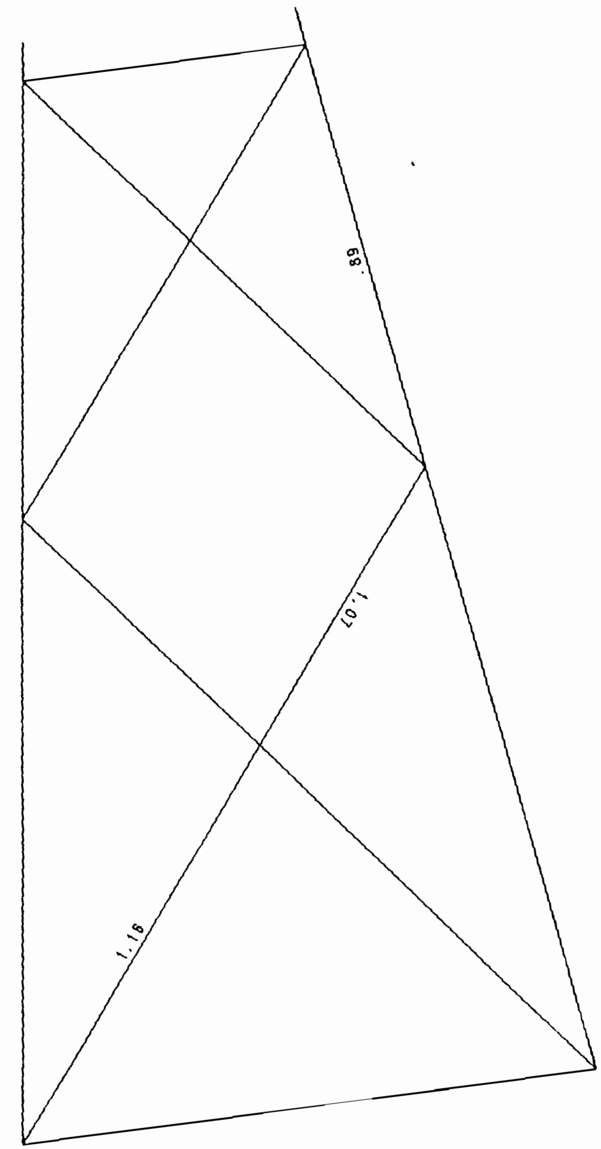
MODEL



PLATFORM L - TRIAL BASIS ANALYSIS

CRITICAL UC

ROW A

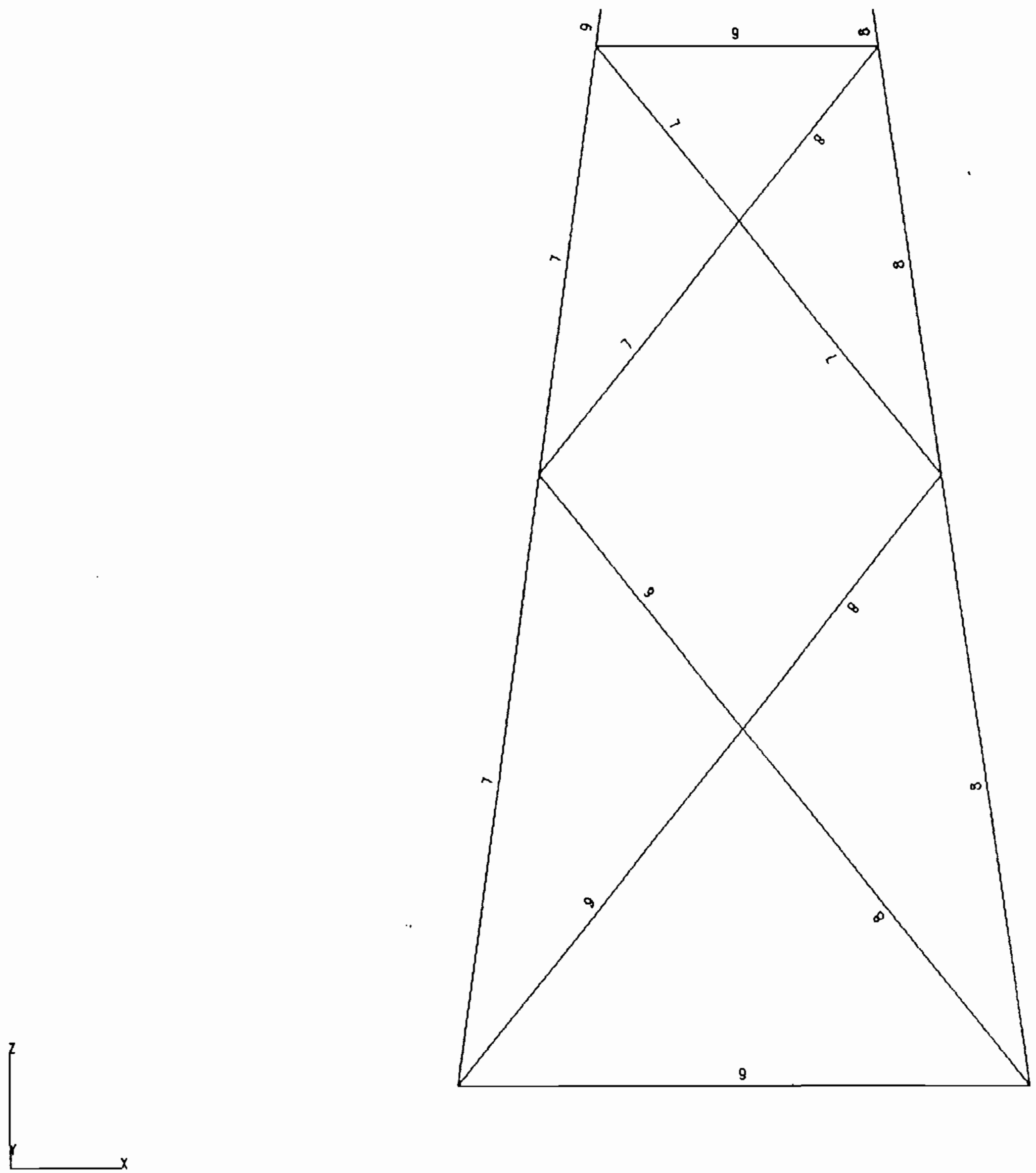


STRUCTURAL SOFTWARE, INC

PLATFORM L - TRIAL BASIS ANALYSIS

ROW A

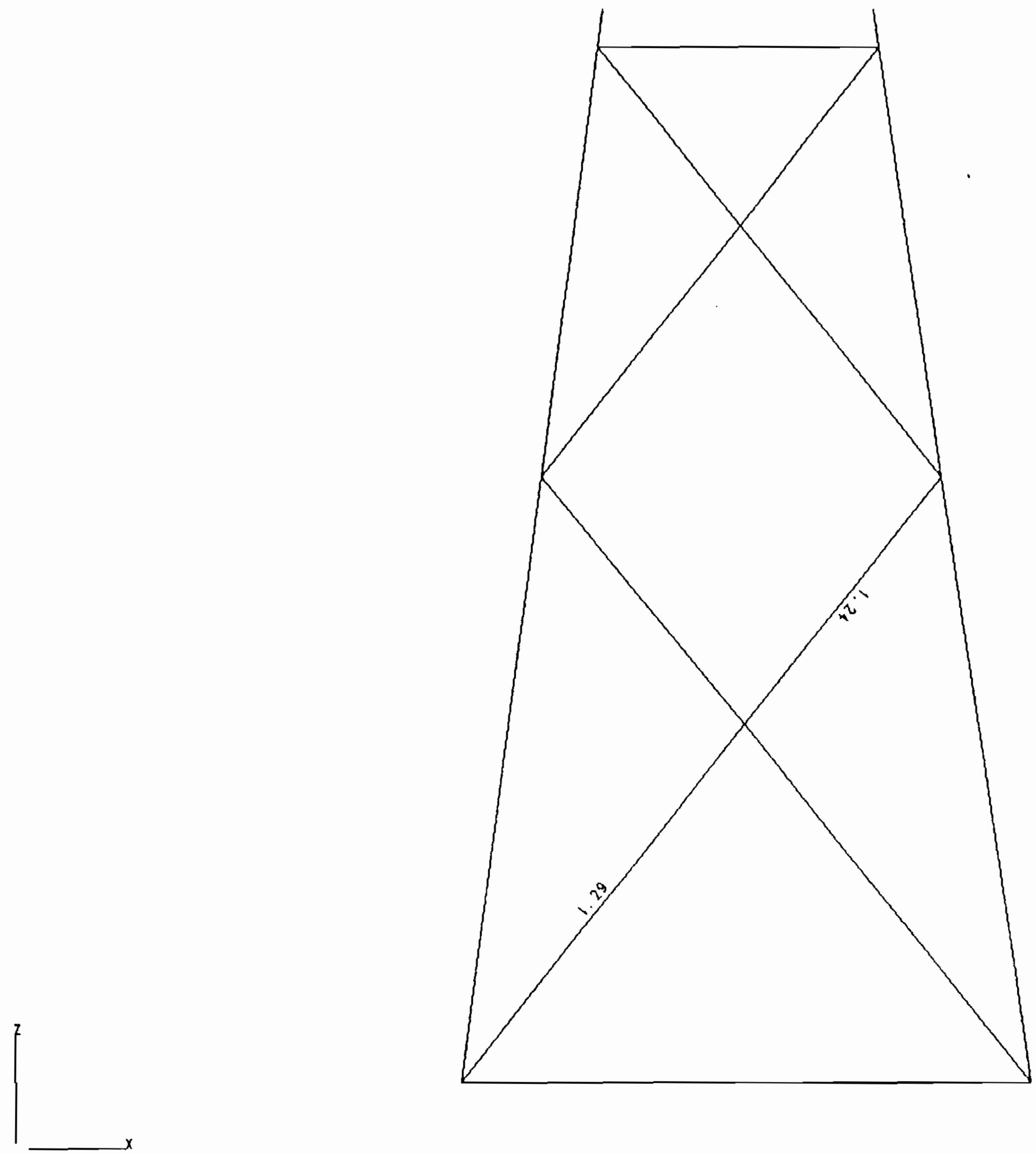
CRITICAL LC



PLATFORM L - TRIAL BASIS ANALYSIS

ROW B

CRITICAL UC



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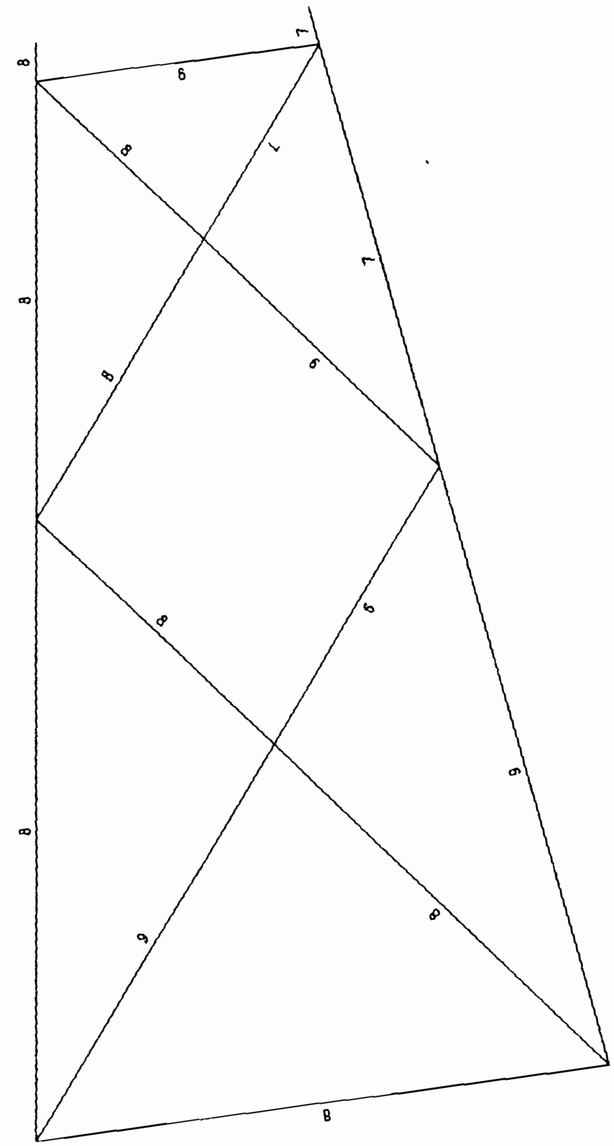
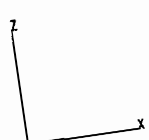
PLATFORM L - TRIAL BASIS ANALYSIS

CRITICAL LC

ROW B





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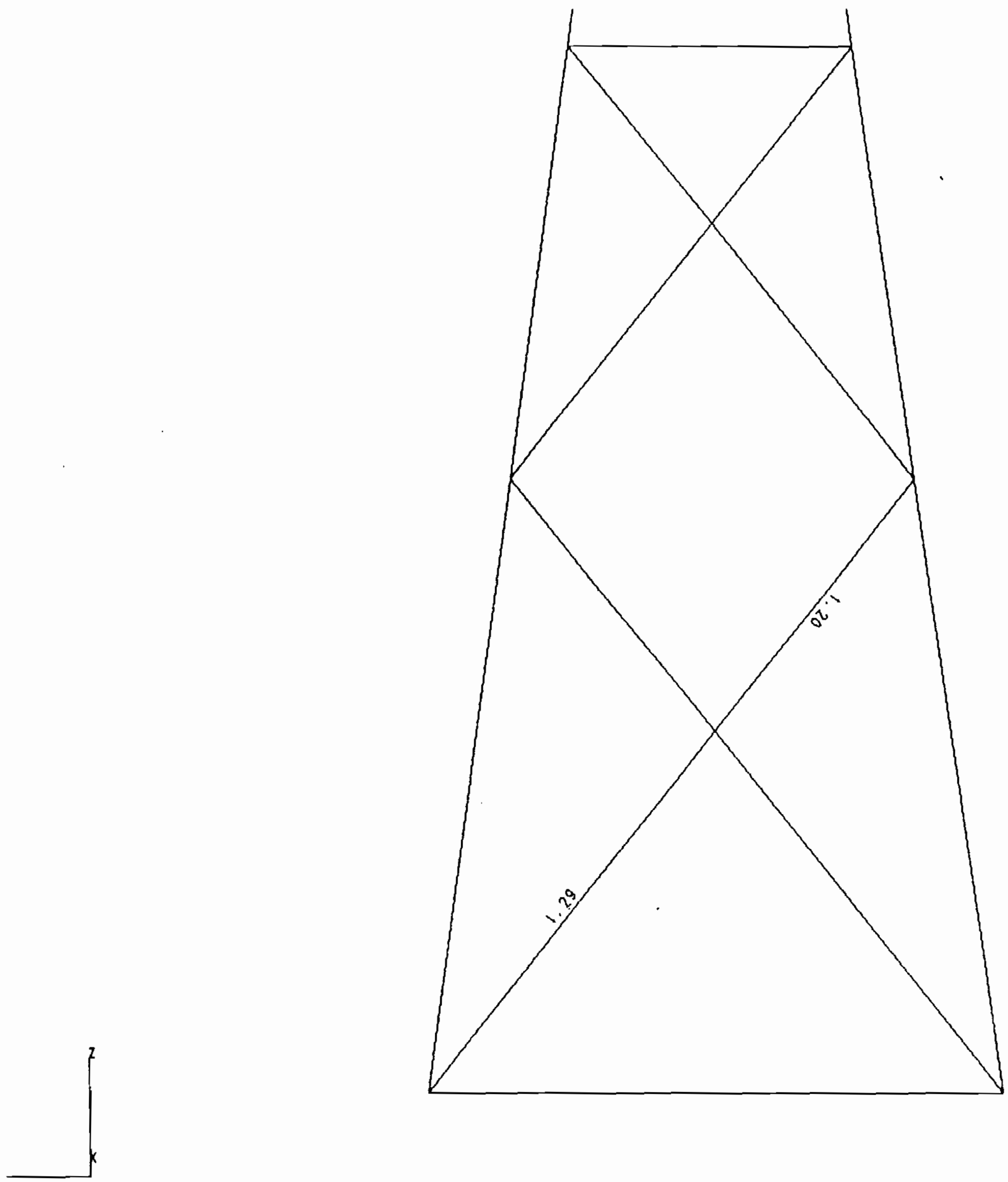
PLATFORM L - TRIAL BASIS ANALYSIS

ROW 1

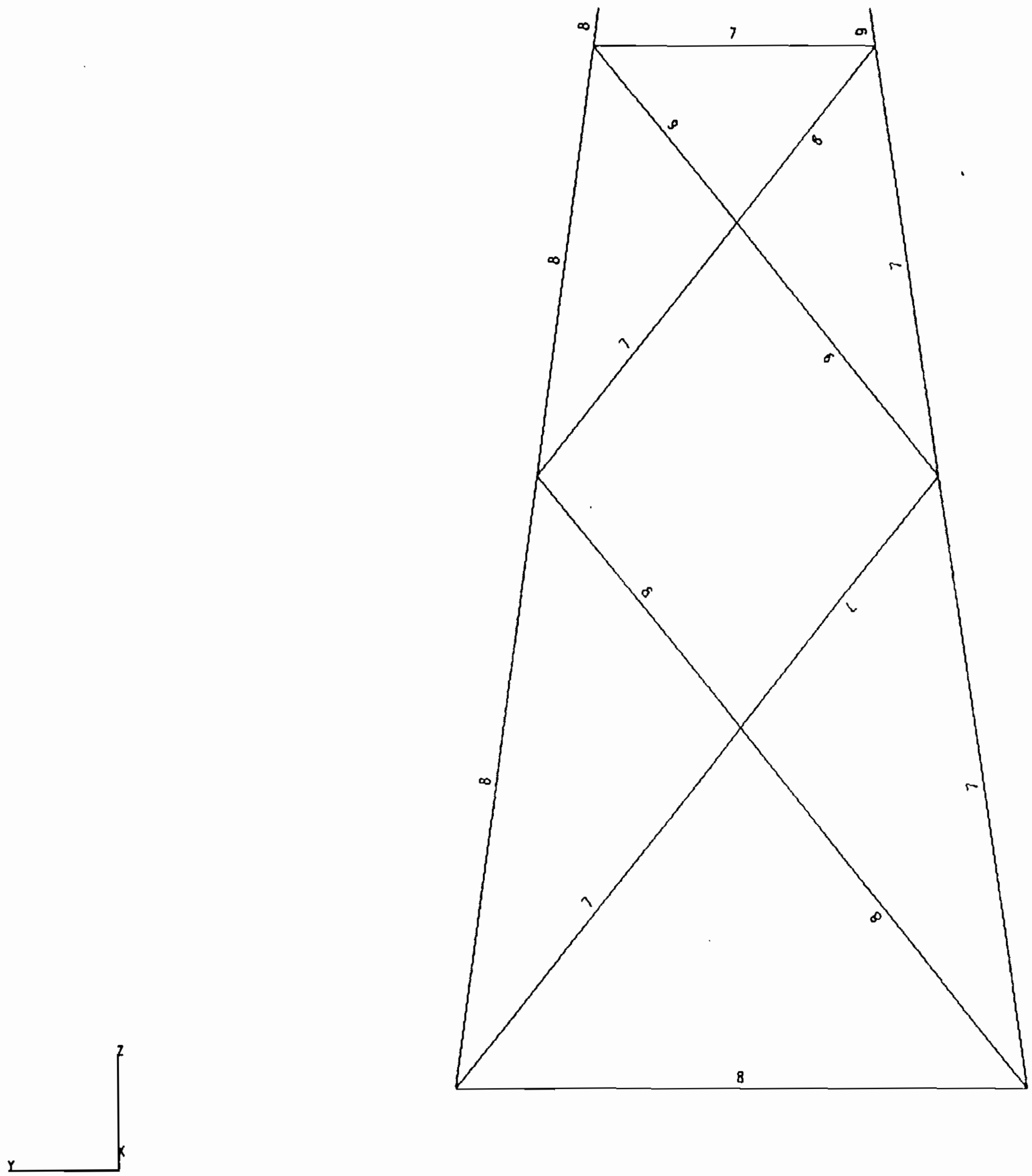
CRITICAL UC



PLATFORM L - TRIAL BASIS ANALYSIS

ROW 1

CRITICAL LC



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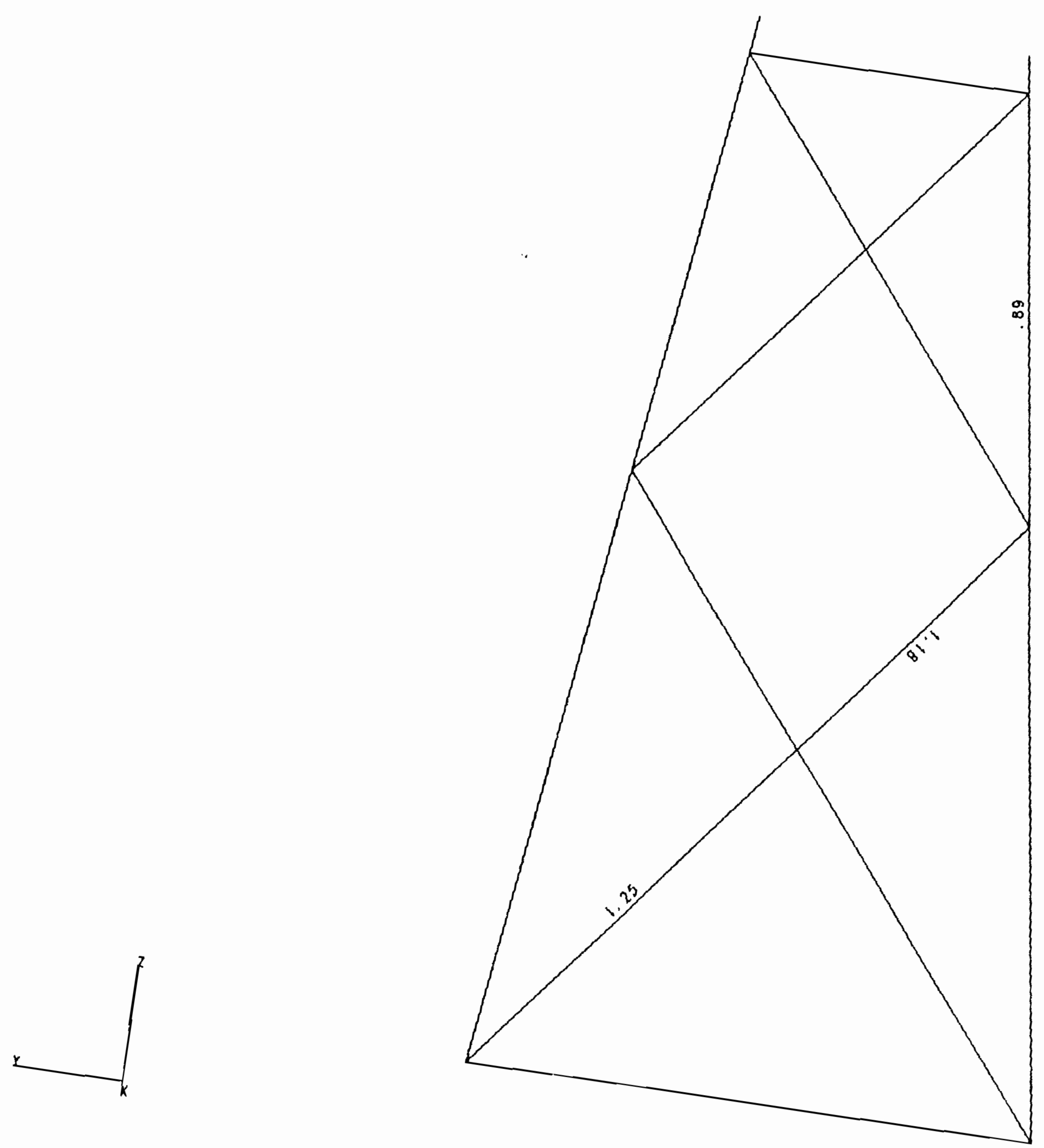
ROW 2

PLATFORM L - TRIAL BASIS ANALYSIS

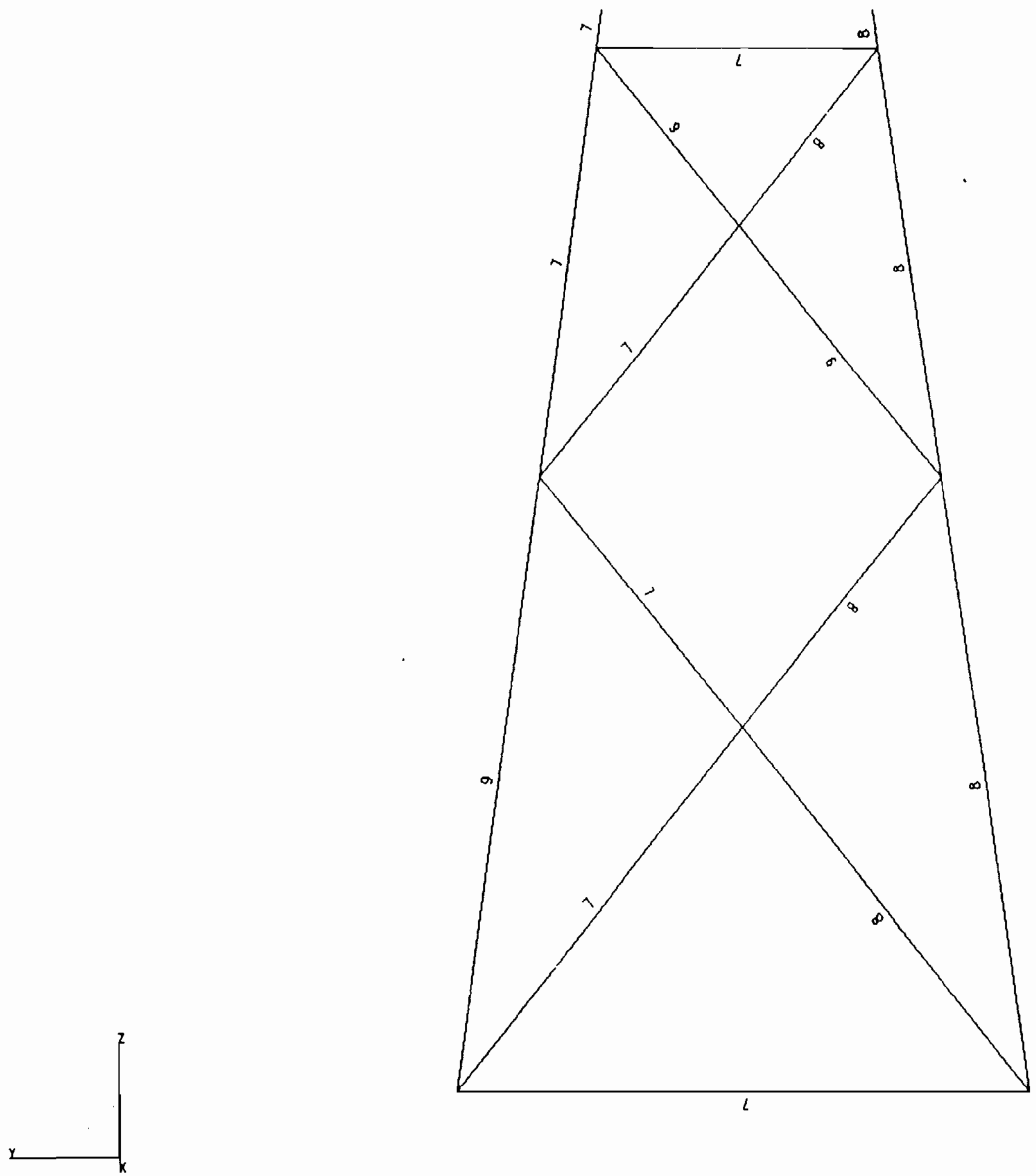
CRITICAL UC



PLATFORM L - TRIAL BASIS ANALYSIS

ROW 2

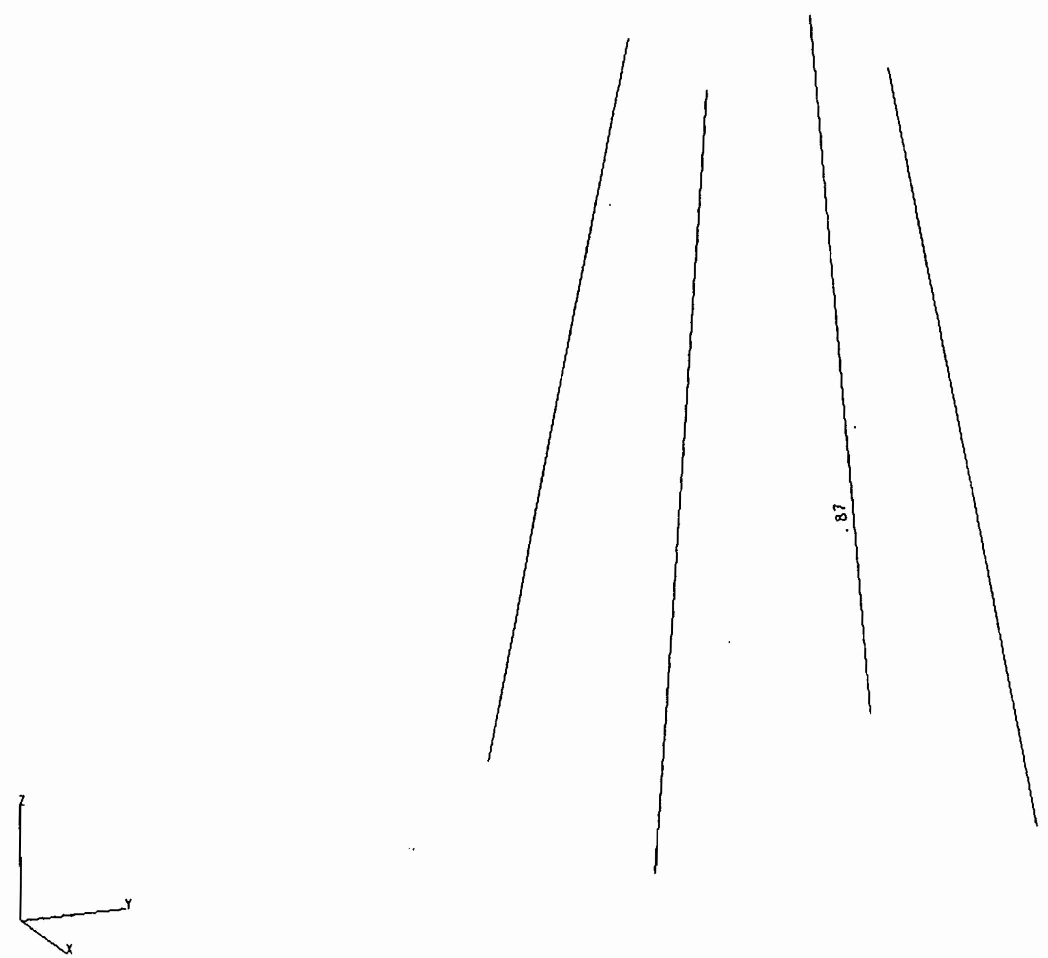
CRITICAL LC



PLATFORM L - TRIAL BASIS ANALYSIS

PILES ABOVE MUDLINE - DESIGN LEVEL

CRITICAL UC



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A.5.4 Ultimate Strength Analysis

A push-over analysis method has been used for the ultimate strength. The platform orientation indicates that one of its diagonal directions is almost parallel to the predominant wave direction as given in Figure 2.3.4-4 of API RP 2A. Therefore, for 135 degree approach, a full wave height of 67.5 ft has been used. For 90 and 180 degree directions, wave heights of 64.125 ft and 60.75 ft have been used. The current speed also has been scaled in the same proportion. Wind velocity remains the same for all directions. For the three wave directions the base shears are:

		Fx	Fy	Resultant
LC7:	90 degree wave:	0.	2358 k	2358 k
LC8:	135 degree wave:	-1824 k	1848 k	2596 k
LC9:	180 degree wave:	-2072 k	0.	2072 k

The push-over analysis has been performed using a linear elastic analysis program but sequentially updating the member stiffness to account for member failure. The procedure is indicated below:

(1) Initial linear analysis for the full ultimate wave load had demonstrated that there would be several member failures. Therefore, the wave load has been scaled down and combined with full gravity load to determine at what level the first member failure occurs.

(2) Member failure has been defined when the unity check according to API code check formulae is equal to 1.0 with all safety factors removed. A stress increase factor of 1.67 has been used for this purpose.

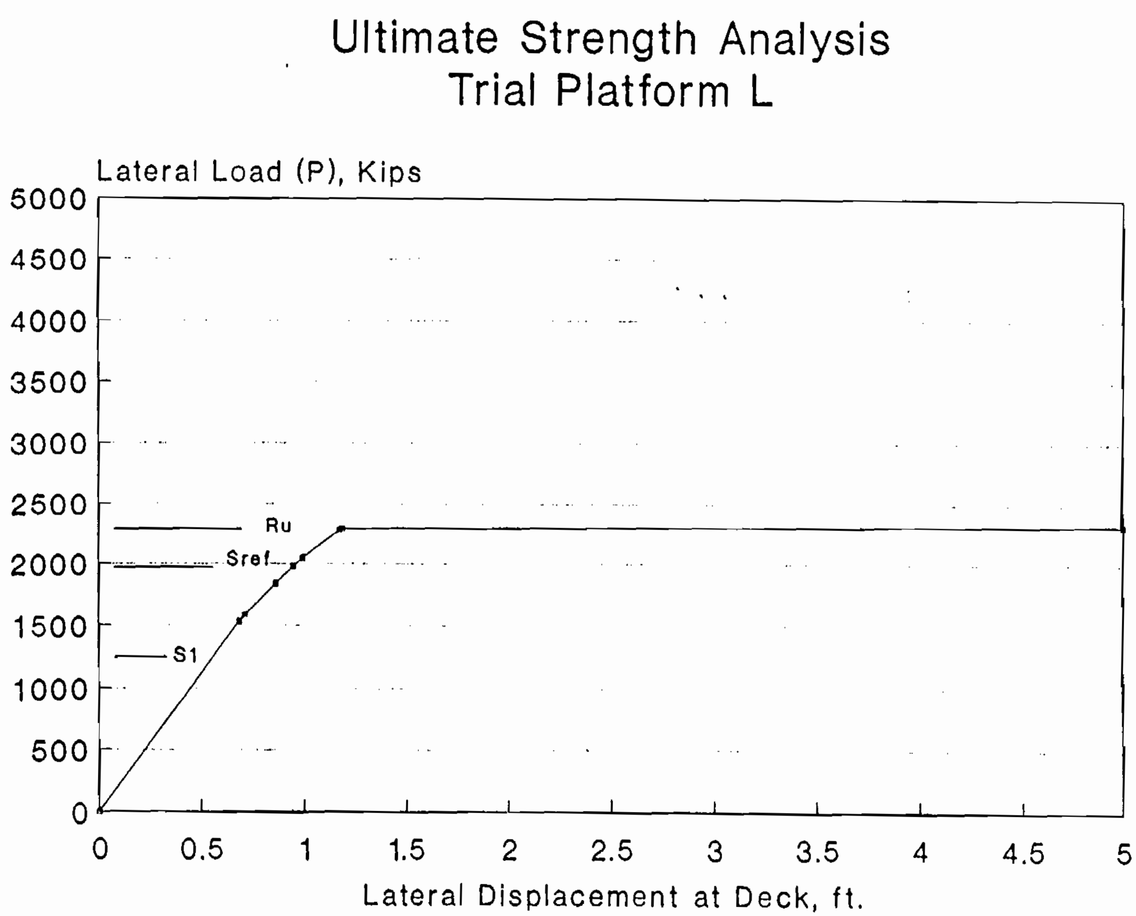
(3) When a member has failed, the member has been replaced by its internal forces and moments at the two ends and the member stiffness reduced to zero. This is equivalent to the assumption that after initial member failure, its load carrying capacity is held constant. In reality the member may still have additional strength, if it strain hardens, or it may unload, if it buckles.

(4) In the subsequent analysis load step, the proportion of wave load is increased slightly and the modified structure with the stiffness of the failed member removed, is analyzed. The load is increased till further member failure occurs.

(5) P-delta effect has been included. A joint check has been performed at each stage. In case a major jacket joint failed, the corresponding member load was limited to that load.

(6) This process is continued till the maximum loading for the structure is reached.

Load cases 7 and 8 have been analyzed for ultimate load. The load deflection plots are shown in the following figures and tables.



Load at First member I.R = 1.0, S_1	=	1226 kips
Reference Level Load, S_{ref}	=	1966 kips
Ultimate Capacity, R_u	=	2321 kips
Reserve Strength Ratio, RSR	=	1.18
Platform Failure Mode:		Jacket collapse

Figure 1: Load - Displacement Rc 's for 90 degrees w.r.t x-axis

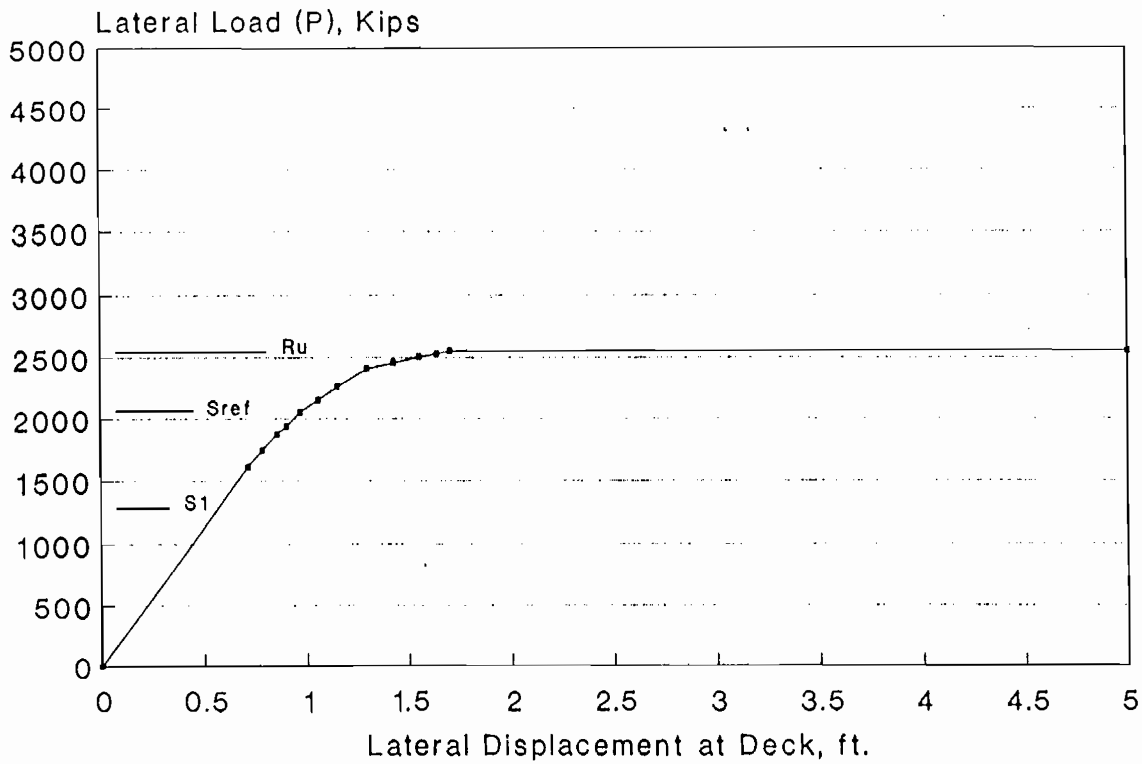
Direction 1 : Analysis with Foundation

Lateral load level for first member with unity check = 1.0 # 1226 Kips

Load Step	Lateral Displacement at Deck Level (+65') ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.69	1533	36	Buckling	
2	0.71	1580	16	Buckling	
3	0.86	1836	35	Buckling	
4	0.95	1975	15	Buckling	
5	0.99	2045	141	Buckling	
6	1.17	2275	37,139	Yielding & Buckling	
7	1.19	2287	38	Yielding	
8	7.01	2321	ALL	Jacket Collapse	
9					
10					

Table 1: Ultimate Strength Analysis for 90 degrees to x–axis

Ultimate Strength Analysis
Trial Platform L



Load at First member I.R = 1.0, S_1	=	1298 kips
Reference Level Load, S_{ref}	=	2160 kips
Ultimate Capacity, R_u	=	2556 kips
Reserve Strength Ratio, RSR	=	1.18
Platform Failure Mode		Jacket collapse

Figure 2: Load - Displacement Results for 135 degrees w.r.t x-axis

Direction 2 : Analysis with Foundation

Lateral load level for first member with unity check = 1.0 # 1298 Kips

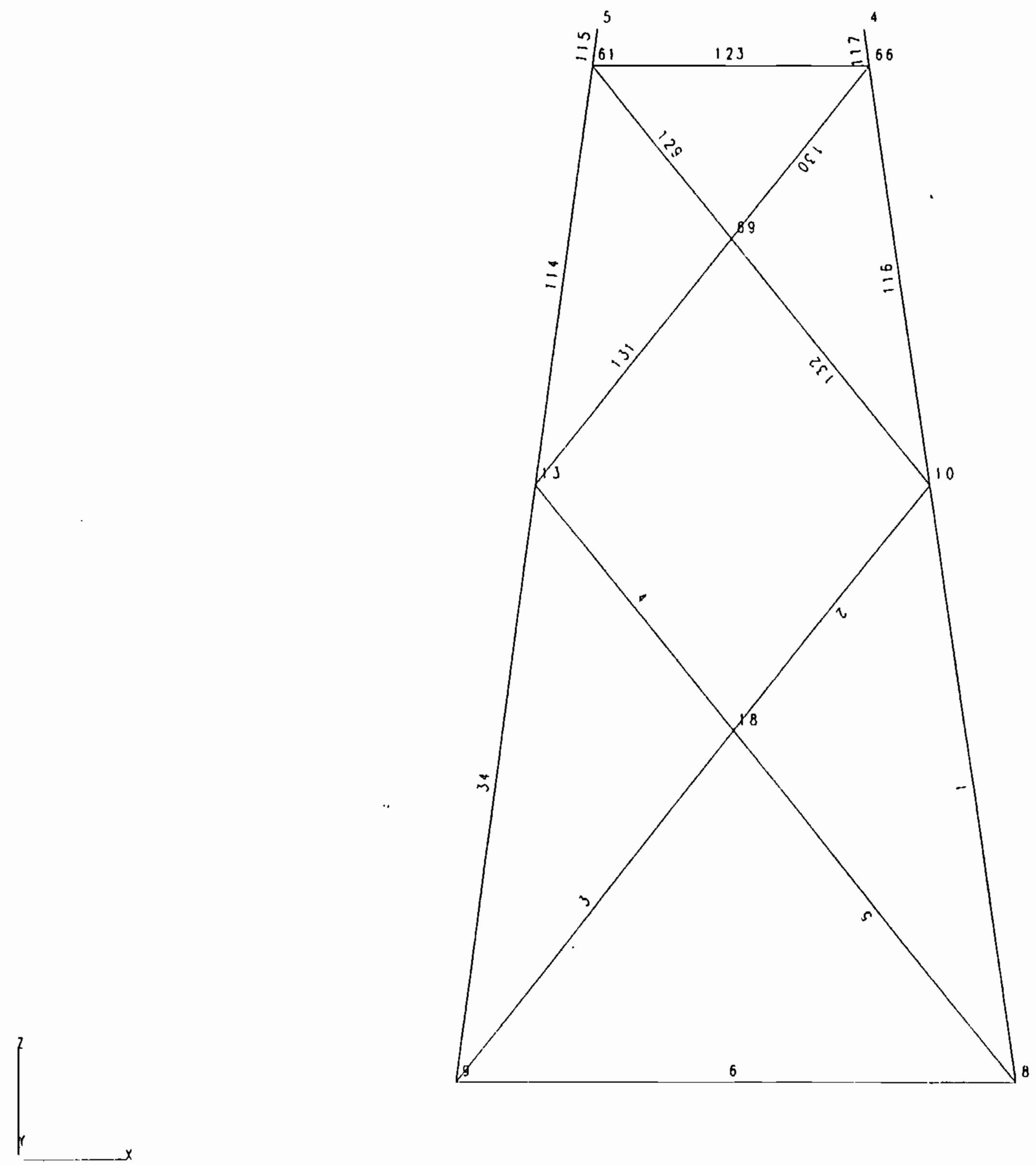
Load Step	Lateral Displacement at Deck Level (+65') ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
1	0.71	1609	15	Buckling	
2	0.78	1740	2	Buckling	
3	0.86	1870	36	Buckling	
4	0.90	1935	27	Buckling	
5	0.97	2051	24	Yielding	
6	1.06	2153	141	Buckling	
7	1.15	2258	131	Buckling	
8	1.30	2407	37, 14	Yielding	
9	1.43	2456	35	Buckling	
10	1.56	2504	5	Yielding	Pile Failure
11	1.64	2528	16, 4	Buckling & Yielding	Conductor Failure
12	1.70	2552	140, 34, 39, 13, 3, 6	Buckling & Yielding	Leg Failure
13	2359.83	2556	ALL	Pile & Jacket Collapse	

Table 2: Ultimate Strength Analysis for 135 degrees to x−axis

TRIAL PLATFORM L

ROW A

JNT # & ELM #



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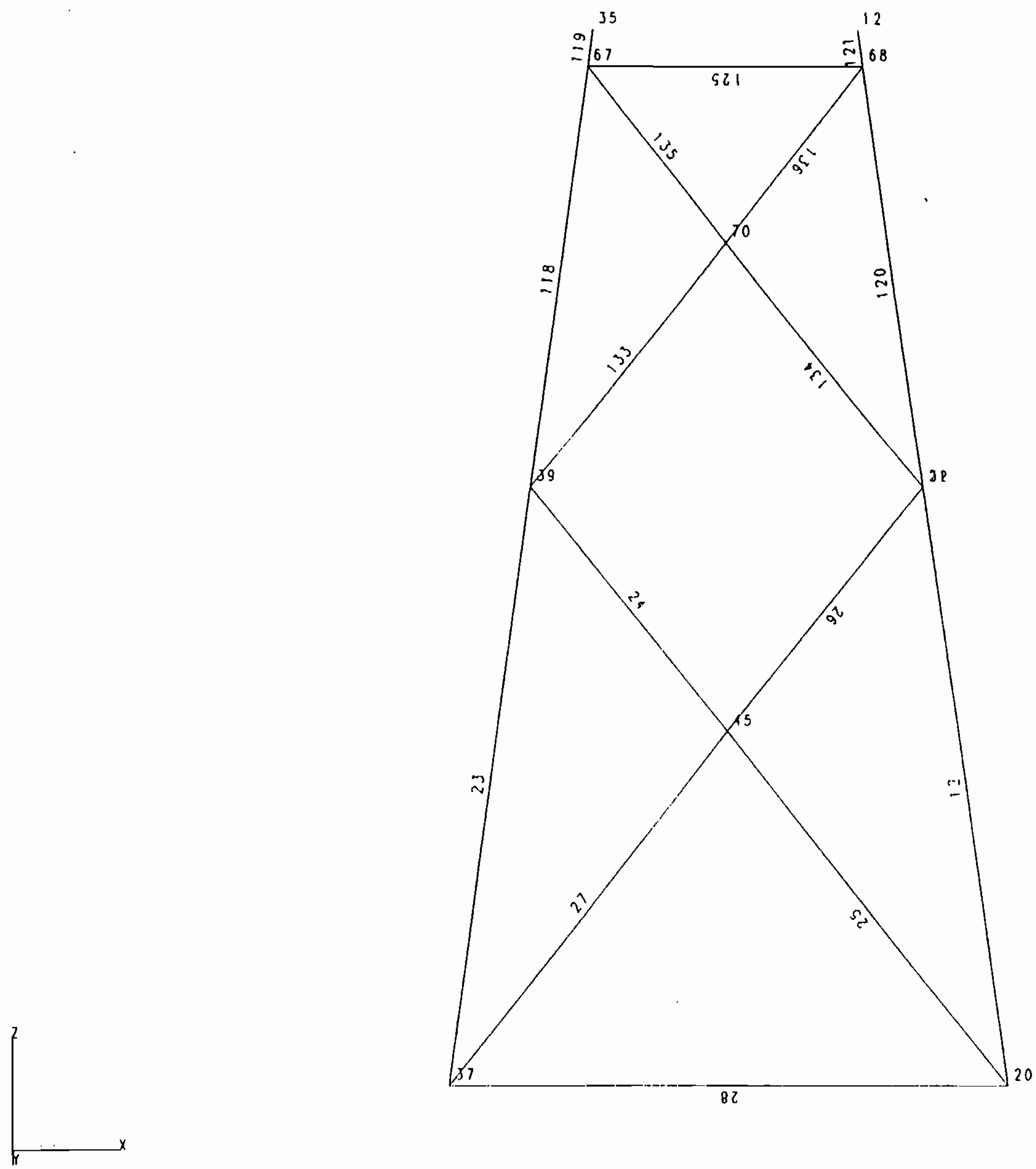
TRIAL PLATFORM L

ROW B

JNT # & ELM #



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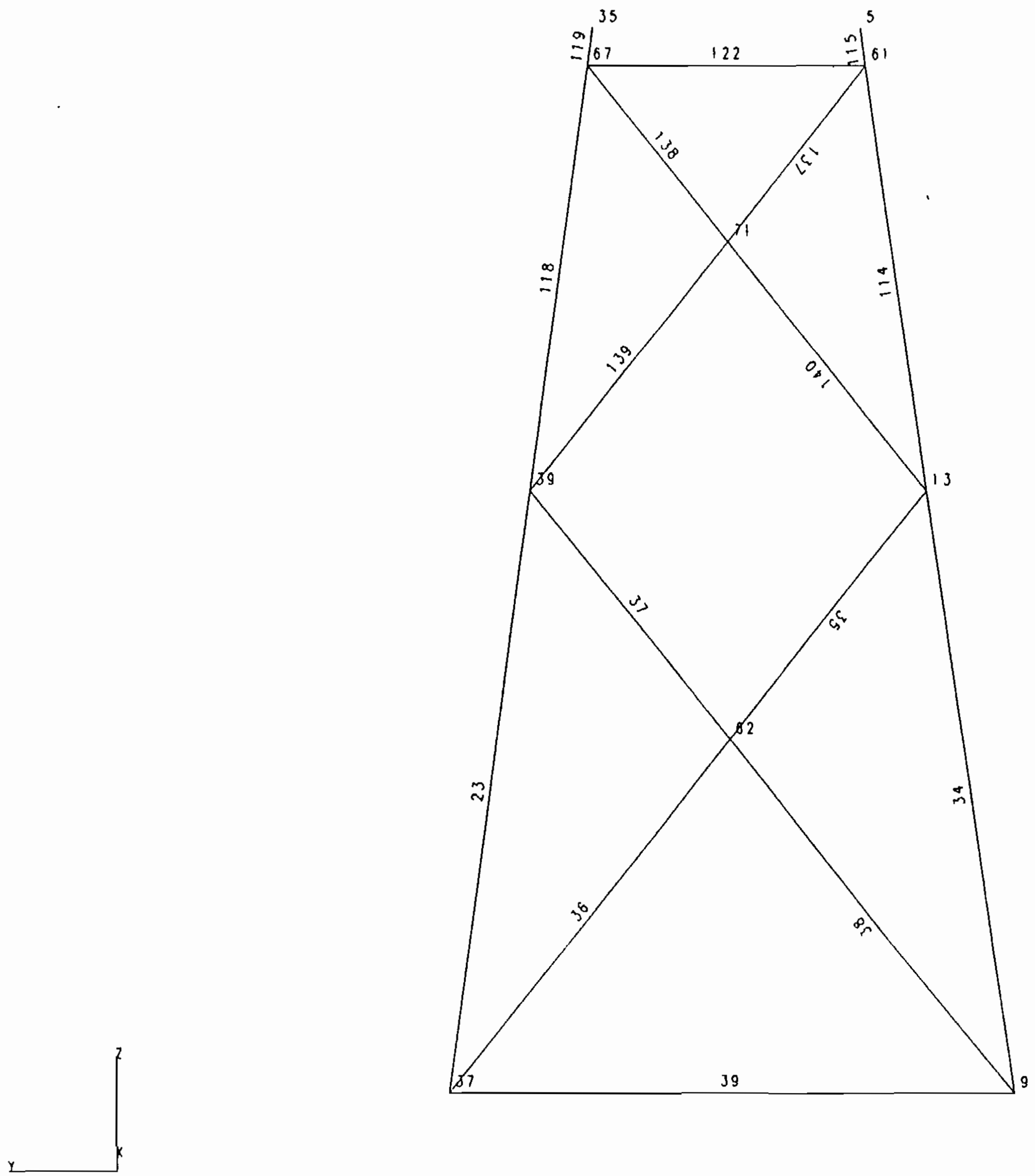
TRIAL PLATFORM L

ROW 1

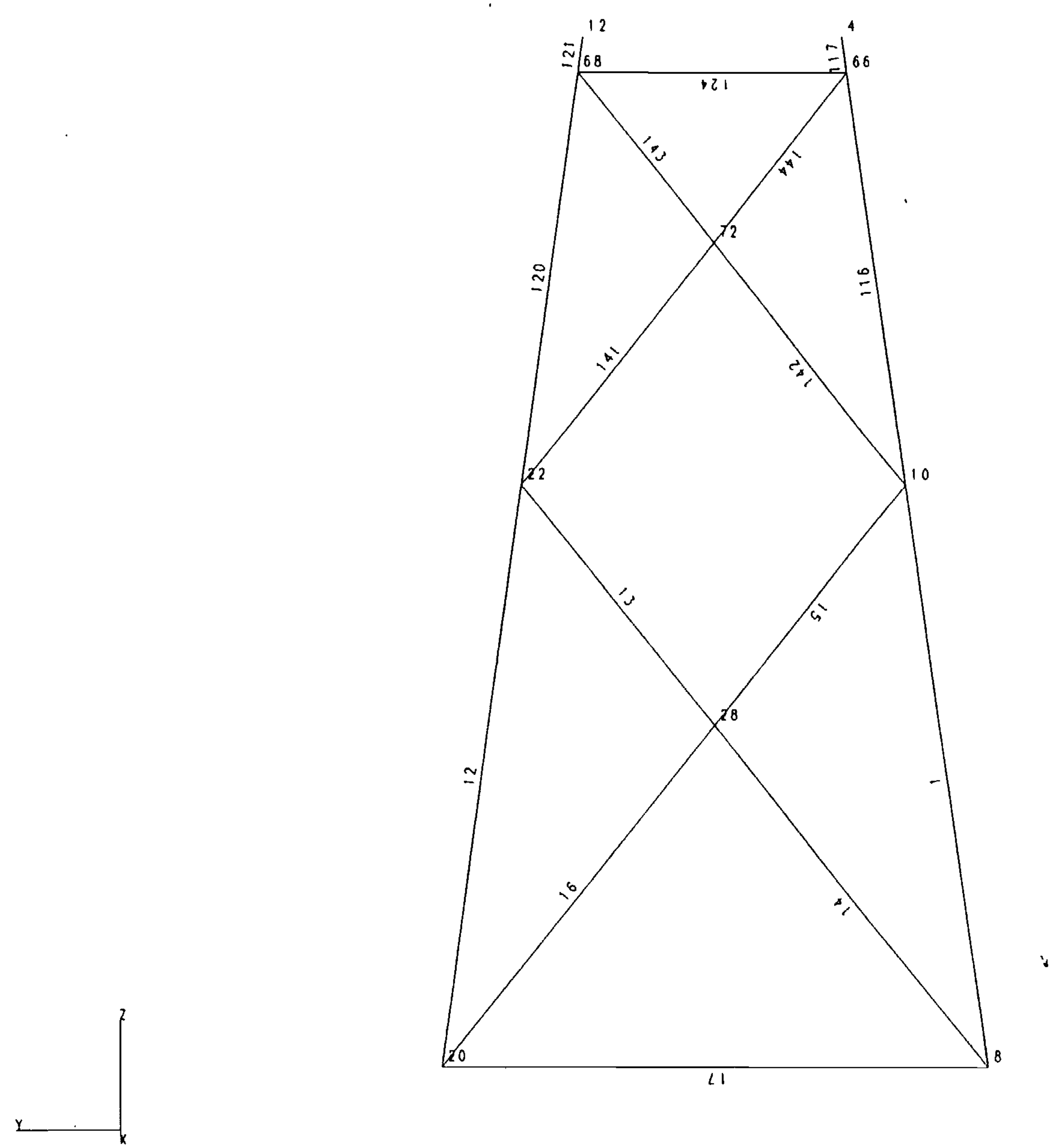
JNT # & ELM #



TRIAL PLATFORM L

ROW 2

JNT # & ELM #



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The above results have shown that the platform almost reached the ultimate capacity demand before collapse. The following percentages of ultimate capacity to load for ultimate wave have been obtained:

Direction 1, 90 degrees	98%
Direction 2, 135 degrees	98%

Therefore, it is our opinion that for all intents and purposes, the platform meets the ultimate criteria.

A.6 Consideration of Mitigant

In case a strict compliance to the Section 17 is demanded, mitigation alternatives need to be investigated. As we had mentioned earlier, the real environmental hazard for the platform failure could not be ascertained for lack of data. It is proposed that as a first step to mitigation, a closer look at the platform hydrocarbon safety features be investigated. If this indicates that there are enough safeguards for hydrocarbon spillage in case of a collapse, we can reclassify the platform as "Insignificant Environmental Impact".

For this case, the required ultimate strength environmental parameters are governed by sudden hurricanes which are much less onerous:

Wave Height H	=	56 ft
Wave Period	=	12.5 sec
Current	=	1.8 knots
Wind	=	70 knots

The principal direction is 90 degrees followed by 45 and 135 degrees. Ultimate strength analyses were performed for 90 and 135 degree directions. For the principal direction of sudden hurricane conditions, the maximum base shear is 1700 kips The ultimate load capacity of the platform, 2300-2550 kips, being larger than this load, the platform will pass the assessment for manned, evacuated / insignificant environmental impact categorization.

A.7 Summary Note - Part A

Name of Platform:	Platform L
Year of Installation	1970
Water depth	160 ft
Location	Gulf of Mexico, East Cameron
Platform Type	4 pile, X-braced
Number of wells	8
Requalification trigger	Addition of equipment, permanent manning
Condition assessment	Drawings available, no damage
Categorization	Manned Evacuated / Significant Environmental Impact

Design Basis Check — Not performed

Design Level Analysis

Metocean Criteria
Wave H=55 ft, T=12.1 sec
Current=1.6 knots, wind =65 knots
Maximum base shear=1440 k
Results : Platform did not pass

Ultimate Strength Analysis

Metocean Criteria
Wave H=67.5 ft, T=13.5 sec
Current=2.3 knots, wind =85 knots
Directionality as per API RP 2A

Direction 1
90 degrees
Maximum base shear=2358 kips
Load at which First Component reaches I.R of 1.0, S_1 = 1226 kips
Reference Level Load (API 20th edition), S_{ref} = 1966 kips
Ultimate Capacity, R_u = 2321 kips
Reserve Strength Ratio = 1.18
Platform Failure Mode: Jacket collapse

Direction 2
135 degrees
Maximum base shear=2596 kips
Load at which First Component reaches I.R of 1.0, S_1 = 1298 kips
Reference Level Load (API 20th edition), S_{ref} = 2160 kips
Ultimate Capacity, R_u = 2556 kips
Reserve Strength Ratio = 1.18
Platform Failure Mode: Jacket collapse
Results: Platform practically met the requirement (98% of the load)

Consideration of Mitigant

In case strict compliance to the ultimate load level is required, we suggest, ways of reducing Environmental Impact should be investigated. In case it is concluded that the platform can be reclassified to have insignificant environmental impact, the platform passes.

PLATFORM "M"

1. SUMMARY OF PLATFORM INFORMATION

As-is Condition

The platform is in the Gulf of Mexico in 184 ft of water. It was originally designed and constructed in 1964 for a water depth of 196 ft and was finally installed at its current location (184 ft water) in 1968. During that period the joint were reinforced by gusset plates following the detail shown in Figure A5 of attachment A. Field measurements conducted in June of 1994 show that the bottom of the cellar deck is 38'11" above MLLW. During the installation process an attempt to push the bottom horizontal braces 12' into the mudline failed to reach its objectives and the bottom braces currently sit at 7'6" below the mudline (this was also checked in June of 94 by field measurements).

The platform has 14 producing wells, but no processing or storage. Oil produced from this platform is processed and stored on an adjacent structure. In case of any structural damage no large quantities of oil will be lost. The platform houses quarters which are evacuated in the case of a forecasted environmental condition.

The reader is referred to Section A.2 of this document for a detailed condition assessment of this structure.

Platform Sketches

Platform sketches showing platform orientation can be found in Attachment A. For example Figure A3 in attachment A shows platform orientation as well as critical loading directions. In Figure A6 the finite element model used in analyzing the structure is shown. Figures A8 - A16 of the same attachment show the vertical and horizontal framing of the structure. Note that elevations shown in these Figures do not agree with field measurements.

The structures foundation consists of 36" piles with wall thicknesses varying from 0.75" to 1.75". Actual penetration records are not available. Design penetrations which amount to 160 ft were used in the analysis.

PART A: PLATFORM ASSESSMENT

A.1 PLATFORM ASSESSMENT INITIATORS

This platform was selected because of its history of being design and constructed in one location and installed in a different location. Moreover, the number of wells on this structure where increased from 11 to 14 wells, the facilities on the decks was also increased.

On the other hand, this structure has quarters on it hence it will help in checking the manned-evacuated section of the draft document, and hence the sudden hurricane criteria will also be checked.

A.1.1 Addition of Personnel

The platform was initially designed to house quarters, and it still does.

This condition <u>will not</u> act as a trigger.

A.1.2 Addition of Facilities

No significant addition of facilities has occurred on this structure. The major change in facilities is the addition of a communication building and a radio building.

- Appurtenances on the structure were slightly increased on the Cellar Deck by adding a communication building and a radio building a total of 17 kips. (7.5% increase to Cellar deck loads, 1.1 % increase to Cellar and Main deck loads)

This is not considered a trigger for the assessment of the structure.

A.1.3 Increased loading on Structure

Following is the list of structural modifications that the structure underwent since its construction:

- The structure was designed and constructed in 1964 for a location which is in 198 ft of water, it stayed onshore until 1968 to be installed at the present location which is in 184 ft of water. Due to the difference in water depth, an attempt to push the structure 14 feet into the mud failed to reach its objectives and the structure's lowest horizontal braces ended up being 7 feet below mudline.

- During the period from 1964 to 1968 the joints were reinforced using gusset plates.

- The number of wells on the structure were increased from 11 in 1968 to 14 in 1987. (an increase of 12.1% to the wave diameter).

Assuming that the 12.1% increase in the wave diameter will increase the wave force on the structure by more than 10%. If this is considered a significant increase in the combined environmental/operational loading it will act as a trigger to assess the structure.

A.1.4 Inadequate Deck Height

According to Section A.1 and as previously mentioned in section A.1.1 of this report the platform is categorized as manned-evacuated. Hence according to section 17.6.2a (4b) and to Figure 17.6.2-3b the deck height needed for a platform in 184 ft of water is 36.5 ft. See Figure A1 in Attachment A.

Hence this condition <u>will not</u> act as a trigger for this structure since the lower deck elevation is 38' 11" above MLLW.

A.1.5 Damage Found During Inspections

Inspection results from the following surveys have indicated no significant damage

- 1993 level I survey

- 1992 level II survey

- 1987 an underwater inspection that included water blasting of the five most highly stressed nodes reported that "all specified nodes found no apparent surface discontinuities in either the heat affected zone or caps of all welds inspected."

Hence this condition <u>will not</u> act as a trigger for in assessing this structure.

A.2 CONDITION ASSESSMENT

A.2.1 General Information

a. Original and current owner: Original
Current

b. Original and current platform use and function: Drilling, production and quarters platform

c. Location, water depth and orientation: Gulf of Mexico, West Delta Block

Latitude:	28^{o} 55' N
Longitude:	89^{o} 40' W
Water depth:	184 ft
Orientation:	N 55^{o} W

d. Platform type: 8-Pile, K-braced in X direction, Diagonally braced in Y direction
Joints strengthened with gusset plates.

e. Number of wells, Risers, and Production Rate:

Number of wells:	14 (13-20" and 1-30")
Number of risers:	previously 1 (4"), currently none.
Production rate:	2277 BOPD; 2.2 MMSCFD

f. Other site specific information, manning level: 10

g. Performance during past environmental events: No known damage

A.2.2 Original Design

a. Design contractor and date of design: J. Ray McDermott Co. Inc. 1964.

b. Design drawings and material specification: Design drawings available but not the material specifications (all steel in the deck is ASTM A36, Jacket steel is unknown)

c. Design code: Pre-API; N/A

d. Environmental criteria: Unknown

e. Deck clearance elevation: 38'11" to the bottom of cellar deck

f. Operational criteria-Deck Loading and equipment arrangement:

Main Deck:

Table 1(a) Main Deck Loads

Description of Loads	(kips)
Wittaker Survival Capsule (28 man)	15
Radio tower, light water unit	10
30 man quarters building, helideck on top	700
Unit crane 20 ton with 60 ft boom	40
Recreation Room	10
Parts, Boxes etc.	10
Bridge Reaction	25
Hydraulic workover Unit	500
Subtotal Main Deck	1310

Cellar Deck:

Table 1(b) Cellar Deck Loads

Description of Loads	(kips)
Sewage treatment plant	10
Miscellaneous	16
Potable water Storage	175
Sump Tank	10
Subtotal Cellar Deck	211

g. Soil data: Geotechnical criteria used in the design is from a soil boring taken in 1963 at the adjacent West Delta XX platform. The geotechnical criteria was updated in 1987 by using another soil boring taken at an adjacent block for the West Delta YY platform. A boring from the adjacent WD ZZ #5 well was taken in early 1993. The results from this soil boring were not used in the assessment.

h. Number, size and design penetration of Piles and Conductors

Piles:
- Number: 8
- Size: 36 inch diameter, wall thickness varies 0.75 to 1.750
- Pile material: NOT AVAILABLE
- Design penetration: 160 ft

Conductors:
- Number: 14
- Size: See Table B1 in Attachment B
- Location: See Figure B1 Attachment B
- Design penetration: See Table B1 Attachment B

i. Appurtenances - list and location as designed:

A.2.3 Construction

a. Fabrication and installation contractor and date of installation

Fabrication contractor:	J. Ray McDermott Co. Inc.
Date of Fabrication:	1964
Installation contractor:	J. Ray McDermott Co. Inc.
Date of installation:	1968

b. "As - built" drawings

Not available.

c. Fabrication, welding, and construction specifications

Not available

d. Material traceability records

Not available

e. Pile and conductor driving records

Actual pile driving records not located

f. Pile grouting records

The BCI (1987) report points out that grouting records taken when the structure was installed do not clearly indicate that grouting of all jacket legs was successful.

No success in locating installation records; extent of grouting unknown

A.2.4. Platform History

a. Environmental loading history: No records of Significant loading instances.

b. Operational loading history: No record of significant loading (e.g., boat strikes)

c. Survey and maintenance records:

Records of surveys performed in 1971, 1987, 1988, 1992 and 1993 are available.

d. Repairs - Descriptions, analysis, drawings, and dates

No record of major repairs

e. Modifications - Description, analyses, drawings, dates

- Analysis done when a fourteenth conductor was added in 1987 are available.
- Analysis for the addition of the twelfth and thirteenth conductor are not available, however their location, size and penetration can be seen in Attachment B.
- The location, size, and weight of the communication and radio buildings are available. See Figure 3B of Attachment B

- A recreational room that was added to the quarters, is also shown in Figure 2B of Attachment B. The time at which this was added is not known however its weight is documented.

A.2.5. Present condition

a. All decks - Actual size, location, elevation

Main deck: Dimensions: 119' x 70' (change in width from 66' is reported, not confirmed)
Elevation: 55'-10.75" (based on field measurements)
Equipment arrangement:
Equipment layout drawings included in Figure 2B of Attachment B.
Loading: Same as original section. (see Table 1(a)).

Cellar deck: Dimensions: 90' x 40'
Elevation: 38'-11" to bottom of Cellar deck (based on field measurements)
Equipment arrangement:
Equipment layout drawings included in Figure 3B of Attachment B.
Loading: A communications building and a radio building were added
Weights provided in the following Table.

Table 2 Modified Cellar Deck Loads
* added loads

Description of Load	(kips)
* Communications building	12
* Radio building	5
Sewage treatment plant	10
Miscellaneous	16
Potable water Storage	175
Sump Tank	10
Subtotal Cellar Deck	228

Other decks: All information acquired can be found in attachment B and in the construction drawings.

b. All decks: - Existing loading and equipment arrangement: See attachment B

c. Field measured deck clearance elevation: As noted earlier

d. Production and storage inventory:

- 2277 BOPD, 2.2 MMSCFD

- No hydrocarbons stored on this structure (processing done on "A" and "E" platforms); the only significant storage is potable water tank, for which an a approximate weight is noted above

e. Appurtenances - current list, sizes and locations:
List: as shown in Table 1 and modified in Table 2.

Boat landings: No change from original design, note that both boat landings are above MLLW because of the jacket being 6'6" higher than intended. See construction drawing set (not included).

Barge Bumpers: No change from original design, note that all barge bumpers are 7' higher than their design level. See construction drawing set (not included).

f. Wells, number, size, location of existing conductors

Number of wells:	14
Size of conductor:	See Table 1B of Attachment B
Locations:	See Figure 1B of Attachment B
Producing ?:	All Production wells

g. Recent above water survey, Level I:
Performed annually since 1988; no significant damage noted

h. Recent underwater platform survey, Level 2 minimum:
Inspected in 1987, 1988 and 1992; no significant damage noted

A.3 PLATFORM SELECTION

The environmental impact and life safety classifications of the structure were determined based on the criteria of the draft document.

A.3.1 Life Safety

The structure has thereon quarters that are evacuated in the case of a forecasted design environmental event.

Hence the structure is to be categorized as **Manned-Evacuated**.

A.3.2 Environmental Impact

Even though there are 14 wells producing from this structure, all oil storage, oil processing and pipeline tie-in's are on or from an adjacent structure.

- Topsides Inventory: No topsides inventory.
- Wells: Uncontrolled flow from wells is not a concern since all wells are equipped with SSSV .
- Pipelines: Pipeline tie-in's are not directly connected to this structure but to an adjacent structure.

The production from this platforms flows to an adjacent structure through pipes attached to a bridge linking the two structures. The produced hydrocarbons go into storage tanks installed on the main deck of the adjacent platform. In the case of damage to the pipes no back flow from the storage tanks is possible. Hence the amount of hydrocarbons that can be released into the ocean is very minimal and it is limited to the amount of hydrocarbons in the pipes.

The platform is in the Gulf of Mexico in a location that is not in the proximity of environmentally sensitive areas.

Hence the platform can be deemed Environmentally-Insignificant.

The platforms is therefore a Manned-Evacuated, Environmentally-Insignificant structure which should be assessed according to the sudden hurricane criteria defined in section 17.6 of the draft document.

A.4 DESIGN BASIS CHECKS

Even though the platform is in the Gulf of Mexico, it was designed in 1964 before any API RP2A was published. Moreover, the metocean criteria to which the platform was designed is not available and hence the design basis check is not a feasible tool.

A.5 ANALYSIS CHECKS

A.5.1 Metocean, seismic, and ice criteria/loads

A.5.1.1 Metocean Criteria

The metocean criteria needed in the assessment of Gulf Of Mexico platforms is explicitly provided in the draft document for the Design Level and the Ultimate Strength analysis.

Metocean criteria needed in the assessment consist of the following items:

- Omni-directional wave height vs. water depth
- Storm Tide (storm surge plus astronomical tide)
- Deck height
- Wave and current direction
- current speed and profile
- Wave period
- Wind speed

The criteria specified in the draft document for the design level and the ultimate strength analysis are defined according to geographical location and are further differentiated according to exposure category. Special provisions are to be considered if the wave inundates the cellar deck, in which case the draft document mandates an ultimate strength analysis.

According to section 17.6.2a of the draft document, for a Gulf of Mexico Manned-Evacuated / Environmentally Insignificant platform in general and for a structure in 184 ft of water in particular the metocean criteria is described below.

A.5.1.1a Metocean Systems

A directional spreading factor 0.88 should be used since the environmental criteria is governed by hurricane loads and not by winter storms.

A.5.1.1b Deck Height Check

The deck heights shown in Figure A1 (Fig. 17.6.2-3b in draft document) are calculated based on ultimate strength analysis sudden hurricane criteria. As shown in Figure A1, the air gap needed for a platform in 184 ft of water is 36'3" which is smaller than the 38'11" air gap found on the platform being considered.

It is worth noting that the deck height required by API RP2A (20th edition) is 45'1" which is higher than the 38'11" air gap measured on the platform. Note that the 45'1" air gap matches the Full population hurricane deck height criteria (Fig. 17.6.2-2b of the draft document) even though the Full population hurricane wave height is higher than the API 100-yr. wave height. The difference is attributed to the additional 5' air gap that API RP2A requires to account for the settlement of the structure.

A.5.1.1c Design Level and Ultimate Strength Analysis Criteria

Section 17.6.2a-b describes the environmental criteria for Insignificant Environmental Impact / Manned Evacuated platforms. The combined sudden hurricane and winter storm population applies. Metocean criteria for this class of platforms is given in Table 17.6.2-1 of the draft document. Relevant parts of the above mentioned table are reproduced in Table A1 of Attachment A for convenience.

The wave height and storm tide are functions of water depth, these are shown in Figure 17.6.2-3a of the draft document which is reproduced in Figure A2 for a water depth of 184 ft. The wave period, wind speed, and current speed do not vary with water depth these are provided in Table A1.

For Design Level analysis the omni-directional criteria is specified in Table A1. The wave height, and associated current are assumed to be constant for all directions. Table 3 summarizes the sudden hurricane metocean criteria for the design level analysis of a platform in 184 ft of water.

Table 3 Design Level Metocean Criteria

True North	290.00	335.00	20.00	65.00	110.00	155.00	200.00	245.00
Platform North (x-axis)	-14.60	30.40	75.40	120.40	165.40	210.40	255.40	300.40
Water Depth	184.00	184.00	184.00	184.00	184.00	184.00	184.00	184.00
H max	46.00	46.00	46.00	46.00	46.00	46.00	46.00	46.00
T max	11.30	11.30	11.30	11.30	11.30	11.30	11.30	11.30
Surface Current (*)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Mudline Current (*)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Tide	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7

On the other hand the wave height and storm tide for ultimate strength analysis are function of direction. Factors used in calculating the ultimate strength wave heights and associated currents are given in Figure 17.6.2-4. Table 4 summarizes the Sudden Hurricane metocean criteria for the ultimate strength analysis of a platform in 184 ft of water.

Table 4 Ultimate Strength Analysis Metocean Criteria

True North	290.00	335.00	20.00	65.00	110.00	155.00	200.00	245.00
Platform North (x-axis)	-14.60	30.40	75.40	120.40	165.40	210.40	255.40	300.40
Water Depth	184.00	184.00	184.00	184.00	184.00	184.00	184.00	184.00
Factor	0.90	1.00	0.95	0.85	0.70	0.70	0.70	0.70
H max	51.75	57.50	54.63	48.88	40.25	40.25	40.25	43.13
T max	12.50	12.50	12.50	12.50	12.50	12.50	12.50	12.50
Surface Current (*)	2.70	3.00	2.85	2.55	2.10	2.10	2.10	2.25
Mudline Current (*)	2.70	3.00	2.85	2.55	2.10	2.10	2.10	2.25
Tide	3	3	3	3	3	3	3	3

For non-critical directions the omni-directional wave heights are larger than the ultimate strength wave heights (compare shaded areas in Tables 3 and 4). In such case the draft document recommends using the ultimate strength criteria for the design level analysis. Since lower storm tides and slower currents are usually associated with smaller waves, the authors have used the smaller value from each criteria for each of the storm tide and the associated current criteria. Hence the ultimate strength wave height and storm tide along with the design level associated current and wave period are to be used in the design level analysis. Table 5 shows this combination.

Table 5 Used Design Level Metocean Criteria

True North	290.00	335.00	20.00	65.00	110.00	155.00	200.00	245.00
Platform North (x-axis)	-14.60	30.40	75.40	120.40	165.40	210.40	255.40	300.40
Water Depth	184.00	184.00	184.00	184.00	184.00	184.00	184.00	184.00
H max	46.00	46.00	46.00	46.00	40.25	40.25	40.25	43.13
T max	11.30	11.30	11.30	11.30	11.30	11.30	11.30	11.30
Surface Current (*)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Mudline Current (*)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Tide	3.7	3.7	3.7	3.7	3.0	3.0	3.0	3.0

The reserve strength ratio (RSR) is defined in the draft document as the ratio of a platform's ultimate lateral load carrying capacity to its 100-yr. extreme wave lateral loading computed using present RP2A procedures. Hence criteria for a 100-yr. extreme wave condition as described by API-RP2A were calculated for 184 ft of water. Table 6 summarizes this data for each of the eight directions defined in Figure 2.3.4-4 of RP2A (note the difference in the factors associated with each direction). The platform being considered is located in the Gulf of Mexico delta area in a water depth of 184 ft requiring site specific current data (see Figure 2.3.4-5 of API RP2A). Therefore currents associated with the three main loading directions were calculated from a hindcast and were included in Table 6.

Table 6 API 100-yr. Environmental criteria

True North	290.00	335.00	20.00	65.00	110.00	155.00	200.00	245.00
Platform North (x-axis)	-14.60	30.40	75.40	120.40	165.40	210.40	255.40	300.40
Water Depth	184.00	184.00	184.00	184.00	184.00	184.00	184.00	184.00
Factor	1.00	0.95	0.85	0.70	0.70	0.70	0.75	0.90
H max	65.00	61.75	55.25	45.50	45.50	45.50	48.75	58.50
T max	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00
Surface Current (*)	1.00	1.50						3.00
Mudline Current (*)	0.50	1.00						2.00
Tide	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5

By comparing the last row in each of Table 3 and Table 6 it can be seen that the storm tide associated with the design level criteria is larger than the storm tide associated with the API 100-yr. extreme wave criteria. Similarly, the last row in each of Table 3 and Table 4, the storm tide associated with design level criteria is larger than the storm tide associated with the ultimate strength analysis criteria. It is the understanding of the authors that smaller storm tides are associated with smaller waves, hence the two storm tide criteria described at the bottom of Figure 17.6.2-3a should not cross each other but rather approach each other asymptotically with the ultimate storm tide criteria being always larger than the design level criteria.

A.5.1.2 Seismic Loading

Not applicable to Gulf Of Mexico (GOM) platforms since the area is categorized as seismic zone 0 (see section C2.3.6c).

A.5.1.3 Ice Loading

Not applicable to Gulf Of Mexico platforms.

A.5.2 Screening

Not applicable to this platform.

The eight legged structure being assessed is symmetrical with respect to both of its major axes. Figure A3 shows the orientation of the platform with respect to true north. Platform north is 304.6 degrees clockwise from true north, hence it is 14.6 degrees clockwise from the 290 degree wave direction also shown in Figures A3 and A4.

Four directions (End-on, Broadside and two diagonals (45 and 22.35 degrees)) were analyzed for each loading case (API 100-yr., Design Level, and Ultimate Strength). The 22.35 degree case passes through the center of the platform and one of its corner piles, hence it will maximize pile loads. By overlaying the platform's orientation on each of the two wave/current figures (Figure 2.3.4-4 of API RP2A and Figure 17.6.2-4 of the draft document) the following combination of load direction/load case were chosen to be in the analysis. See Table.

Table 7 Load combinations to be analyzed

Wave direction cw from true North	Wave direction from platform north	API RP2A 100-yr.	Design Level	Ultimate Strength
290	0	X	X	X
290	22.35	X	X	
335	22.35			X
335	45.0	X	X	X
20	90.	X	X	X

The KARMA Finite Element Models
The KARMA program is the common users version of the program INTRA, which was developed through joint industry cooperation among offshore operators over many years. The program is a general purpose structural analysis program with capabilities for linear and nonlinear static and dynamic analyses.

The attached Figure A6 illustrates the KARMA computer model used in the analysis.

A.5.3 Design Level

Design level analysis procedures as defined in Section 3, 4, 5 and 6 of API RP2A and in the draft document were used to assess the structure. The loads combinations defined in Table 7 under "Design Level" along with the metocean criteria given in Table 5 were used in the design level analysis. Base shears and overturning moments for each load combination are summarized in Table 8.

Table 8 Overturning Moment and Base Shear calculated
in the Design Level analysis

Wave Heading	Wave Height	Analysis Type	Direction	Overturning Moment	Base Shear	Equivalent Base Shear
0	46	Design Level	x	5478.00	1562.00	1562.00
			y	4918.00	1.00	
22.35	46	Design Level	x	82600.00	1136.00	1237.57
			y	199300.00	491.00	
45	46	Design Level	x	184400.00	1098.00	1614.76
			y	5667.00	1184.00	
90	46	Design Level	x	232600.00	10.00	1453.03
			y	2462.00	1453.00	

The Karma design level analysis model used linear beam-column elements for all of the deck, jacket and pile elements. Soil structure interaction was modeled using near field soil elements consisting of two orthogonal lateral springs and one axial spring. These soil elements are capable: of performing very generalized analyses of axial and lateral soil-pile interactions; of characterizing static, fully cycled , strain rate, and cycle-by-cycle interactions; and of being used in single static, time-step analyses, and cycle-by-cycle interactions.

Wall thickness changes in jacket legs were not modeled in KARMA; the smaller leg wall thickness for each leg element was used in the model. Leg pile stiffness was modeled by using an effective leg cross section to model both the axial and bending stiffness of the combined leg/pile section.

API RP2A-WSD member and joint checks as defined in sections 3 and 4 respectively, were used in calculating interaction ratios. Effective length (K-factors) and Reduction factors (Cm factors) defined in Section 3.3.1d of RP2A-WSD were used for member checks. Only K joints that were not reinforced with gusset plates were checked. All leg joints were reinforced using double gusset plates as illustrated in Figure A5 after the platform was constructed and before it was installed.

A.5.3.1 Structural Steel Design

Member sizes and wall thicknesses were not modified to account for any possible deterioration or any rust since all underwater surveys have shown little evidence of any deterioration.

In three out of the four load cases only one member failed to satisfy the API member checks. Table 9 lists all members with unity check ratios exceeding 0.85. It is noticed that all members listed in Table 9 are conductor framing members at the +13' elevation above MLLW except for member 526 which is also a conductor framing member at the -24'4" elevation. Moreover, for all members listed in Table 9 yield ratios ranged between 0.723 and 0.512. Considering the above mentioned results and the fact that the conductor framing model uses longer members than the actual conductor framing system, member 616 is not considered to have failed the API member check. And the structure would pass the design level analysis if it passes the joint check.

Table 9 Members with unity check ratios exceeding 0.85

Wave Direction from Platform North	0	22.35	45	90
Failed member name and its API interaction ratio	616 (1.004)	616 (0.90)	616 (1.148)	616 (1.220)
			630 (0.969)	630 (0.943)
			526 (0.945)	615 (0.896)
			615 (0.868)	629 (0.862)

Figure A7 in Attachment A shows the location of members listed in Table 9.

A.5.3.2 Connections

All critical joints in the structure are reinforced with double gusset plates as illustrated in Figure A5 of attachment A. Preliminary analysis on these joints showed that member failure will occur before any joint failure. Hence no detailed joint checks were performed.

A.5.3.3 Fatigue

Level III surveys in 1987 and 1992 that were aimed at assessing fatigue critical joints have shown no apparent surface discontinuities in either the heat affected zone or caps of all welds inspected. Hence no additional analytical effort is required.

A.5.4 Ultimate Strength

Based on the draft document's definition of a Reserve strength ratio (RSR), the first step in an ultimate strength analysis is to calculate the base shear for a 100-yr. extreme environmental event. The load combinations defined in Table 7 along with the metocean criteria summarized in Table 6 resulted in the overturning moments and base shears shown in Table 10. The design level analysis finite element model was used for the API-100 yr. base shear calculation.

Table 10 API 100-yr. Overturning Moments and Base Shears

Wave Heading	Wave Height	Analysis Type	Direction	Overturning Moment	Base Shear	Equivalent Base Shear
0	65	API 100-yr	x	5449.00	2492.00	2492.00
			y	392600.00	2.00	
22.35	65	API 100-yr	x	165200.00	2501.00	2712.47
			y	397300.00	1050.00	
45	61.75	API 100-yr	x	283000.00	1739.00	2528.11
			y	284300.00	1835.00	
90	55.25	API 100-yr	x	331000.00	13.00	2176.04
			y	24430.00	2176.00	

The same soil elements described in the previous section were also used in the ultimate strength analysis finite element model. However for ultimate strength analysis the jacket legs, the deck legs, and the piles were modeled using large displacement non-linear beam-column elements. The vertical X-braces were modeled using strut elements having post buckling behavior based on tests of tubular frames performed by Zayas and Popov at the University of California Berkeley. Conductors, and boat landings were modeled using wave load elements, these elements are not included in the stiffness matrix. There contribution to the structural analysis is calculating wave loads and projecting these loads to user defined structural nodes. The conductor supporting frame was approximated by a cross braced rectangular frame with all conductor loads applied to four corner nodes and a center node. Horizontal braces were modeled using a three dimensional plastic hinge beam-column elements. A simplified deck framing pattern was used to model the deck stiffness. Similar to the design level analysis deck elements were modeled using linear beam-column elements.

As in the design level analysis, wall thickness changes in jacket legs were not modeled in KARMA; the smaller leg wall thickness for each leg element was used in the model. Leg/pile stiffness was modeled by using an effective leg cross section to model both the axial and bending stiffness of the combined leg/pile section. Even though all piles are grouted, no increase in the pile/leg stiffness is attributed to the grout.

Loads combinations defined in Table 7 under "Ultimate Strength analysis" along with the metocean criteria summarized in Table 4 were used in the ultimate strength analysis.

For connection checks the reader is reffered to Section A.5.3.2

Table 11 Overturning Moments and Base Shear values from Ultimate Strength Analysis

Wave Heading	Wave Height	Analysis Type	Direction	Overturning Moment	Base Shear	Equivalent Base Shear	RSR
0	51.75	Ultimate Strength	x	5620.00	3975.00	3975.00	1.60
			y	600100.00	2.00		
22.35	57.5	Ultimate Strength	x	253500.00	3892.00	4219.55	1.56
			y	594800.00	1630.00		
45	57.5	Ultimate Strength	x	510900.00	3154.00	4588.02	1.81
			y	488800.00	3332.00		
90	54.63	Ultimate Strength	x	713700.00	31.68	4767.11	2.19
			y	17620.00	4767.00		

A pushover analysis was performed for each of the four loading directions. Load-Displacement curves for each load direction are found in Figures 1-4.

The first set of member failures as well as the load and the displacement of a deck node at which each element failed can be found in Tables 12-15. By studying the order and the type of elements that are reaching in-elastic behavior, it is clear that the structure fails in its foundation. In a couple of load cases the failure starts in the bottom level horizontal braces. This is due to the fact that these braces are under the mudline, see Figure A6 in Attachment A. Soil pressure on the legs which would typically lead to larger pile displacements is in this case restrained by the bottom horizontal braces. No special consideration was given to model this level of horizontal braces as a mudmat, or to model the soil structure interaction caused by the braces being under the mudline. It is believed that modeling the soil/braces interaction will slightly increase the structures stiffness, however its effect on the braces' member capacity will be more.

The Reserve strength Ratios (RSR's) for each load case were calculated and are shown in Table 11. All RSR's have a value that is larger than 1.0, hence the platform passes the ultimate strength analysis.

A.6 MITIGATION ALTERNATIVES

Not needed

A.7 SUMMARY NOTE - PART A

Name of Platform:	Platform M
Year of Design and Construction:	1964
Year of Installation:	1968
Water Depth:	184 ft
Location:	Gulf Of Mexico, West Delta
Platform Type:	8 piles, K and Diagonal bracing.
Number of Wells:	14
Re qualification Trigger:	Addition of Conductors
Condition Assessment:	No damage.
Categorization:	Manned-Evacuated / Insignificant Environmental Impact
Design Basis Check:	Not performed

Design Level Analysis:

Metocean Criteria

H = 46 ft, T = 11.30 sec
Current = 2.0 knots, Wind = 55 knots
Maximum Base Shear = 1615 kips
Results: Platform Passes Assessment

API RP2A-WSD 100-yr.

Metocean Criteria

H = 65 ft, T = 13 sec
Current as defined in Table 6, Wind = 80 knots
Directionality as per API RP2A-WSD

Ultimate Strength Analysis:

Metocean Criteria

H = 57.5 ft, T = 12.5 sec
Current = 1.8 knots, Wind = 70 knots
Directionality as per Draft Document

End-On Direction

0 degrees
Maximum Base Shear = 3975
Load at which first component reaches I.R =1.0
S_1 = 1562 kips
API reference Level (API 20th edition)
S ref = 2492 kips
Reserve Strength Ratio = 1.60
Platform Failure Mode: Foundation

22.35 Diagonal

22.35 degrees
Maximum Base Shear = 4219
Load at which first component reaches I.R =1.0
S_1 =
API reference Level (API 20th edition)
S ref = 2712 kips
Reserve Strength Ratio = 1.56
Platform Failure Mode: Foundation

45 Diagonal	45 degrees Maximum Base Shear = 4588 Load at which first component reaches I.R =1.0 S_1 = 1615 kips API reference Level (API 20th edition) S ref = 2528 kips Reserve Strength Ratio = 1.81 Platform Failure Mode: Foundation
Broadside Direction	90 degrees Maximum Base Shear = 4767 Load at which first component reaches I.R =1.0 S_1 = 1453 kips API reference Level (API 20th edition) S ref = 2176 kips Reserve Strength Ratio = 2.19 Platform Failure Mode: Foundation
Results:	Platform PASSES Assessment

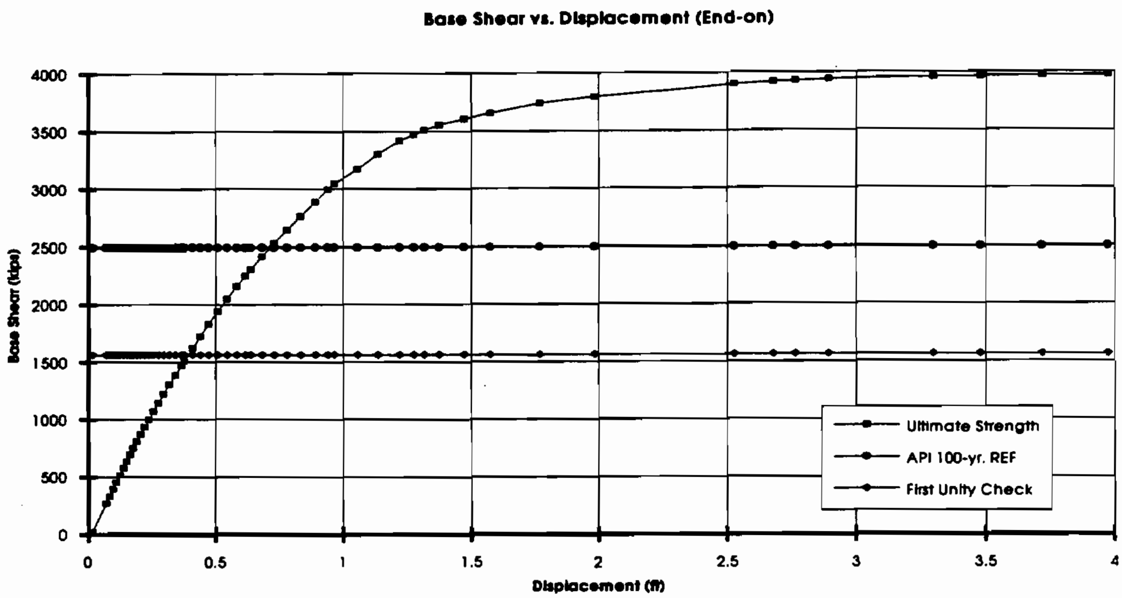


Figure 1 Load Displacement Result for End-on Case

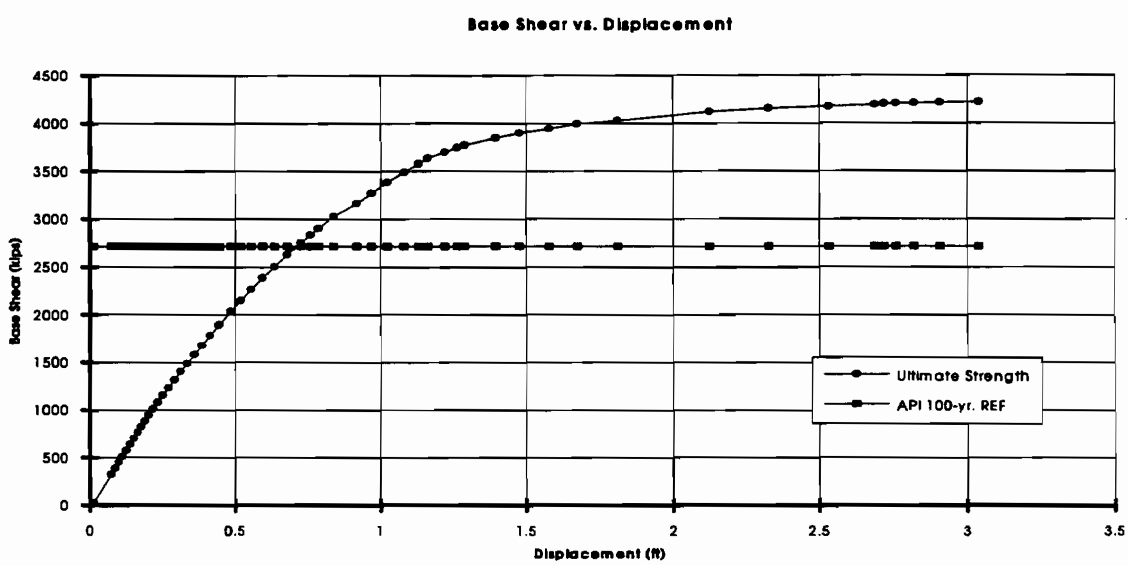


Figure 2 Load Displacement Result for 22.35 Diagonal Case

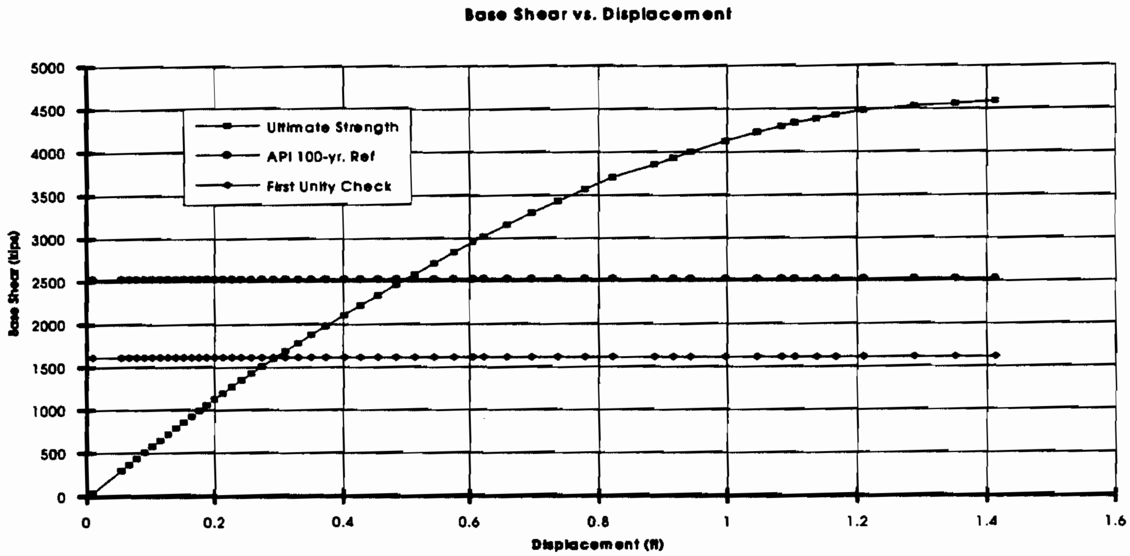


Figure 3 Load Displacement Result for 45 Diagonal Case

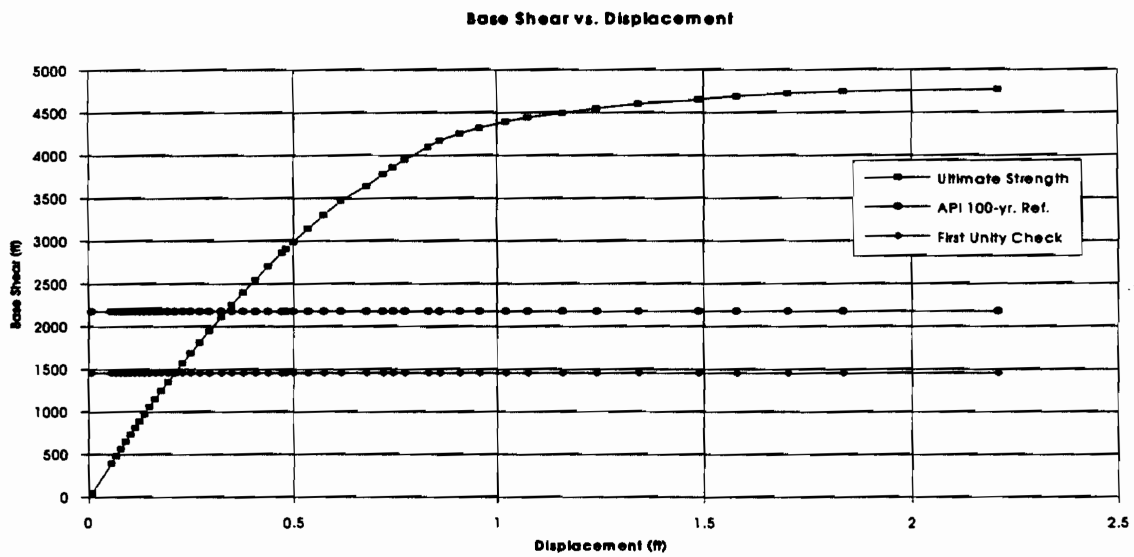


Figure 4 Load Displacement Result for Broadside Case

Load Step	Lateral Displacement at Deck Level	Lateral Load	Number of Element Failing	Element Type
	ft.	kips		
34	0.89	2889	1	Horiz. Brace
35	0.94	2992	1	Horiz. Brace
35	0.94	2992	1	Pile
36	0.96	3044	3	Pile
37	1.06	3171	1	Horiz. Brace
37	1.06	3171	4	Pile
38	1.136	3303	6	Leg
38	1.136	3303	4	Pile
39	1.22	3415	2	Horiz. Brace
39	1.22	3415	2	Leg
39	1.22	3415	4	Pile
40	1.27	3470	1	Horiz. Brace
40	1.27	3470	2	Leg
41	1.31	3508	1	Horiz. Brace
41	1.31	3508	2	Pile
42	1.37	3552	2	Leg
42	1.37	3552	2	Pile
43	1.47	3603	5	Pile
56	3.93	3975		

Table 12 Ultimate Strength Analysis Results

Load Step	Lateral Displacement at Deck Level	Lateral Load	Number of Element Failing	Element Type
	ft.	kips		
35	0.92	3160	2	Horiz. Brace
35	0.92	3160	3	Pile
36	0.96	3269	1	Pile
37	1.02	3382	2	Leg
37	1.02	3382	5	Pile
38	1.08	3488	2	Pile
38	1.08	3488	2	Leg
39	1.13	3574	2	Horiz. Brace
39	1.13	3574	1	Leg
40	1.16	3635	2	Pile
40	1.16	3635	2	Leg
41	1.22	3696	1	Horiz. Brace
41	1.22	3696	3	Leg
41	1.22	3696	4	Pile
42	1.26	3744	1	Leg
42	1.26	3744	1	Pile
43	1.29	3769	1	Horiz. Brace
44	1.39	3843	1	Leg
44	1.39	3843	4	Pile
45	1.48	3895	1	Horiz. Brace
45	1.48	3895	4	Pile
57	3.04	4219		

Table 13 Ultimate Strength Analysis Results (22.35 Diagonal)

Load Step	Lateral Displacement at Deck Level	Lateral Load	Number of Element Failing	Element Type
	ft.	kips		
37	0.73	3430	1	Pile
38	0.78	3566	2	Pile
39	0.82	3706	1	Leg
39	0.82	3706	1	Pile
40	0.89	3851	1	Horiz. Brace
40	0.89	3851	2	Leg
40	0.89	3851	2	Pile
41	0.92	3929	1	Pile
42	0.94	3998	1	Leg
42	0.94	3998	2	Pile
43	0.99	4121	3	Leg
43	0.99	4121	3	Pile
44	1.05	4225	1	Leg
44	1.05	4225	2	Pile
45	1.08	4295	2	Pile
46	1.1	4337	1	Horiz. Brace
46	1.1	4337	1	Leg
46	1.1	4337	1	Pile
47	1.14	4382	1	Horiz. Brace
47	1.14	4382	1	Pile
48	1.17	4424	2	Pile
52	1.41	4588		

Table 14 Ultimate Strength Analysis Results (45 Diagonal)

Load Step	Lateral Displacement at Deck Level	Lateral Load	Number of Element Failing	Element Type
	ft.	kips		
30	3468	0.61	1	Pile
31	3628	0.68	3	Pile
32	3778	0.72	1	Leg
32	3778	0.72	4	Pile
33	3858	0.74	2	Leg
33	3858	0.74	1	Pile
34	3947	0.77	2	Leg
34	3947	0.77	1	Pile
35	4096	0.83	3	Leg
35	4096	0.83	5	Pile
36	4167	0.86	1	Pile
37	4250	0.91	4	Leg
37	4250	0.91	2	Pile
38	4319	0.96	1	Leg
38	4319	0.96	2	Pile
39	4389	1.02	1	Leg
39	4389	1.02	4	Pile
40	4437	1.07	4	Pile
41	4490	1.16	2	Horiz. Brace
41	4490	1.16	2	Pile
50	2.21	4767		

Table 15 Ultimate Strength Analysis Results (Broadside)

PART B: REVIEW AND FEEDBACK TO THE API TG 92-5

Comments , Suggestions.

Comments and suggestions included in this section are mainly areas that the authors had to spend quite some time to find the information required.

- It is suggested in section 17.6.2a-4b, paragraph 3, that the third sentence which currently starts with "For some non-critical directions, the omni- ... " be modified to include the notes that are found at the bottom of Table 17.6.2-1 and to explicitly state that if the wave height or current vs. direction calculated for the omni-directional criteria exceeds that required by the ultimate strength analysis the smaller of each wave height or current from both criteria will be used.
- In the draft document in general, all references should be numbered or labeled and only the reference number/label included in the body of the text. This will make reading the document much easier.
- References to the 20th edition of API-RP2A should be changed to current edition of RP2A. After all this section will first occur in the 21st edition, and should reference the 21st and not the 20th edition.
- The criteria for Gulf of Mexico platforms passing Ultimate Strength Analysis should be clearly stated in the text. Not only in the flow chart.

Questions related to draft Document

Questions that are not clear to the authors of this memo are listed below in no specific order:

- In section 17.6.2a-4b which wave period and storm tide are to be used in the Design Level analysis if the Ultimate wave analysis wave height governs. Normally smaller wave periods and smaller storm tides are associated with smaller wave heights.
- For the 184 ft water depth Sudden Hurricane Criteria the Storm tide for ultimate strength analysis, (larger wave height) is higher than the storm tide of the Design Level analysis (smaller wave height). Should these two curves be asymptotic with the ultimate strength storm tide always being larger that the design basis storm tide.
- For the same structure in 184 ft of water, the storm tide for the Design Level analysis is higher than the storm tide for the API 100-yr. extreme environmental criteria.
- By comparing Figure 17.6.2-4 to Figure 2.3.4-4 it can be seen that the factors used for the Ultimate Strength analysis are shifted from the factors used in API 100-yr. extreme load criteria by 45 degrees. This will clearly effect the reassessment of structures that in the case of the 335 degree angle, for example, will be assessed for a much higher environmental criteria. Specially in the case of a Manned non-evacuated structure were the 95% of the API 100-yr. wave is to be compared to the 100% Full Population Hurricane Load which is already 6-7% higher.

Typo Errors in Draft date April 1, 1994.

Typo mistakes that were found in reviewing the draft document dated April 1, 1994.

- Page 5, section 17.5.2 environmental is written environ-mental.
- Table 17.6.2-1, Design Level Analysis written Design Level Level Analysis. (Level written twice). In two instances.
- In paragraph 3 of section 17.6.2a-4b non- critical should be written non-critical. No space between the hyphen and the letter "c".
- The first paragraph in Section 17.6.2a-4c ends with two periods.

PLATFORM "N"

1. INTRODUCTION

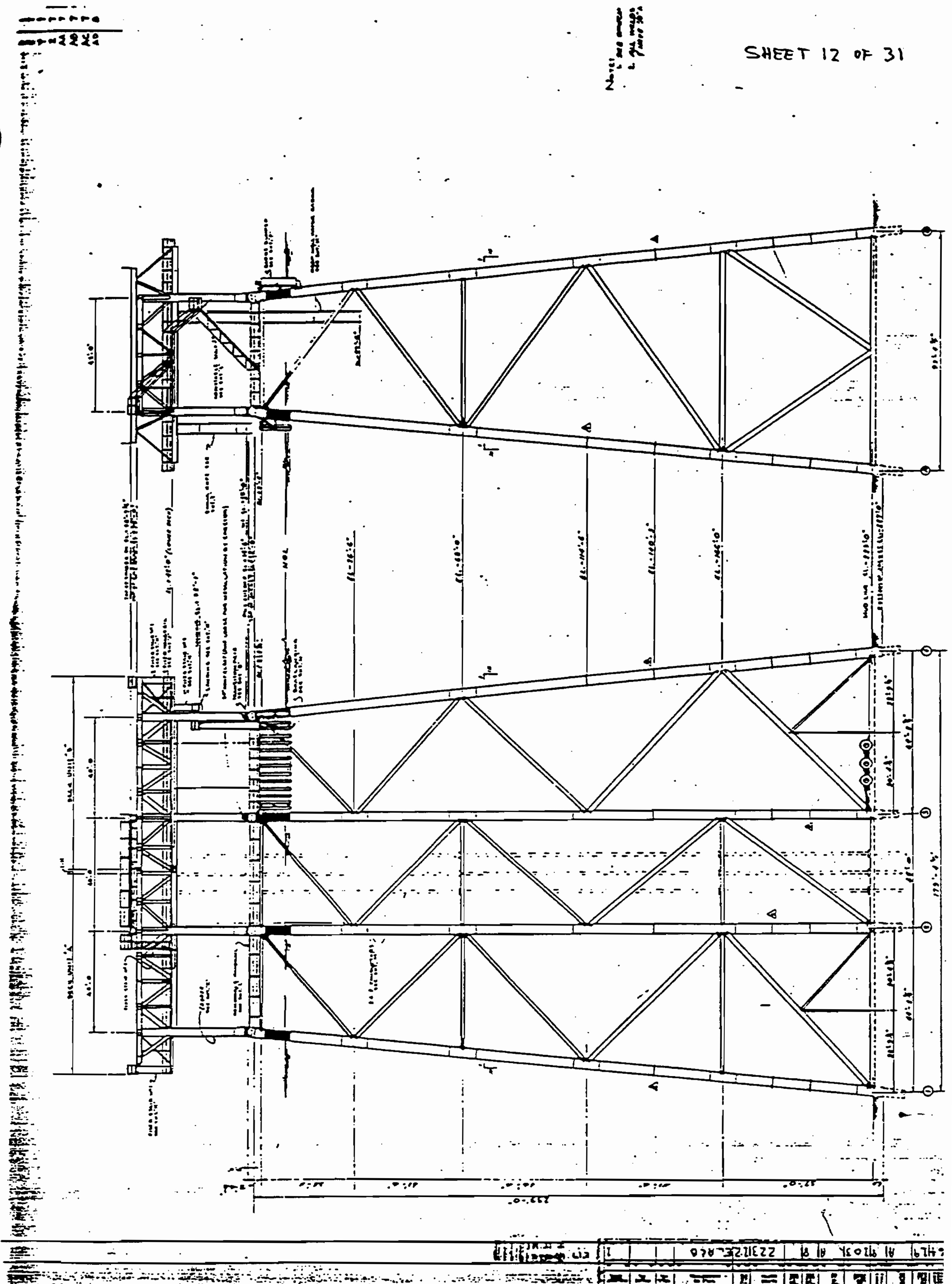
1.1 Scope & History

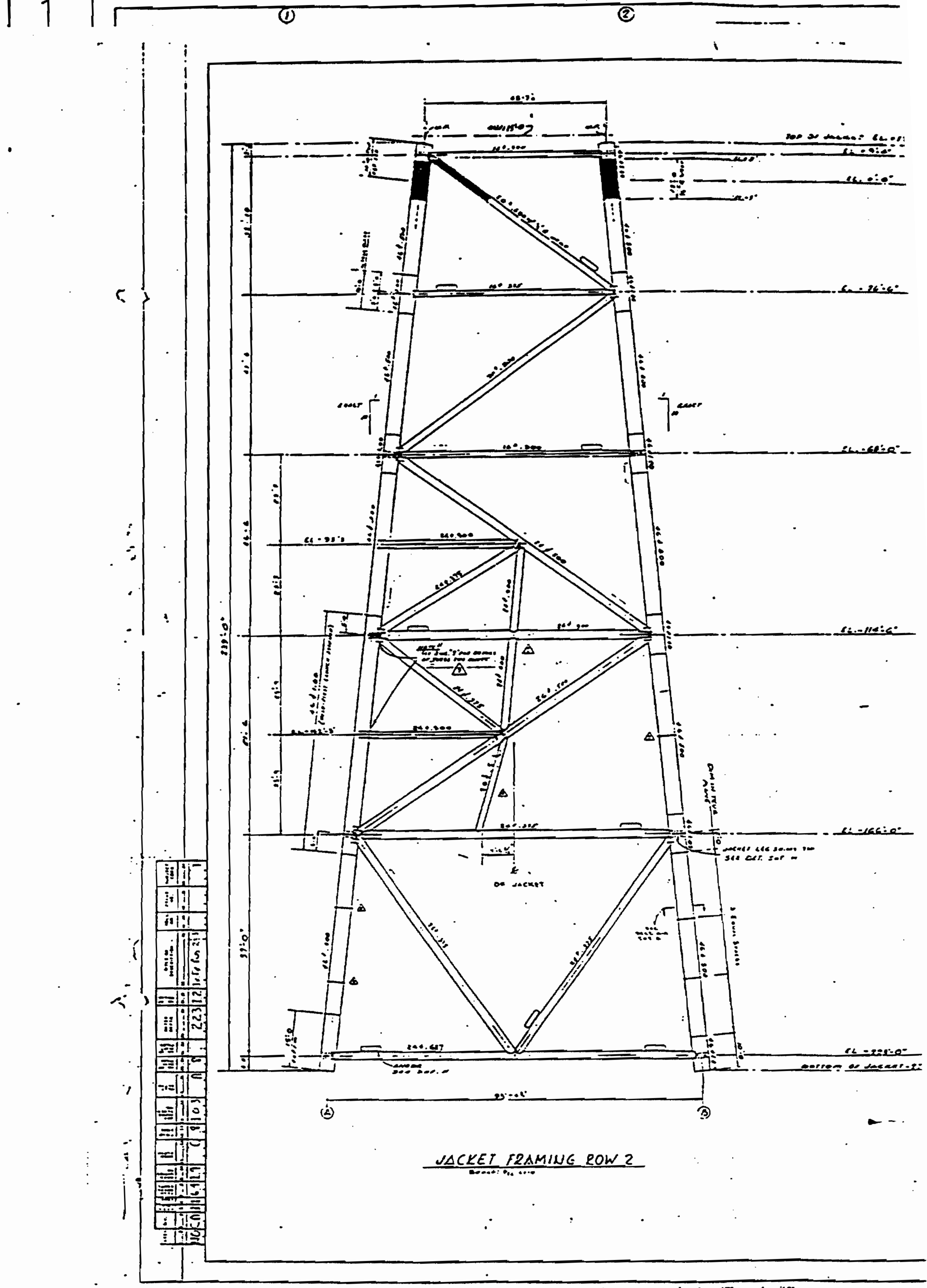
This study deals with the structural assessment of Platform West Delta 103 "A" according to API guidelines (Ref. 4.1).

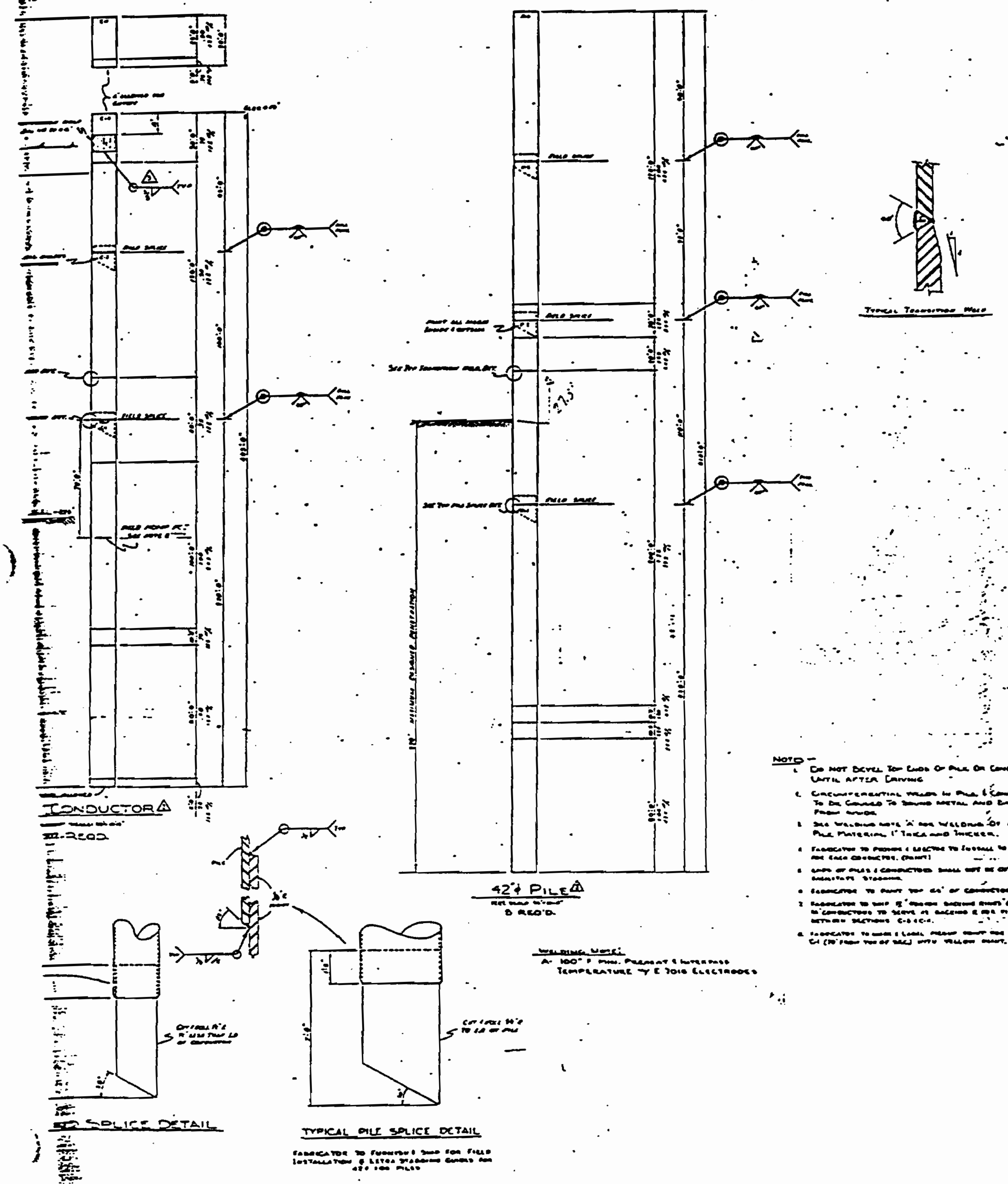
The platform is located in Louisiana offshore waters, in 223 ft. of water. It has 8 legs, double battered and K-braced.

The platform design drawings were signed on 9/30/64 (Ref. 4.2) and were released for construction on 3/24/65.

On 9/20/90, the walkway at elevation (+) 9' - 4 5/8" was repaired to replaced 12 3/4" dia. x ½" horizontal jacket member between the column axes A2, A3, A4 and B2, B3, B4, as well as to cover 6 ft. long section of horizontal jacket member with ½" thick rolled plate between A1, A2, and B1, B2 (see Drawing No. D-64-19AD of Ref. 4.2).

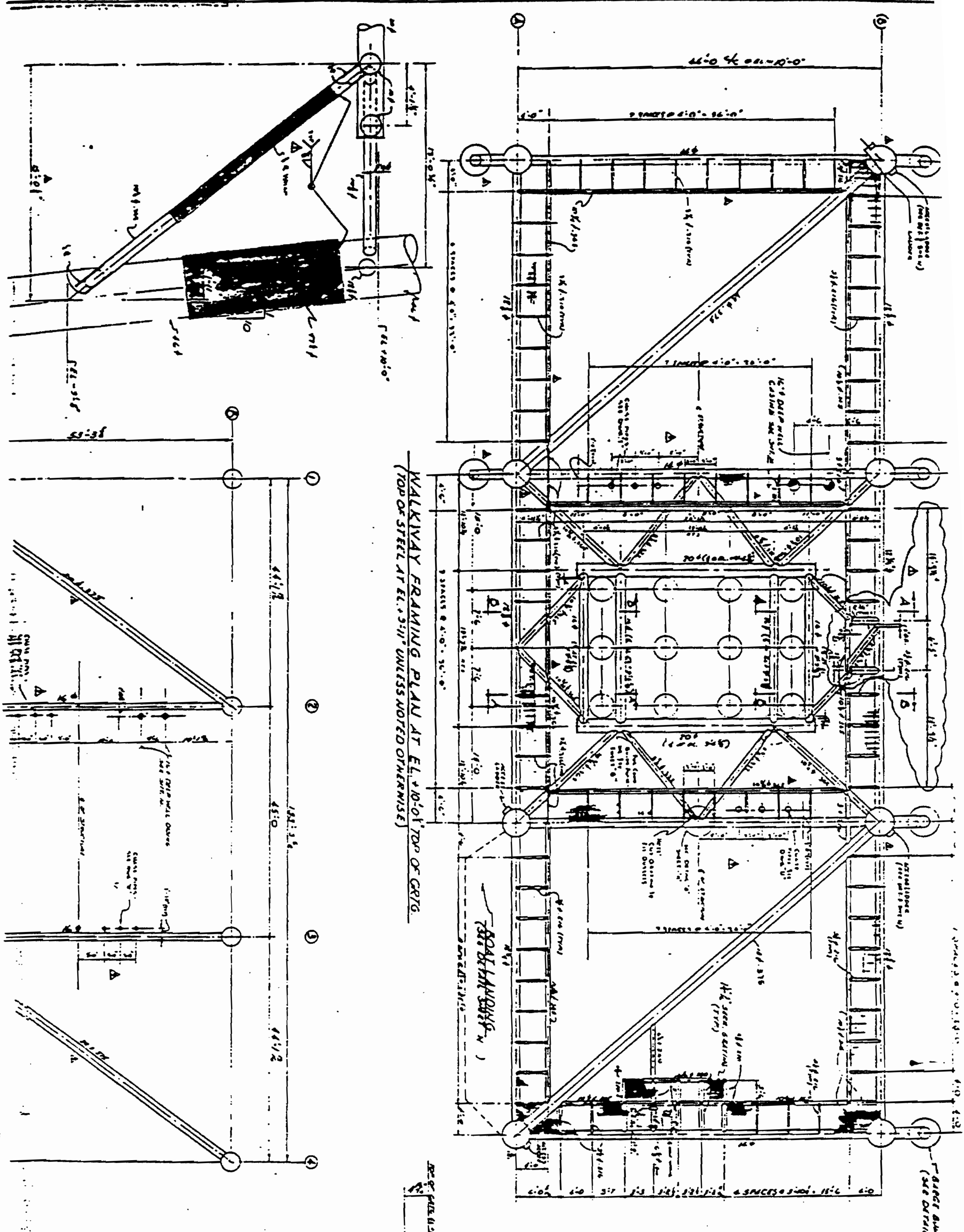


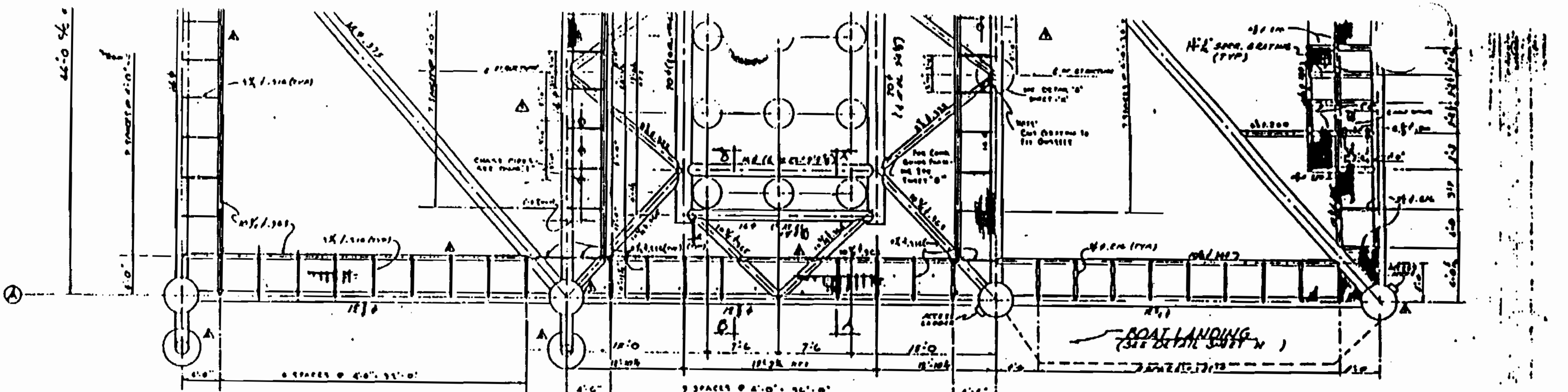

JACKET FRAMING ROW 2



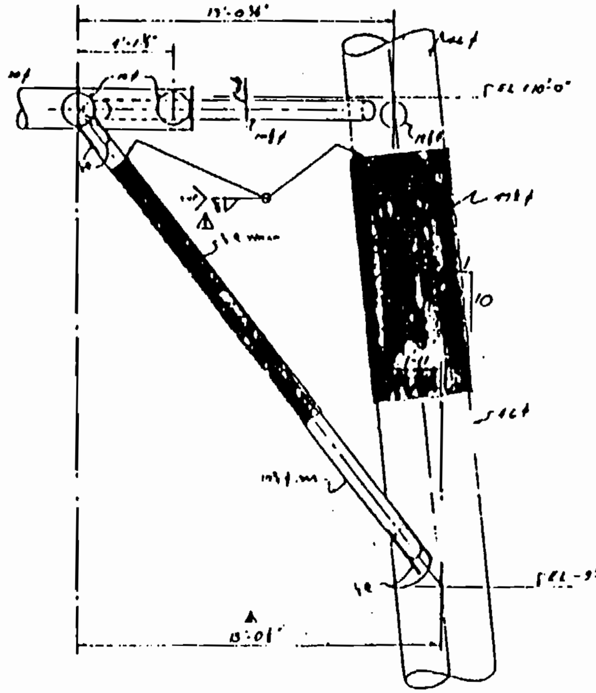
NOTES –
1. DO NOT BEVEL TOP ENDS OF PILE OR CONDUCTORS UNTIL AFTER DRIVING
2. CIRCUMFERENTIAL WELDS IN PILE & CONDUCTORS TO BE GOUGED TO SOUND METAL AND BACK WELDED FROM INSIDE.
3. SEE WELDING NOTE 'A' FOR WELDING OF ALL PILE MATERIAL 1" THICK AND THICKER.
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]
8. [illegible]

WELDING NOTE:
A- 100° F MIN. PREHEAT & INTERPASS TEMPERATURE W/ E 7018 ELECTRODES

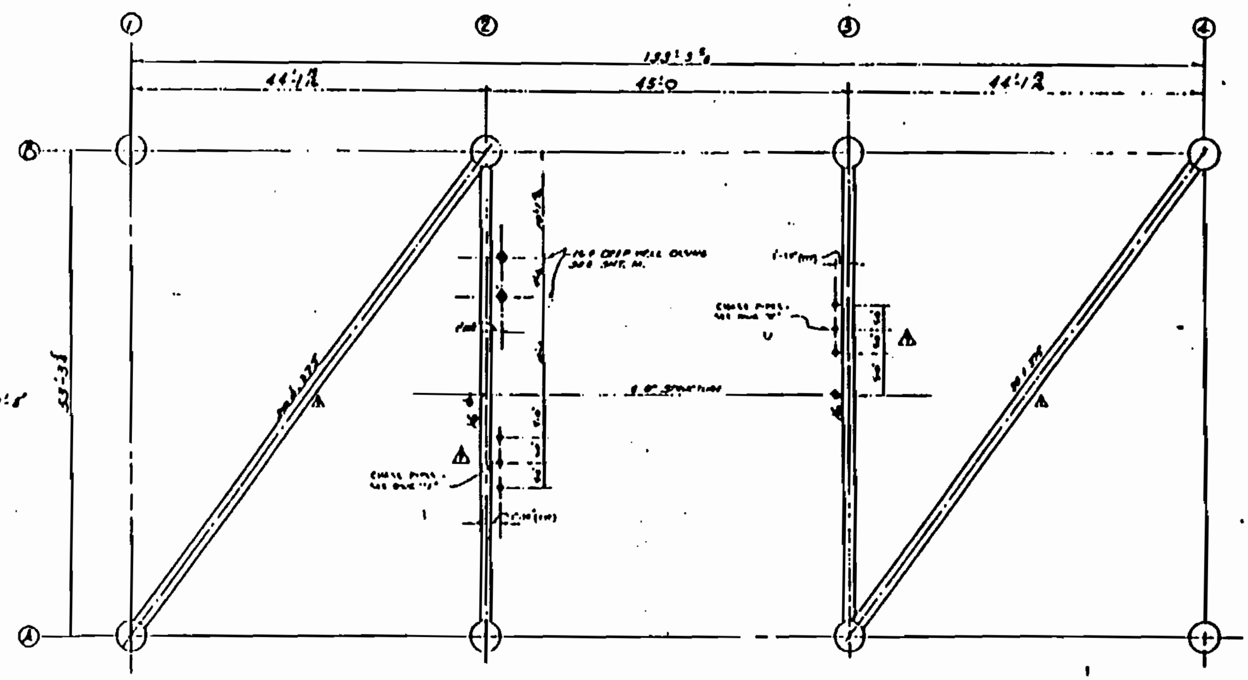

WALKWAY FRAMING PLAN AT EL.+10'-0" TOP OF GRTG.
(TOP OF STEEL AT EL.+9'-11" UNLESS NOTED OTHERWISE)



WALKWAY FRAMING PLAN AT EL. +10'-0½" TOP OF GRTG.
(TOP OF STEEL AT EL. +9'-11" UNLESS NOTED OTHERWISE)

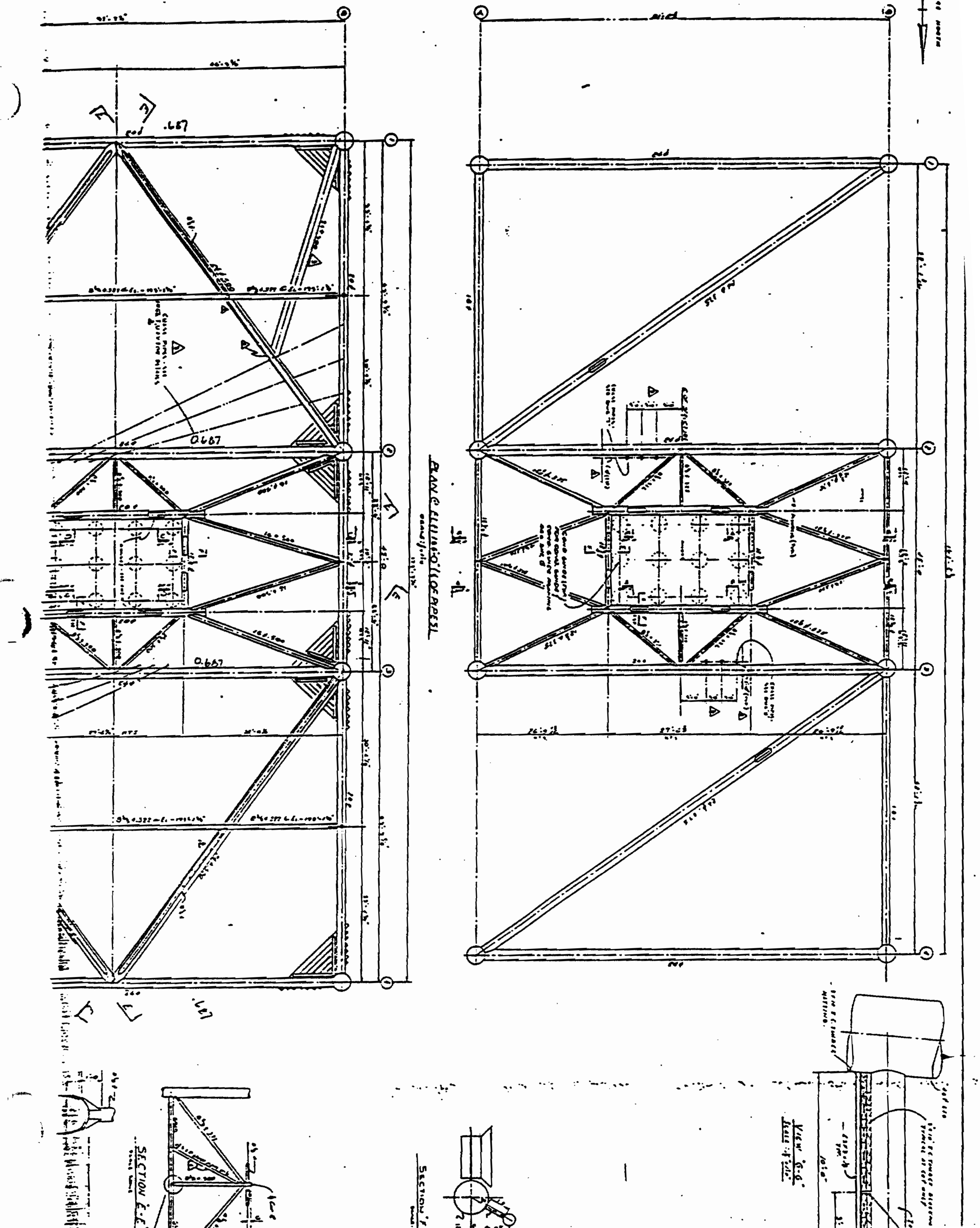


SECTION 'A-A' (SHOWN)
SECTION 'B-B' (OPPOSITE HAND)
$\frac{3}{8}$" = 1'-0

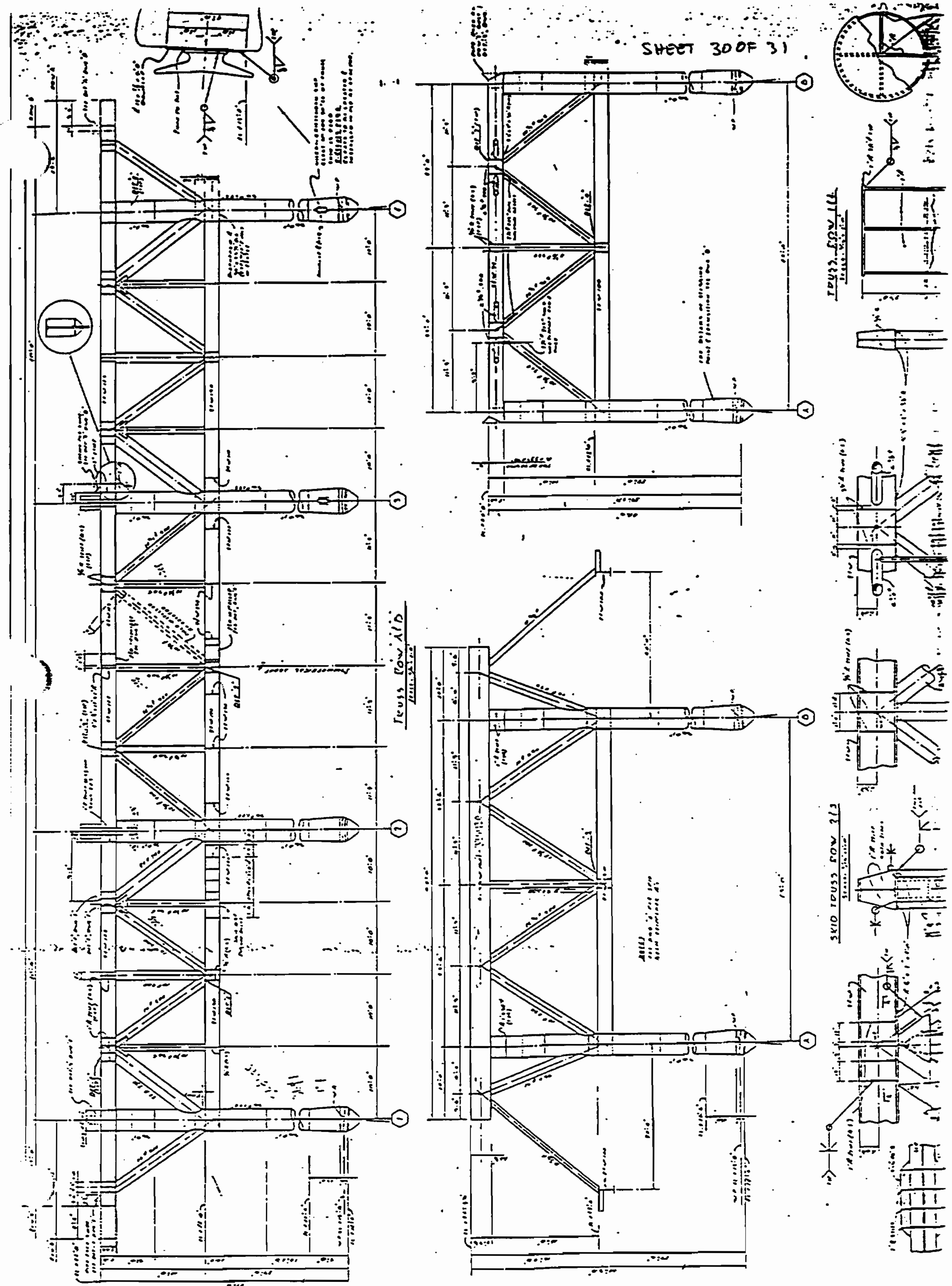


FRAMING PLAN AT EL. -26'-6"
$\frac{3}{8}$" = 1'-0

SHEET 20 OF 31



SHEET 30 OF 31



2. PURPOSE OF STUDY

The purpose of this study is to demonstrate the application of the draft API assessment process (Ref. 4.1) which gives guidelines to prove the fitness for purpose of existing platforms.

The study was initiated by the Minerals Management Service (MMS) due to concern of the adequacy of older structures and recent occurrence of powerful hurricanes.

According to draft API guidelines (Ref. 4.1, Section 17.2), existing platforms shall undergo the assessment process if one or more of the conditions noted below exist:

a) No assessment is required for platforms decommissioned or ready to be removed (e.g. wells plugged and abandoned).

b) Assessment is required if:
   1) Manning condition is changed to a more restrictive level.
   2) Addition of new facilities to the platform increases the original operational loads.
   3) The structure is altered such that new combined environmental and operational loading is significantly increased beyond original design loading.
   4) During any inspection which is required and defined (per Ref. 4.1, Section 14.4), a significant damage to primary structure is found.

   (Significant is defined as a 10% increase in loading or decrease in total capacity.)

Selection of West Delta 103 "A" platform for this assessment is not based on any of the requirements listed above. It was merely based on the availability of platform data and permission to use it for the assessment exercise.

3. SUMMARY OF RESULTS

West Delta 103 "A" platform does not pass the assessment requirements of the draft Section 17 of API RP2A-WSD (20th Edition).

The assessment process took place at the following consecutive levels and the findings are summaries accordingly:

a) Condition Assessment
b) Design Basic Check
c) Design Analysis Check
d) Ultimate Strength Check
e) Pushover Strength Check I (for the purpose of overall stability)
f) Pushover Strength Check II (with failing diagonals in truss planes 1 through 4 removed and 90° loading only)

It is believed that failures are the result of:

a) weak soil layers at the top 50' below the mudline
b) unfavorable rough surface assumptions for jacket members (with C_D = 1.05 and C_M= 1.4), which develop high wave forces.

3.1 Condition Assessment (see Section 6.3)

3.1.1 Platform is not damaged.

3.1.2 Deck height is inadequate - bottom elevation of 40' is less than required elevation of 46'.

3.2 Design Basic Check (see Section 6.4)

3.2.1 Design basic check is bypassed due to 3.1.2.

3.3 Design Analysis Check (see Section 7.2)

Structure fails due to the following failures:

3.3.1 Pile Related Results (Section 7.2.3)

3.3.1.1 Soil Failure (Axial Compression)

No soil failure.

3.3.1.2 Pile Failure (Bending & Compressive Stress)

Stress U.C. > 1 only @ Pile Head Joint 182 for Load Cases 6, 7, and 8.

3.3.2 Jacket Member Results (Section 7.2.4)

3.3.2.1 Diagonal Member Buckling

a) Jacket Plane Rows 1, 2, 3, 4

Compression diagonals from mudline (Elev. -223') to Elev. -166' are all failing for L.C. 8 (90° direction).

KL/r = 99.6, D/t = 64

b) Jacket Plane Rows 1 & 3

Compression diagonals from Elev. -68' to Elev. -26.5' are failing for L.C. 8 (90° direction).

KL/r = 120, D/t = 40

c) Jacket Plane Row 4

Compression diagonal from Elev. -14.5' to Elev. -68' is failing for L.C. 8 (90° direction).

KL/r = 113.7, D/t = 64

3.3.2 Joint Can Analysis Results (Section 7.2.5)

No punching shear failure in design level analysis.

3.4 Ultimate Strength Check (see Section 7.3)

Structure fails due to the following failures:

3.4.1 Pile Related Results (Section 7.3.5.7)

3.4.1.1 Soil Failure (Axial Compression) (Section 7.3.5)

Soil failure in skin friction & end bearing for pile head 182 in L.C. 6 (45° direction).

3.4.1.2 Pile Failure (Bending & Compressive Stress) (Section 7.3.6)

All pile head joints develop plastic hinges in critical load case (L.C. 6).

Pile 182 develops further plastic hinges at pile depths 50' to 90' below the mudline, making itself unstable. For this reason, a pushover analysis was performed by replacing Pile 182 with reaction loads assumed to happen at the time of U.C. = 1 occurring at Joint 182 (see Section 7.3.6.4).

3.4.2 Jacket Member Results (Section 7.3.7)

a) Jacket Plane Rows 1, 2, 3, 4

Compression diagonals from mudline (Elev. -223') to Elev. -166' are all failing for L.C. 8 (in Plane 2, L.C. 7).

KL/r = 99.6, D/t = 64

b) Jacket Plane Rows 1 & 3

Compression diagonals from Elev. -68' to Elev. -26.5' are failing - Truss Row 1 in L.C. 7 (67.5° direction), and Truss Row 3 for L.C. 8 (90° direction).

KL/r = 120, D/t = 40

c) Jacket Plane Row 4

Compression diagonal from Elev. -14.5' to Elev. -68' is failing for L.C. 8 (90° direction).

KL/r = 113.7, D/t = 64

d) Jacket Planes A & B

Jacket Legs - 1A and 4B are failing in inelastic buckling.

Jacket Diagonals (see Section 7.3.7) - Truss Planes A and B1 are failing in compression and bending.

3.4.3 Joint Can Punching Shear Results (Section 7.3.8)

a) Punching shear failure in chord members at all mudline level joints (except 181) where chord is 1" thick.

b) Further failures in five other chord joints at higher elevations where chord thickness is ½" thick.

3.4.4 Conclusion of Ultimate Strength Analysis

3.4.4.1 Platform Assessment in Ultimate Strength Analysis fails due to:

a) Pile 182 collapses (foundation failure).
b) Jacket diagonal and leg buckle.
c) Punching shear failure in jacket leg chords.

3.4.4.2 Pushover Analysis will be performed for L.C. 6 (45° direction) to simulate load distribution that will occur in jacket / pile structure before Pile 182 collapses.

3.5 Pushover Strength Check I (see Section 7.4)

Intended to demonstrate the load redistribution when the ultimate strength of Pile 182 is reached (45° direction). Results showed that overall pile stability remains intact, but jacket local members and joints are still failing.

3.5.1 Pile Related Results (Section 7.4.3)

3.5.1.1 Soil Failure (Axial Compression) (Section 7.4.3.1)

No soil failure in skin friction and end bearing of piles.

3.5.1.2 Pile Failure (Bending & Compressive Stress) (Section 7.4.3.2)

All piles except conductors fail by forming plastic hinges at pile head regions. No secondary hinge formation occurs along the pile depth. The failures are therefore considered local in nature and the platform foundation will remain intact.

3.5.2 Jacket Member Failure

See Section 3.4.2 (Ultimate Strength Analysis considers more critical load direction).

3.5.3 Joint Can Failure

See Section 3.4.4 (Ultimate Strength Analysis produces more failed joints).

3.5.4 Conclusion of Pushover Strength Analysis I

3.5.4.1 Pushover Strength Analysis I shows that there is not a foundation failure, as indicated by Ultimate Strength Analysis.

Before Pile 182 collapses, a redistribution of loads and deformations will take place.

In Pushover Analysis I, no piles collapse.

3.5.4.2 The failure of the jacket is due to local member and joint can punching shear failures.

3.6 Pushover Strength Check II (Failing Diagonals in Truss Planes 1 through 4 Removed)

The intent of this check was to find out if the platform has any reserve strength left when failing diagonals in 90° direction truss planes are removed.

3.6.1 Pile Related Results (see Section 7.5.3)

3.6.1.1 Soil Failure (Axial Compression) (Section 7.5.3.1)

No soil failure in skin friction and end bearing of piles.

3.6.1.2 Pile Failure (Bending & Compressive Stress) (Section 7.5.3.2)

Pile Numbers 162, 172, and 182 collapse due to double plastic hinge formation.

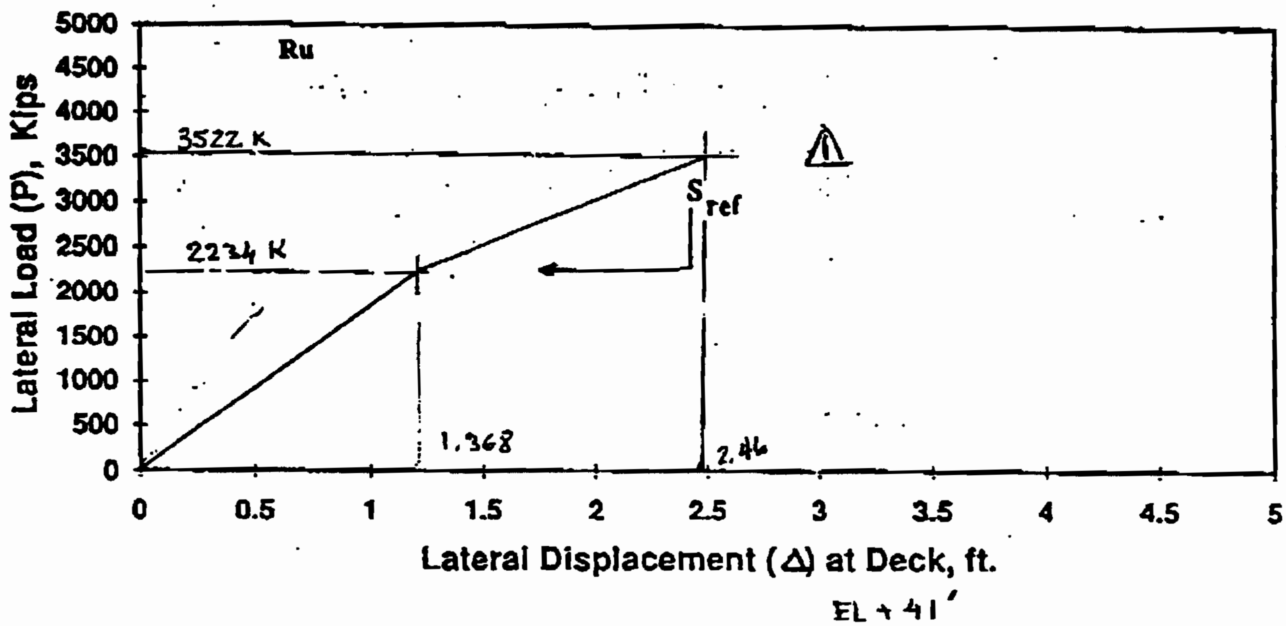
3.6.2 Jacket Member Failure

Jacket collapses due to failure of:

a) Jacket legs between El. -223' (mudline) and El. -114.5'
b) Piles inside legs between El. -223' and El. -114.5'
c) Jacket diagonals in vertical planes between El. -223' and El. -68'
d) Jacket horizontals in vertical planes El. -223' and -166'
e) Jacket diagonals in horizontal planes El. -223', -166', -114.5', -68'

PROJECT NO. 1836M01
WEST DELTA BLK 103A
PLATFORM ASSESSMENT
BY MEC DATE 7/30/94
CHK. DATE
SHEET 3 OF 31

Ultimate Strength Analysis- Minimum Required Results



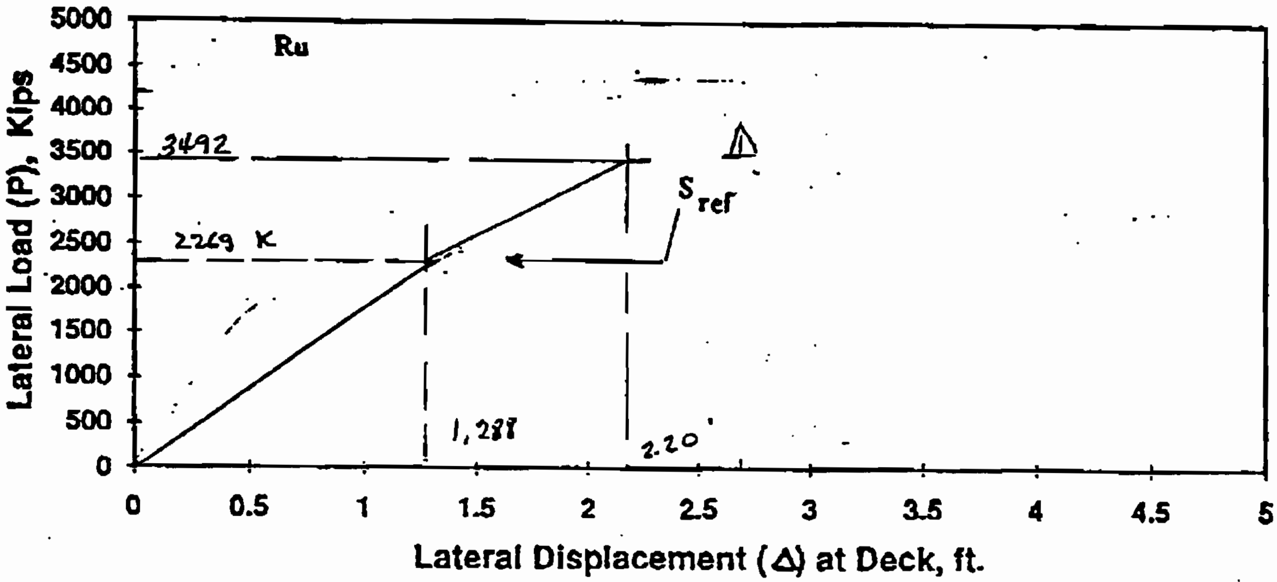
Orig. Design Reference Level Load ([redacted])	2234	Kips
Ultimate Capacity (R_u)	3522	Kips
Reserve Strength Ratio (RSR)	0.6	Kips
Platform Failure Mode: Jacket, Pile, Soils, etc.	Pile, Jacket	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 45°

11/30/94
REVISED GRAPH & Ru, RSR VALUES
BASED ON LATEST COMPUTER ANALYSIS

PROJECT NO.	W30M06
WEST DELTA BLK 103 A	BY MEC DATE 7/30/94
PLATFORM ASSESSMENT	CHK. ___ DATE ___
	SHEET 4 OF 31

Ultimate Strength Analysis- Minimum Required Results



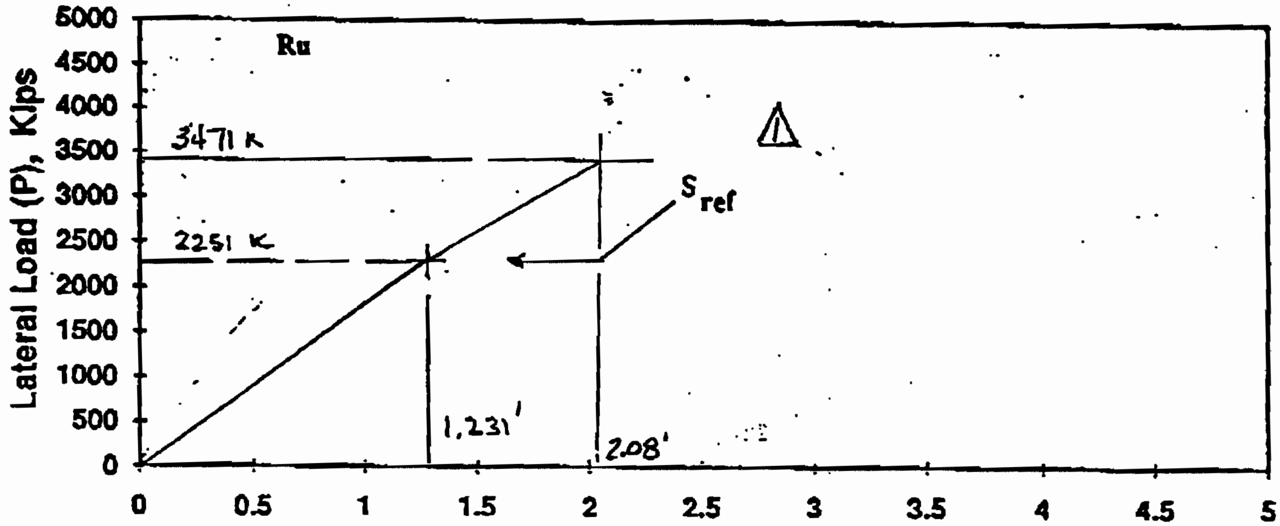
Orig. Design Reference Level Load (~~Sref~~)	2269	Kips
Ultimate Capacity (R_u)	3492	Kips
Reserve Strength Ratio (RSR)	0.6	Kips
Platform Failure Mode: Jacket, Pile, Soils, etc.	Jacket	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 67.5°

11-30-94
REVISED GRAPH & Ru, RSR VALUES
BASED ON LATEST COMPUTER ANALYSIS

WEST DELTA BLK 103 A	PROJECT NO. 1836 MBI
	BY MEC DATE 7/30/94
PLATFORM ASSESSMENT	CHK. ___ DATE ___
	SHEET 5 OF 31

Ultimate Strength Analysis- Minimum Required Results



EL + 41'

Orig. Design	Reference Level Load ~~(S ref)~~	2251	Kips
	Ultimate Capacity (R_u)	3471	Kips
	Reserve Strength Ratio (RSR)	0.6	Kips
	Platform Failure Mode: Jacket, Pile, Soils, etc.	JACKET	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 90°

Δ 11-30-94
REVISED GRAPH & Ru, RSR VALUES
BASED ON LATEST COMPUTER ANALYSIS

Analysis case: 3-D Model

45° Direction (L.C. 6) SHEET 1 OF 3

Load Step ULTIMATE DESIGN LEVEL	Lateral Displacement at Main Deck Level (+40') SEE SECTION 8.2.3 " " 8.1.3 ft.	Lateral Load SEE SECTION 8.2.1 P. 96 L.C. 6 Kips	Element Failure Number* (optional)	Component Failure Mode ** (optional)	Remarks
	UPPER DECK EL +55'		FOR JACKET PLANE A & B FAILURES SEE FOLLOWING TWO SHEETS		
1	JNT 154 X = 27.935" = 2.328'	Fx = 2908			
2	P. 12 Y = 21.154" = 1.763'	Fy = 2864			
3	LOWER DECK EL +41		PILE HEAD FAILURES	SEE SECTION 8.2.2	P. 234
4	JNT 38 X = 28.135" = 2.344'	$(F_x^2+F_y^2)^{1/2}$ = 4082	PILE HEAD		
5	P. 3 Y = 21.612" = 1.801'		112	BENDING	U.C = 1.23
6	$(X^2+Y^2)^{1/2}$ = 35.478" = 2.95'		122		= 1.09
7		P. 96 SECTION 8.1.1	132		= 1.15
8	JNT 154 X = 12.504" = 1.042'	Fx = 1580 K	142		= 1.26
9	Y = 10.463" = .872'	Fy = 1579 K	152		= 1.04
10	JNT 38 X = 12.525" = 1.044		162		= 1.31
	Y = 10.619" = .885	$(F_x^2+F_y^2)^{1/2}$ = 2234 K	172		= 1.29
	$(X^2+Y^2)^{1/2}$ = 16.421" = 1.368'		182	DOUBLE HINGE @	= 1.54
				PILE HEAD & 66' BELOW	= 1.16

* Failure number of element from analysis and as marked on the platform sketches

** Identify failure mode of a component: buckling, yielding, double hinge, etc.

STRUCTURE NORTH

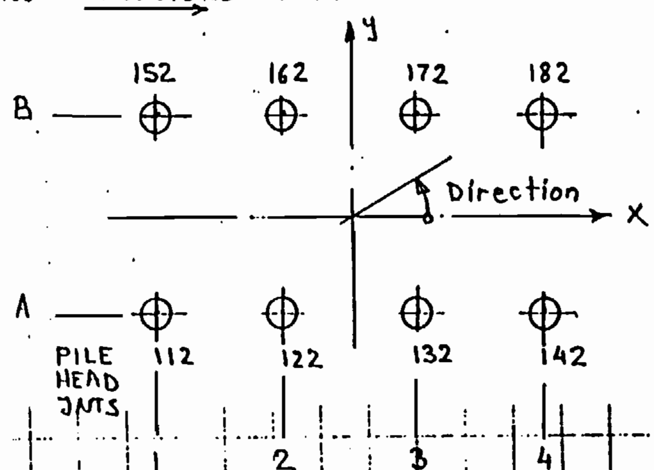


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

(7.3.6 Continued) USED ALSO FOR TRIAL DOCUMENT TABLE 2
SHEET 2 OF 3

WEST DELTA BLK 103A 223 FT WATERS

BROADSIDE VIEW ROW A

SEE COMPUTER OUTPUT WD103US. OT3 PAGES FOR STRESS & U.C. COMPONENTS

P. 84 STRESS KSI — P. 36 U.C. COMPONENTS

$f_a = -38.13$ — 1.016 — KL/r = 26.7

$f_{by} = 6.92$ — .137 — U.C. = 1.158

$f_{bz} = -1.74$ — .035

P. 84 STRESS KSI — P. 29 U.C. COMP

$f_a = -13.62$ — .616

$f_{by} = -3.08$ — .121

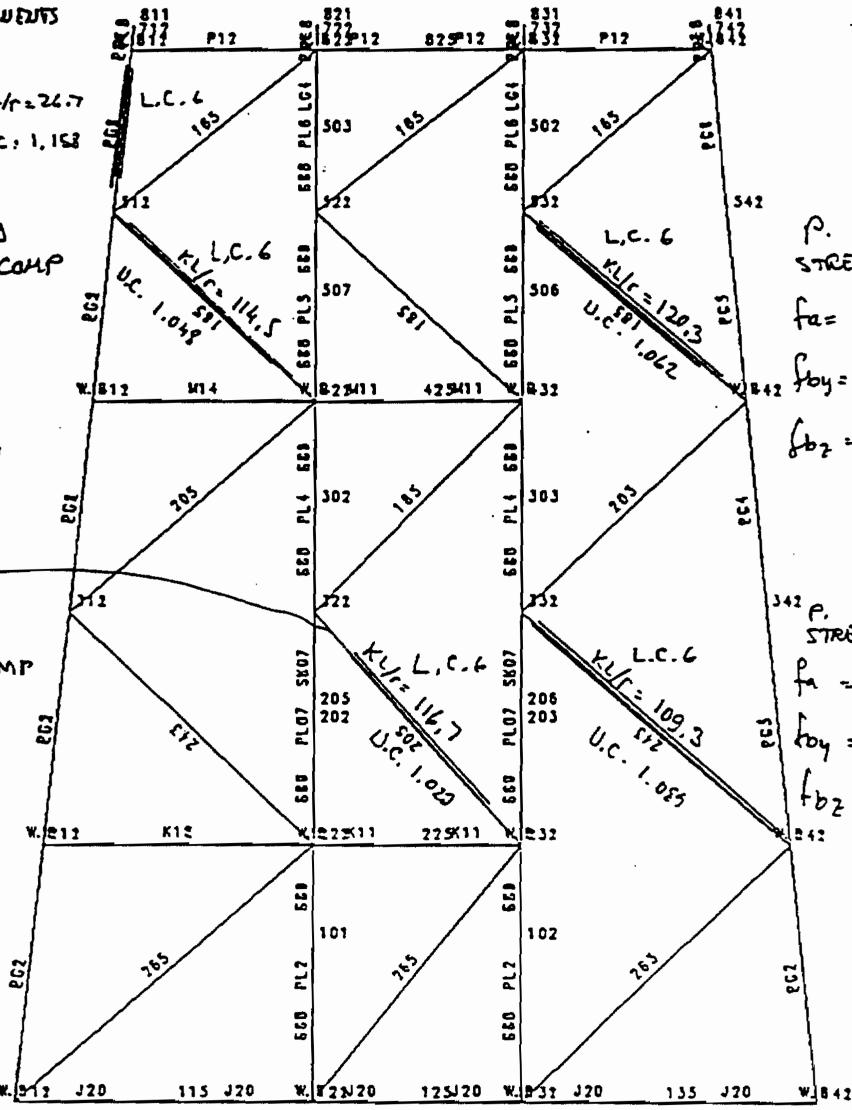
$f_{bz} = -10.58$ — .414

P. 84 STRESS KSI — P. 29 U.C. COMP

$f_a = -11.41$ — .558

$f_{by} = -6.02$ — .212

$f_{bz} = -12.98$ — .457



P. 84 STRESS KSI — P. 29 U.C. COMP

$f_a = -15.40$ — .717

$f_{by} = -2.14$ — .113

$f_{bz} = -5.32$ — .281

P. 84 STRESS — P. 30 U.C. COMP

$f_a = -15.00$ — 0.640

$f_{by} = -5.57$ — 0.234

$f_{bz} = -9.10$ — 0.383

LEG MEMBER FAILURE : BUCKLING IN INELASTIC REGION

TRUSS DIAGONAL : BUCKLING + BENDING (U.C ARE SLIGHTLY ABOVE 1.0)

PROJECT NO. 1830M01

WEST DELTA BLK 103A — BY MEC DATE 7/23/96

PLATFORM ASSESSMENT — CHK. ____ DATE ____

(7.3.6 Continued) — USED ALSO FOR TRIAL DOCUMENT TABLE 2 SHEET 3 OF 3 — SHEET B OF 31

WEST DELTA BLK 103A 223 FT WATERS

BROADSIDE VIEW ROW B

NOTE: STRESS & U.C COMPONENTS FROM WD103US.OT3 output

OUTPUT p. 84 STRESS — p. 29 U.C COMPONENTS

f_a = −10.28 KSI — .468

f_{by} = −8.97 KSI — .259

f_{bz} = −16.64 KSI — .481

p. 36 U.C Components

Axial .921

Y-Axis .119

Z-Axis .035

KL/r = 26.7

UC = 1.053

p. 84

f_a = 40.12 KSI

f_{by} = −5.61 "

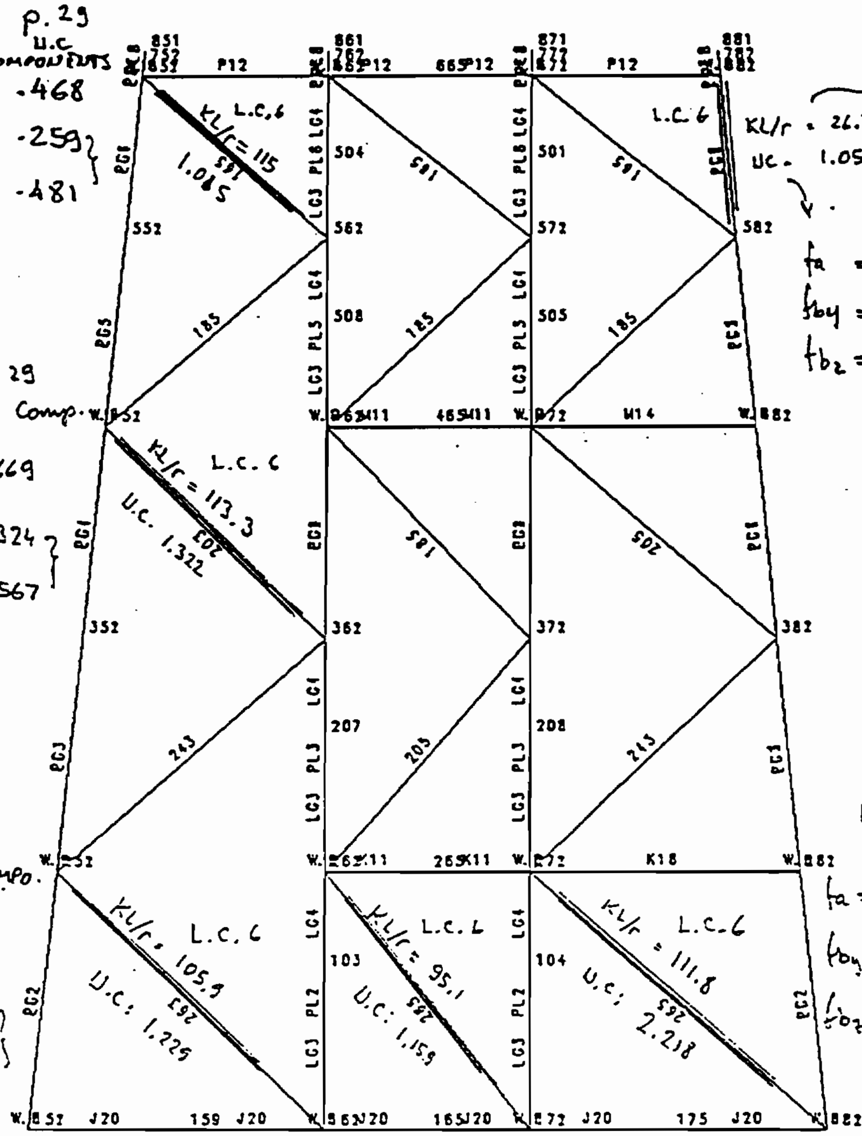
f_{bz} = 1.64 "

p. 84 STRESS — p. 29 U.C Comp.

f_a = −15.00 KSI — .669

f_{by} = −7.06 " — .324

f_{bz} = −12.34 " — .567



p. 84 STRESS — p. 31 U.C. COMPO.

f_a = −19.30 KSI — .798

f_{by} = −4.64 " — .288

f_{bz} = −5.16 " — .320

p. 83 STRESS — p. 31 U.C. COMP.

f_a = −21.84 — .956

f_{by} = −5.60 — 1.039

f_{bz} = −3.63 — .686

p. 84 STRESS — p. 31 U.C COMPONENTS

f_a = −22.56 KSI — 0.830 — 0,830

f_{by} = −6.26 " — 0,320

f_{bz} = −1.54 " — 0,079 — 0,330

1,160

FAILURES

LEG MEMBER ; INELASTIC BUCKLING

TRUSS DIAGONALS ; BUCKLING + BENDING

	PROJECT NO. 1830Md1
WEST DELTA BLK 103A	BY MEC DATE 7/30/94
PLATFORM ASSESSMENT	CHK. ____ DATE ____
	SHEET 9 OF 31

Analysis case: 3-D Model

67.5° Direction (L.C. 7)

Load Step ULTIMATE DESIGN LEVEL	Lateral Displacement at Main Deck Level (+40') SEE SECTION 8.2.3 " " 8.1.3 ft.	Lateral Load SEE SECTION 8.2.1 p.96 LC. 7 Kips	Element Failure Number* SECTION 7.3.6 JACKET (optional)	Component Failure Mode ** (optional)	Remarks
1	EL + 55' JNT 154 X = 14.924" = 1,244'	Fx = 1588 K	PLANE ① JA-JB 451-515	Mainly Buckling + Bending	U.C = 1,221 JACKET DIAGONAL
2	p.12 y = 27.445" = 2.287'	Fy = 3786 K			
3	EL + 41'		JACKET PLANE ②		
4	JNT 38 x = 14,992" = 1,249'	$(F_x^2+F_y^2)$ = 4106 K	JA-JB 155-221	Mainly Buckling + Bending	U.C = 215.791 "
5	p.3 y = 28.020" = 2,335'				
6	$(x^2+y^2)^{1/2}$ = 31.779 = 2.648		PILE HEAD FAILURES	SEE 8.2.2 p.234	
7	p.	96 SECTION 8.1.1	PILE 112	BENDING	U.C = 1.08
8	JNT 154 x = 6,822" = .569'	Fx = 869 K	122		= 1.02
9	y = 13,683" = 1,140'	Fy = 2096 K	132		= 1.07
10	JNT 38 x = 6.803" = .567'		152		= 1.03
	y = 13,875" = 1.156'	$(F_x^2+F_y^2)^{1/2}$ = 2269 K	162		= 1.06
	$(x^2+y^2)^{1/2}$ = 15.453 = 1.288'		172		= 1.25
			182		= 1.43

* Failure number of element from analysis and as marked on the platform sketches

** Identify failure mode of a component: buckling, yielding, double hinge, etc.

STRUCTURE NORTH

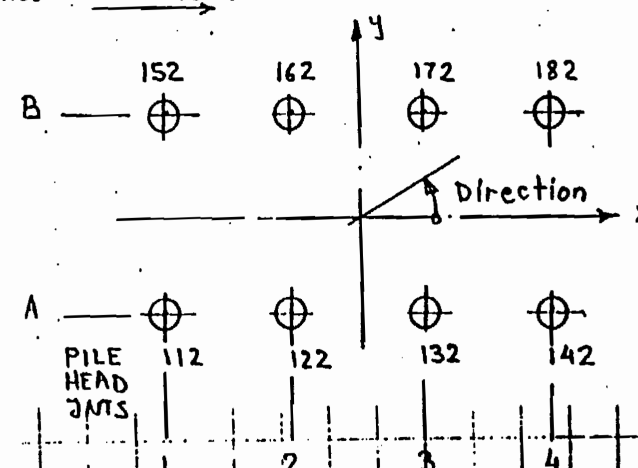


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

Analysis case: 3-D Model

90° Direction (L,C,8) SHEET 1 OF 2

Load Step ULTIMATE DESIGN LEVEL	Lateral Displacement at Main Deck Level (+40') SEE SECTION 8.2.3 " " 8.1.3 ft.	Lateral Load SEE SECTION 8.2.1 p. 97 L.C. 8 Kips	Element Failure Number* SEE SECTION 7.3.6 (optional)	Component Failure Mode ** (optional)	Remarks
	EL + 55'		JACKET PLANE ①	Mainly	
1	JNT 154 X = .532" = .044'	Fx = 57	JA-JB 145-211	Buckling + bending	U.C = 1.768
2	p.12 Y = 29.210" = 2.434'	Fy = 4072		"	
3	EL + 41'		JACKET PLANE ③		
4	JNT 38 X = .469" = .039'	$(F_x^2+F_y^2)$ = 4072	JA-JB 146-231	"	U.C = 142.681 JACKET
5	p.3 Y = 29.816" = 2.485'		JA-JB 471-531	"	U.C = 1.284 DIAGONALS
6	$(X^2+Y^2)^{1/2}$ = 29.820" = 2.485'		JACKET PLANE ④		
7		p.97 SECTION 8.1.1	JA-JB 148-241	"	U.C = 2.814
8	JNT 154 X = .380" = .032	Fy = 30 K	JA-JB 381-441	"	U.C = 2.722
9	Y = 14.567" = 1.214	Fy = 2251 K			
10	JNT 38 X = .320" = .027		JA-JB 152-252	Bending	U.C = 1.14 PILE HEAD
	Y = 14.770" = 1.231	$(F_x^2+F_y^2)^{1/2}$ = 2251 K			
	$(X^2+Y^2)^{1/2}$ = 14.775" = 1.231		SEE NEXT PAGE	FOR PILEHEAD JOINT	FAILURES →

* Failure number of element from analysis and as marked on the platform sketches

** Identify failure mode of a component: buckling, yielding, double hinge, etc.

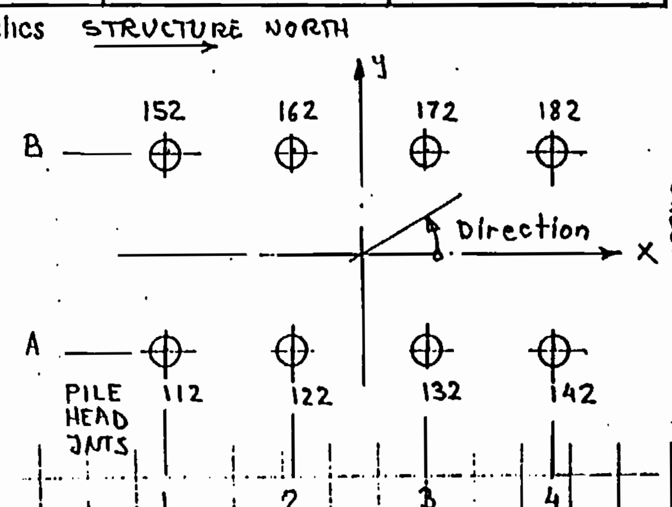


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

Analysis case: 3-D Model

90° Direction (L.C. 8) SHEET 2 OF 2

continued for Pile Head Failures

Load Step	Lateral Displacement at Main Deck Level (+40') ft.	Lateral Load Kips	Element Failure Number* SEE SECTION 8.2.2 P. 234 (optional)	Component Failure Mode ** (optional)	Remarks
1			Pile 132	BENDING	U.C = 1.02
2			142	"	U.C = 1.01
3			152	"	U.C = 1.14
4			162	"	U.C = 1.23
5			172	"	U.C = 1.26
6			182	"	U.C = 1.33
7					
8					
9					
10					

* Failure number of element from analysis and as marked on the platform sketches

** Identify failure mode of a component: buckling, yielding, double hinge, etc.

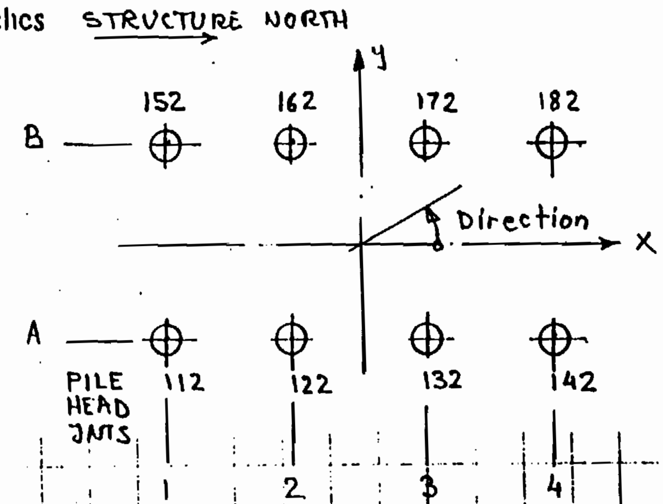


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

4.0 REFERENCES

4.1 API RP2A-WSD 20th Edition with Draft Section 17 (April 1, 1994)

4.2 Drawing Set No. D-64-19 and 22 for / by Company (New Orleans Area), "Self-Contained Drilling Platform "A", West Delta Block 103, 8-Pile, 12-Well, 223' Water."

4.3 Soil Report No. 64-237-5 dated February 22, 1965, by McClelland Engineers, 1018 Richard Bldg., New Orleans, LA.

4.4 Applied Hydraulics in Engineering, Henry M. Morris & James M. Wiggert, 2nd Edition, John Wiley & Son, 1972.

4.5 Answers marked on faxed questions, by ., New Orleans, LA. Fax from
Offshore, dated 6/17/94, 08:46 (See Appendix).

4.6 StruCAD*3D Program Version 3.42 (September 1993), Structural Software, Inc., Houston, TX.

4.7 Drawing Set No. 255A-90-4A & 4B by , "Equipment Layout, Upper Deck & Lower Deck, West Delta Block 103 Platform "A", West Delta 105 Field."

5. ASSUMPTIONS

The following assumptions used in this study still require verification:

5.1 Load intensities for deck & equipment (Section 7.1.3.3), uniform deck loads.

5.2 Rough surface assumptions for jacket members (with C_D = 1.05 and C_M = 1.40).

5.3 Marine growth distribution (Section 7.1.3.5.1).

6. METHODOLOGY

The assessment process is done following the guidelines in Section 17.5 of Ref. 4.1.

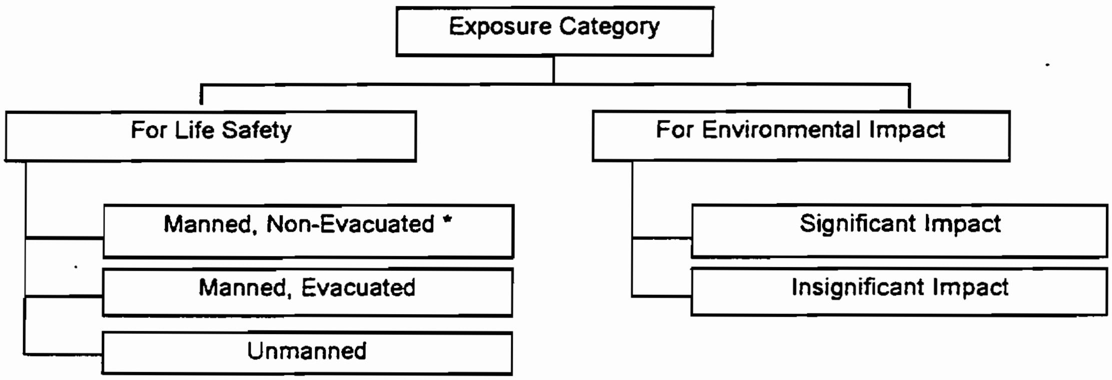
There are six major components used during this process:

6.1 Select Platform for Assessment

When one or more conditions noted in Ref. 4.1, Section 17.2 exist(s). See also Section 2 of this study.

6.2 Establish the Exposure Category

Per Ref. 4.1, Section 17.3, there are six categories (as shown below:



* As per industry practice in Gulf of Mexico not applicable.

I West Delta 103 "A" platform is categorized as follows (Ref 4.5):

For Life Safety	⟶	Manned Evacuated
For Environmental	⟶	Insignificant Impact

6.3 Collect Information and Perform Condition Assessment

Per Ref. 4.1, Section 17.4, the following information is required:

6.3.1 General

6.3.1.1 Current inventory of platform's structural condition.

6.3.1.2 Current inventory of facilities.

6.3.1.3 All information gathered shall be up-to-date, accurate, and shall reflect actual conditions. Any assumptions made shall be specified and reasonable.

6.3.2 Surveys

6.3.2.1 Topside Survey - Level I per Section 14.3.1

a) Accuracy of drawings should be verified.

b) If not accurate, or drawings do not exist, additional walk-around surveys may be required for topside structure, facility, framing details, and exposure category.

6.3.2.2 Underwater Survey

a) As minimum Level II per Section 4.3.2 (existing or new survey).

b) Additional surveys Level III & IV per Sections 14.3.3 and 14.3.4 to verify suspected damage, deterioration due to age, lack of joint cans, major modifications, lack of / suspect accuracy of platform drawings, poor inspection records or analytical findings.

6.3.2.3 Soil Data

Many older platforms were installed based on soil boring information at considerable distance away from the installation site.

Available or near-site soil borings and geophysical data should be reviewed and considered assessment.

6.3.4 Results of Condition Assessment for W.D. 103 "A"

6.3.4.1 Platform is not damaged.

6.3.4.2 Deck height is inadequate. Ref. 4.1, p. 30, Fig. 2.3.4-8 requires a deck height of +46'. Bottom of lower deck beams are at elevation +40'.

6.3.4.3 Topside survey has not been received.

6.3.4.4 Underwater survey has not been received.

6.3.4.5 Soil data used (Ref. 4.3) is for actual platform site.

6.4 Design Basis Check

This screening process is performed per Ref. 4.1, Section 17.5 and 17.6, also shown in Fig. 17.5.2.

If the platform passes the Design Basis Check, the assessment process ends. If not, the assessment continues into analysis level.

6.5 Design Level Analysis

Design Level Analysis is performed in 3D computer model. (See Section 7, "Calculations," and Sub-Section 7.2).

6.6 Ultimate Strength Analysis

Ultimate Strength Analysis is performed by removing safety factors and using mean yield stress instead of nominal yield stress (see Section 7.3).

6.7 Pushover Strength Analysis I

Pushover Strength Analysis I is performed to assess the global strength of the platform so that the pile foundations do not collapse (as was appearing in the Ultimate Strength Analysis). The methods used in Ultimate Strength and Pushover Analyses are the same, except for the representation of "ultimate capacity of failing pile" in Ultimate Strength model (see Sections 7.3.5.2.4 and 7.4).

6.8 Pushover Strength Analysis II

Pushover Strength Analysis II is performed to establish the reserve strength of the jacket after its vertical plane diagonals failed in 90° direction Ultimate Strength Loading. Since the failing diagonal members were compression members (i.e. buckling failure) they were removed in the computer model used for Pushover Strength Analysis II. The loading in Pushover analysis is the same as in the Ultimate Strength Analysis in 90° direction (see Section 2.5).

PLATFORM "O"

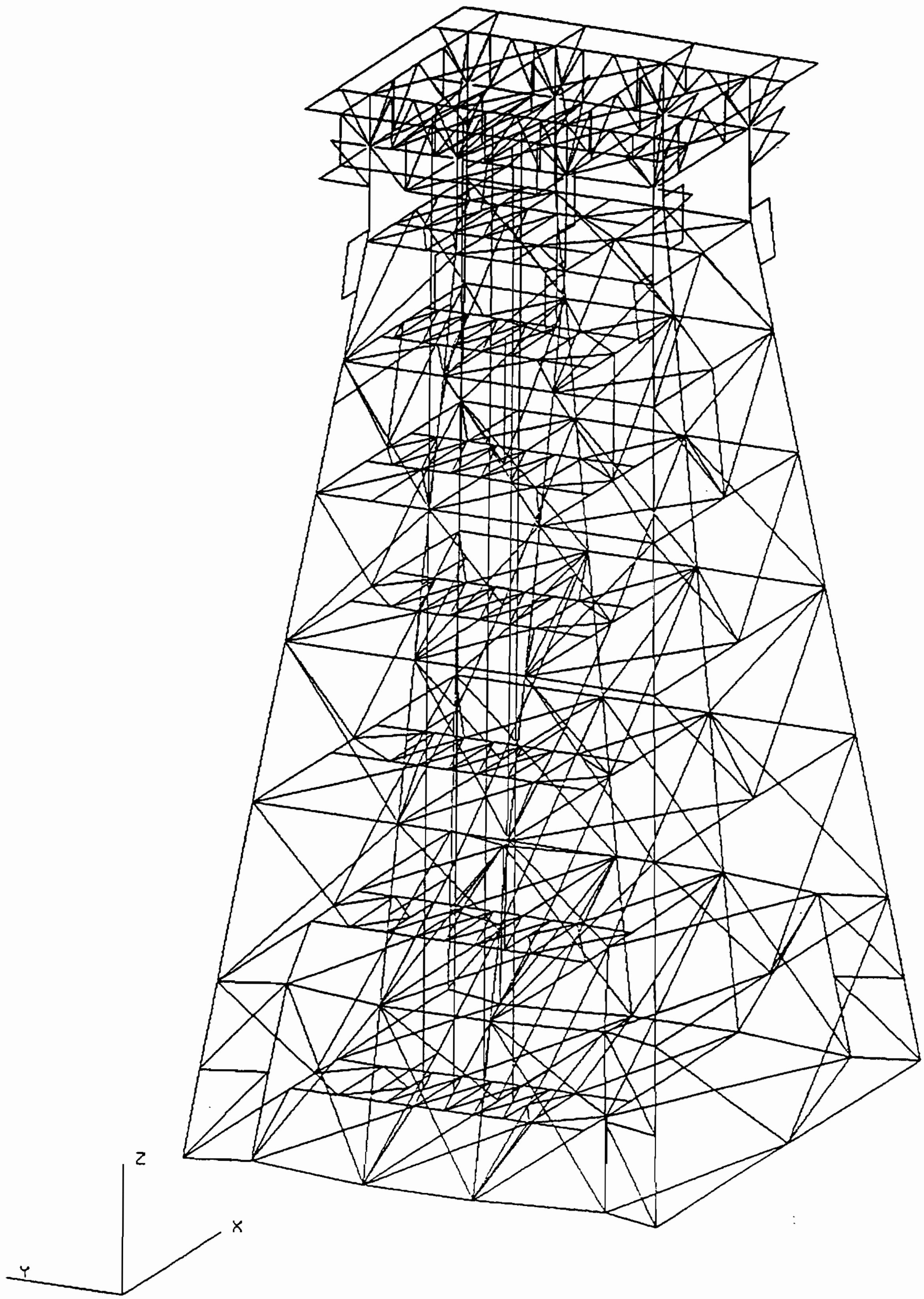
1.0 PLATFORM INFORMATION

Platform "O" is located in the Gulf of Mexico. The following are the salient features of the platform.

Water Depth	-	300 ft.
No. of Legs	-	8
Batter of Legs		
- Longitudinal	-	1:7.86
- Transverse	-	1:7.86
No. of Piles		
- Main	-	8 - Grouted
- Skirt	-	4 - Grouted
Type of Platform	-	Drilling/Production/Quarters Platform Manned and Evacuated during Hurricane
Brace Type in Vertical Frames	-	K Braces
Cellar Deck Elevation	-	46'-6"
No. of Wells	-	24 - (30" Dia. X 0.50)
Year Designed	-	1973
Year Installed	-	1973 in 300 ft. water depth
Pile Size		
- Main	-	48" Dia.
- Skirt	-	42" Dia.
Pile Penetration		
- Main	-	340.00 ft.
- Skirt	-	320.00 ft.
Barge Bumpers	-	8

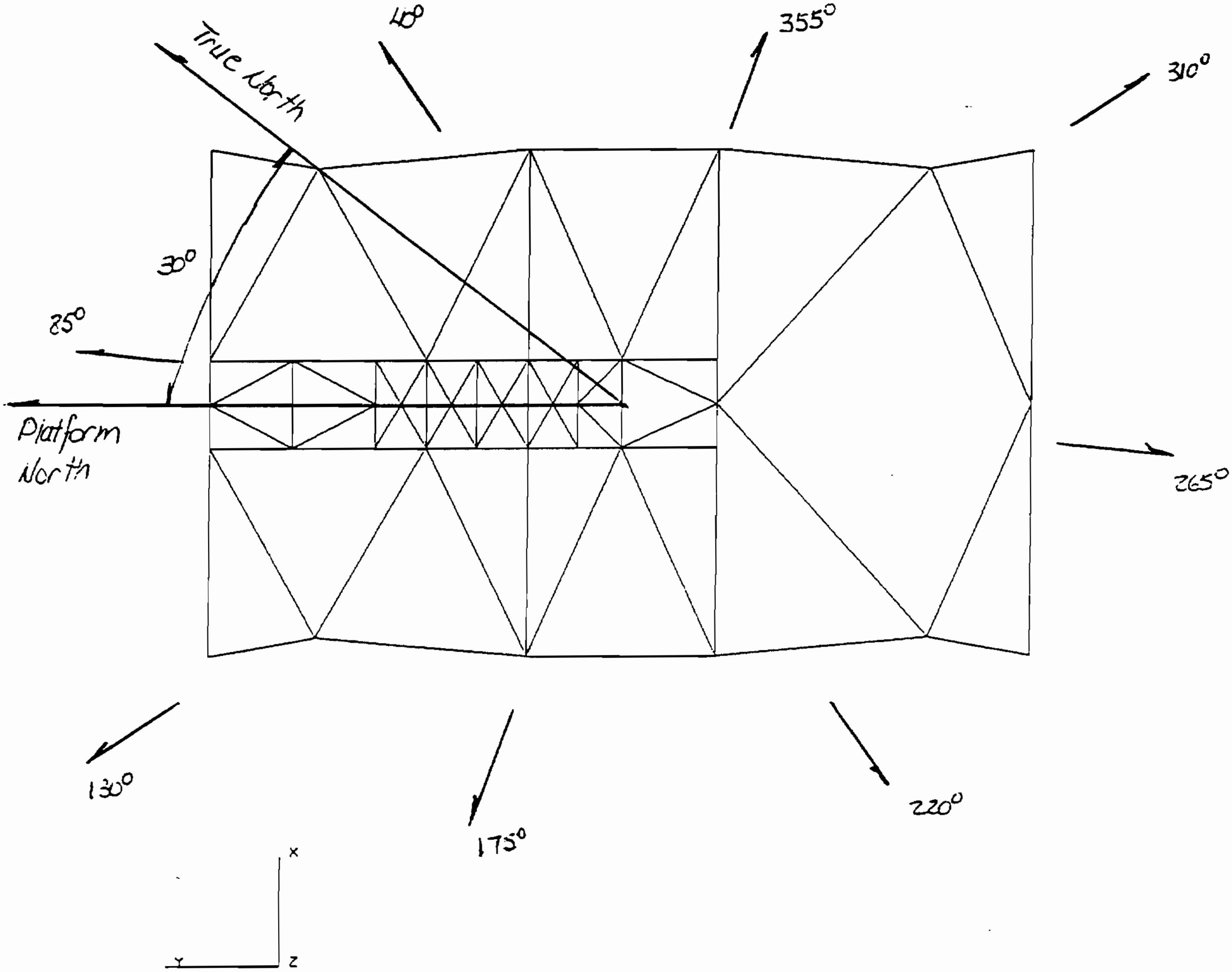
The soil boring at the platform site was used for development of the soil characteristics considered in the assessment.

JOINT INDUSTRY PROJECT - EUGENE ISLAND 349 "A" - 229 FT. WATER - FINAL I



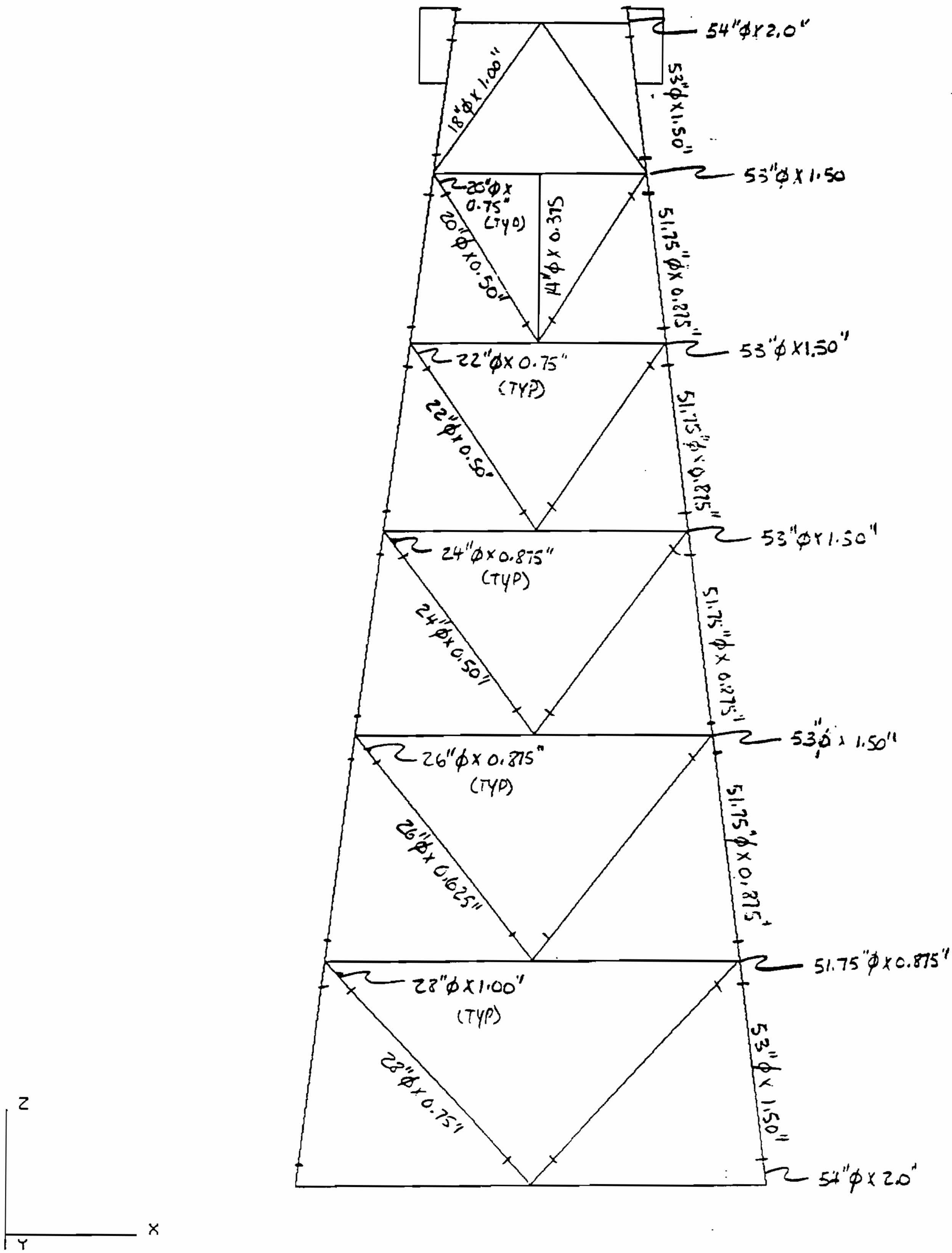
Platform "O"

JOINT INDUSTRY PROJECT - PLATFORM "O"   PLAN AT ELEV. (-) 295'-0"

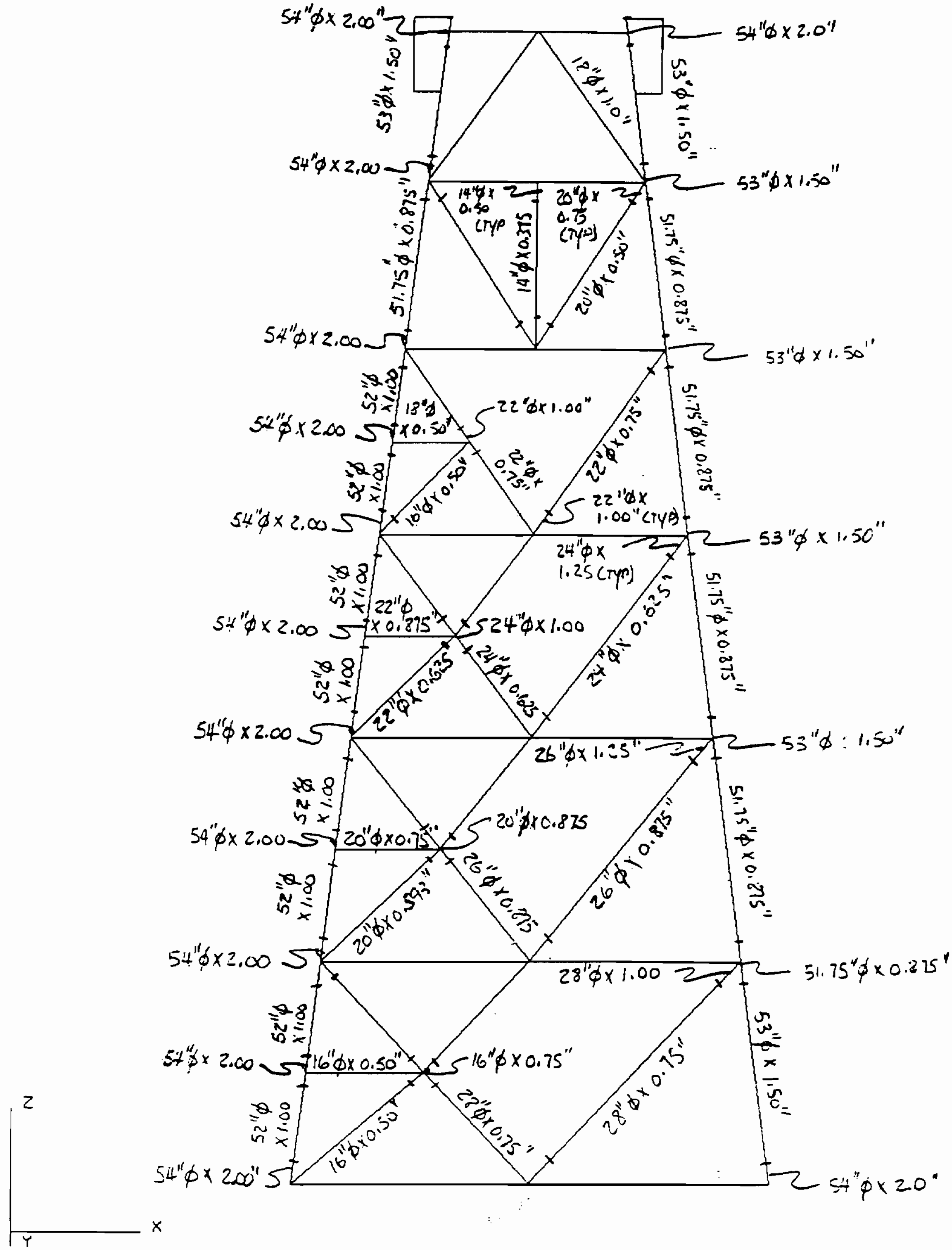


Platform Orientation

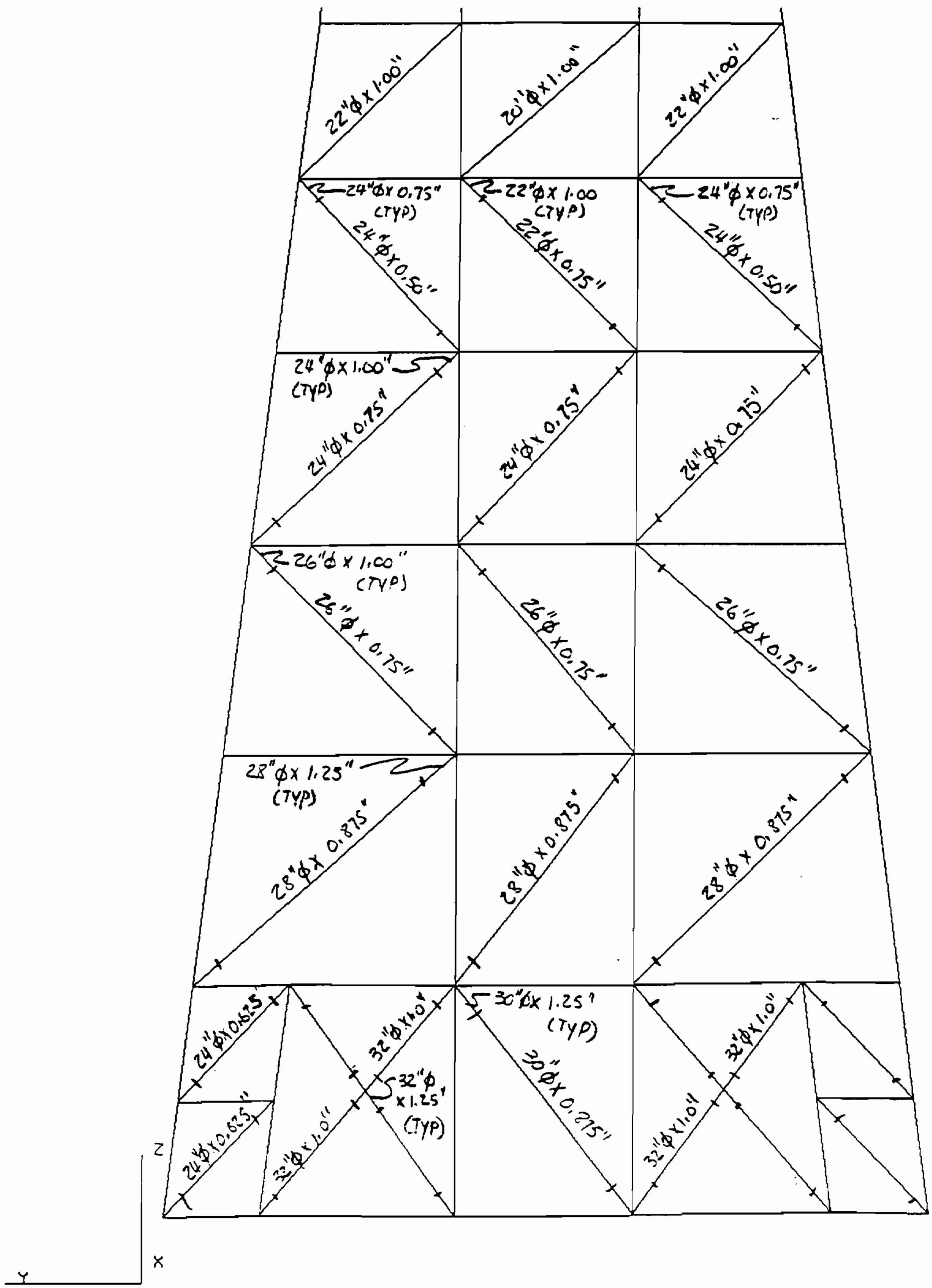
JOINT INDUSTRY PROJECT - PLATFORM "O" JACKET AT ROW 1



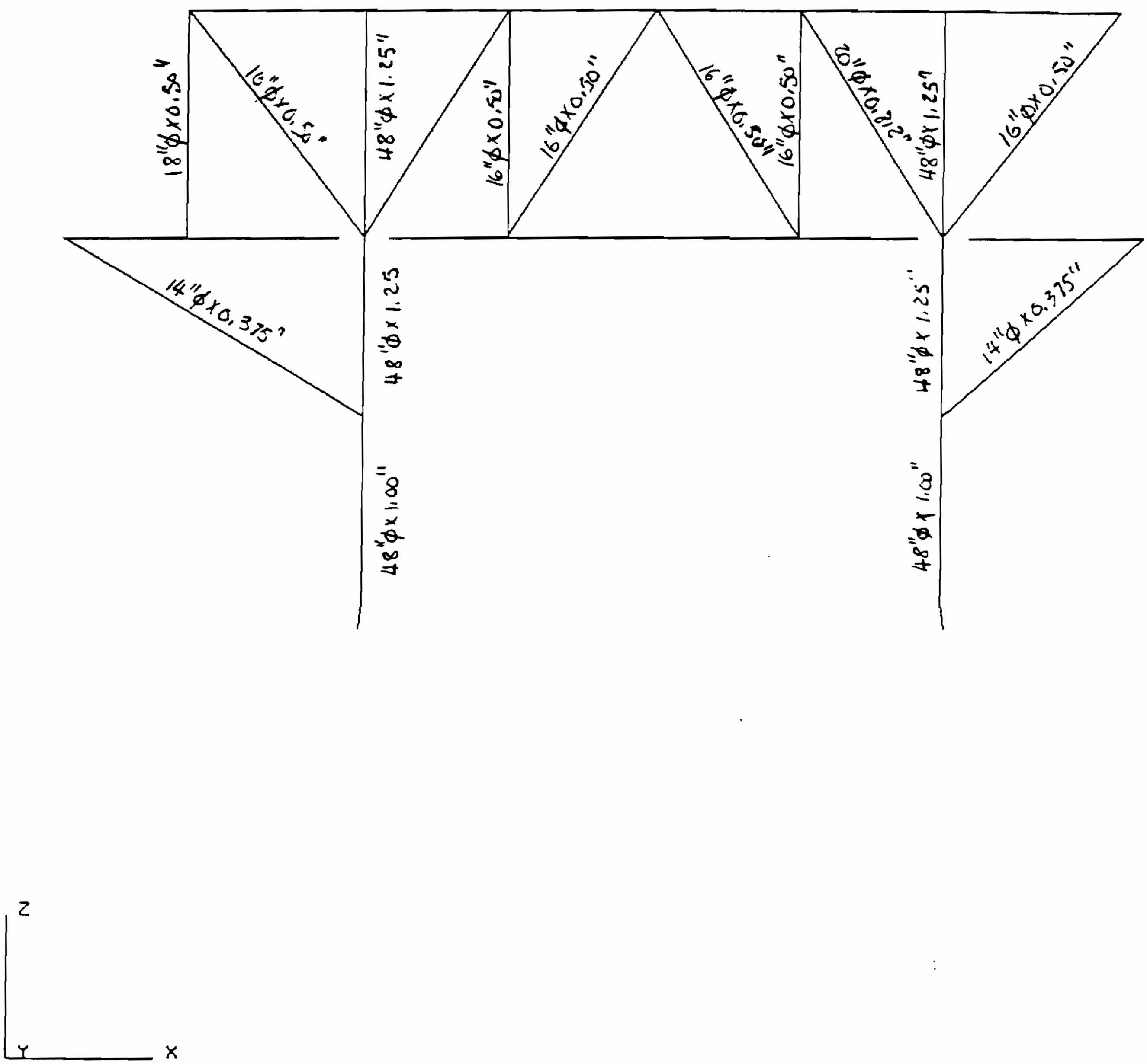
JOINT INDUSTRY PROJECT - PLATFORM "O"   JACKET AT ROW 2



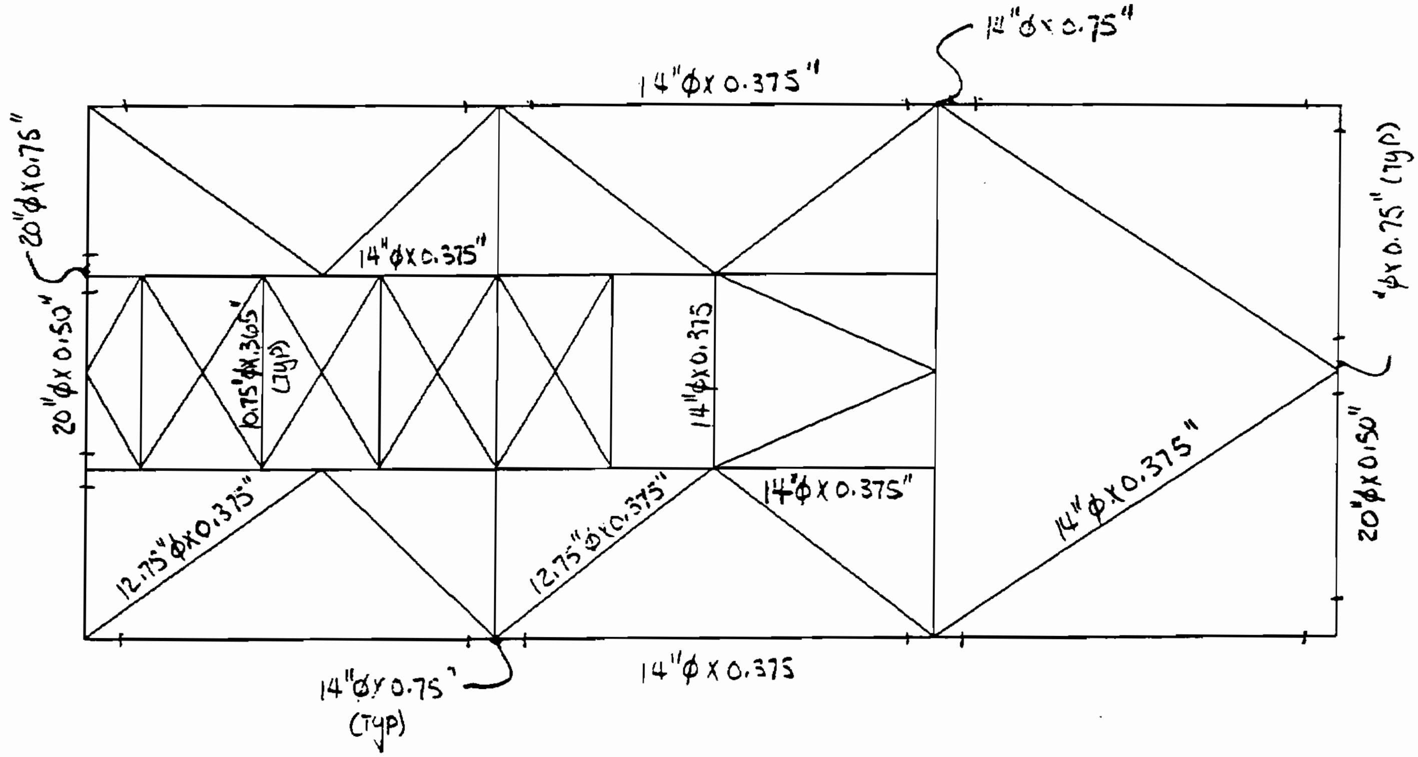
JOINT INDUSTRY PROJECT - PLATFORM "O"   JACKET AT ROW A

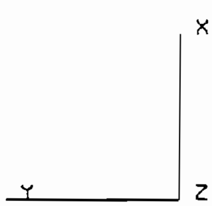


JOINT INDUSTRY PROJECT - PLATFORM "O"   DECK AT ROW 1

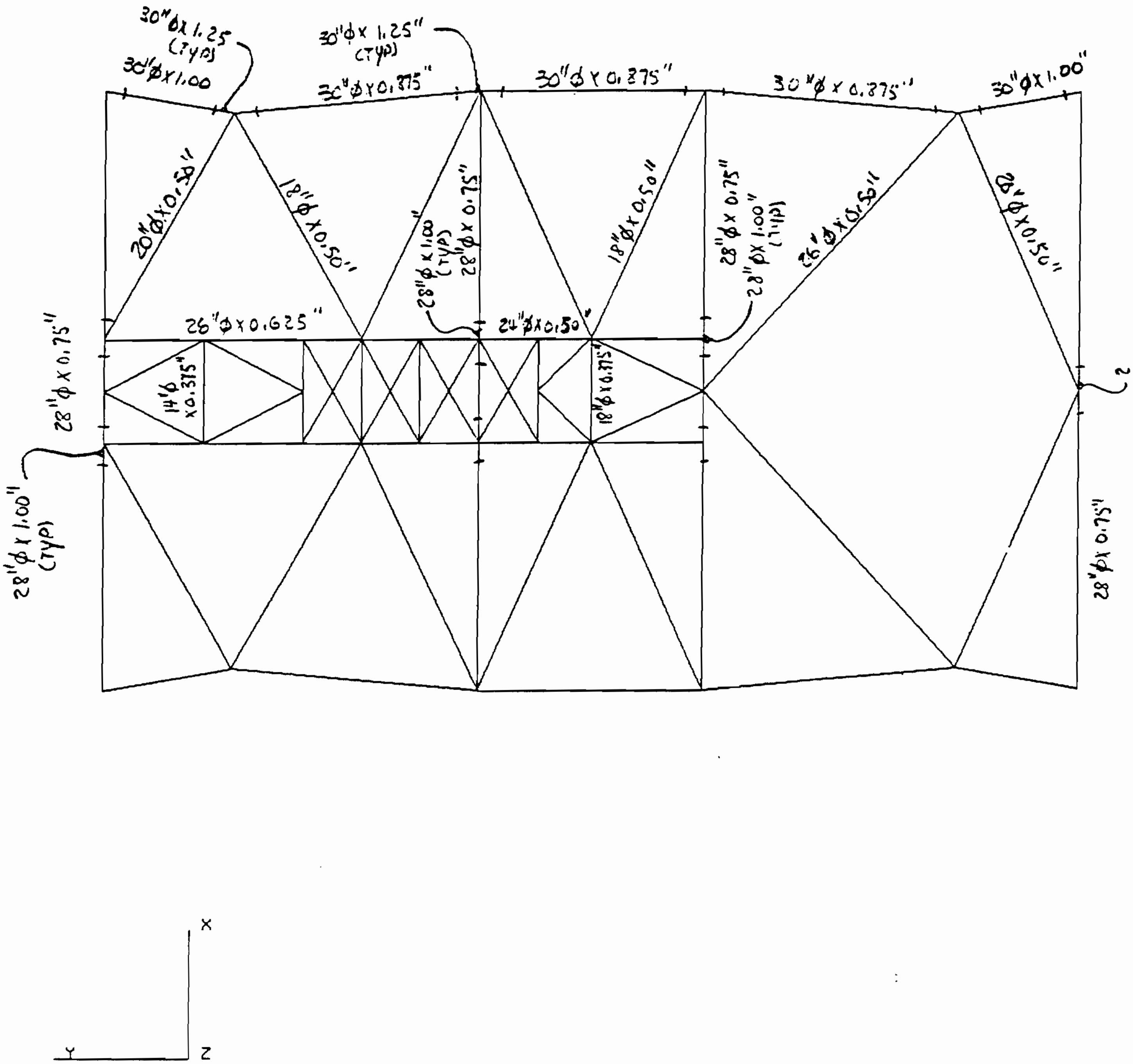


JOINT INDUSTRY PROJECT - PLATFORM "O"   PLAN AT ELEV. (-) 25'-0"





JOINT INDUSTRY PROJECT - PLATFORM "O"   PLAN AT ELEV. (-) 295'-0"



PART A: PLATFORM ASSESSMENT

A.1 **Platform Selection:** The platform should undergo assessment if one or more of the conditions below exists and are satisfied:

A.1.1 Addition of Personnel (Section 17.2.1): The manning condition has not changed.

This condition is not satisfied.

A.1.2 Addition of Facilities (Section 17.2.2): No significant addition of facilities relative to the original operational loads have been made.

This condition is not satisfied.

A.1.3 Increased Loading on Structure (Section 17.2.3): No addition or changes to the facility has resulted in an increase of loading by more than 10% beyond the original design loads using the original design criteria.

This condition is not satisfied.

A.1.4 Inadequate Deck Height (Section 17.2.4): The minimum deck height required from Fig. 17.6.2-3B for the platform is 36'-6" ft. The elevation of the lowest point on the cellar deck is 46'-6".

This condition is not satisfied.

A.1.5 Damage Found During Inspection (Section 17.2.5): Information regarding results of the most recent inspection were not available. Assessment of the structural integrity for the platform due to possible effects of corrosion and/or damage to the structure could not be completed due to absence of the inspection records.

This condition is satisfied.

A.2 **Condition Assessment (Section 17.4):** Assessment of the platform structural condition could not be verified. Information regarding the annual required Level 1 survey (topside) and any other subsequent and additional survey levels (underwater) which may have been performed was not available.

The soil boring log developed for the original platform site location in 1973 was used for the review and assessment process.

A.3 **Categorization:** For assessment purposes, the platform was categorized as manned, evacuated with insignificant environmental damage.

A.4 **Design Basis Check (Section 17.2):** The original design basis was reviewed to determine if initiators were satisfied which would require the platform to be assessed. The initiators included a review of the possible addition of personnel and/or facilities, a review of the minimum deck elevation against the current criteria and a review of any reported damage and/or corrosion.

Information on reported damage and/or corrosion was not available so a complete review of this category could not be accomplished. Due to the absence of this information and because the structure was designed prior to the 9th Edition of API RP2A (1977), a detailed assessment is required.

A.5 **Analysis Checks:** A design level analysis was performed.

A.5.1 Design Level Analysis: The platform model developed for the design level analysis included twenty four conductors and all appurtenances. The reduced thickness of members due to corrosion was not modelled. This was done to check initially whether the platform passes the design level analysis in the intact state.

SACS program was used for the analysis.

The design level wave, current and wind were applied to the platform in eight different directions all 45° apart. Wave kinematic factor and current blockage factor were considered in the analysis. The following sudden hurricane metocean criteria was used for the design level analysis:

Wave Height	-	47 ft. (Fig. 17.6.2-3A) (Omni-directional unless it was found greater than ultimate analysis wave height for certain directions)
Wave Period	-	11.3 secs. (Table 17.6.2.1)
Current Speed	-	1.20 knots (Omni-directional unless greater than ultimate analysis current for certain directions)
Storm Tide	-	3.5 ft. (Fig. 17.62-3A)
Wind Speed (1 hr. @ 10m)	-	55 knots (Table 17.6.2-1)
Marine Growth	-	1.5"
Minimum Deck Height Required	-	36.5 ft. (Fig. 17.6.2-3B)

The bottom of the cellar deck beams is 46'-6". Since minimum deck height required is satisfied, it is not mandatory to perform ultimate strength analysis.

The results from the analysis are summarized below:

Maximum Base Shear (wave direction)	-	2,422 kips 355 degrees
Maximum Overturning Moment (wave direction)	-	606,503 (ft. kips) 85 degrees
Max. Compressive Pile Load (wave direction)	-	15,180.2 kips 85 degrees
Min. Safety Factor on Pile Compression Capacity	-	2.61
Max. Tensile Pile Load (wave direction)	-	285.5 kips 85 degrees
Min. Safety Factor on Pile Tension Capacity	-	23.94

Results of the inplace analysis for the design level criteria failed to produce a single member in the jacket with a unity check greater than 0.85. Typically, the unity checks throughout the jacket were classified as low.

Several members within the main and secondary deck framing resulted in unity check calculations greater than 1.00. Those members were all found to be overstressed for the drilling condition in which maximum gravity loading occurred. No environmental loading resulted in any member from within the deck with unity checks greater than 0.85.

A punching shear analysis for in-place conditions was performed. A single can in the platform developed a unity check greater than 0.85. The maximum unity check was observed to be 0.95 and occurred in the conductor framing region. The load condition which resulted in the maximum unity check was a dead load condition associated with drilling.

Figure 1 illustrates the joint can location in the jacket where the maximum unity check occurred.

A.5.2 Ultimate Strength Analysis: The platform model developed for the design level analysis was used for the ultimate strength level analysis. The model included all twenty four (24) conductors and appurtenances used in the design level analysis. The affects of possible reduced thickness upon members due to corrosion were not considered since evidence of such affects were not made available from the client.

SACS program was used for the analysis. A new non-linear routine currently listed as "under development" was used for the analysis.

The ultimate strength level wave, current and wind was applied to the structure in two (2) independent directions. The approach angles considered were in the direction of the longitudinal and transverse axis of the platform or 0° and 90° degrees with respect to platform north orientation.

The following metocean criteria was used for the ultimate strength analysis:

Wave Height	54.9 feet 0° direction 61.0 feet 90° direction (Fig. 17.6.2-3A & Fig. 17.6.2-4)
Wave Period	12.5 Sec. (Table 17.6.2-1)
Current Speed	1.8 kts. (Table 17.6.2-1 & Fig. 17.6.2-4)
Storm Tide	3.5 feet (Figure 17.6.2-3A)
Wind Speed (1 hr. @ 10m)	70 kts. (Table 17.6.2-1)
Marine Growth	1.5"
Wave Kinematics Factor	0.88

The ultimate strength analysis was performed through SACS using a two step approach method. The first step consisted of a linear analysis of the computer model to establish the Reference Level Load (S_{ref}) and the load level at which the first component reaches an IR = 1.0. This subsequent level is referred to as the First Failure Load (S_i).

The subsequent step involved a non-linear analysis in which the reference load level (S_{ref}) was increased by increments until a total collapse of the structure was observed.

Both the linear and non-linear analysis portions were considered for the platform's longitudinal and transverse directions.

A number of significant observations occurred throughout the non-linear load increment analysis. The first is evidenced in Figures 2 and 3 and revealed that the ratio of the incremental load increase was consistent wit the ratio of the deflection increase. Also, as the increased load began to come very near the ultimate capacity or ultimate load limit, a rapid rise in the ratio rate of deflection increase was observed when compared to the ratio rate of the load increase. This rapid non-linear effect occurred until a total collapse of the structure was triggered.

As illustrated in Figures 2 and 3, the Reserve Strength Ratio (RSR) was 3.18 and 3.24 respectively for the platforms longitudinal and transverse directions. The Reserve Strength Ratio was determined as:

$$RSR = \frac{R_u\text{; (Ultimate Capacity)}}{S_{ref}\text{ (100 yr., 20th Edition Storm Load)}}$$

The ultimate capacity (R_u) was chosen as the last incremental load step before the load resulting in platform collapse occurred.

The platform failure mode for both wave load directions occurred within the jacket. The platform passes the assessment since the ultimate capacity (R_u) for both principal directions significantly exceeds the Reference Load Level (Sref) capacity. This outcome is expectant and consistent with the results of the design level analysis.

A.6 **Mitigation Alternatives:** The platform is considered to have passed the assessment analysis based upon results of the design level analysis and the ultimate strength level analysis. This determination is conditional upon favorable reports of inspections as to the actual structural condition for the jacket. Due to the age of the structure it is assumed the corrosion life of the anodes has either been depleted for some cases or near depletion for most actual anode locations. The affects this may have as to the actual corrosion which has occurred to the structure is unknown. Also, it is unknown as to any occurrence or the level of possible damage which may have resulted unto the structure during the history of operation. Any damage which has not been repaired would warrant further analysis specific to the type of damage which has occurred. An assessment of the knowledge of such damage could alter the results outcome of the ultimate strength analysis.

In addition, the following recommendations are suggested:

1) Replace any anodes which have been depleted through sacrifice during the platform's life.

2) Reduce the load on the structure by removing either any partial or complete non-producing wells.

A.7 **Summary Note:** The assessment process for Platform "O" was initiated as a result of the omitted inspection reports. Absence of the reports precluded all of the conditions to be considered which determines whether an assessment is required.

The platform was categorized as manned, evacuated and with insignificant environmental damage.

The platform was determined to pass the design level analysis and ultimate strength level analysis subject upon the structural condition which can only be answered after reviewing the inspection reports.

JOINT INDUSTRY PROJECT - PLATFORM "O"   PLAN AT ELEV. (+) 15'-0"

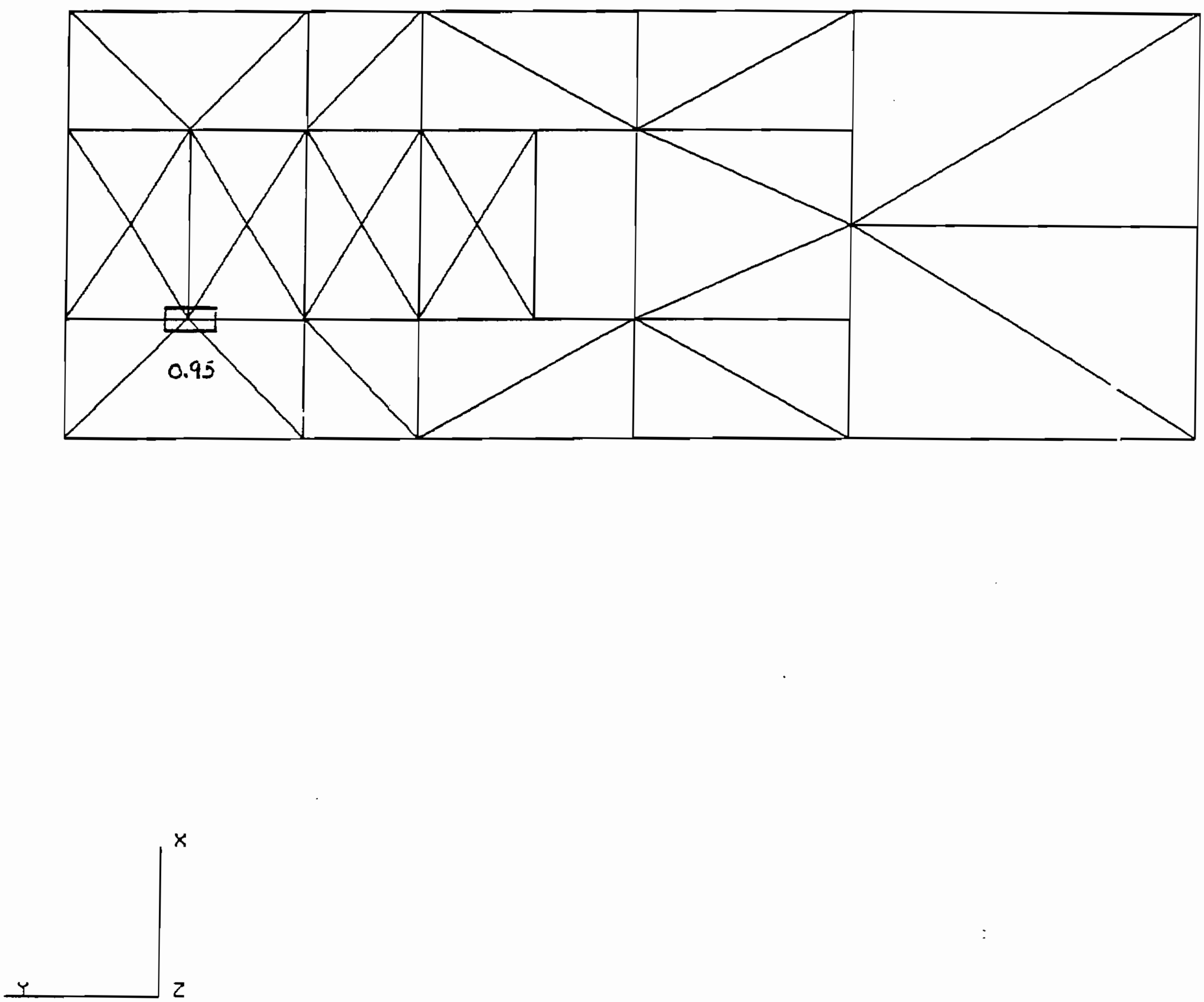
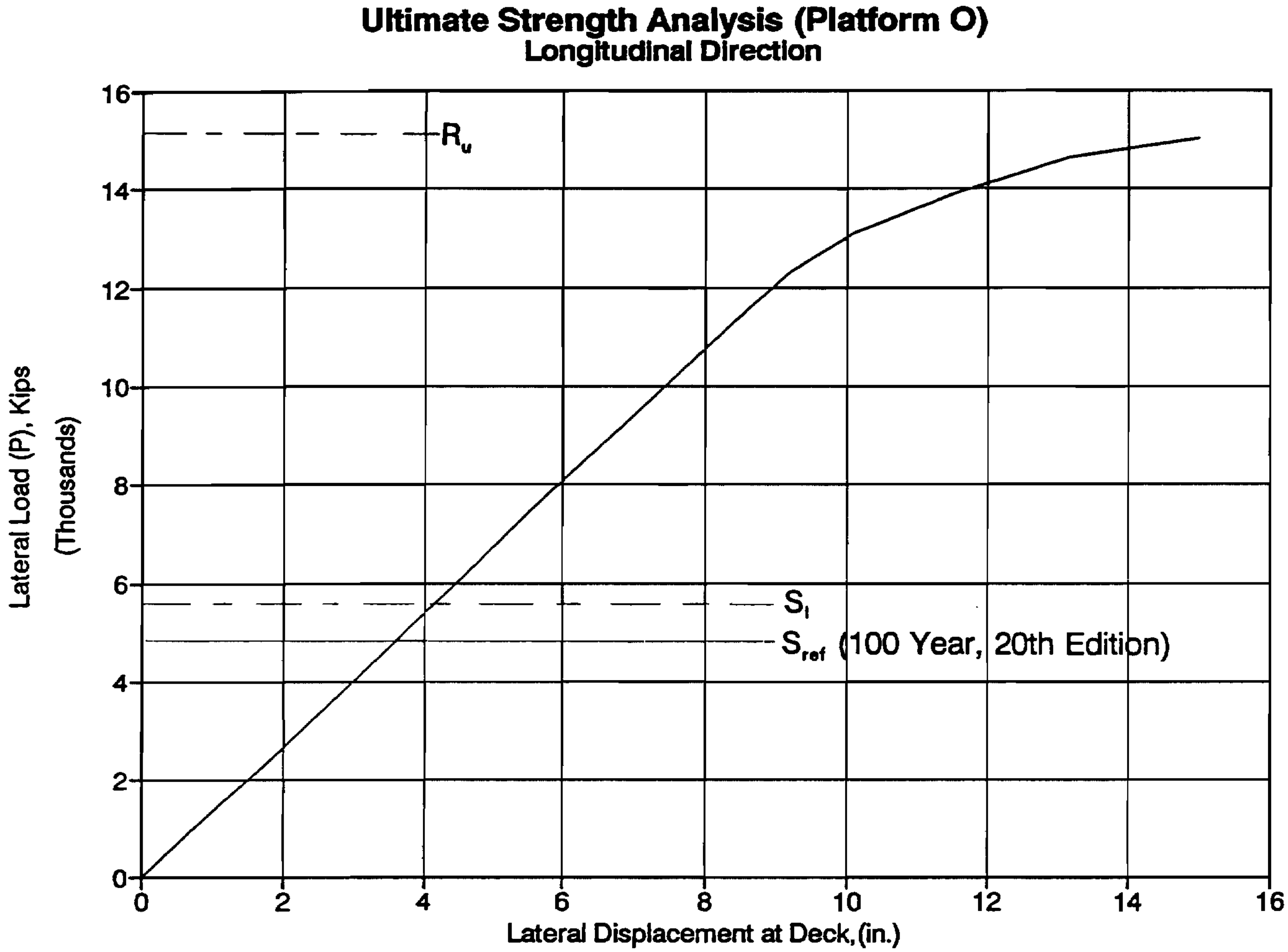


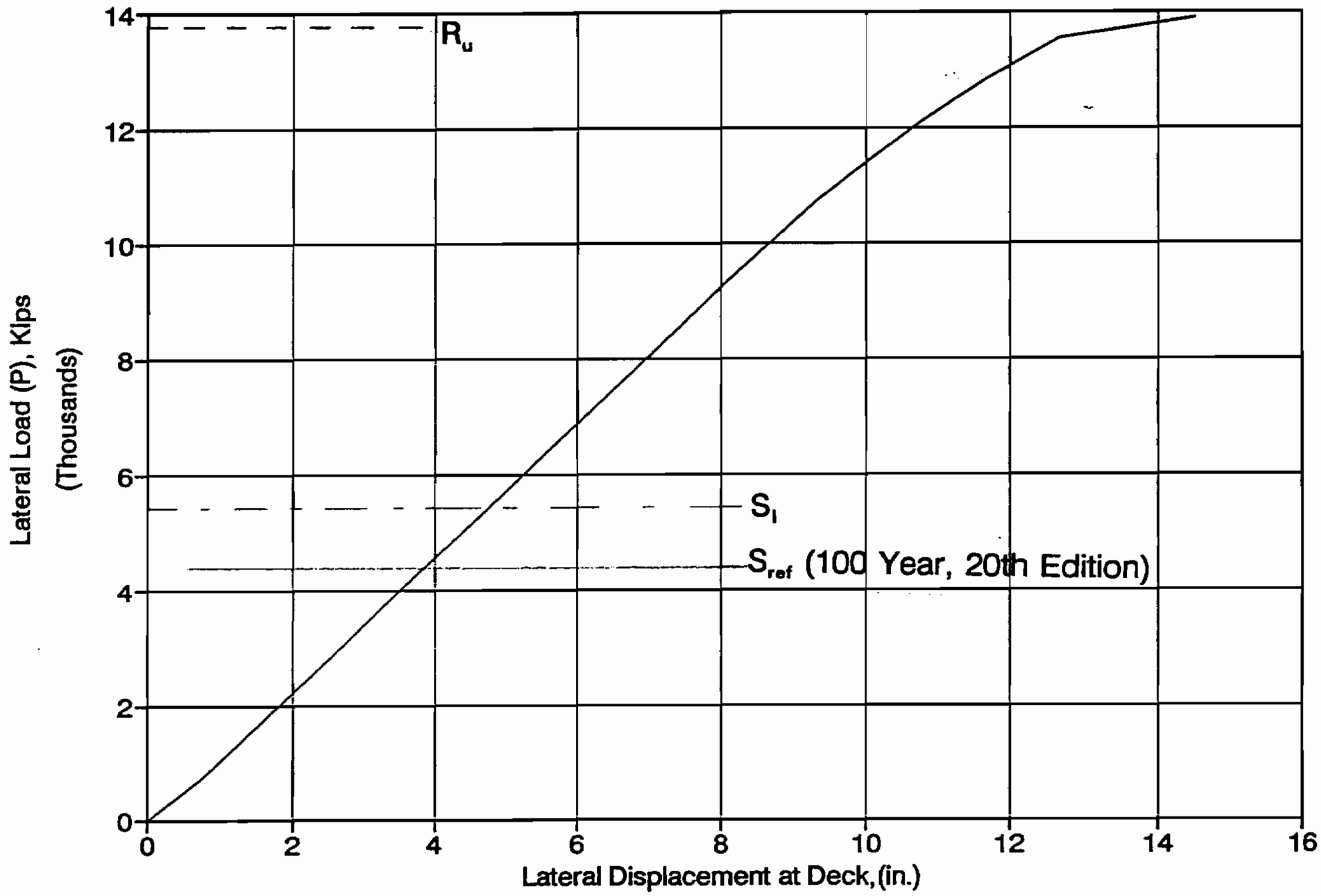
Figure 1
Punching Shear Unity Checks >0.85



Load Level at which first Component Reaches I.R. of 1.0 (S_i)	5780 Kips
Reference Level Load (S_{ref})	4726 Kips
Ultimate Capacity (R_u)	15,029 Kips
Reserve Strength Ratio (RSR)	3.18
Platform Failure Mode:	Jacket

Figure 2

Ultimate Strength Analysis (Platform O)
Transverse Direction



Load Level at which first Component Reaches I.R. of 1.0 (S_I)	5344 Kips
Reference Level Load (S_{ref})	4288 Kips
Ultimate Capacity (R_u)	13,894 Kips
Reserve Strength Ratio (RSR)	3.24
Platform Failure Mode:	Jacket

Figure 3

PLATFORM "P"

Section 1
Executive Summary

This document presents the results of an assessment of Platform-P in Eugene Island Block 339. This assessment was performed in accordance with the recommendations of the API RP2A WSD Draft Section 17 document. Platform-P is an 8-leg drilling and production platform installed in 1973 in 263 ft. The work performed fulfills the contribution requirements of Trials JIP, which is being conducted by PMB Engineering for the Minerals Management Services (MMS).

Under an assessment situation, Platform-P would not need detailed assessment per Section 17, as it passes all five Platform Assessment Initiators of Section 17.2. However, the platform was assessed anyway to determine conformance with Section 17 recommended performance targets to meet the project requirements of the Trials JIP.

The platform falls under "manned evacuated" and "significant environmental impact" categories, thus the "full population hurricane" metocean criteria are applicable. The flow chart in Section 17.5 indicates that the platform requires "Analysis Checks" per Section 17.6 and 17.7. The determining factor for this requirement was that the platform was designed in 1972 prior to the 9th Edition of RP2A.

Three storm approach directions were identified as sufficient for analysis of this platform for Section 17 Design Level and Ultimate Strength requirements. Both series of analyses were performed using PMB Engineering's CAP and Seastar computer programs.

All diagonal braces of the platform are oriented in the same direction in its two longitudinal frames. This places them all in compression with an end-on loading. From a structural design viewpoint, this condition is not optimum. Also it was observed that the structural strength of the lowest bay is weaker than the next bays in the transverse framing, causing a weak link in the structural system.

The platform does not pass assessment at the Design Level analysis stage for any of the three directions. The Interaction Ratios exceeded 1.0 for a large number of braces in the vertical frames for each loading direction.

The ultimate strength is dictated by buckling of a K-brace for the broadside loading condition and by failure of a third diagonal brace for the diagonal and end-on direction loading conditions.

The reserve strength ratio (RSR) of a platform is used as a check of ultimate strength and provides a measure of capacity beyond the requirements of present RP2A requirements. RSR is defined in Section 17 as the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading (based on RP2A, 20th

Edition criteria). The RSR for Platform-P is in the range of 1.14 to 1.26 for an extreme hurricane approaching from platform East to South.

There is no specific value given in Section 17 for an acceptable minimum RSR. The calculation of the RSR in this analysis is not meant to indicate if the platform passes or fails Section 17. It is largely given as a value required by the API Task Group for its use.

Whether a platform passes Section 17 ultimate load criteria can be more appropriately described by its ultimate capacity ratio (UCR). This ratio is not discussed in this way elsewhere, but is used in this report for discussion purpose. UCR is defined as the ratio of platform's ultimate lateral load carrying capacity to its Section 17 ultimate strength analysis lateral loading, computed using present RP2A procedures. A UCR equal to or greater than 1.0 indicates that the platform has adequate capacity against global failure or collapse.

For Platform-P, the UCR values are above 1.0 (1.05 max.) for the diagonal and broadside loading conditions and slightly below 1.0 (0.94) for the end-on loading condition. However, considering the levels of uncertainties and biases involved in the overall process of load and strength computation, the platform marginally meets the Section 17 criteria for practical purposes.

Section 2
Platform Information

The available information for the Platform-P located in the Eugene Island (EI) Block 339 is summarized in this section. This information was used to establish the as-is state of the platform for its assessment per API RP2A WSD, Draft Section 17. The data reflects information gathered from original design drawings, recent underwater inspection reports and future operational plans.

Platform-P is an 8-leg drilling and production platform installed in 263 ft. water depth, with a 2-level deck. The jacket has a K-brace framing pattern in the end-on frames and diagonal brace pattern in the broadside frames. The piles penetrate 300 ft. below the seabed. Selected drawings are given in Figures 2.1 to 2.5 to summarize the platform framing type and configuration. Details of the pile foundation are given in Figure 2.6.

McClelland Engineers' soil and foundation investigation report of July 1973 for the EI 339 block was used to define the soil parameters for the foundation analysis. Figure 2.7 presents the design shear strength and submerged unit weight profiles. The ultimate pile capacity profile is given in Figure 2.8. This capacity for 300 feet of penetration was calculated by McClelland to be 5,500 kips. However, the capacity was recalculated for this study using the current API formulation. This capacity was calculated to be 8,210 kips.

The deck module and area loading is given in Figure 2.9.

Table 2.1 presents a comparison of the available platform data with the suggested information per Section C17.4.1 (Section 17), which is needed for assessment of a platform.

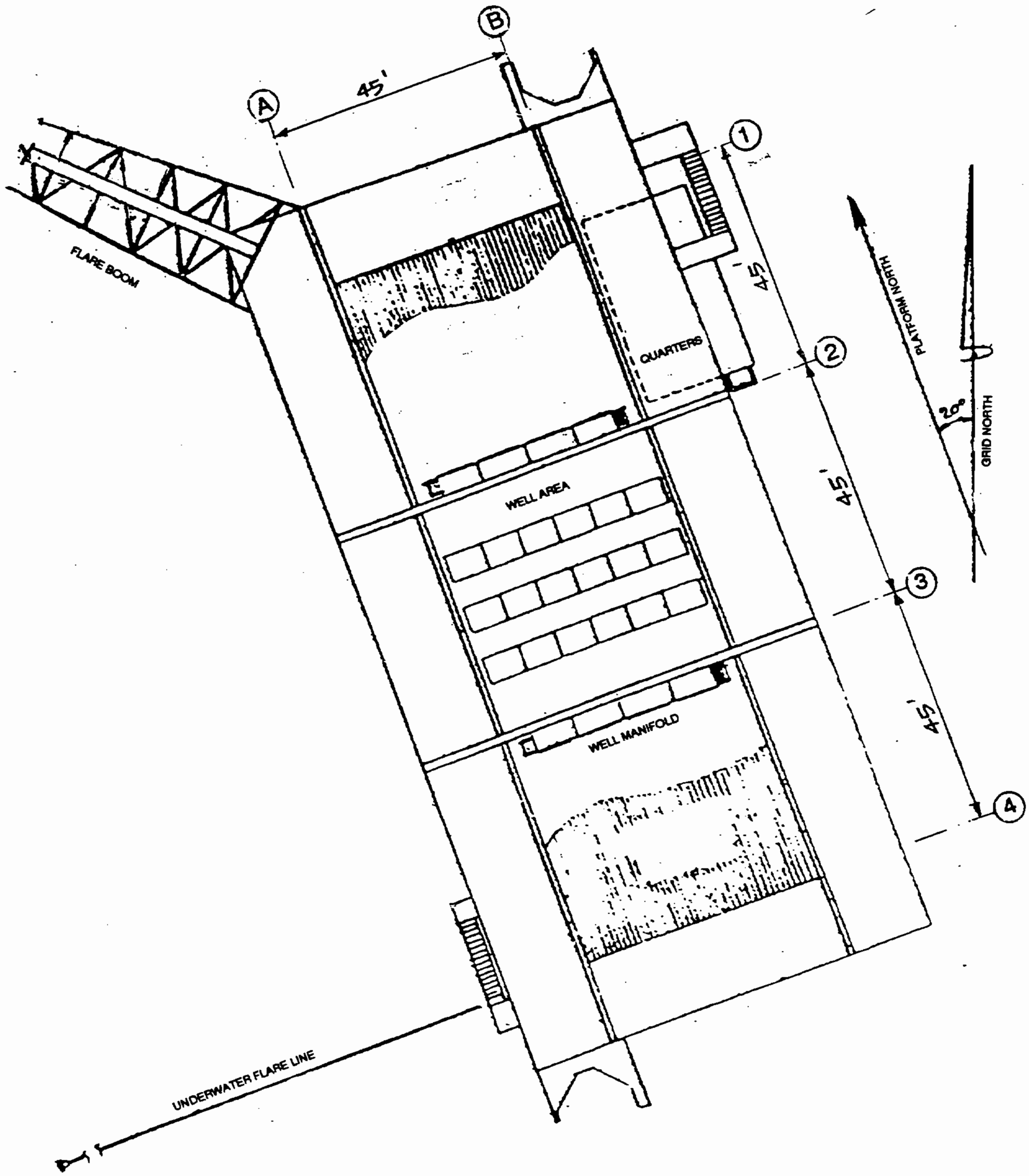


Fig. 2.1 KEY PLAN - PLATFORM P

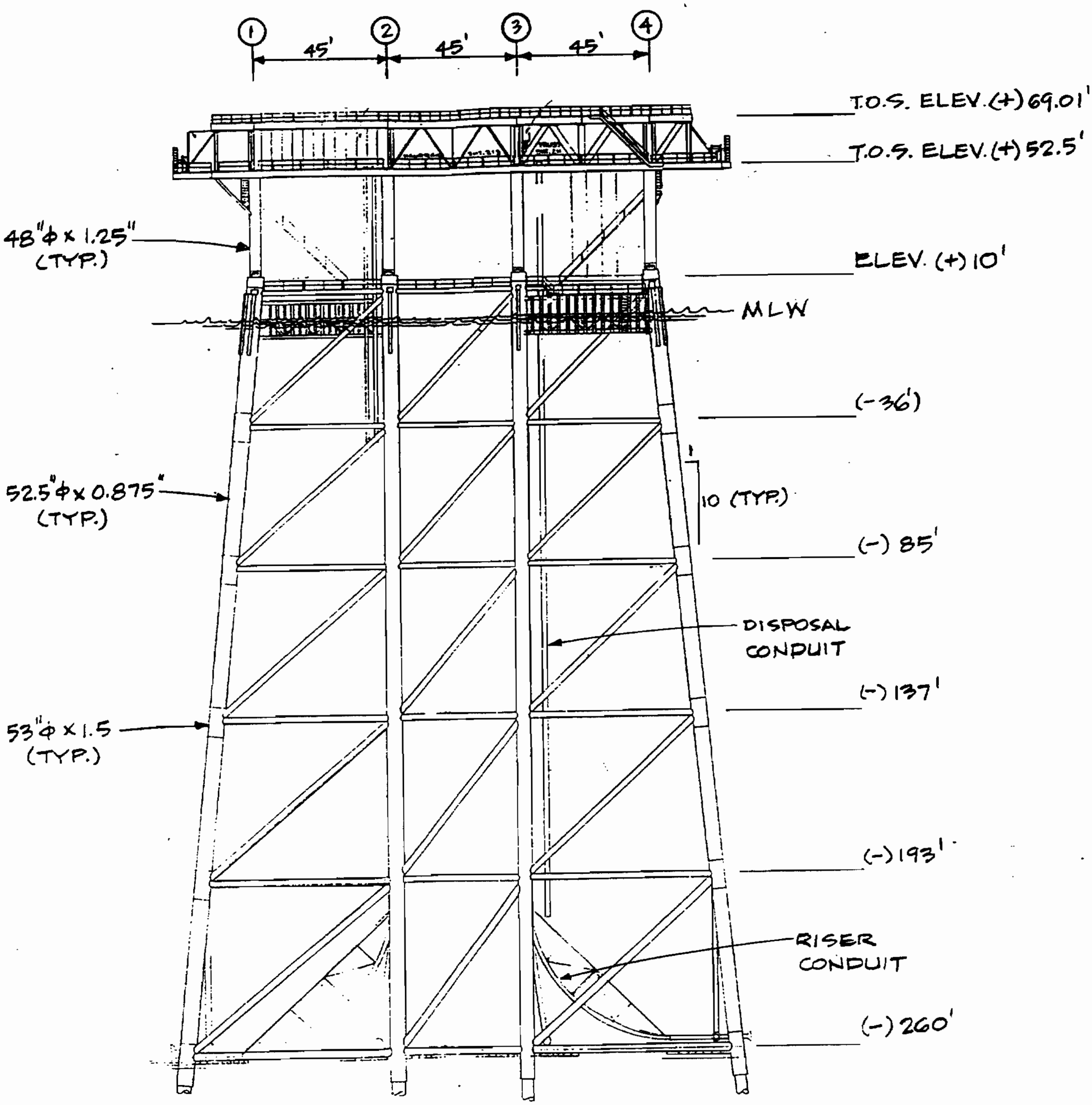


Fig. 2.2 BROADSIDE ELEVATION ROW A AND ROW B
(As per Original Design Drawings)

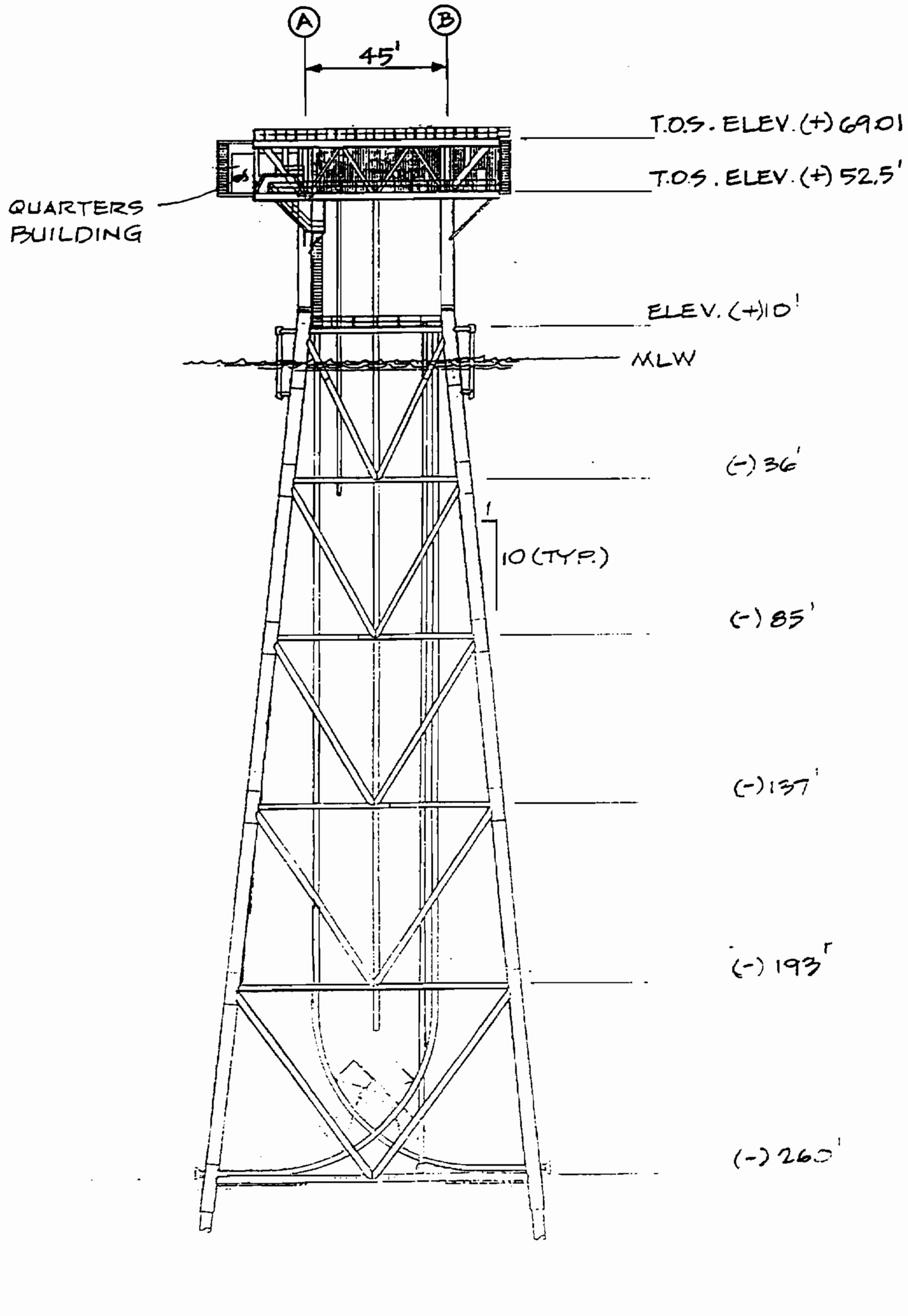


Fig. 2.3 ENDON ELEVATION - ROW 1 and 4

(As per Orignial Design Drawings)

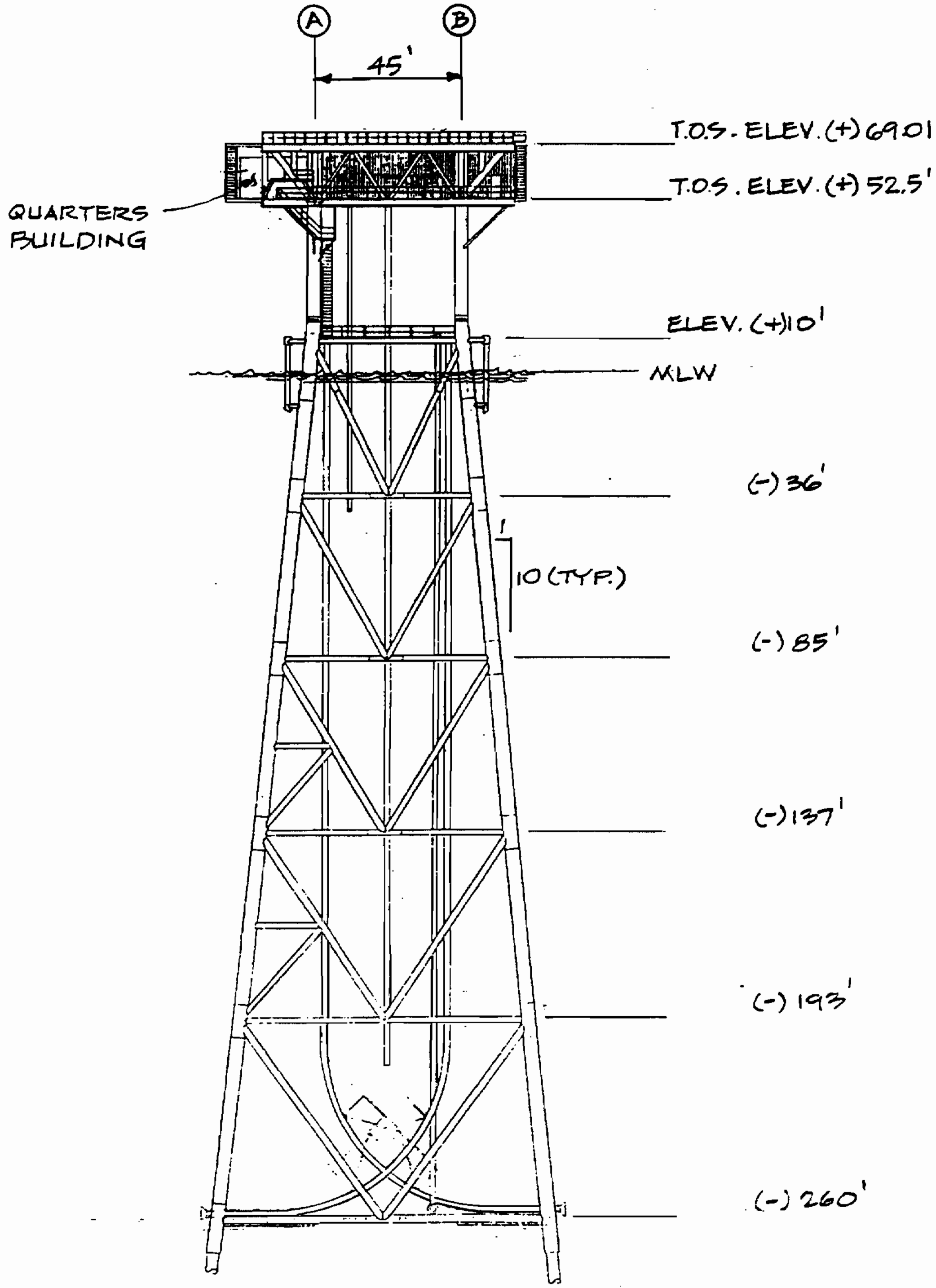


Fig. 2.4 ENDON ELEVATION - ROW 2 and 3

(As per Orignial Design Drawings)

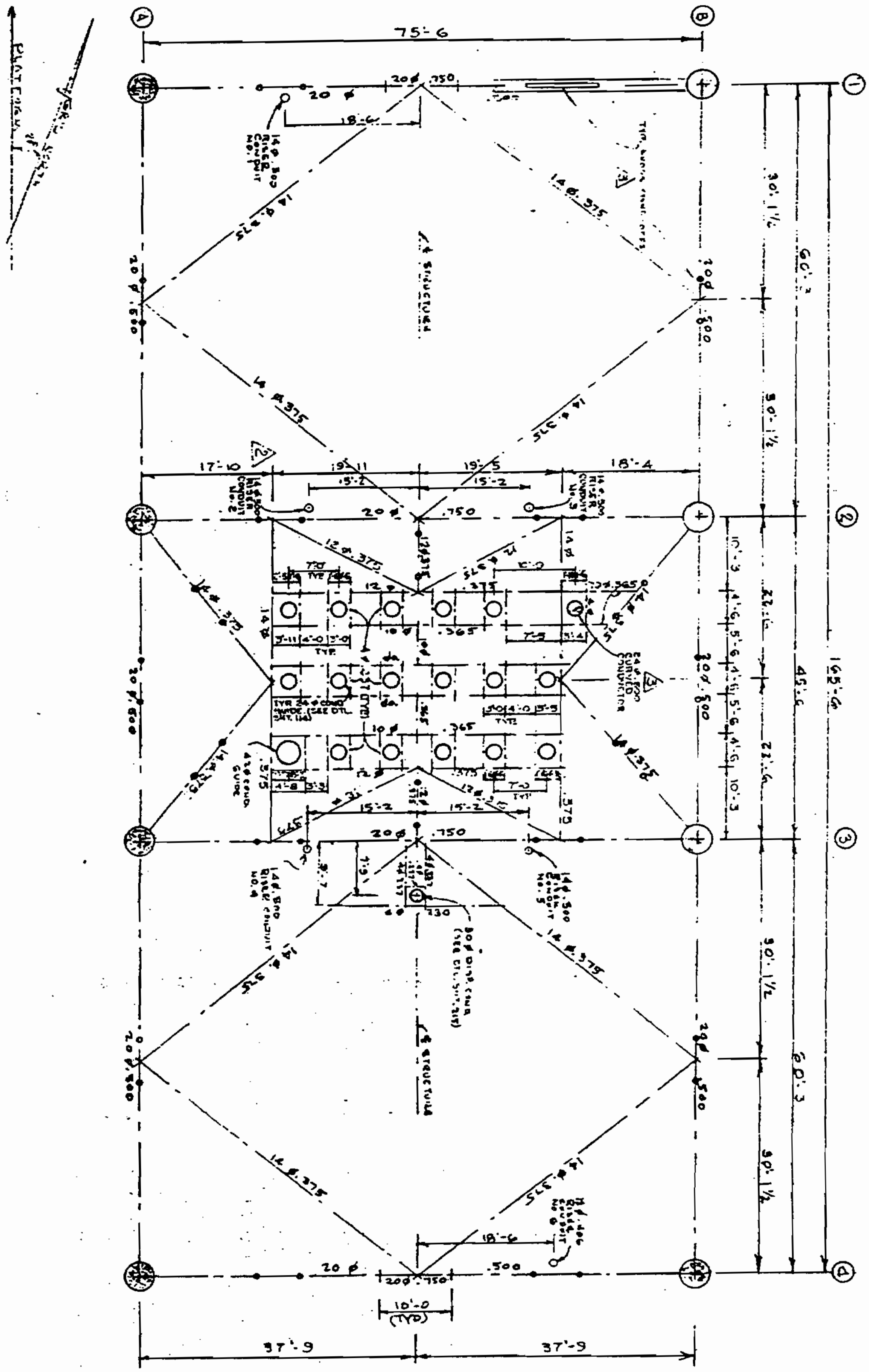


Fig. 2.5 PLAN AT ELEV. (-) 137'
(Similar at Other Elevations)

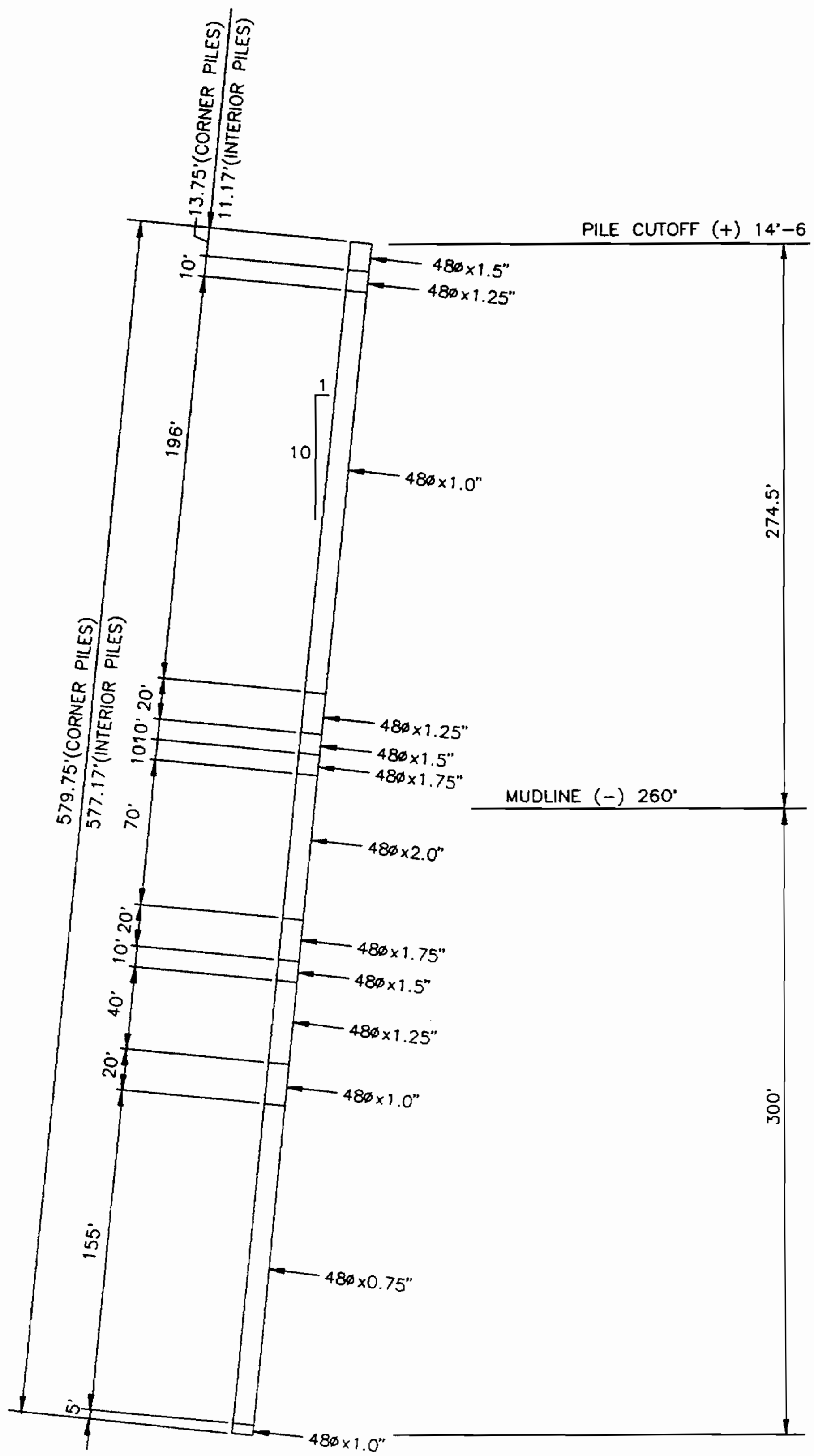


Fig. 2.6 PILE DETAILS

(As per Orignial Design Drawings)

FIG-1-6 AH 08-15-94 PLOT SCALE: 1=900

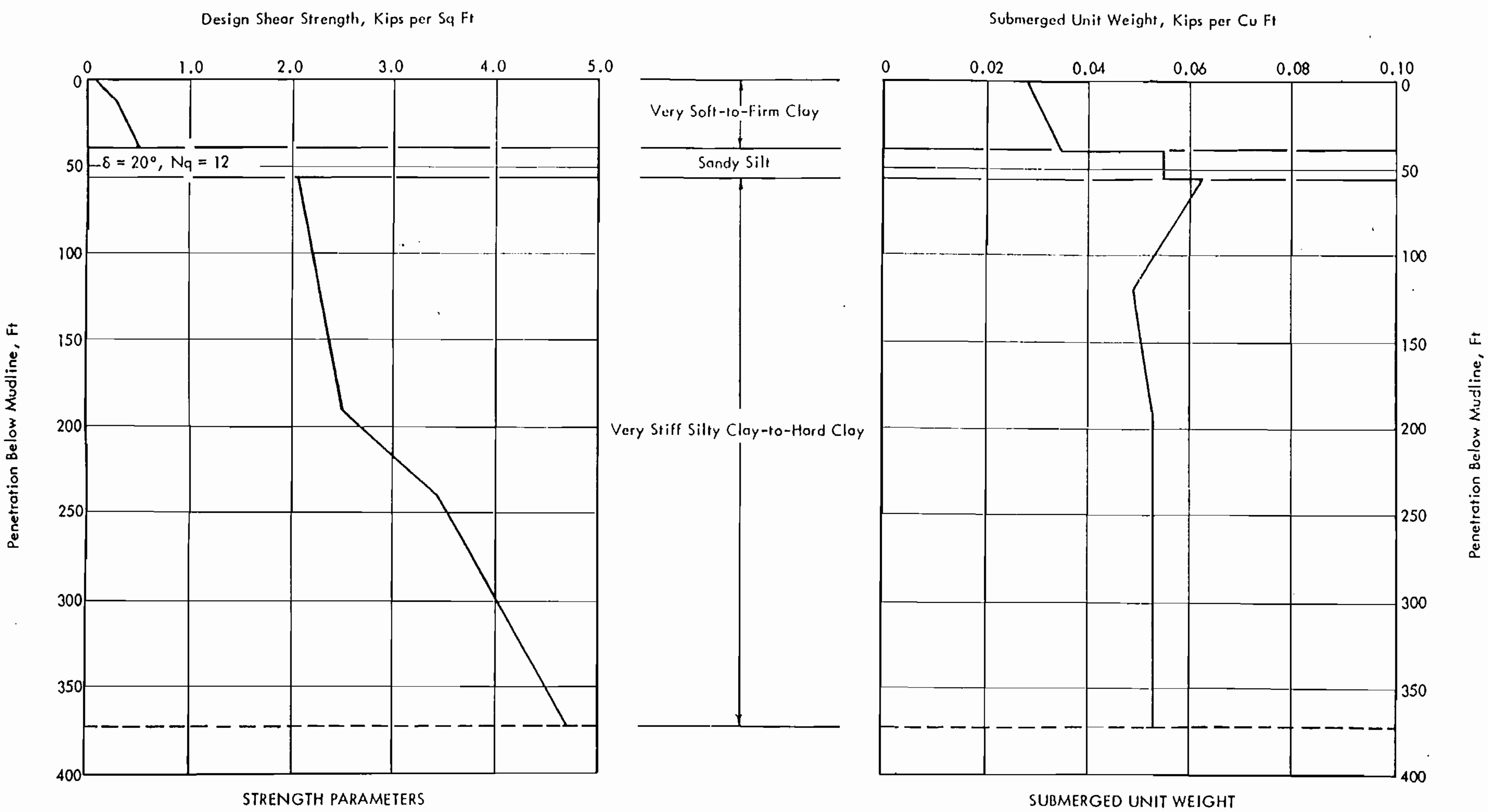
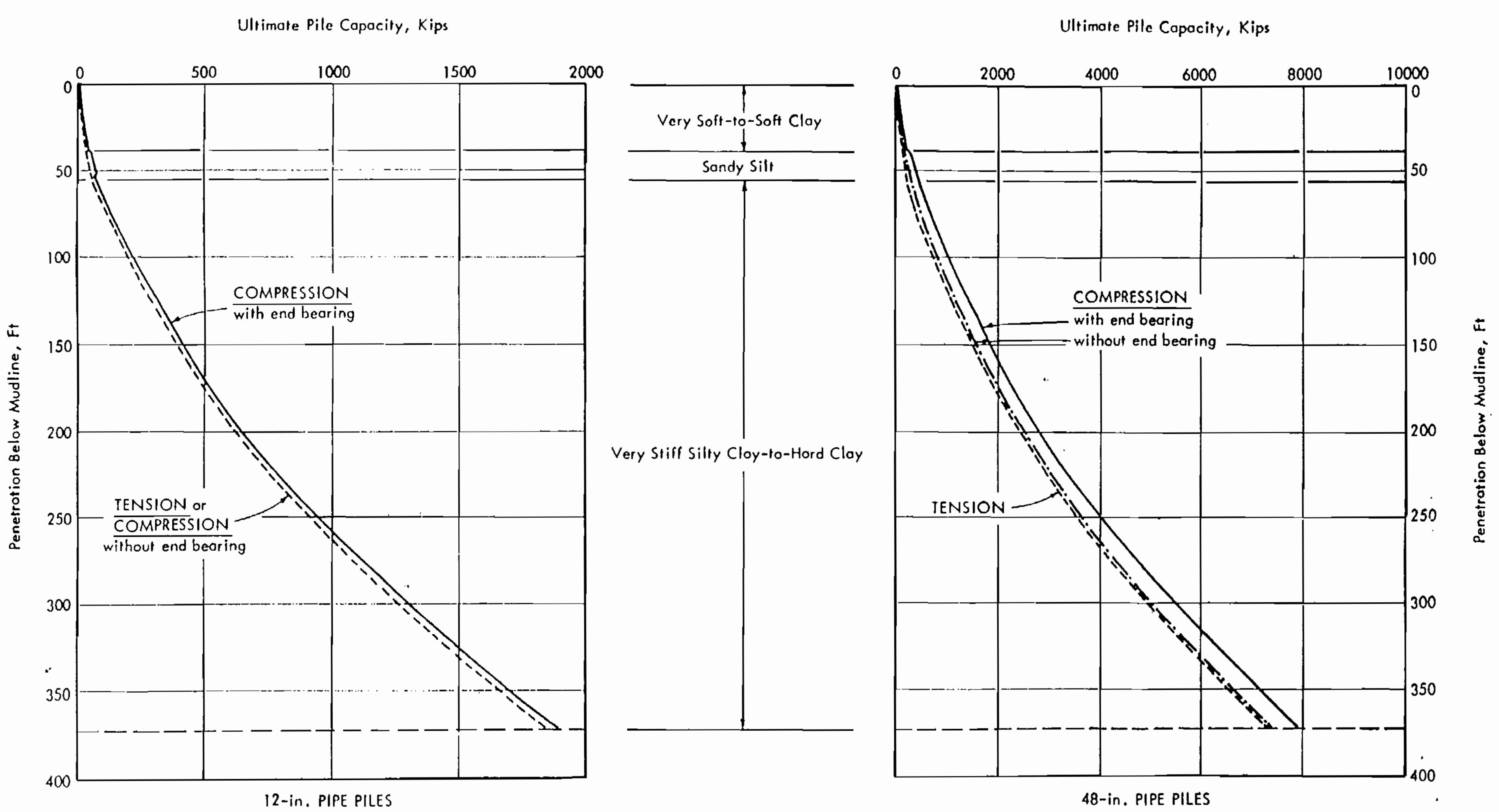


Fig. 2.7 SOIL SHEAR STRENGTH AND UNIT WEIGHT PROFILES (from McC ɹnd Report)



MCCLELLAND ENGINEERS

Fig. 2.8 PILE CAPACITY CURVES (from McClelland Report)

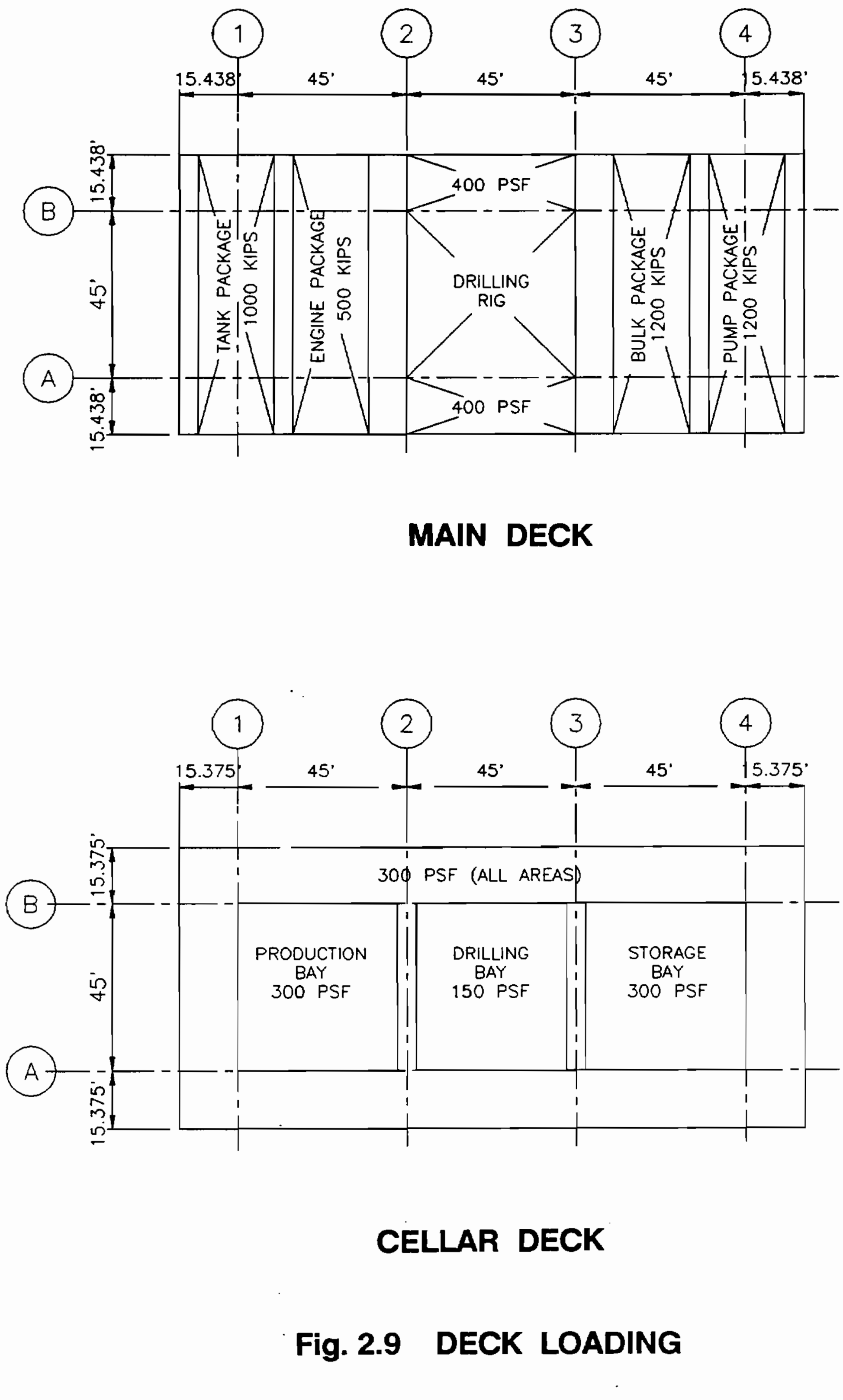


Fig. 2.9 DECK LOADING

FIG-1-9 AH 08-15-94 PLOT SCALE: 1=480

1. General Information		
a.	Original and current owner	Same
b.	Original and current platform use and function	Drilling and Production Manned, evacuated during storm
c.	Location Latitude/ Longitude	Eugene Island 339, Gulf of Mexico 91° 39' 23" W 28° 12' 3" N
	Water Depth	as designed: 260 ft. as-is state: 263 ft.
	Orientation	N20W
d.	Platform type	8-legged steel jacket platform. Grouted leg/ pile annulus. K-braces in end-on frames. Diagonal braces in broadside frames.
e.	Number of wells Number of risers	18 as originally designed. 6 risers, as originally designed 1-12" dia. and 5-14" dia.
	Production rate	Information not available
f.	Other site specific information Manning level	 5 people
g.	Performance during past environmental events	No known damage
2. Original Design		
a.	Design contractor Date of design	Owner 1972
b.	Design drawings Material specifications	Available Not available. Assumed A36 steel for all main steel.
c.	Design code	RP2A, probably 4th Edition
d.	Environmental criteria	See Table 2.2
e.	Deck clearance elevation:	Cellar Deck B.O.S. Elev. (+)49'- as designed
f.	Operational criteria Deck loading	 Drilling rig and modules. Load information available
	Equipment arrangement	Complete information not available. For cellar deck, area loads used.

Table 2.1(a): Available Information vs. Section C17.4.1 Suggested Data

2. Original Design (Continued)	
g. Soil data: McClelland Engineers report available.	
h. Number of piles	8
Pile size:	48 in. dia.
Pile penetration	300 ft. below the seabed
Number of conductors:	18
Conductor size:	1-40" dia., 1-24" dia., 16-22" dia.
Conductor penetration:	As planned
i. Appurtenances	2- Boat landings on broadside frames 8- 18" dia. Barge bumpers 2-14" dia. Fire water pump casings 1-30" dia. Disposal conduit
3. Construction	
a. Fabrication contractor:	Information not available
Installation contractor:	Information not available
Date of installation:	1973
b. As-Built drawings:	Not available
c. Fabrication, welding, and construction specifications:	Not available
d. Material traceability records:	Not available
e. Pile and conductor driving records:	Not available Assumed design penetration reached
f. Pile grouting records:	Not available. Leg-pile annulus assumed properly grouted.
4. Platform History	
a. Environmental loading history:	Information not available.
b. Operational loading history:	Subjected to drilling phase loads
c. Survey, maintenance records:	Platform inspected periodically. At least Level II survey within last 5 years. No significant damage to the platform The marine growth profile recorded. Some corrosion noted, which was cleaned and painted
d. Repairs description:	None performed.
e. Modifications descriptions:	None performed.

Table 2.1(b): Available Information vs. Section C17.4.1 Suggested Data

5. **Present Condition**

a.	Deck size and elevations:		
	Main deck	Plan size:	166' x 76'
		T.O.S. Elev.	(+) 66.01'
	Cellar deck	Plan size:	166' x 76'
		T.O.S. Elev.	(+) 49.5'
b.	Deck: existing loading and equipment arrangement		
	Main deck	Available (similar to as design)	
	Cellar deck	Unit area loading (as design) available.	
c.	Field measured deck clearance elevation:	**Cellar deck: B.O.S. Elev.(+) 47'**	
d.	Production and storage inventory:	Detail information not available, 100 bbl crude storage on cellar deck	
e.	Appurtenances:	All installed as in the original design.	
f.	Wells:	Assumed all installed as in original design	
	Risers	5 out of 6 conduits utilized.	
g.	Recent above water survey:	see item 4.c	
h.	Recent underwater platform survey	see item 4.c	

Table 2.1(c): Available Information vs. Section C17.4.1 Suggested Data

Maximum Storm Conditions

Wind Data: Velocity = 199 ft/sec from 295 degree (from True North) for each wave case

Wave Data:

Direction* (degree)	Hmax (ft.)	Period (sec)
250	53.1	12.2
295	63.2	12.1
340	66.4	12.1

* From True North

Current Data:: at 295 degree (from True North) for each wave case

Depth (ft.)	Velocity (ft./sec)
0	5.3
55.5	1.9
111.0	1.0
166.0 +	0.8

Marine Growth:

Depth (ft.)	Marine Growth (inch on radius)
0	1.5
218	1.5
218 +	0.0

Operating Storm Conditions

Wind Data: Velocity = 88 ft/sec from 295 degree (from True North) for each wave case

Wave Data:

Direction* (degree)	Hmax (ft.)	Period (sec)
250	22.4	7.5
295	26.8	8.3
340	25.3	7.9

* From True North

Current Data:: at 295 degree (from True North) for each wave case

Depth (ft.)	Velocity (ft./sec)
0	2.6
55.5	1.5
111.0	1.1
166.0	0.9
222.0 +	0.8

Marine Growth: Same as for maximum storm case

Table 2.2: Original Design Criteria - Platform P

PART A PLATFORM ASSESSMENT

Section A.1 Platform Selection (Section 17.2)

The following platform information is summarized for the Platform Assessment Initiators.

Addition of Personnel	No change from the original design
Addition of Facilities	The present facilities on the main deck are reduced as the drilling rig and modules are gone. The rig may return for further drilling in the future. Therefore, the loading assumed for this assessment uses the same as that for the original design.
Increased Loading on Structure	The platform did not have any modifications to increase loads more than 10%.
Inadequate Deck Height	The exposure category of the platform is "Full Population". The minimum deck elevation for this category is (+) 44.3' per Figure 17.6.2-2b. This elevation is lower than the actual cellar deck B.O.S. Elevation of (+)47'. Therefore, the assessment wave does not reach the deck level.
Damage Found During Inspections	No significant damage was found during inspections.

Based upon Section 17.2, none of the assessment indicators would trigger a need for assessment of this platform. However, this platform was selected by the operating company for assessment to meet the requirements of the Trials JIP submittal partly because this platform is typical of the 8-leg vintage of platforms installed in the 1970's. In addition, this platform is being reviewed to assess the feasibility of drilling new wells. Thus a full assessment procedure was conducted.

Section A.2 Categorization (Section 17.3)

The platform has 18 conductors with the assumption that all are present. The wellheads are located between rows 2 and 3 of the cellar deck, and the platform has six riser conduits.

It also has a quarters building located on the cellar deck. The quarters building side of the platform has diaphragm walls provided along Rows 3 and 4 to provide safety from the wellhead area. Access to the platform is by helicopter and by boats with two boat landings located on the longitudinal frames of the platform. The survival crafts for emergency escape are provided at the two end-on sides of the platform.

Considering the above operational features, this platform falls under the following category:

Manned, Evacuated
Significant Environmental Impact

Therefore, full population metocean criteria will be applicable for assessment of the platform.

Section A.3 Condition Assessment (Section 17.4)

The available information for the platform was compared to that suggested in Section C17.4.1 and is given in Table 2.1 of this report.

The construction and installation records indicated the following changes from the original design drawings:

Water depth:	263 ft. M.L.W. instead of 260 ft. shown on drawings.
Cellar deck:	T.O.S. elevation as (+)49.5' instead of (+)52.5'.
Marine growth:	Original design criteria of 3" on diameter down to 218 ft. below M.W.L., thus exceeding the 150 feet below M.W.L. per RP2A.

The underwater survey performed in the last five years indicated no significant damage to the jacket or deck structures. Reviewing the platform information against that given in Figure 17.5.2 of Draft Section 17, the following observations are noted:

Is the platform damaged?	No
Is the deck height inadequate?	No
Has the loading increased?	No
Is platform unmanned?	No
Does it have insignificant environmental impact?	No

The flow chart of Figure 17.5.2 indicates that based on the first three responses above, the platform does not need to be evaluated relative to "Analysis Checks". The last two responses indicate that the platform is required to be evaluated relative to the "Design Basis Check".

Section A.4 Design Basis Checks

The tests under design basis checks given in Figure 17.5.2 provide the following observations:

Is platform located in G.O.M.?	**Yes**
Is platform designed to API RP2A 9th Ed. or later?	**No**

This platform was designed in 1973 based upon an edition of RA2A earlier than the 9th. It is most likely designed to the RP2A, 4th Edition (1972). Hence, the design basis check is not passed and is required to be evaluated relative to the "Analysis Checks."

Section A.5 Analysis Checks (Section 17.6 and 17.7)

Section A.5.1 Metocean Criteria/Loads (Section 17.6.2a)

The full population hurricane criteria given in Table 17.6.2-1 were used to develop the basis for the analysis checks of this platform. The pertinent metocean information for a water depth of 263 ft. is summarized in Table 3.1 The metocean criteria per RP2A, 20th Edition, needed to determine the reference level force for computation of the RSR, are also presented in this Table.

The wave force procedure per RP2A, 20th Edition was followed for this study, using a wave kinematic factor for a hurricane condition of 0.88 and current blockage factors of 0.8, 0.85, and 0.7 for the broadside, diagonal, and end-on storm approach directions, respectively. The shielding factor for the conductor array was computed as 0.85 for the broadside approach direction and 1.0 for the diagonal and end-on directions. However, no conductor shielding effect was directly included in the structural analysis.

Section A.5.2 Screening

No screening analysis was performed for this platform.

Section A.5.3 Design Level Analysis (Section 17.7.2)

A complete 3-dimensional computer model was generated for the platform using PMB Engineering's computer program, CAP. A 3-dimensional perspective view of the computer model is given in Figure 3.2. The computer model development included the linear behavior of the deck legs, jacket structure, and piles. The soil-structure interaction for the foundation was characterized by non-linear soil spring elements.

The primary structural elements of the deck were also modeled as linear elastic beam elements. Boat landings, barge bumpers, conductors, risers, pump casings, disposal caisson were modeled by equivalent wave load elements. The modeling of the conductor guide framing was idealized to include primary load carrying members with adjusted hydrodynamic coefficients to accurately estimate the wave load acting upon the conductor guide framing. The influence of minor appurtenances such as handrails, stairs, anodes was neglected in this study.

The deck loads were estimated for drilling and production scenarios. The loads during drilling operation with the drill rig and associated modules on top of the main deck governed the analysis. The total loads considered during a storm condition on the main deck and cellar deck were 5,190 kips and 2,490 kips, respectively. The deck structural weight was computed as 1,200 kips.

Waves approaching from the north east to the south were found to be the most critical for this platform. Three wave directions, shown in Figure 3.1, were selected based upon consideration of platform orientation and configuration. The metocean parameters per Section 17 design level analysis for each of the three directions are given in Table 3.2.

The analysis was performed for three wave approach directions using CAP and Seastar. Code checks were performed for the deck legs, primary members of the jacket, and piles.

The leg/ pile annulus of the platform is grouted and joint cans are provided at the brace to leg connections. At many of the K-joints in vertical frames, can sections are provided. These features indicate that the leg-brace joints and K-joints were probably not be critical in most cases and emphasis was first placed upon the brace stresses and I.R.'s. Procedurally, if member unity checks indicate acceptable I.R.'s, then critical joints would be reviewed for their strength acceptability.

The analysis results for the three directions are summarized in Table 3.3. In general, the results indicate the following:

Broadside Direction (250 degrees): Figure 3.3 presents the IR plot, highlighting the members with IR's greater than 0.85. This limit was exceeded by 15 K-braces in the four transverse direction vertical frames. Of these braces, 7 had IR's more than 1.0. The maximum compressive pile force was 3,578 kips, giving a factor of safety of 2.29.

Diagonal Direction (295 degrees): Figure 3.4 presents the IR plot for this loading condition. An IR of 0.85 was exceeded for 17 diagonal braces in the two longitudinal vertical frames of the jacket and one pile section. Of these, 8 braces had IR's greater than 1.0. The maximum compressive pile force was 4,058 kips, giving a factor of safety of 2.02.

End-on Direction (340 degrees): Figure 3.5 presents the IR plot highlighting the members with ratios greater than 0.85. 27 diagonal braces in the two longitudinal frames exceeded this threshold. Of these, 22 braces exceeded 1.0. The maximum compressive pile force was 3,206 kips, giving a factor of safety of 2.56.

Overall, the braces in the two broadside frames have high slenderness (kl/r) ratios, thereby reducing their axial and flexure capacities. Resulting I.R.'s for many diagonal braces were much greater than 1.0 (maximum of 2.3), using node-to-node member lengths. Refining the I.R. calculation to include face-to-face member lengths would reduce the values, but only marginally because of the high axial load component. Hence, no joint strength calculations were pursued.

Hence, the platform does not pass the Design Level Analysis assessment due to the overstressing of braces in the transverse and longitudinal frames. At this stage, either mitigation measures could be identified to reduce the external loading or the platform analyzed to establish its ultimate strength. It is assumed for this study that no mitigation alternatives will be undertaken and, per Figure 17.5.2 (Section 17), an ultimate strength analysis is required.

Section A.5.4 Ultimate Strength Analysis (Section 17.7.3)

The ultimate strength analysis criteria for the three directions selected for analysis are given in Table 3.4. These directions were selected giving consideration to wave directionality per Fig. 2.3.4-4 of RP2A, 20th Edition and the structural configuration and orientation of the platform. The 20th Edition criteria for these directions are also given in Table 3.5.

The capacity analysis consisted of a nonlinear structural analysis (static pushover analysis) using PMB's programs CAP and Seastar. The 3-dimensional computer model developed for the design level analysis was updated to reflect inelastic behavior of primary members of the deck, jacket, and piles. Special nonlinear elements were used to model the nonlinear behavior of the jacket bracing, joints, legs and piles. Platform load and resistance estimates were based primarily upon the RP2A, 20th Edition using several modifications as required in Section 17. Some of the important modifications are identified below:

Factors of safety and other known sources of conservatism were removed in order to compute unbiased platform capacity. A mean yield strength of 42 ksi was assumed for the A36 steel used. The mean value represents increase from nominal due to strain rate effects (rapid loading to storms).

The brace capacity was defined per Equation D.2.2-2 of API RP2A LRFD. The brace capacity was also modified to account for the effect of lateral wave and current loads.

The effective length factor, K, for the diagonal and K braces was taken as 0.65, based on recent laboratory tests and analytical studies. The brace lengths were taken as node-to-node in the computer model.

The leg/pile annulus for the platform is grouted and cans are provided at the joints of K-braces with the horizontals (Rows 1 to 4). The mean joint capacity was estimated per OTC Paper 4189, 1982. The computed joint capacities were compared with the brace buckling capacity and used in the brace formulation if lower than the brace buckling capacity. The strut buckling capacity was found to govern in most cases. The K-joint capacity was found to be most critical for the upper two bays of the jacket.

The static lateral and axial soil strength (without degradation) were used for this analysis. Recent analyses have shown that for pushover analysis static soil-pile capacity is more applicable [OTC Paper 7473, 1994].

The results obtained for each of the three directions are presented in Figures 3.6 to 3.14 and are discussed below:

Broadside Direction (267.5 degrees): The failure mechanism of the platform is given in Figure 3.6. This indicates that the collapse mechanism forms with buckling of several K-braces and full yielding of leg and pile sections. Figure 3.7 presents the load-displacement plot using the deck displacements as the discriminator. This indicates that the ultimate capacity of platform in this direction is 6,298 kips. The pushover load level is 6,235 kips and the 20th Edition reference level load is 5,256 kips. Hence, the RSR in this direction is estimated as 1.2.

The inelastic events of selected key elements, which are descriptive of failure patterns and describe the load-displacement behavior, are shown in Figure 3.8. The first inelastic event occurs at a pushover load level of 6,298 kips when a K-brace in the bottom bay of Row-3 frame buckles. Subsequent to its buckling, three other braces in the same bay also buckle. Thereafter, a K-brace in Row 1 in the bay above buckles and the pile sections below the sand layer at the intersection of the reduced pile wall thickness begin yielding. This is followed by buckling of K-braces in the second and third bays above the mudline and yielding of leg sections at Elev. (-)193' in Rows 2 and 3.

It is noted that the buckling capacity of the K-braces in the lowest bay is lower than the average buckling capacity of the bay above. Therefore, the K-braces in the lowest bay provide a weak link for the platform. Also, the buckling capacity of K-braces in Rows 1 and 4 of the second and third bays from the mudline is much lower and also propagates failure events.

Diagonal Direction (295 degrees): The failure mechanism of the platform for this storm direction is reached by failure of diagonal braces in the two broadside frames of the jacket and yielding of sections of all piles as shown in Figure 3.9. The load-displacement plot is shown in Figure 3.10. This indicates that the ultimate capacity of the platform in this direction is 6,587 kips. The pushover load level is 6,291 kips and the 20th Edition reference level load is 5,232 kips. The resulting RSR in this direction is estimated as 1.26.

Figure 3.11 presents the successive formation of selected inelastic events. The first group of inelastic events occur by first yielding of sections in three piles below the sand layer. The first brace failure occurs at a pushover load level of 6,108 kips with the buckling of a diagonal brace in the bottom bay of the Row-B frame. Subsequent to its buckling, two diagonal braces in the bottom bay of the Row-A frame buckle and first yielding of pile sections occur up to a load level of 6,587 kips. Thereafter, sections below the sand layer in other piles at the intersection of the reduced pile wall thickness start yielding. Also, all diagonal braces buckle one after the other in the Row-A and Row-B frames.

The analysis indicates 8% higher strength beyond the load level at first yield of pile sections and buckling of a first brace. The buckling of a third diagonal brace in the bottom bay of the jacket dictates the ultimate capacity of the platform.

End-on Direction (340 degrees): The failure mechanism of the platform in this direction is shown in Figure 3.12. This indicates that the mechanism forms by buckling of all diagonal braces in two frames and yielding of deck leg sections. The load-displacement plot given in Figure 3.12 indicates that the ultimate capacity of platform in this direction is 4,660 kips. The pushover load level is 4,964 kips and the 20th Edition reference level load is 4,154 kips. The RSR in this direction is estimated as 1.14.

The first inelastic event occurs with buckling of a diagonal brace in the bottom bay of Row-A. Its pushover load level is 4,302 kips. Subsequently, buckling of two other diagonal braces occurs in the bottom bay of Row-B up to a load level of 4,660 kips, representing the ultimate capacity load level. The analysis indicates an 8% higher strength beyond the load level of first brace buckling. Subsequent to buckling of the three braces, all remaining 27 diagonal braces in the Row-A and Row-B buckle. Next, the deck leg sections below the cellar deck for all eight legs show first yielding followed by full yielding of three of them.

It is important to note that all of the diagonal braces in both broadside frames are oriented in the same direction. Therefore, all braces are in compression due to an end-on loading giving limited capability to absorb load redistribution from the buckled braces. For example, upon failure of a third diagonal brace, the remaining braces fail in sequence due to overloading. Similar behavior was observed for the diagonal loading direction case.

The ultimate strength analysis results indicate that the platform can be classified as a "marginal case" in meeting the Section 17 requirements. Even though the ultimate capacity ratio (defined as the ratio of ultimate lateral load carrying capacity to Section 17 ultimate strength analysis lateral loading) for the end-on loading condition is slightly below 1.0 (0.94), the level of uncertainties and biases involved in the overall process of load and strength computation are indicative that for practical purposes the platform is marginally acceptable.

Figures 3.7, 3.10 and 3.13 indicate loads levels when the first member reaches an IR of 1.0 per RP2A, 20th Edition. These results were obtained using the linear elastic analysis performed in Section A.5.3 and are being provided for use by the Trials JIP project and API Task Group.

Section A.5.5 Fatigue

An assessment of accumulated fatigue degradation effects was examined per Section 17.7.2 and Section 17.7.3 of Draft Section 17. An alternative to analytical demonstration is to perform Level III and/or Level IV surveys per Section 14.3 of RP2A. This demonstrates that any known damage is assessed or repaired. Periodic surveys have shown no damage exist.

An analytical fatigue assessment was not performed in this study.

Section A.6 Mitigation Alternatives (Section 17.8)

In order to evaluate possible modifications for future platform uses, various scenarios were identified for reducing load level and increasing the ultimate capacity of the platform. These include:

Reduction of topside loads: The buckling of braces has been found to govern the ultimate capacity estimates for the three directions. Therefore, reduction in topside loads would have only a minimal influence on increasing the ultimate capacity level for this platform.

Reduction in wave load elements: Removal of appurtenances such as boat landings etc. will help in reducing wave and current load levels, but only slightly.

In further evaluations, these and other possible scenarios may be considered. Alternatively, the characterization of element strength estimates may be improved by reducing bias. The hurricane Andrew JIP (OTC Paper 7473, 1994) indicated bias in ultimate strength estimates, which were based on comparison of analytically predicted behavior with the observed behavior during Andrew.

Part B: Review and Feedback to the API TG 92-5

The API RP2A, 9th Edition metocean criteria for the Gulf of Mexico may be provided in the commentary section to help in the assessment process.

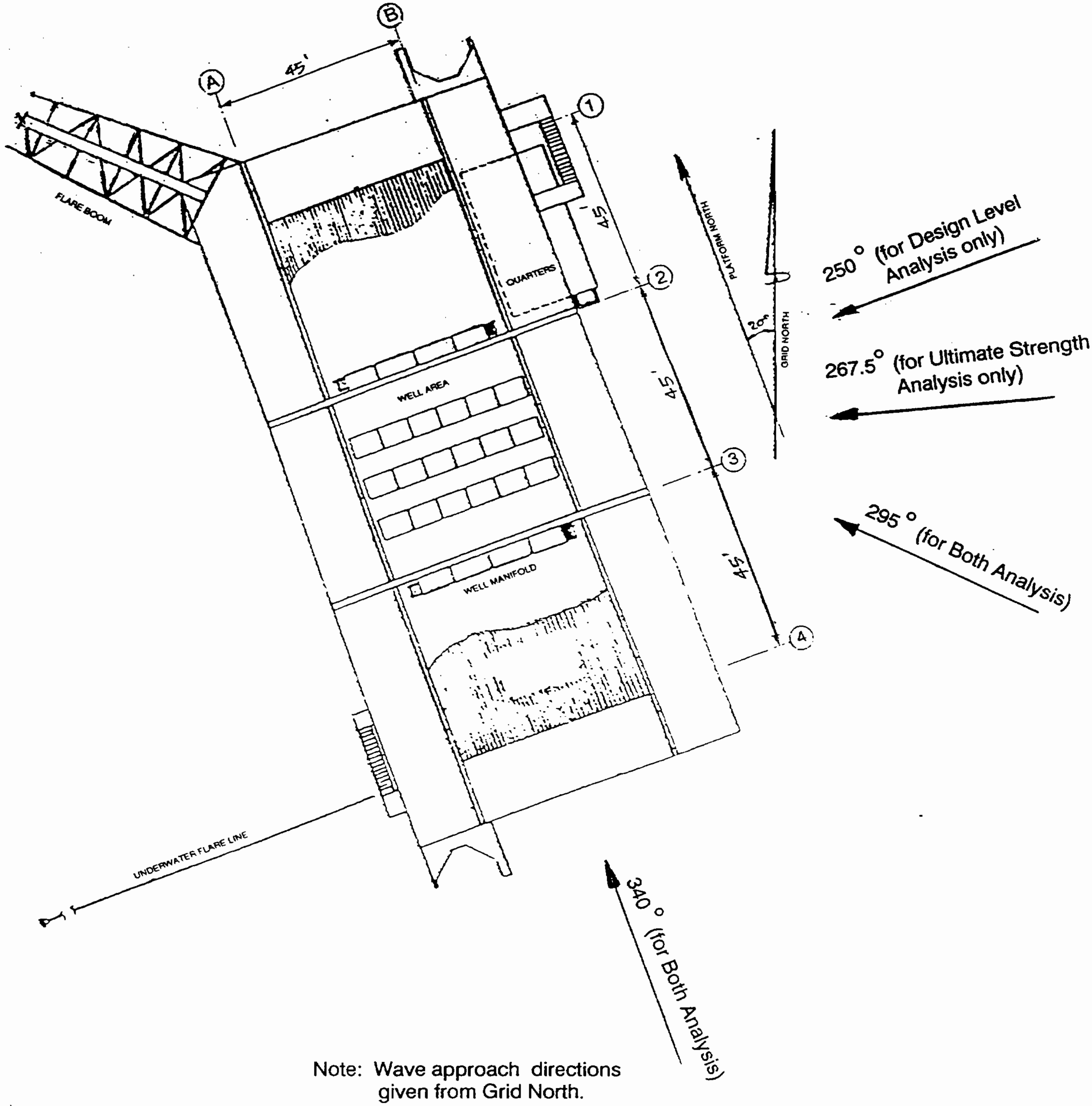


Fig. 3.1 WAVE APPROACH DIRECTIONS - PLATFORM P

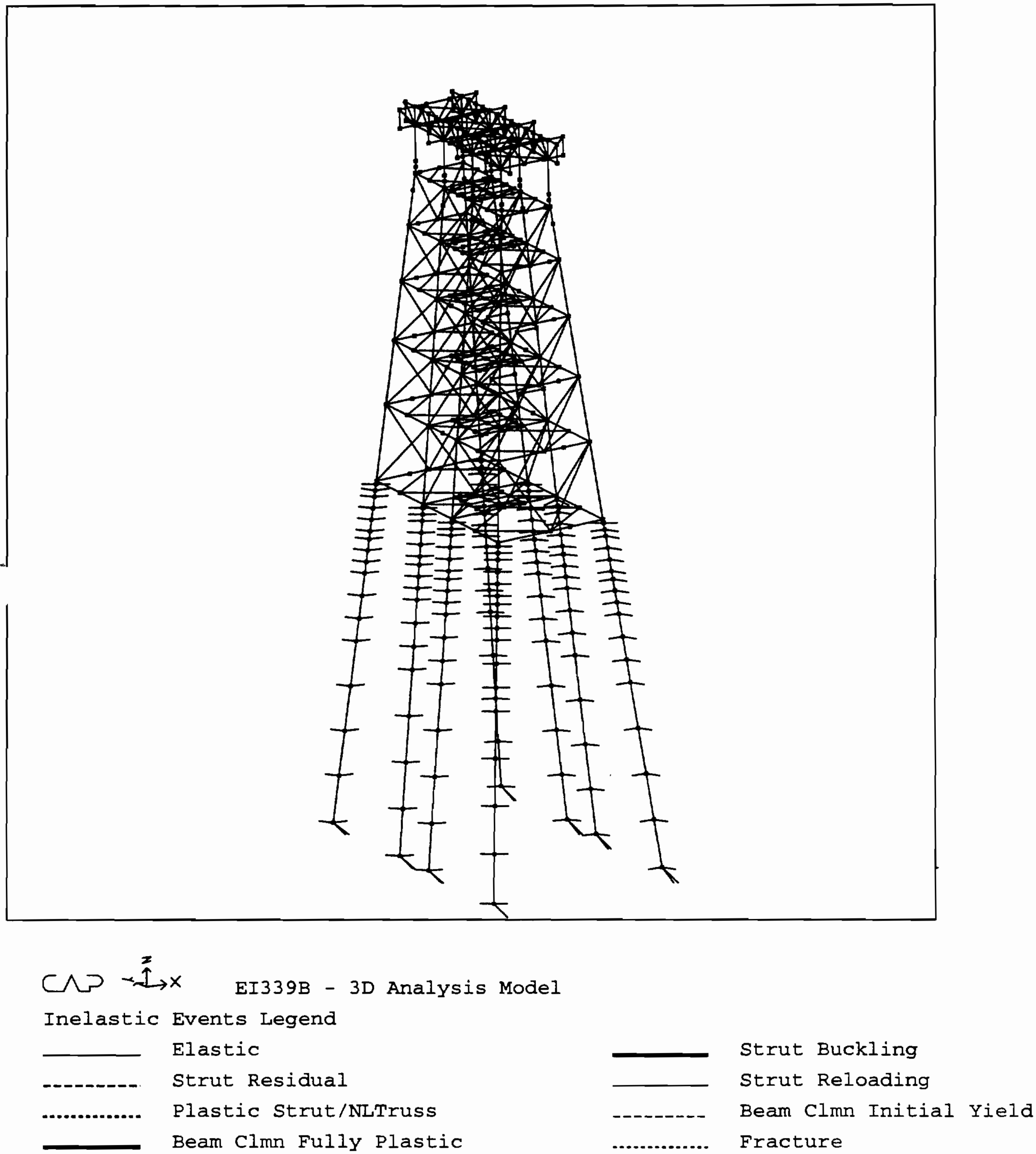
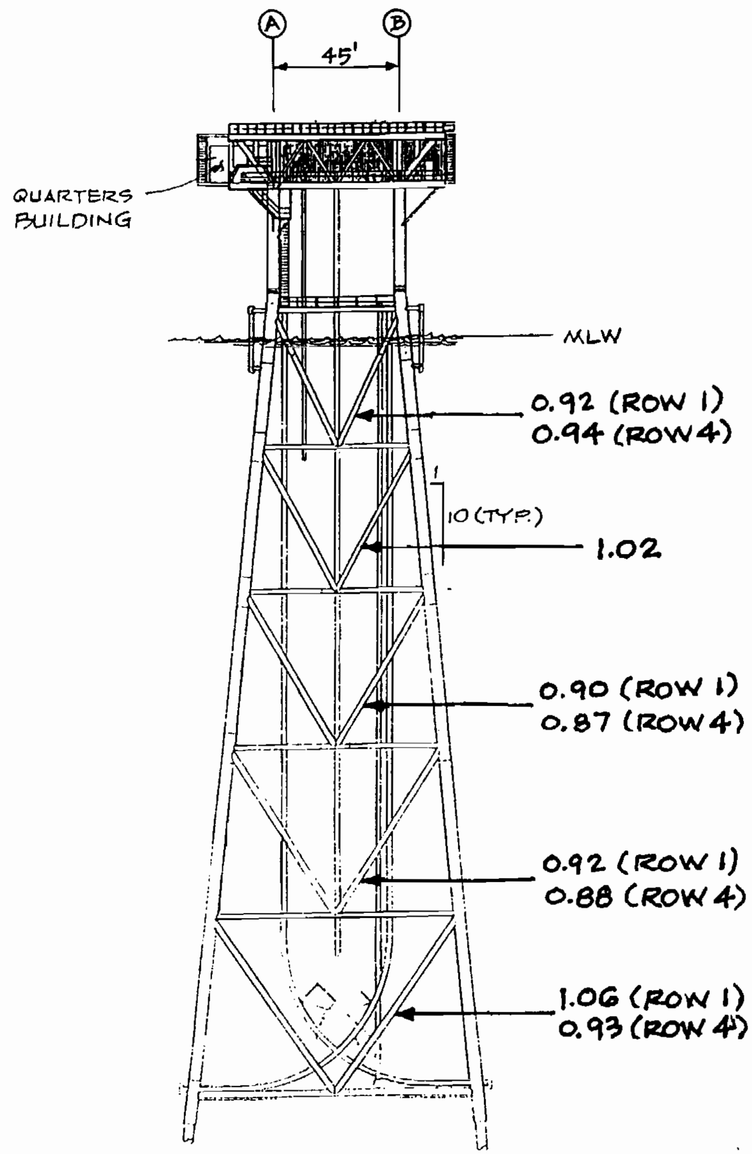
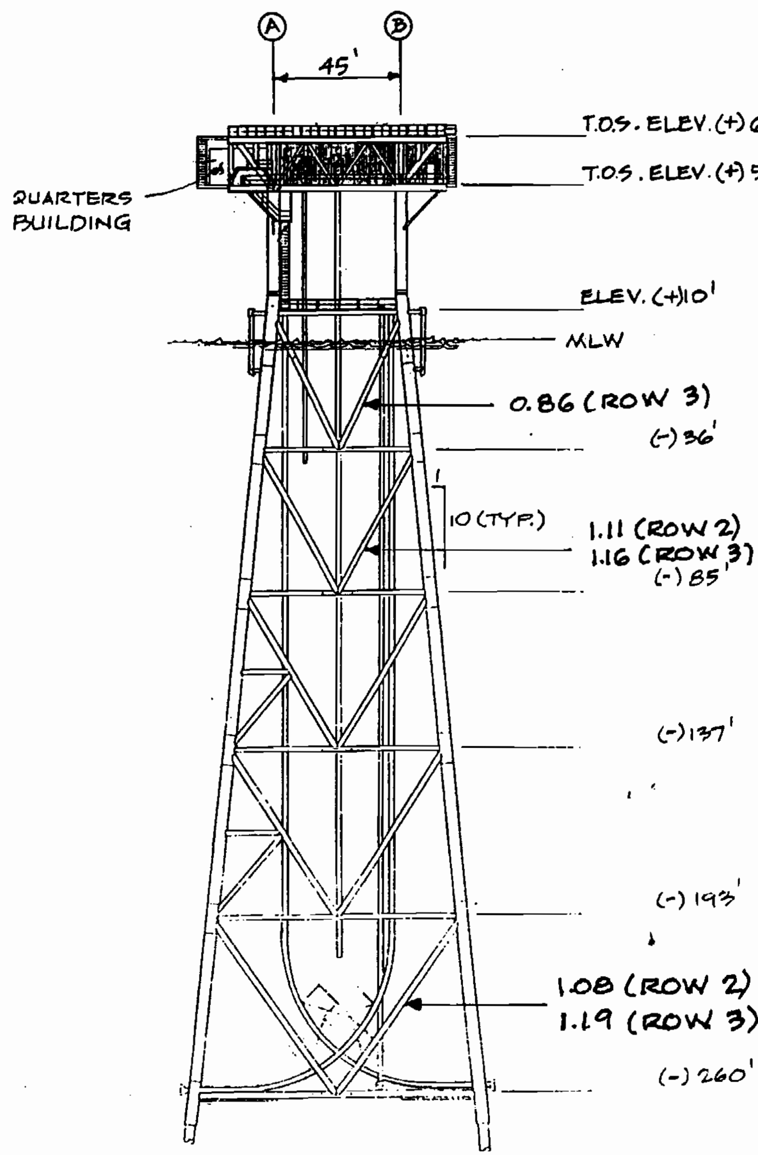


Fig. 3.2: 3-D Analysis Model - Platform P



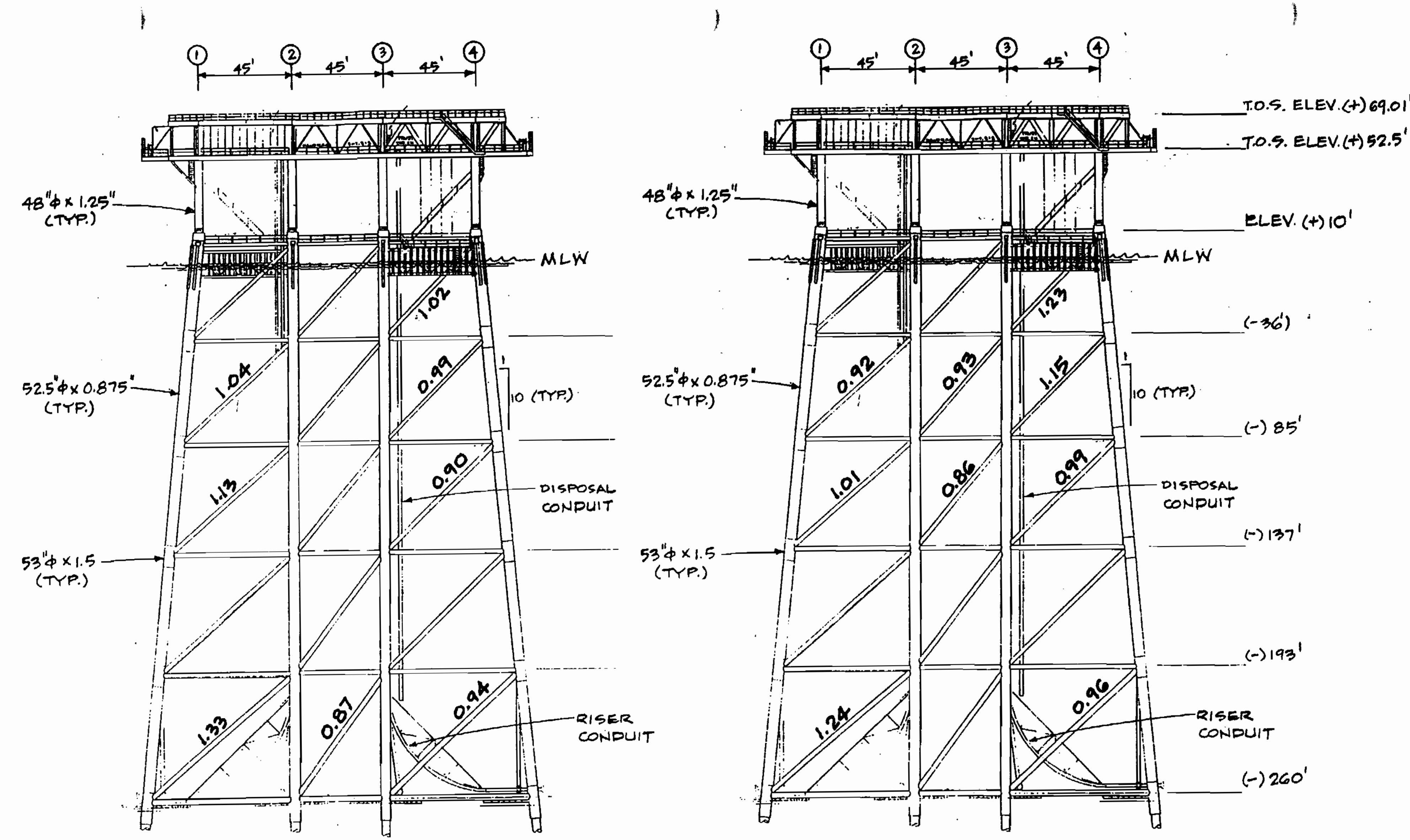
Row-1 & Row-4 Frames

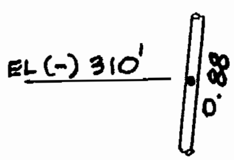


Row-2 & Row-3 Frames

Note: I.R.'s exceeding 0.85 shown

Fig. 3.3: Design Level Analysis - Broadside Direction



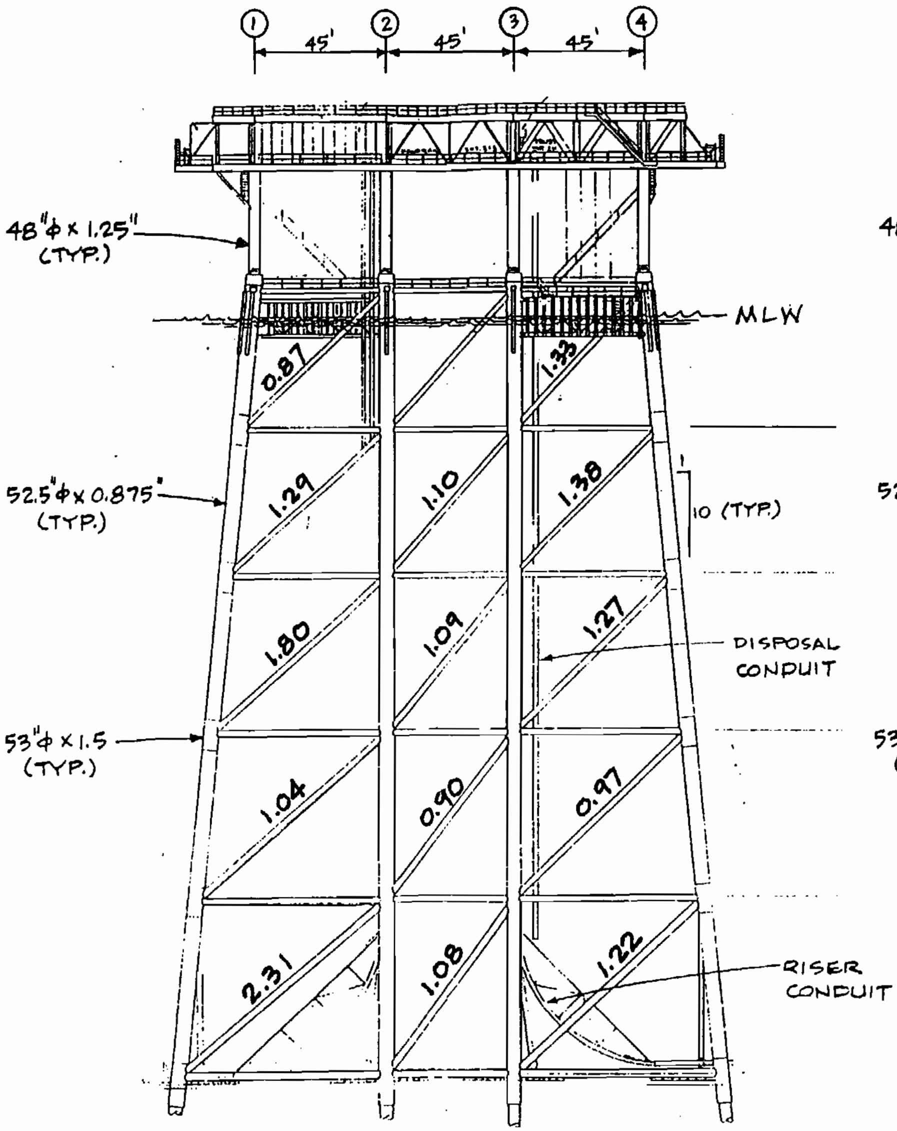


Row-A Frame

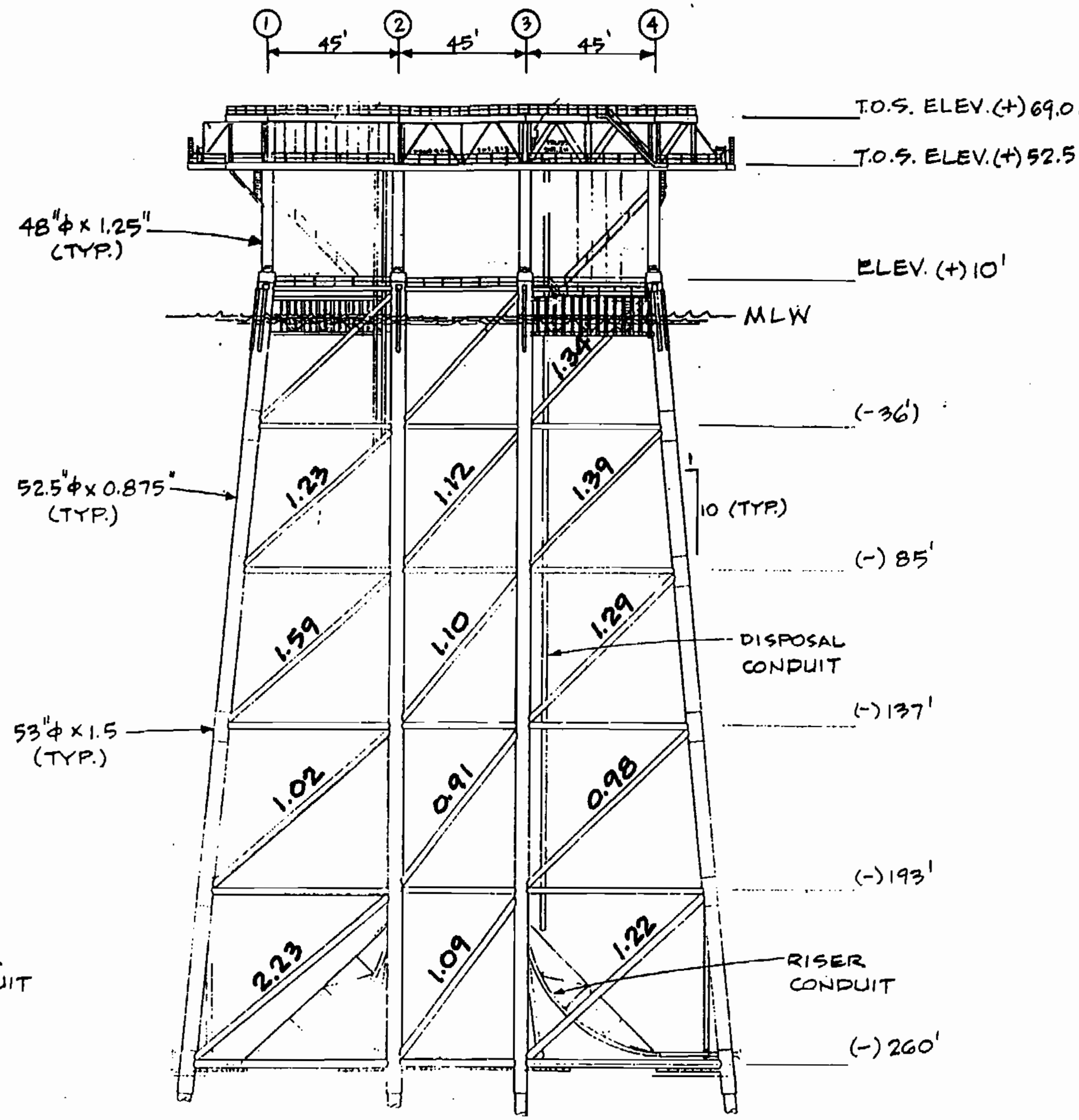
Row-B Frame

Note: I.R.'s exceeding 0.85 shown

Fig. 3.4: Design Level Analysis - Diagonal Direction



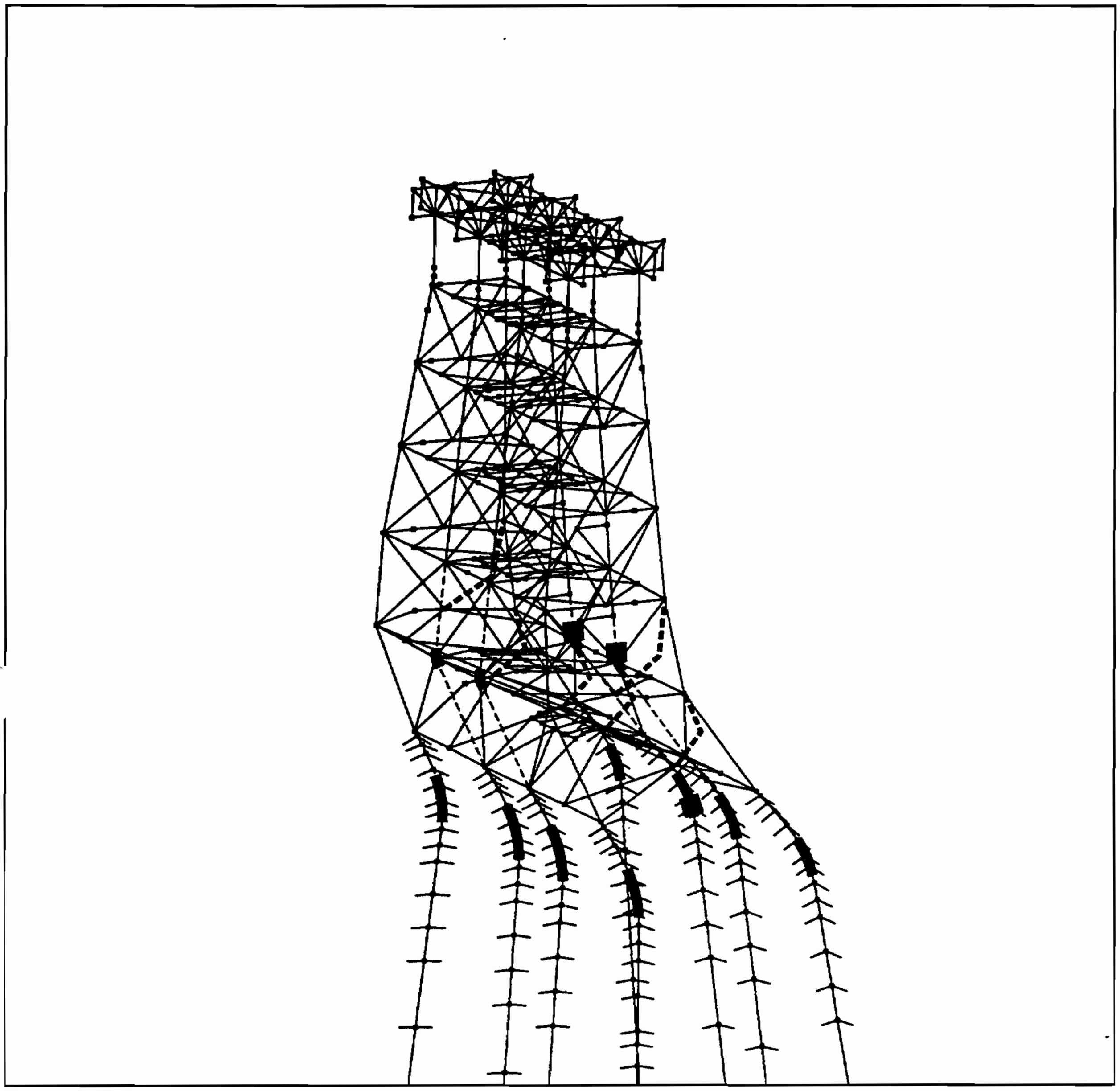
Row-A Frame



Row-B Frame

Note: I.R.'s exceeding 0.85 shown

Fig. 3.5: Design Level Ar^lysis - End On Direction

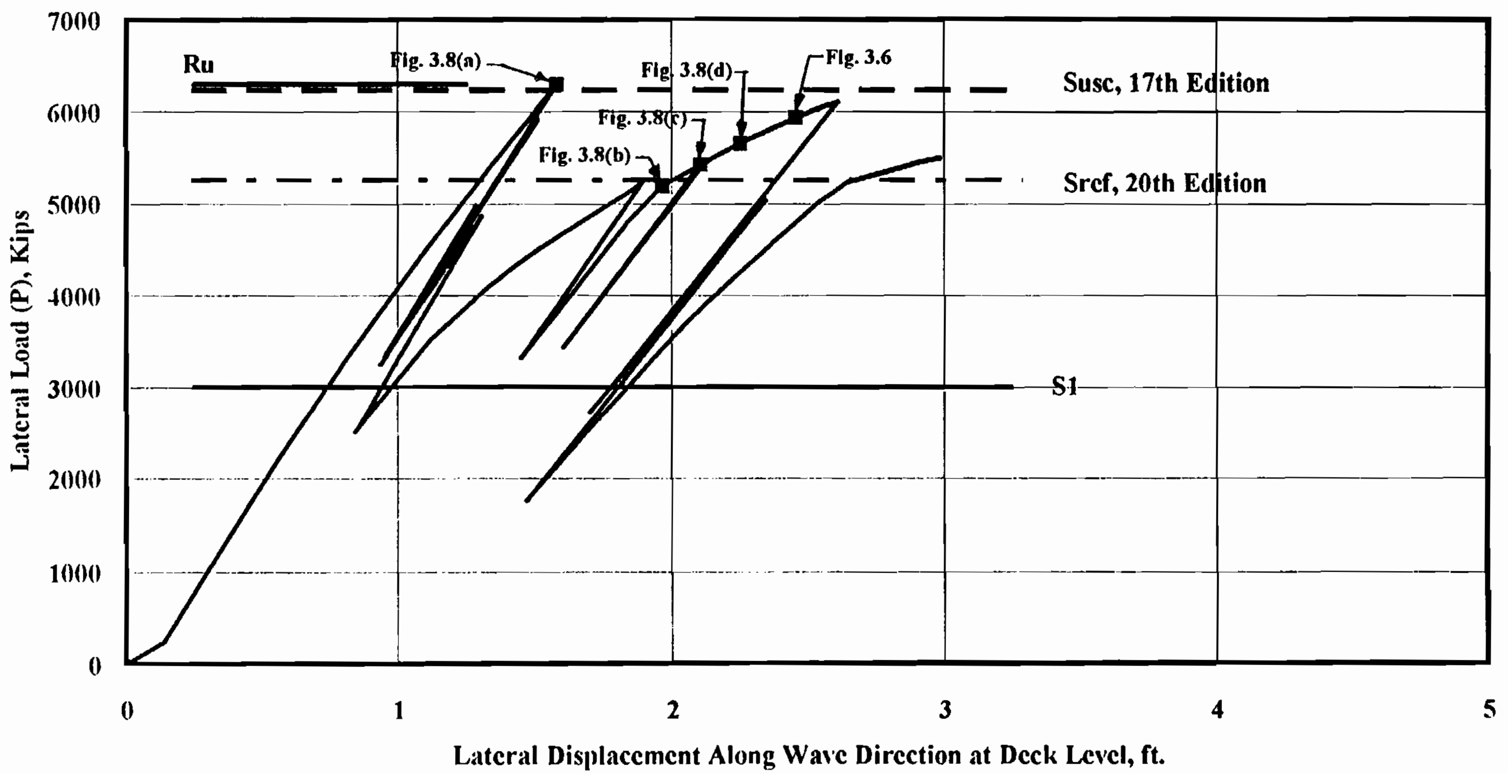


CAP EI339B Pushover Analysis - X Dir. LS154

Inelastic Events Legend

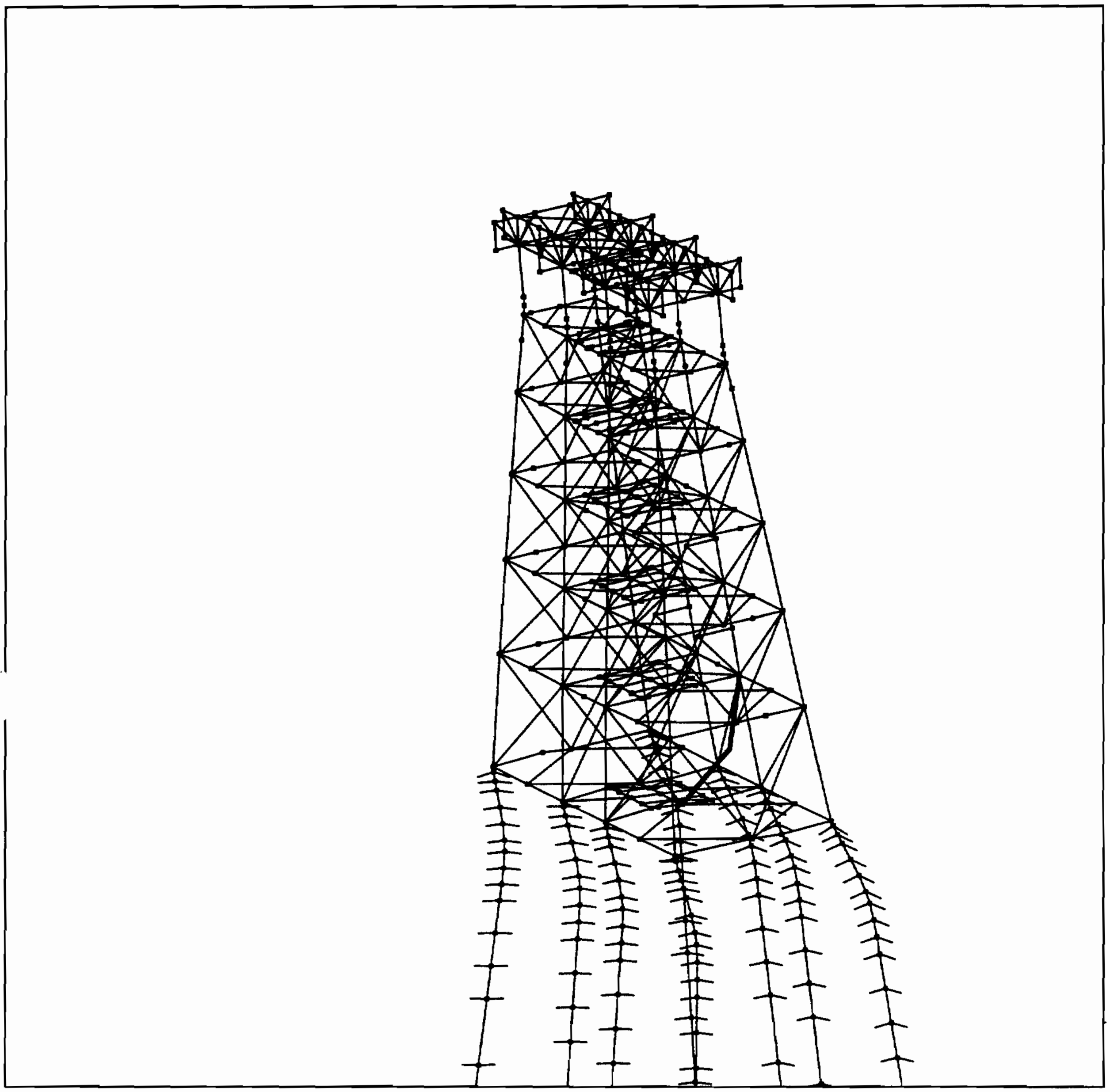
Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Fig. 3.6: Failure Mechanism - Broadside Direction



Load Level at which First Component Reaches I.R. of 1.0 (S1)	3,000 Kips
Load Level, Section 17 Ultimate Strength Criteria (Susc)	6,235 Kips
Reference Level Load (Sref)	5,256 Kips
Ultimate Capacity (Ru)	6,298 Kips
Reserve Strength Ratio (RSR)	1.2
Platform Failure Mode: Jacket, Pile, Soils, etc.	Jacket Braces

Fig. 3.7: Load-Displacement Results -Pushover Analysis (Broadside Direction)

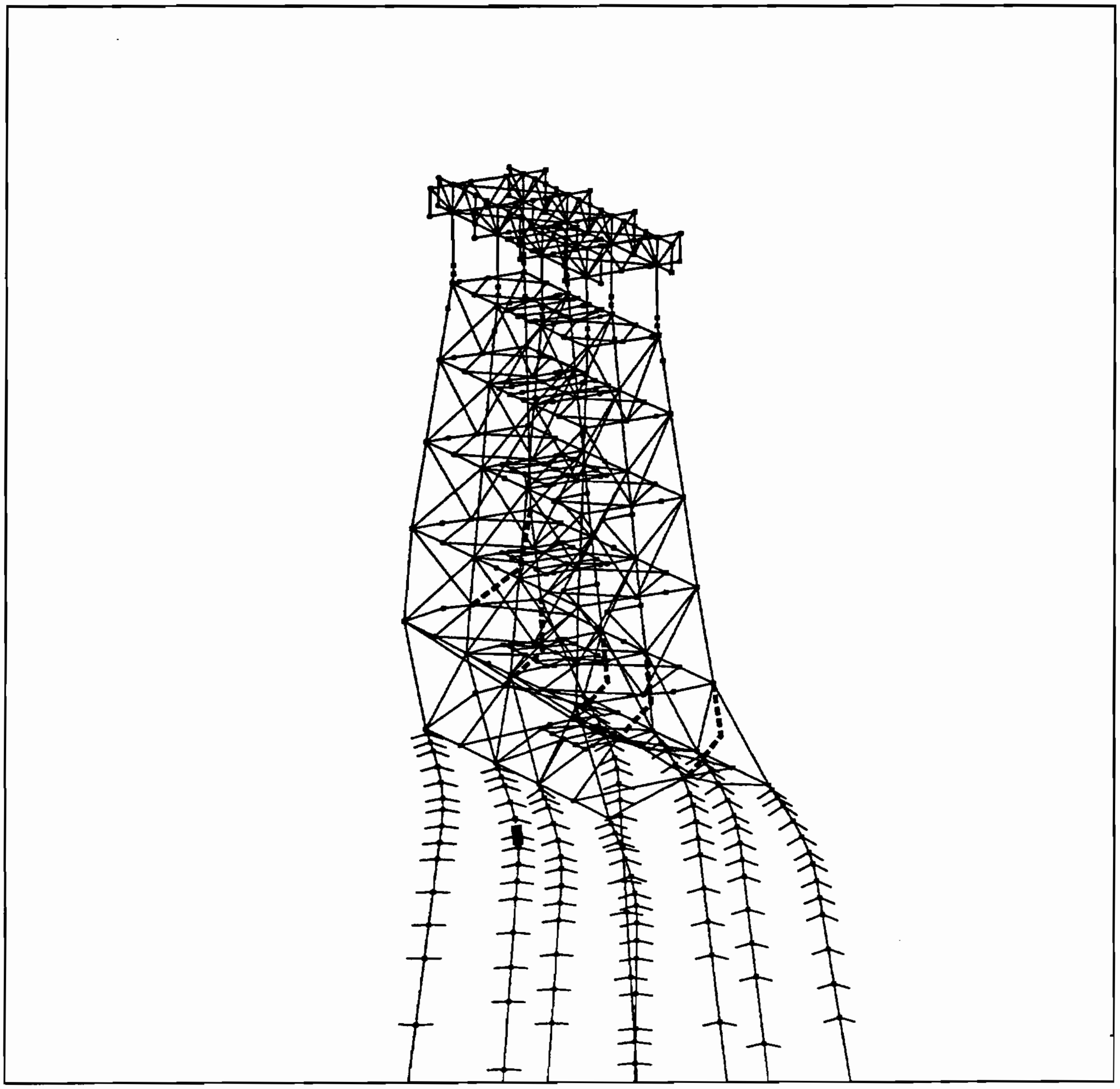


CAP EI339B Pushover Analyssi - Broadside Direction

Inelastic Events Legend

————	Elastic	━━━━	Strut Buckling
---------	Strut Residual	————	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
━━━━	Beam Clmn Fully Plastic	....	Fracture

Fig. 3.8(a): Inelastic Events at Stage per Fig. 3.7 - Broadside Direction



CAP EI339B Pushover Analysis - X Dir. LS122

Inelastic Events Legend

————	Elastic	- - - - - -	Strut Buckling
- - - - - -	Strut Residual	....	Strut Reloading
....	Plastic Strut/NLTruss	- - - - - -	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic	- - - - - -	Fracture

Fig. 3.8(b): Inelastic Events at Stage per Fig. 3.7 - Broadside Direction

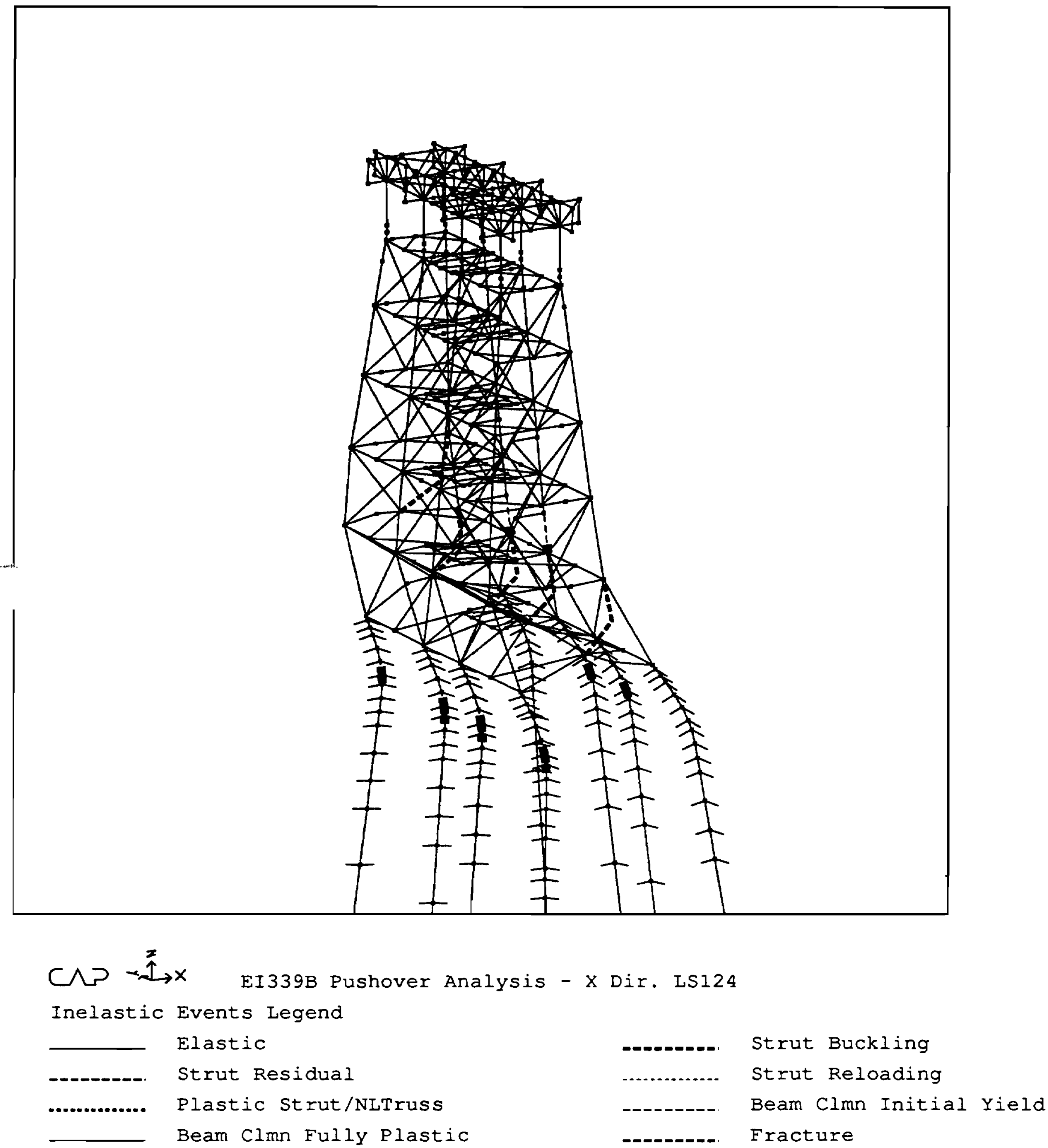


Fig. 3.8(c): Inelastic Events at Stage per Fig. 3.7 - Broadside Direction

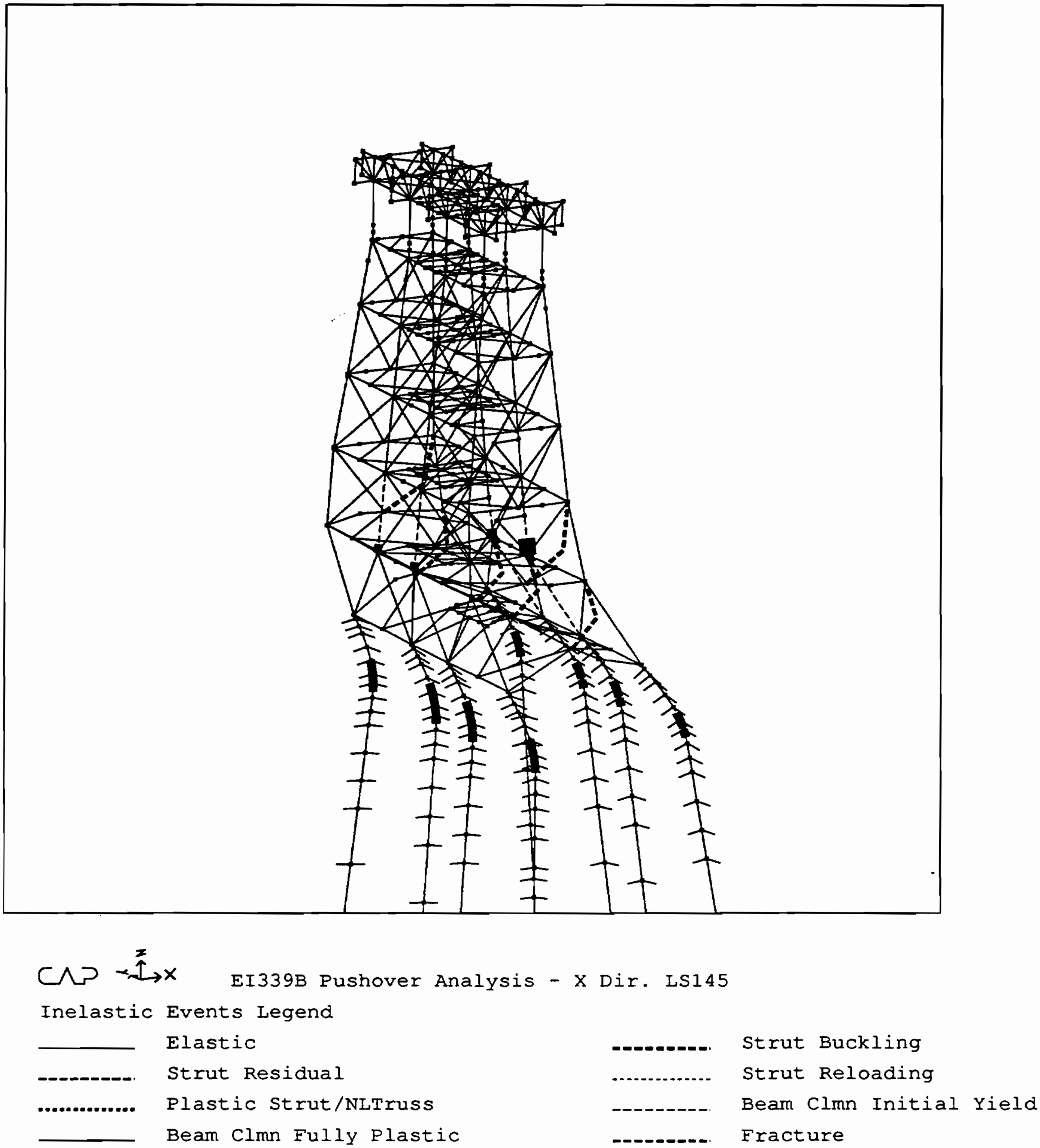
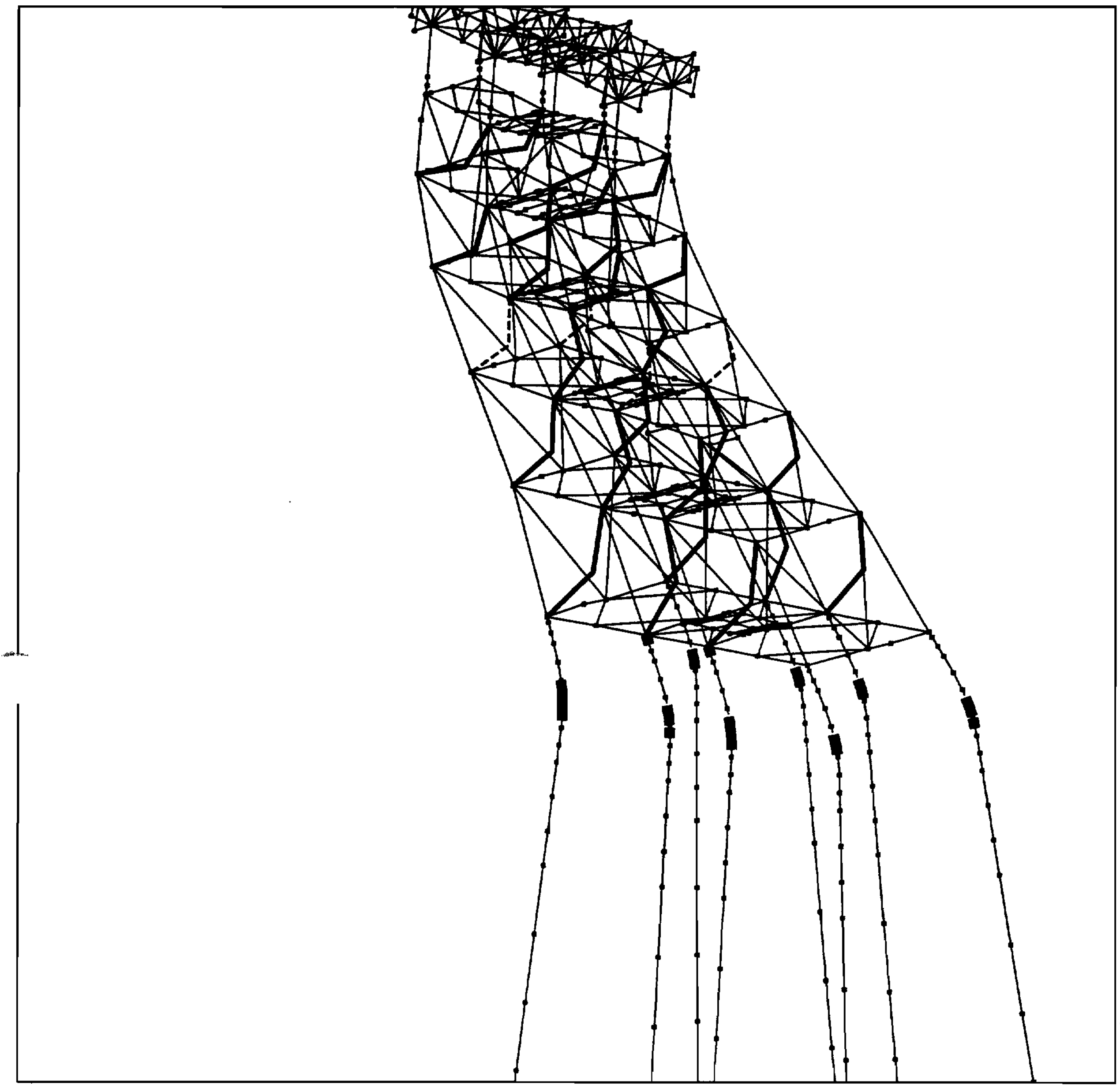


Fig. 3.8(d): Inelastic Events at Stage per Fig. 3.7 - Broadside Direction

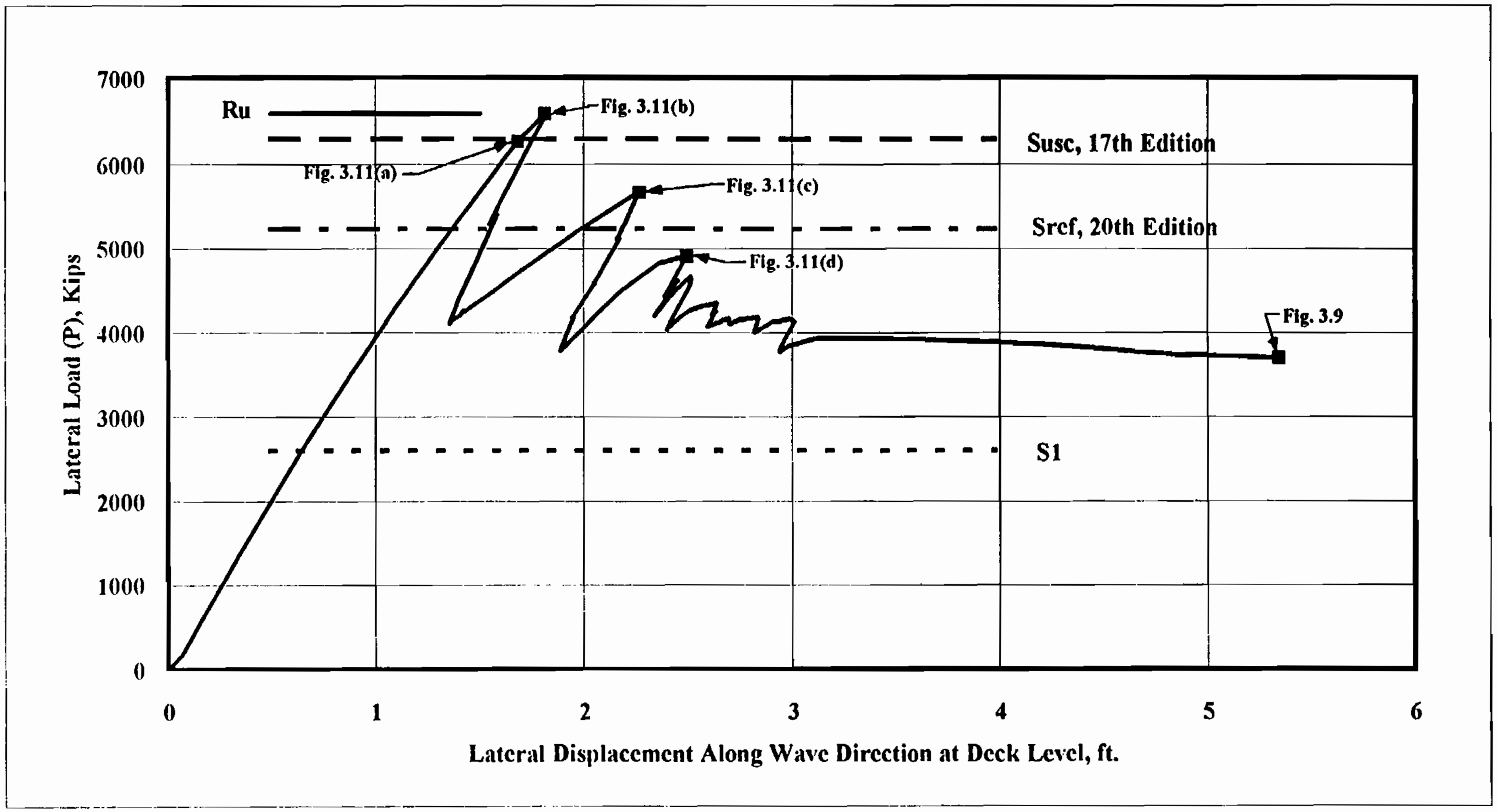


CAP Pushover Analysis- Diagonal Dir. Step 289

Inelastic Events Legend

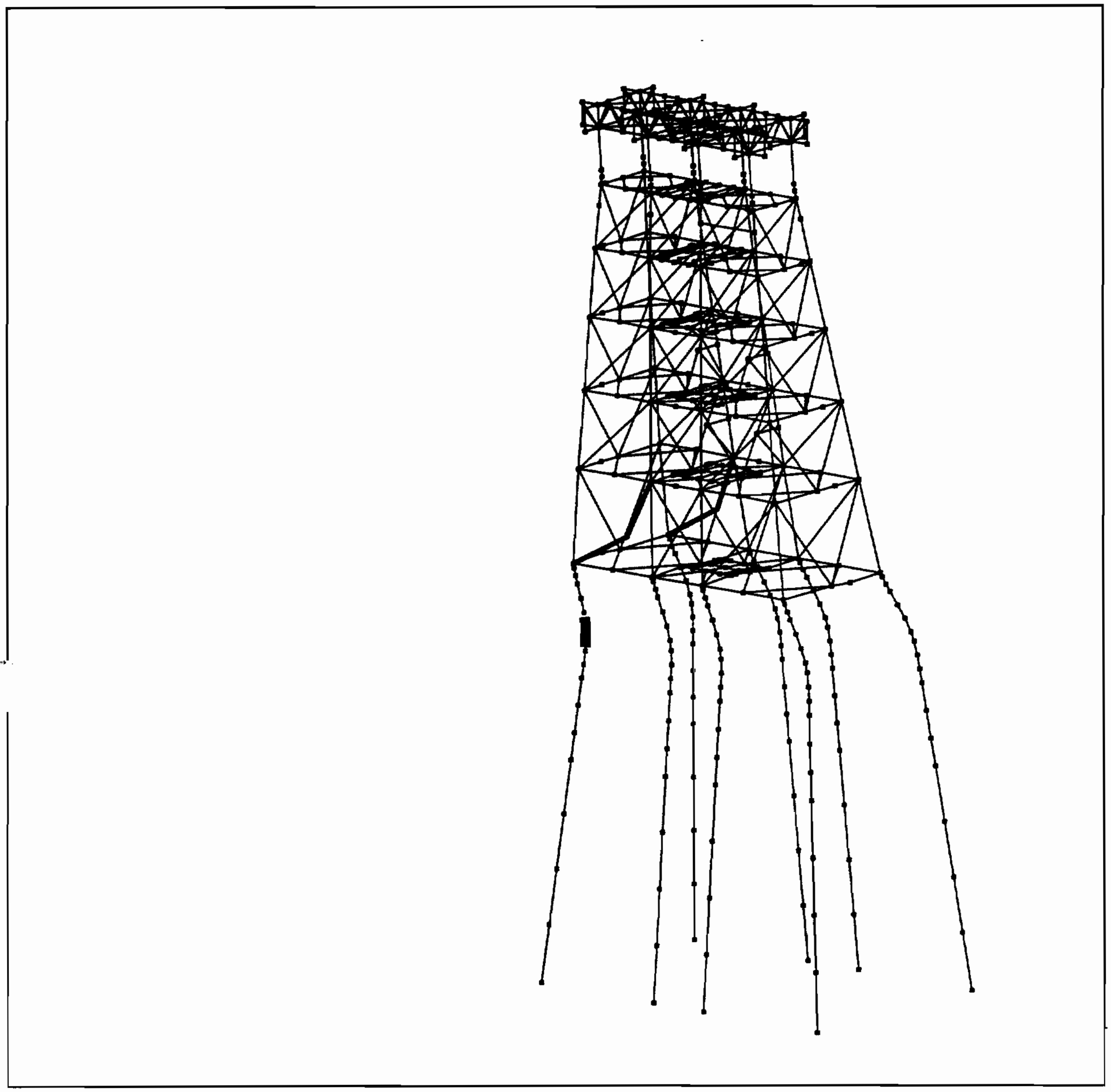
Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Fig. 3.9: Failure Mechanism - Diagonal Direction



Load Level at which First Component Reaches I.R. of 1.0 (S1)	2,600 Kips
Load Level, Section 17 Ultimate Strength Criteria (Susc)	6,291 Kips
Reference Level Load (Sref)	5,232 Kips
Ultimate Capacity (Ru)	6,587 Kips
Reserve Strength Ratio (RSR)	1.26
Platform Failure Mode: Jacket, Pile, Soils, etc.	Jacket Diagonals

Fig. 3.10: Load-Displacement Results -Pushover Analysis (Diagonal Direction)



CAP Pushover Analysis- Diagonal Dir. Step 9

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Fig. 3.11(a): Inelastic Events at Stage per Fig. 3.10 - Diagonal Direction

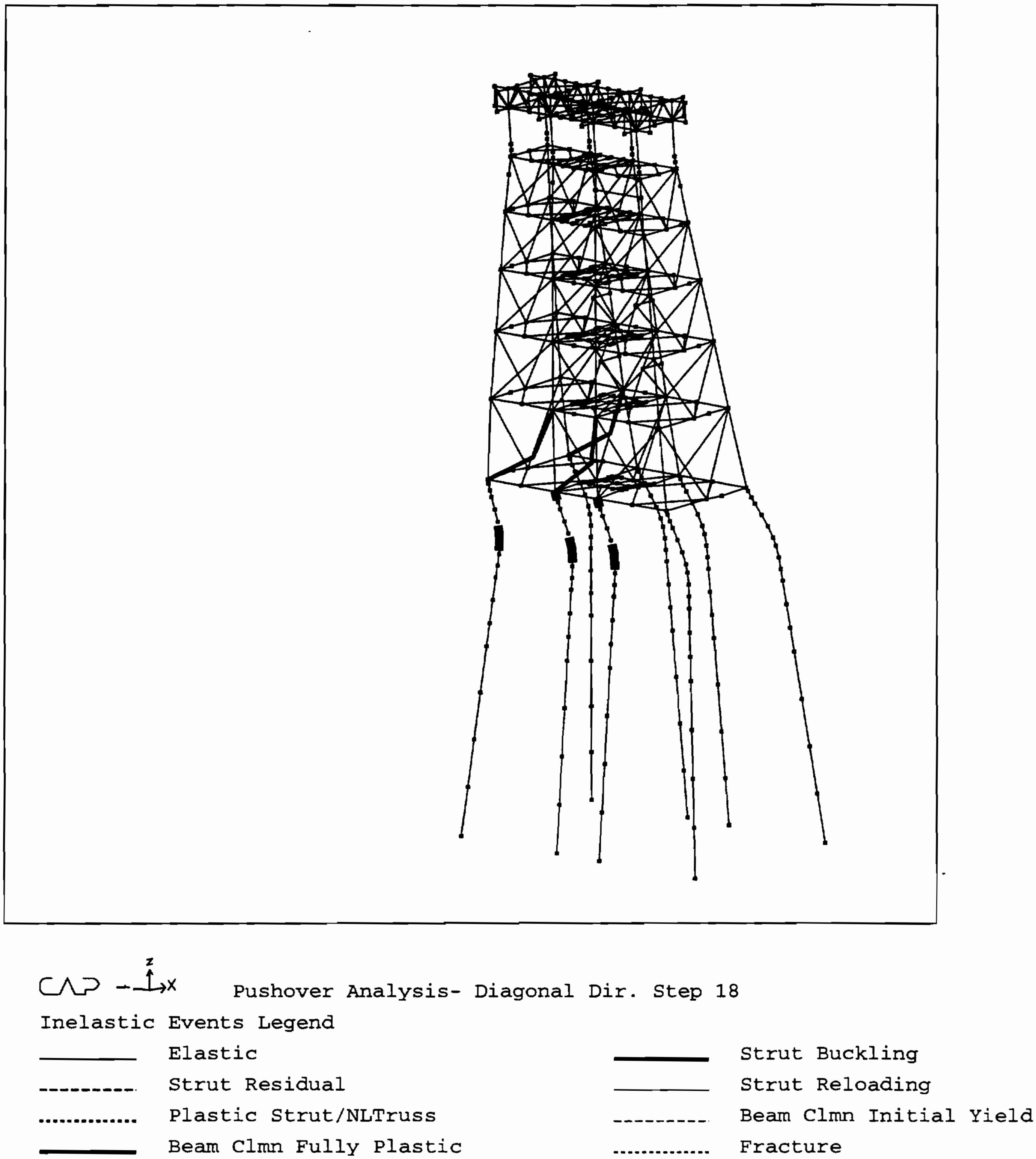
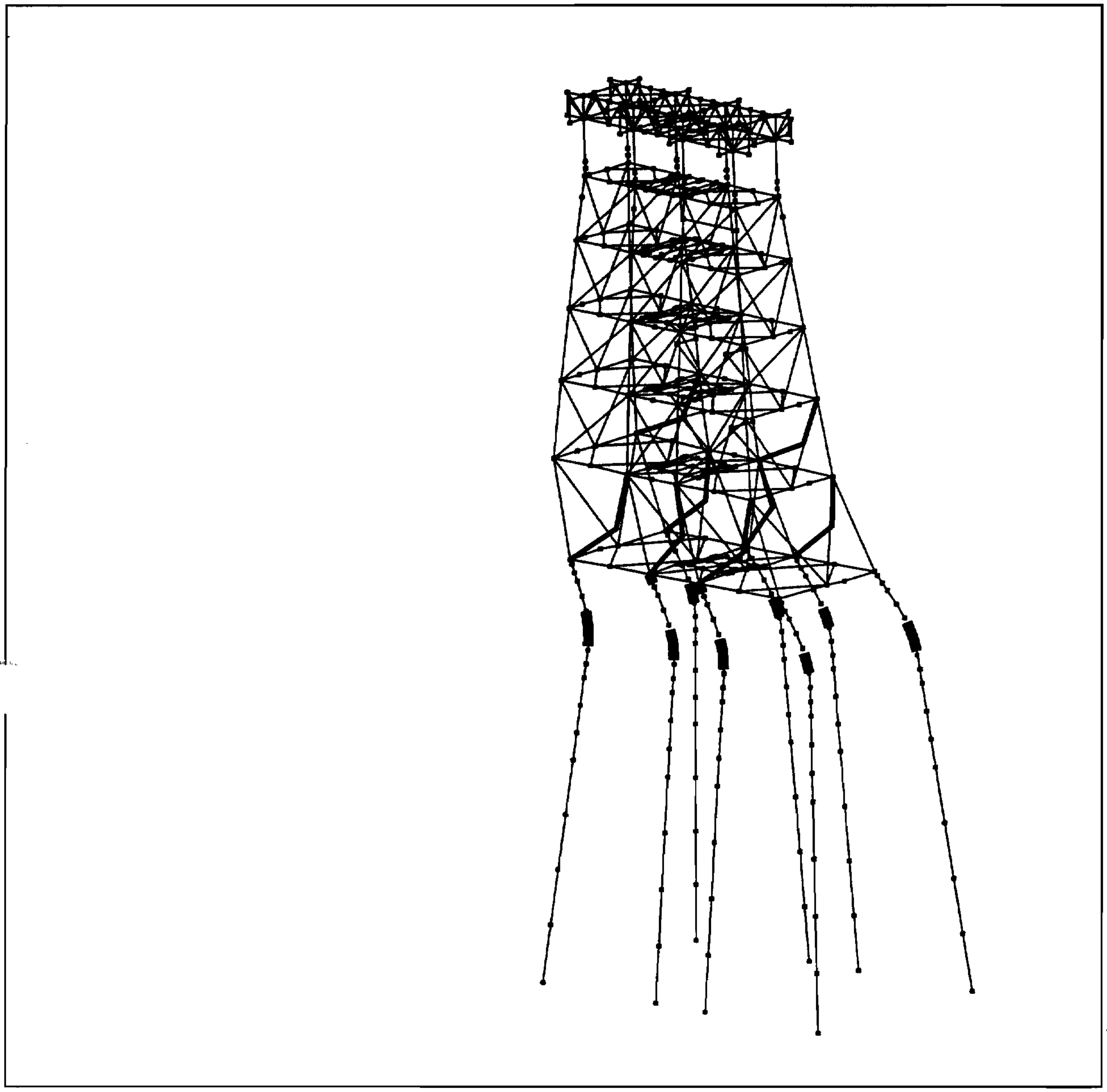


Fig. 3.11(b): Inelastic Events at Stage per Fig. 3.10 - Diagonal Direction



CAP z x Pushover Analysis- Diagonal Dir. Step 76

Inelastic Events Legend

Elastic

Strut Residual

Plastic Strut/NLTruss

Beam Clmn Fully Plastic

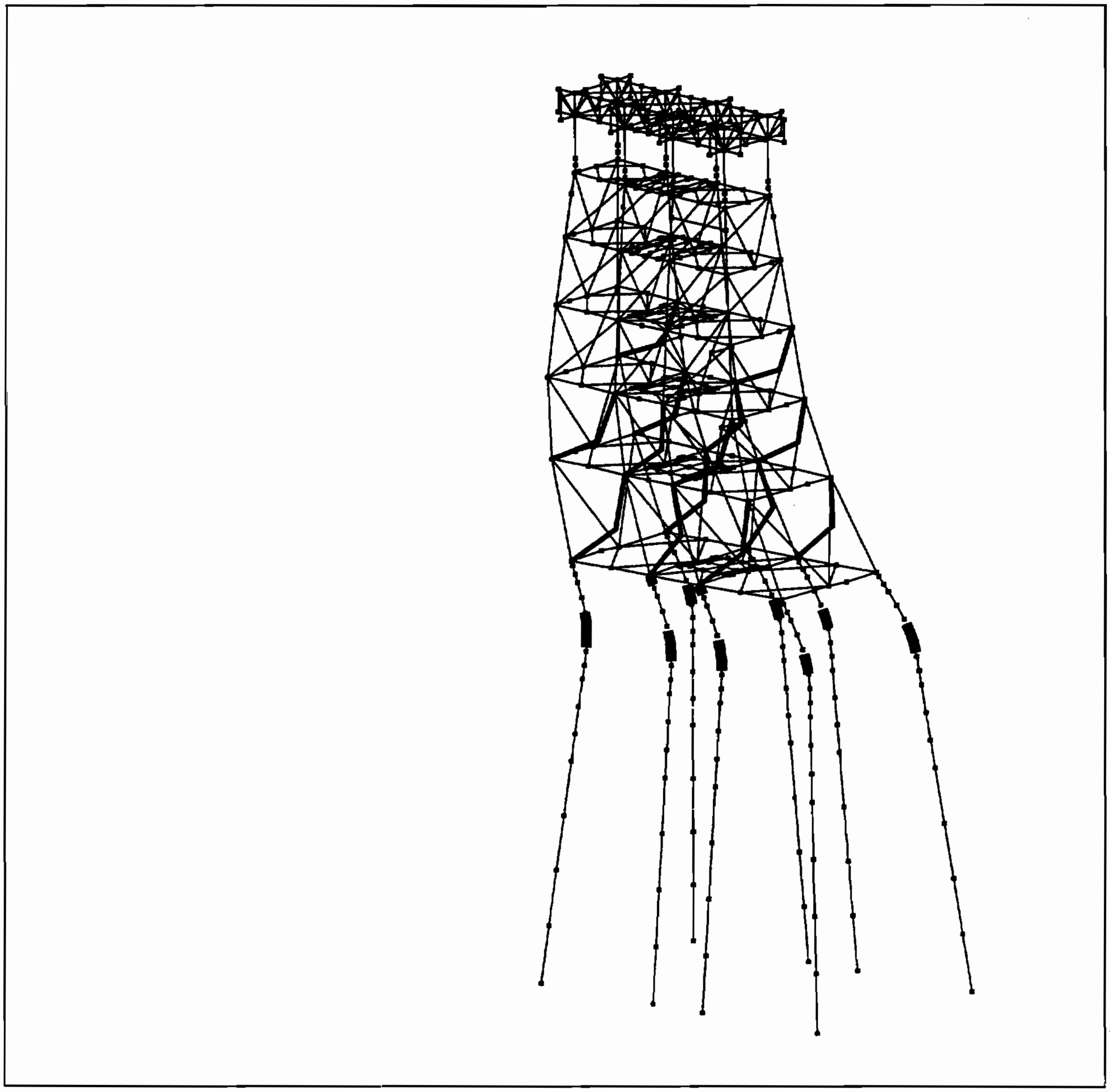
Strut Buckling

Strut Reloading

Beam Clmn Initial Yield

Fracture

Fig. 3.11(c): Inelastic Events at Stage per Fig. 3.10 - Diagonal Direction



CAP z x Pushover Analysis- Diagonal Dir. Step 127

Inelastic Events Legend

________	Elastic	________	Strut Buckling
---------	Strut Residual	________	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
________	Beam Clmn Fully Plastic	....	Fracture

Fig. 3.11(d): Inelastic Events at Stage per Fig. 3.10 - Diagonal Direction

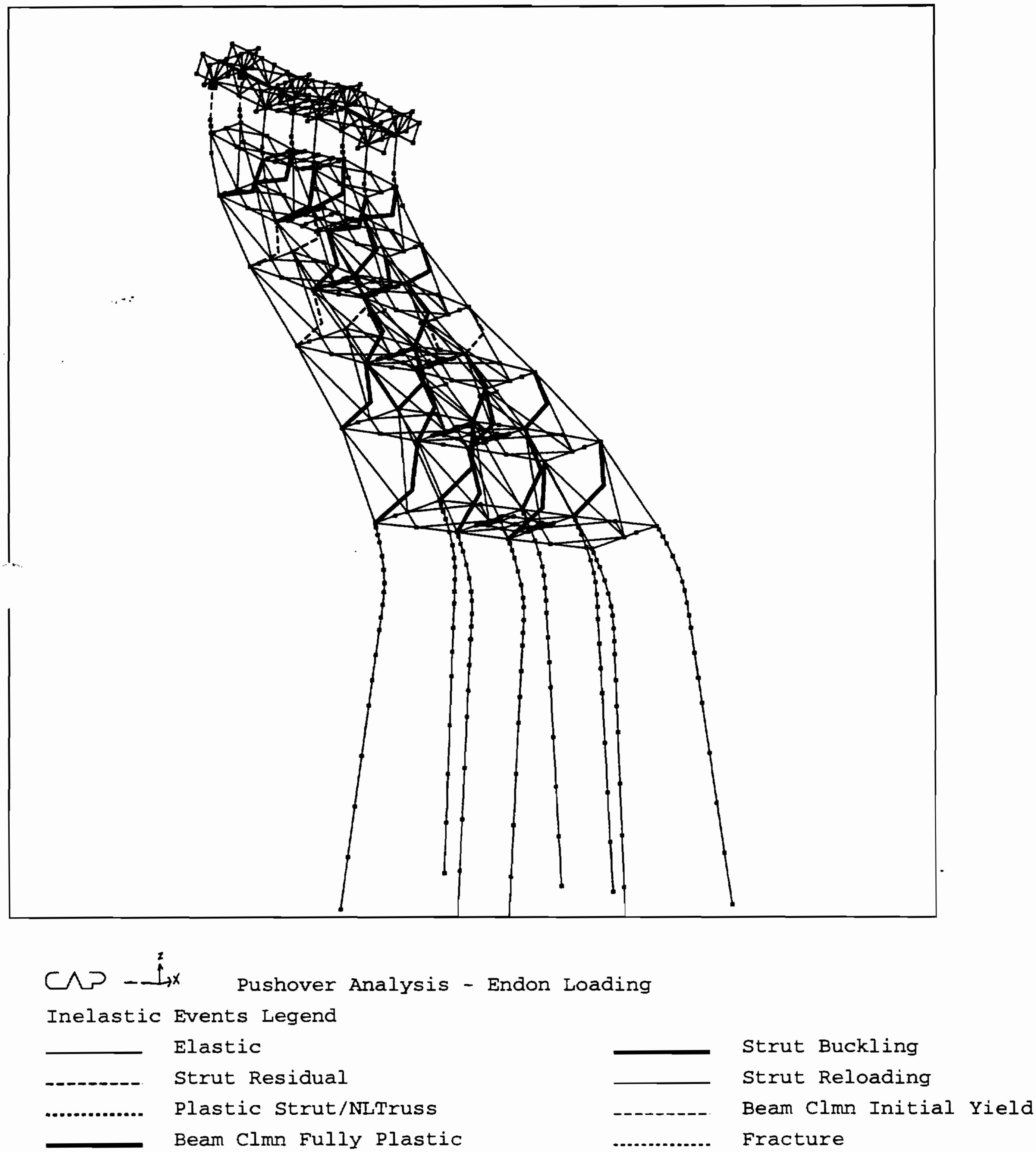
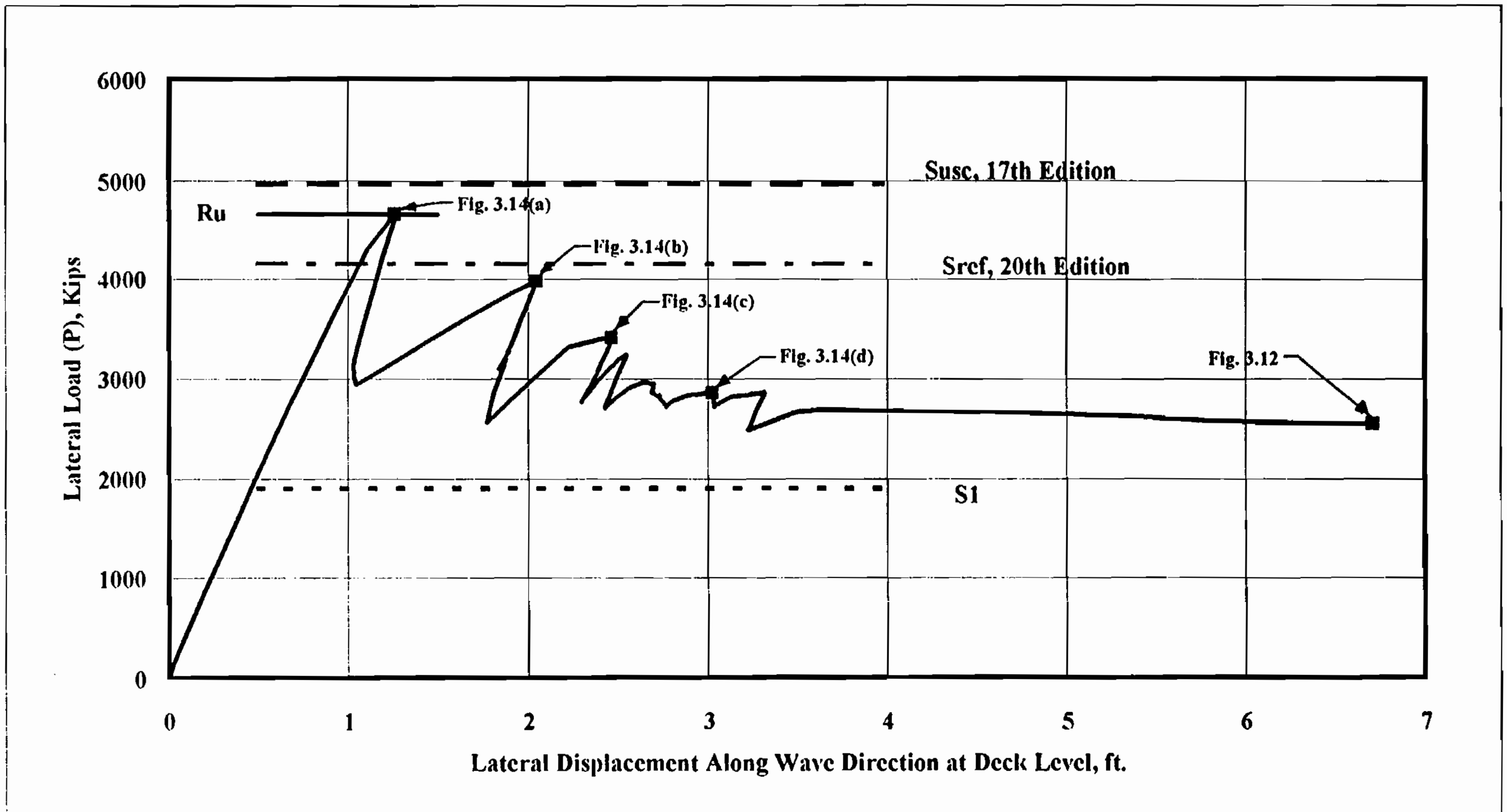


Fig. 3.12: Failure Mechanism - End On Direction



Load Level at which First Component Reaches I.R. of 1.0 (S1)	1,950 Kips
Load Level, Section 17 Ultimate Strength Criteria (Susc)	4,964 Kips
Reference Level Load, 20th Edition Criteria (Sref)	4,154 Kips
Ultimate Capacity (Ru)	4,660 Kips
Reserve Strength Ratio (RSR)	1.14
Platform Failure Mode: Jacket, Pile, Soils, etc.	Jacket Diagonals

Fig. 3.13: Load-Displacement Results -Pushover Analysis (End-On Direction)

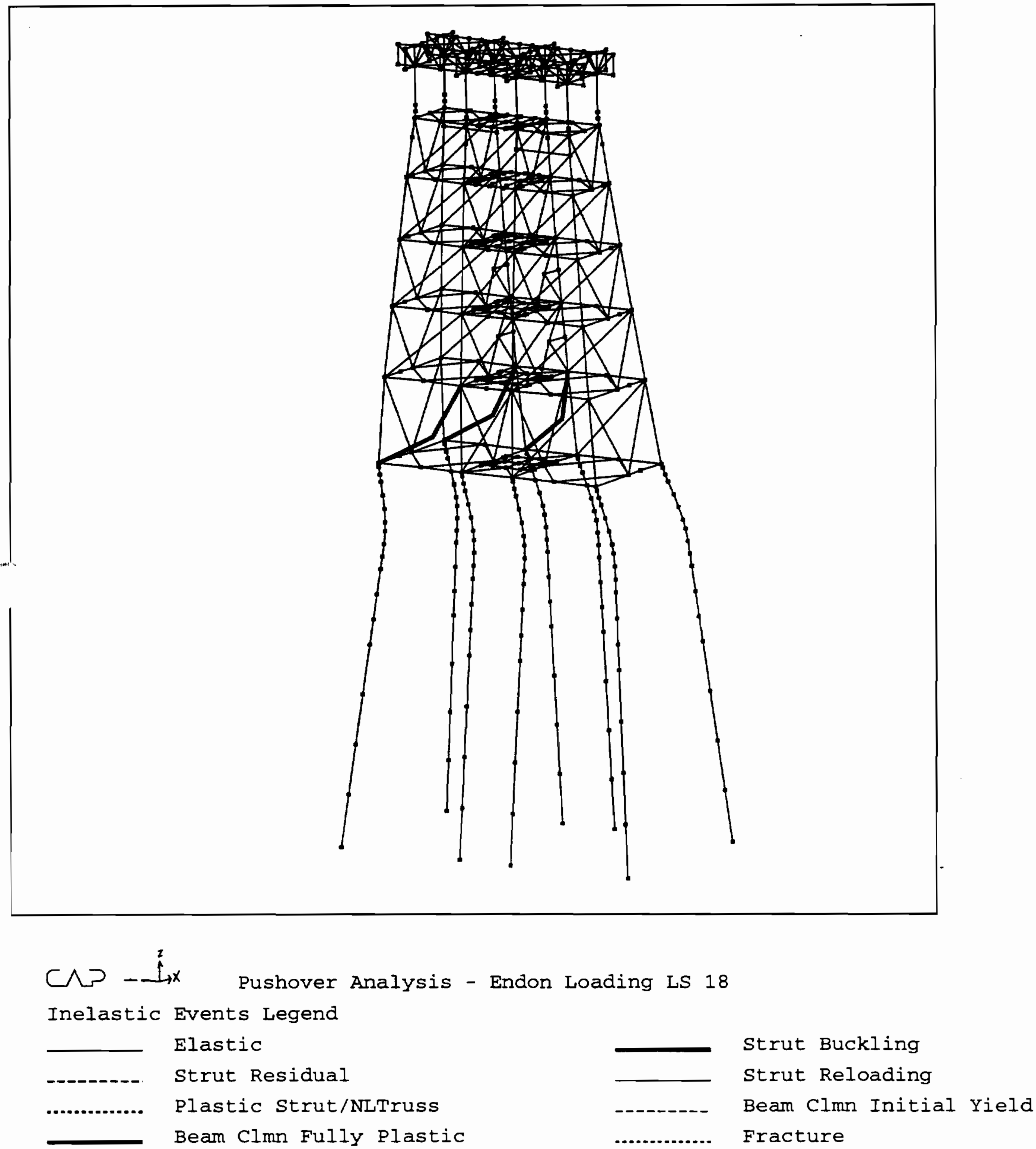
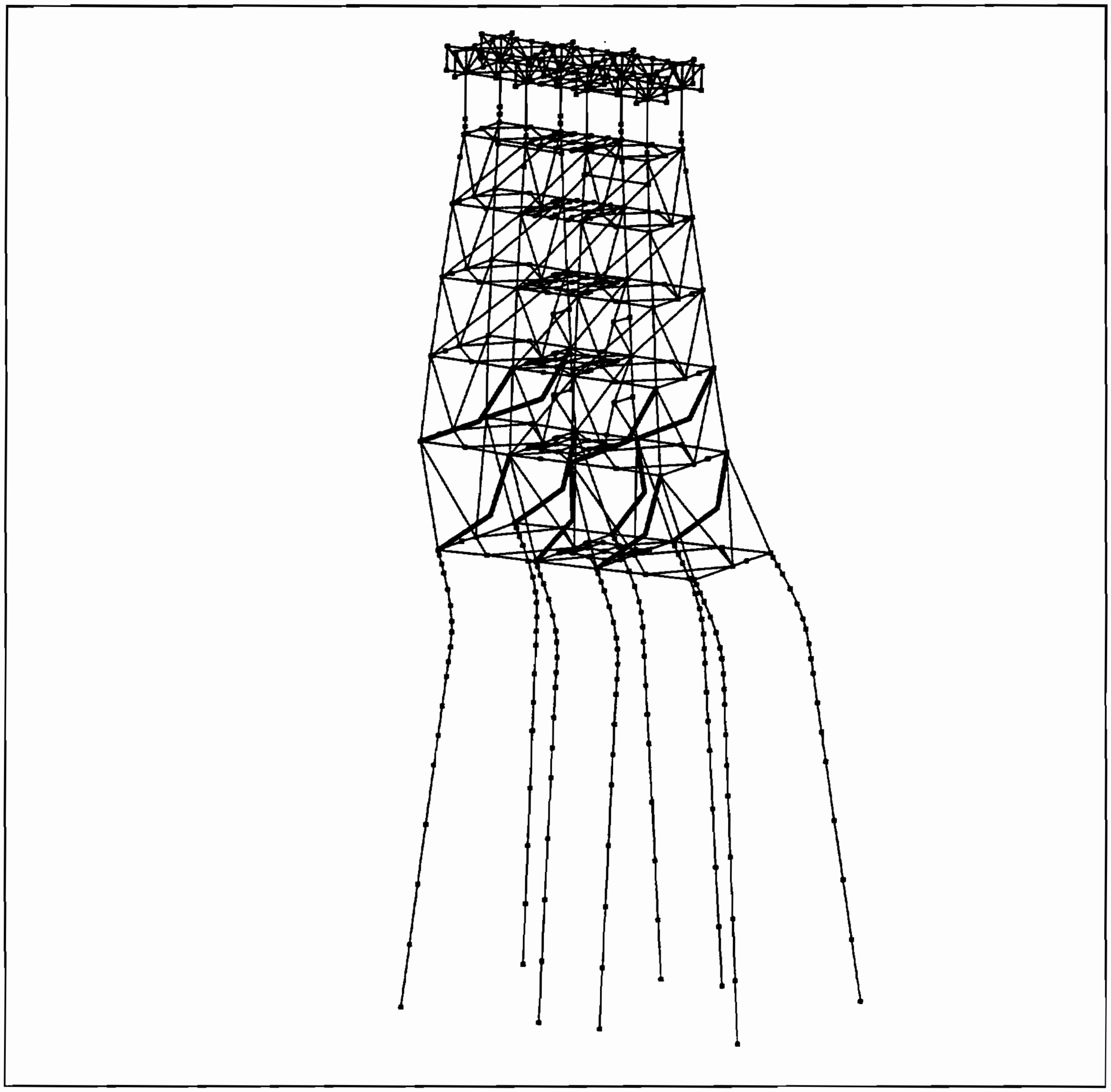


Fig. 3.14(a): Inelastic Events at Stage per Fig. 3.13 - End On Direction



CAP Pushover Analysis - Endon Loading LS 58

Inelastic Events Legend

Elastic	Strut Buckling
Strut Residual	Strut Reloading
Plastic Strut/NLTruss	Beam Clmn Initial Yield
Beam Clmn Fully Plastic	Fracture

Fig. 3.14(b): Inelastic Events at Stage per Fig. 3.13 - End On Direction

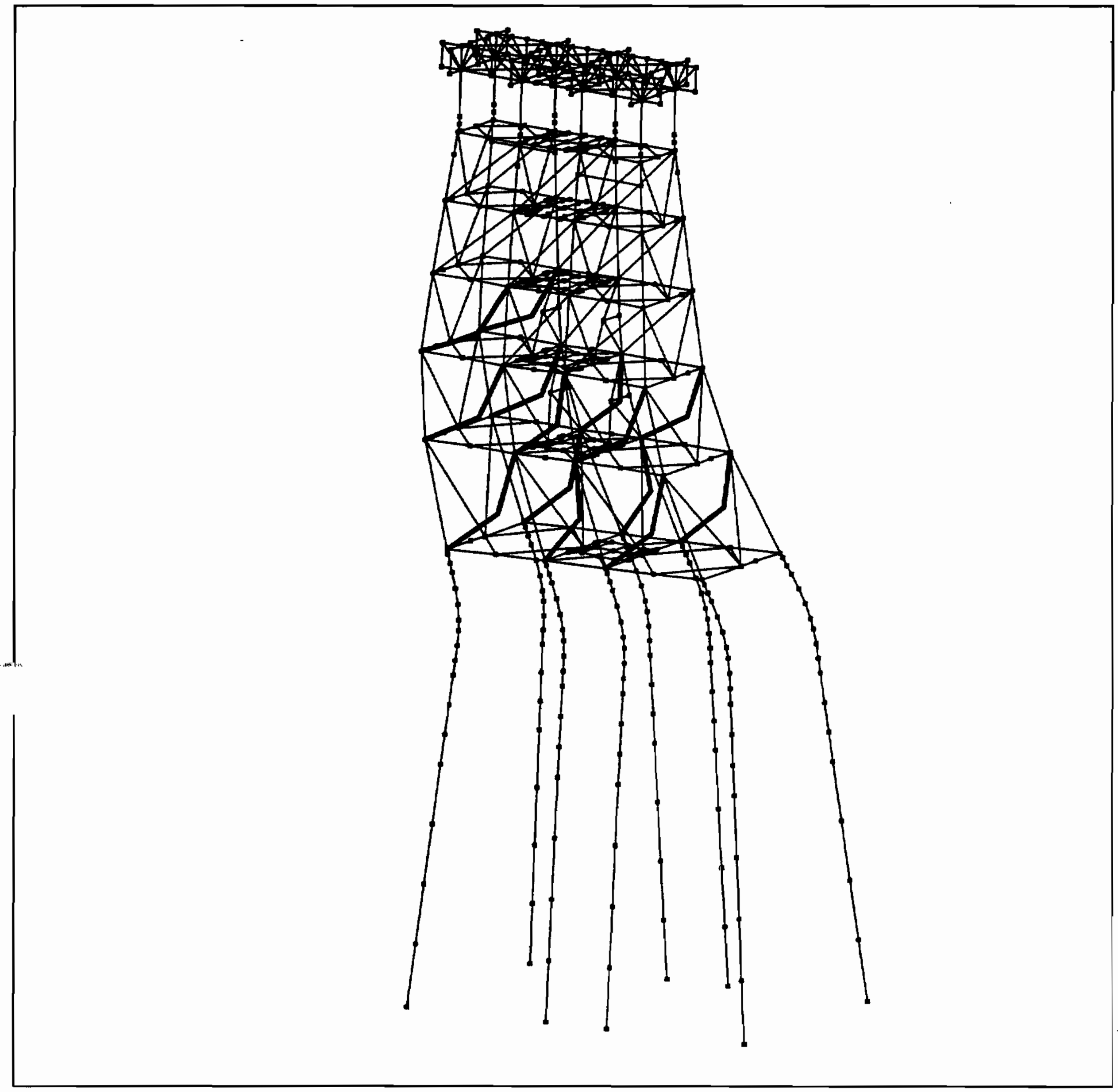
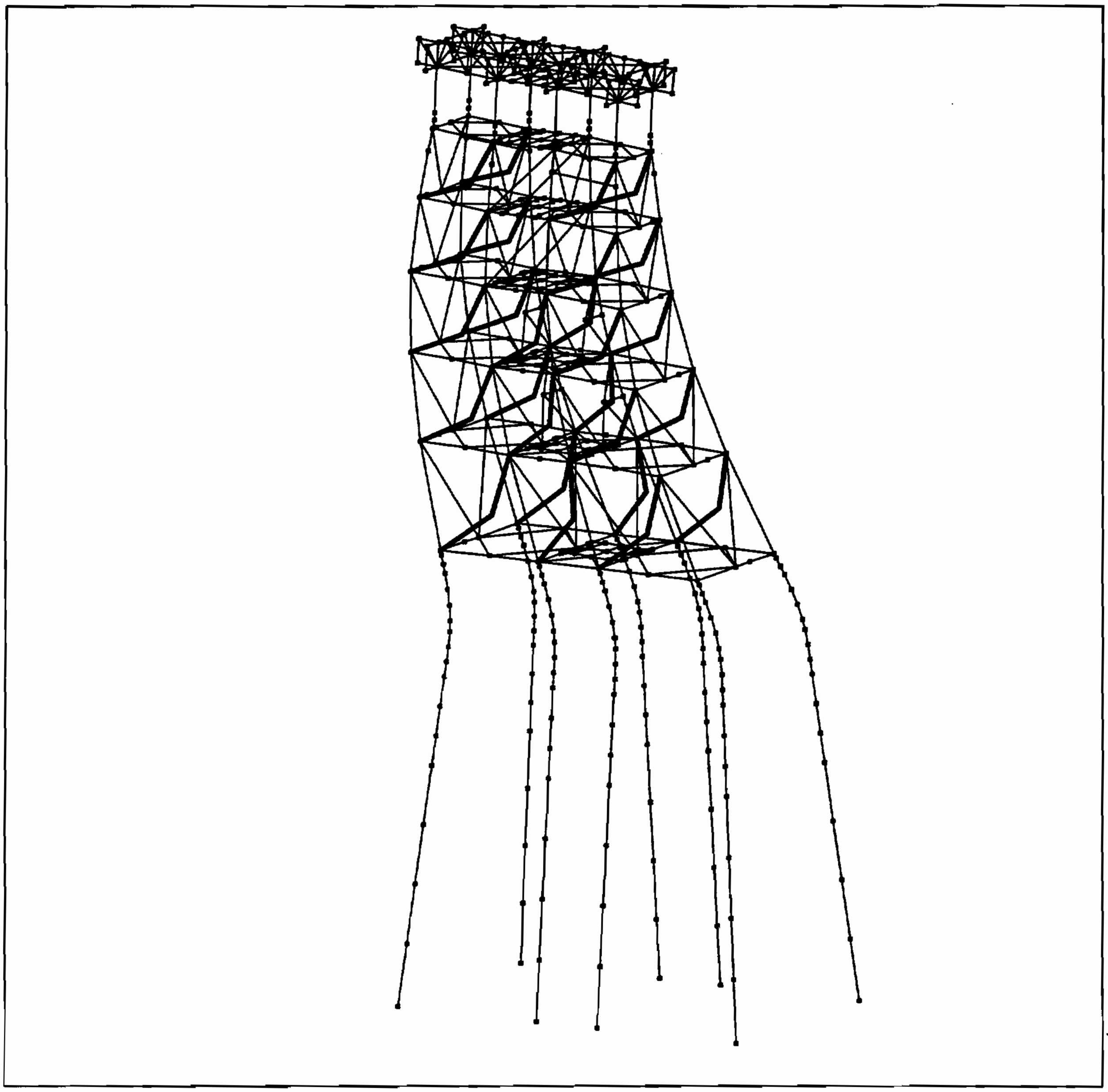


Fig. 3.14(c): Inelastic Events at Stage per Fig. 3.13 - End On Direction



CAP Pushover Analysis - Endon Loading LS 193

Inelastic Events Legend

————	Elastic	━━━━	Strut Buckling
---------	Strut Residual	————	Strut Reloading
....	Plastic Strut/NLTruss	---------	Beam Clmn Initial Yield
━━━━	Beam Clmn Fully Plastic	....	Fracture

Fig. 3.14(d): Inelastic Events at Stage per Fig. 3.13 - End On Direction

Table 3.1: Basic Metocean and Wave Force Computation Parameters

Item	Units	Section 17 Criteria (Full Population)		RP2A, 20th Edition
		Design Level Analysis	Ultimate Capacity Analysis	
Metocean Criteria				
Wind Speed, 1 hr. @ 10 m	knots	65.00	85.00	80.00
Storm Tide	ft.	2.50	2.50	3.20
Wave Height	ft.	57.00	72.00	67.50
Wave Direction	degrees	Omni-directional	Directional per Section 2	Directional per Section 2
Wave Period	sec	12.10	13.50	13.00
Current Magnitude at Surface	knots	1.60	2.30	2.10
Current Direction	degrees	Along wave	Interpolated for depth	Interpolated for depth
Marine Growth	inch	3" on dia.	3" on dia.	3" on dia.
Deck Height	ft.	-	44.30	48.70
Wave Force Procedure				
Wave Kinematic Factor	-	0.88	0.88	0.88
Current Blockage Factor (#1)	-	0.7 - 0.85	0.7 - 0.85	0.7 - 0.85
Wave Force Coefficients				
Drag Coefficient, Cd	-	0.65 (smooth tubulars)	0.65 (smooth tubulars)	0.65 (smooth tubulars)
	-	1.05 (rough tubulars)	1.05 (rough tubulars)	1.05 (rough tubulars)
Inertia Coefficient, Cm	-	1.6 (smooth tubulars)	1.6 (smooth tubulars)	1.6 (smooth tubulars)
	-	1.2 (rough tubulars)	1.2 (rough tubulars)	1.2 (rough tubulars)
Order of Stream Function Theory	-	9th	9th	9th

Notes:
#1: Values used depended on approach direction

Table 3.2: Metocean Parameters - Section 17 Design Level Criteria

Item	Units	Wave Direction		
		250 Degree (Broadside)	295 Degree (Diagonal)	340 Degree (End On)
Metocean Criteria				
Wind Speed	knots	65.00	65.00	65.00
Storm Tide	ft.	2.50	2.50	2.50
Wave Height	ft.	57.00	57.00	57.00
Wave Period (Apparant)	sec	12.80	12.80	12.80
Current Magnitude at MWL	ft/sec	3.54	3.54	3.54
Marine Growth	inch	3 inch on dia.	3 inch on dia.	3 inch on dia.

Table 3.3: Summary of Results - Section 17 Design Level Analysis

Item	Units	Wave Direction		
		250 Degree (Broadside)	295 Degree (Diagonal)	340 Degree (End On)
Base Forces				
Shear Force	kips	3,622	3,494	3,166
Overturning Moment (OTM)	kips-ft.	7.63E+05	6.98E+05	6.08E+05
Vertical Load	kips	-10,430	-10,370	-10,300
Displacements				
at Deck Level (+) 65.875'	ft.	0.85	0.80	0.70
at Seabed, (-) 263.0'	ft.	0.32	0.33	0.33
Code Check (Primary Members):				
Maximum I. R.	-	1.19	1.33	2.31
Member Category with Max. I.R.	-	K-braces	Diagonal braces	Diagonal braces
Number of members with:				
0.85 < I.R. < 1.0	-	8	10	5
I.R. > 1.0	-	7	8	22
Maximum Pile Forces				
Maximum Bending Moment	kips-ft.	3,200	3,366	3,550
at Elevation below seabed	-	50 ft. below	50 ft. below	50 ft. below
Maximum I.R.	-	0.79	0.88	0.78
Maximum Axial Force	kips	3,578	4,058	3,206
Pile F.O.S.	-	2.29	2.02	2.56

Table 3.4: Metocean Parameters and Base Shear- Section 17 Ultimate Capacity Criteria

Item	Units	Wave Direction		
		267.5 Degree (Broadside)	295 Degree (Diagonal)	340 Degree (End On)
Metocean Criteria				
Wind Speed	knots	85.00	85.00	85.00
Storm Tide	ft.	2.50	2.50	2.50
Wave Height	ft.	72.00	72.00	68.40
Wave Period (Apparant)	sec	14.30	14.31	14.18
Current Magnitude at MWL	ft/sec	3.84	3.87	3.47
Marine Growth	inch	3 inch on dia.	3 inch on dia.	3 inch on dia.
Wave Load Analysis Results				
Wind Loads	Kips	217	181	122
Wave and Current Loads	Kips	6,262/ 6,018 *	6,110	4,842
Base Shear	Kips	**6,479/ 6,235 ***	**6,291**	**4,964**

* with conductor shielding factor included

Table 3.5: Metocean Parameters and Base Shear - 20th Edition (Section 2) Criteria

Item	Units	Wave Direction		
		267.5 Degree (Broadside)	295 Degree (Diagonal)	340 Degree (End On)
Metocean Criteria				
Wind Speed	knots	80.00	80.00	80.00
Storm Tide	ft.	3.20	3.20	3.20
Wave Height	ft.	67.50	67.50	64.13
Wave Period (Apparant)	sec	13.77	13.78	13.65
Current Magnitude at MWL	ft/sec	3.50	3.53	3.17
Marine Growth	inch	3 inch on dia.	3 inch on dia.	3 inch on dia.
Wave Load Analysis Results				
Wind Loads	Kips	193	163	111
Wave and Current Loads	Kips	5,268/ 5,063 *	5,069	4,043
Base Shear	Kips	**5,461/ 5,256 ***	**5,232**	4,154

* with conductor shielding factor included

Pushover Analysis - Broadside Direction:

Lateral load level for first member with unity check = 1.0 3,000 Kips

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
16	1.580	6,298	Row3-328	K-brace buckling	Bottom bay
31	1.512	5,914	Row4-330	K-brace buckling	Bottom bay
53	1.286	4,986	Row2-326	K-brace buckling	Bottom bay
82	1.308	4,872	Row1-324	K-brace buckling	Bottom bay
107	1.891	5,222	Row1-312	K-brace buckling	2nd bay from bottom
122	1.966	5,192	Pile Sections	First yielding	Row2 piles
123	2.017	5,275	Pile sections	First yielding	Row 3 and 4 piles
124	2.105	5,419	Pile and leg sections	First yielding	
127	2.122	5,447	Row4-321	K-brace buckling	2nd bay from bottom
145	2.252	5,656	Leg sections	Full yielding	
153	2.457	5,931			
154	2.459	5,933	Pile sections	Full yielding	
187	2.613	6,105	Row3-317	K-brace buckling	

Table 3.6: Ultimate Strength Analysis - Broadside Direction

Pushover Analysis - Diagonal Direction:

Lateral load level for first member with unity check = 1.0 2,600 Kips

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
7	1.629	6,108	Pile-Row1	First yield of pile section	Section below sand layer
			RowB-361	Diagonal buckling	First strut failure
9	1.685	6,265	RowA-345	Diagonal buckling	2nd strut- bottom bay
12	1.732	6,384	Pile-Row2	First yield of pile section	Section below sand layer
14	1.793	6,539	Pile-Row3	First yield of pile section	Section below sand layer
18	1.812	6,587	RowA-346	Diagonal buckling	**Governs Ultimate Capacity**
22	1.811	6,542	RowA-347	Diagonal buckling	
37	1.578	5,393	RowB-362	Diagonal buckling	
40	1.584	5,410	RowB-363	Diagonal buckling	
69	2.226	5,615	RowB-357	Diagonal buckling	
76	2.267	5,668	RowB-360	Diagonal buckling	8-diagonals buckled
other steps			other failures		
127	2.494	4,901	RowB-356	Diagonal buckling	14-diagonals buckled
other steps			other failures		
289	5.346	3,709	RowB-349	Diagonal buckling	All diagonals buckled

Table 3.7: Ultimate Strength Analysis - Diagonal Direction

Pushover Analysis - Endon Direction:

Lateral load level for first member with unity check = 1.0 1,950 Kips

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures	Component Failure Mode	Remarks
9	1.107	4,302	RowA-345	Diagonal buckling	Bottom bay
10	1.112	4,318	RowB-361	Diagonal buckling	Bottom bay
18	1.256	4,660	RowB-362	Diagonal buckling	**Governs ultimate capacity**
19	1.257	4,657	RowA-346	Diagonal buckling	Bottom bay
21	1.256	4,602	RowA-347, RowB-363	Diagonal buckling	All bottom bay diag. buckled
other steps			other braces buckle		
58	2.048	3,982	RowB-360	Diagonal buckling	10-diagonals buckled
other steps			other braces buckle		
102	2.465	3,410	RowB-354	Diagonal buckling	14-diagonals buckled
other steps			other braces buckle		
132	2.546	3,243	RowA-339	Diagonal buckling	
146	2.648	2,963	RowB-353	Diagonal buckling	
193	3.024	2,858	RowB-348	Diagonal buckling	26-diagonals buckled
other steps			other failures		
294	9.729	2,443	Deck Leg-596	First yield of section	All diagonals buckled

Table 3.8: Ultimate Strength Analysis -End On Direction

PLATFORM "Q"

SECTION 1.0 -- PLATFORM INFORMATION

Platform Q is an operating 12-pile drilling, production, and quarters platform located in 189 feet of water offshore Santa Barbara, California, and was installed in 1968. The first section of this document contains as-is condition details pertaining to Platform Q. A 3-dimensional view of the platform computer model is shown in Figure 1.

As-Is Condition Details

The following is a compilation of information based on the list in the draft API RP 2A Section 17 commentary, Section C17.4:

1. **General Information**
   a. Original and Current Owner
   (confidential).
   b. Original and Current Platform Use and Function
   Drilling, Production, and Quarters Platform.
   c. Location, Water Depth, and Orientation
   Offshore Santa Barbara, California; 189 feet;
   Rows 1-4 are essentially oriented North -South – refer to sketches in Appendix.
   d. Platform Type
   Twelve-leg template structure.
   e. Number of Wells, Risers, and Production Rate
   59 wells; 3 risers; *(confidential).*
   f. Manning Level
   (confidential).
   g. Performance During Past Environmental Events
   No damage observed or reported.

2. **Original Design**
   a. Design Contractor and Date of Design
   Brown & Root, Inc., 1968.
   b. Design Drawings and Material Specifications
   Sketches (see Appendix) based on design drawings.
   Primary material is A36 steel.
   c. Design Code
   Unknown.
   d. Environmental Criteria
   Unknown.
   e. Deck Clearance Elevation
   Bottom of Production Deck Steel approximately (+)40'.
   f. Operational Criteria – Deck Loading and Equipment Arrangement
   Unknown.

2. Original Design (continued)

g. Soil Data

Dames & Moore reports for platform site.

h. Number, Size, and Design Penetration of Piles and Conductors

Piles = 12, 40-in. diam., 85-foot design penetration.

Conductors = 56, 20-in. diam., 35 to 60-foot design penetration.

i. Appurtenances

2 boat landings	5 barge fenders
3 risers	10 caissons

3. Construction

a. Fabrication & Installation Contractors, Date of Installation

Unknown; September 1968.

b. "As-Built" Drawings

Not available.

c. Fabrication, Welding, and Construction Specifications

Not available.

d. Material Traceability Records

Not available.

e. Pile and Conductor Driving Records

Not available.

f. Pile Grouting Records

Not available.

4. Platform History

a. Environmental Loading History

Not available.

b. Operational Loading History -- Collisions and Accidental Loads

None.

c. Survey and Maintenance Records

Not available.

d. Repairs

Not available.

e. Modifications

Not available.

5. Present Condition

a. All Decks -- Actual Size, Location, and Elevation

See sketches located in the Appendix.

b. All Decks -- Existing Loading and Equipment Arrangement

Refer to Figures 2 through 5. The total topsides weight (including structural steel) is 5,200 kips.

c. Field Measured Deck Clearance Elevation (Bottom of Steel)

Assumed to be (+)40' to bottom of production deck.

5. Present Condition (continued)

d. Production and Storage Inventory
Not available.

e. Appurtenances – Current List, Sizes, and Location
See "original design" information, above.

f. Wells – Number, Size, and Location of Existing Conductors
Eighteen at 24-inch diameter (slanted).
Thirty-one at 20-inch diameter (vertical).

g. Recent Topsides Survey (Level I) with Platform Condition Report
Not available.

h. Recent Underwater Platform Survey (Level II Minimum)
Not available.

i. Updated Soil Data
Staal, Gardner & Dunne report, September 1991.
See Appendix for foundation details.

PART A: PLATFORM ASSESSMENT

A.1 Platform Selection (Section 17.2)

Per Section 17.2, an existing platform should undergo the assessment process if one or more of the following conditions exists:

- Addition of Personnel
- Addition of Facilities
- Increased Loading on Structure
- Inadequate Deck Height
- Damage Found During Inspections

For the sake of this exercise, the selection of Platform Q was based on "increased loading on structure" due to new seismic hazard information collected in the region.

A.2 Condition Assessment (Section 17.4)

The most recent Level I (topsides) and Level II (subsea) inspections showed no damage to the deck or jacket structure. A marine growth profile (shown in Figure 6) was generated based on measurements taken during the last subsea inspection. A marine growth management program is being implemented, and is intended to produce a maximum reduced profile, also shown in Figure 6.

An updated (1991) foundation analysis was performed by Staal, Gardner & Dunne to determine best estimate pile capacities for Platform Q. These updated capacities vary from 2,050 kips to 2,080 kips; foundation details appear in the Appendix.

A.3 Categorization (Section 17.3)

Platform Q is continuously manned. Because this structure is located in a seismically active region, it is considered unevacuated during an environmental design event. Therefore, Platform Q is categorized as *Manned, Non-Evacuated.*

Platform Q is also an oil producing platform, located in a region that will be assumed to be environmentally sensitive for the purposes of this exercise. Therefore, Platform Q is categorized as having *Significant Environmental Impact.*

A.4 Design Basis Checks (Sections 17.5 and 17.6)

Design basis checks for metocean conditions are only applicable for Gulf of Mexico platforms; therefore, this section is not relevant to Platform Q. As new seismic criteria has been determined for the site, it is assumed that Platform Q will not satisfy the design basis check for this loading condition.

A.5 Analysis Checks (Sections 17.6 and 17.7)

Platform Q will be checked for both storm waves and seismic events. Ice loading is not applicable to this region.

A.5.1 Metocean and seismic criteria / loads

Meteocean Criteria: For storm waves, refer to Table 17.6.2-2 in the draft of Section 17, API RP 2A. Platform Q has a longitude of 119.6 degrees, placing it in the bottom category for the Santa Barbara Channel. The pertinent storm wave data is as follows:

Wave Height	34 feet
Current	1 knot
Wave Period	12 seconds
Storm Tide	6 feet
Wind Speed	45 knots (1-hour speed @ (+)33 feet)

A directional spreading factor of 1.0 is recommended, as extreme storm waves offshore Southern California are dominated by extratropical storms.

Seismic Criteria: The site seismicity used for this analysis is documented in Fugro-McClelland's report of April 1992. A plot of the linear response spectra with 5 percent damping is shown in Figure 7 for return periods of 200 and 1,000 years, and is specific to the platform site.

A.5.2 Screening

Platform Q does not pass the screening requirements. Design level and ultimate strength analyses are required.

A.5.3 Design level analysis

Storm wave analysis

The total maximum wind, wave, and current load on the platform was measured to be approximately 2,300 kips for the 100-year return period storm wave criteria for all directions analyzed. Under this loading, the vertical diagonals in Rows 1 through 4 between El(-)24' and El(-)63' had interaction ratios of approximately 0.85, for the wave in the North-South direction. The minimum pile factor of safety was calculated to be 1.47 in the North-South direction, 1.42 in the East-West direction, and 1.33 in the diagonal direction, as compared to the target of 1.50. A summary of all pile loads and corresponding factors of safety are shown in Figure 8. No other members or joints were found to have interaction ratios greater than 0.85.

Seismic analysis

From 17.6.3, #2: "For seismic assessment purposes, the design level check is felt to be an operator's economic risk decision and thus is not applicable. An Ultimate Strength Analysis is required if the platform does not pass the design basis check or screening."

The 200-year response spectrum was applied to Platform Q for purposes of determining a reference-level load only. The results of that analysis were 4,100 kips in the North-South direction, and 4,300 kips in the East-West direction. While no code checks were performed on the jacket structure, pile loads were reviewed against ultimate capacity. Eight of twelve piles were calculated to have factors of safety less than 1.25, with two of those eight having factors of safety less than 1.00. A summary of all pile loads and corresponding factors of safety are shown in Figure 8.

Platform structural periods were also calculated, and are as follows:

Sway (N-S)	1.64 seconds
Sway (E-W)	1.72 seconds
Bending (N-S)	0.69 seconds
Bending (E-W)	0.75 seconds
Torsion	1.16 seconds

A.5.4 Ultimate strength analysis

Two ultimate strength analyses were performed on Platform Q -- a pushover analysis to simulate ultimate wave loading, and a seismic time history analysis to simulate the 1,000-year earthquake event. The A36 steel was assumed to have a yield strength of 43 ksi, a 20% increase; a 10% increase was taken to better estimate the actual yield stress of the material, and an additional 10% increase was taken to account for strain rate effects under dynamic loading conditions. The results of the ultimate strength analyses are as follows:

<u>Pushover</u>
Two pushover analyses were performed on Platform Q -- the first in the North-South direction, and the second in the East-West direction. The results of these two analyses are displayed graphically in Figures 9 and 10, respectively.

For the North-South direction, the first components to reach a utilization ratio of unity were the piles located on Row C; the axial capacities of these piles became fully utilized at a lateral load level of 3,750 kips. The platform reached its ultimate capacity at 5,600 kips, when all piles exceeded their calculated axial capacities. Based on the reference-level load of 2,300 kips, the reserve strength ratio (RSR) was calculated to be 2.43.

For the East-West direction, the first components to reach a utilization ratio of unity were the piles located on Row 4; the axial capacities of these piles became fully utilized at a lateral load level of 3,400 kips. The platform reached its ultimate capacity at 5,700 kips, when all piles exceeded their calculated axial capacities. Based on the reference-level load of 2,300 kips, the reserve strength ratio (RSR) was calculated to be 2.48.

<u>Seismic Time History</u>

The seismic time history record used for this analysis was taken from the 1971 San Fernando Earthquake, as measured at the Lake Hughes #4 Station. The three orthogonal components of this record were frequency modulated by the seismologist in an attempt to match the horizontal and vertical 1,000-year seismic spectra. Plots of these time history acceleration records are shown in the Appendix.

Global response quantities for the 1,000-year event are shown in Figure 11. Maximum base shear was measured to be 3,600 kips in the North-South direction, 5,300 kips in the East-West direction, with a combined maximum base shear of 5,600 kips. Deck displacement time histories showed that the platform suffered lateral plastic deformation of approximately 4 inches in the North-South direction and 12 inches in the East-West direction. Vertical plastic set was measured to be approximately 4 inches. Several vertical diagonal and horizontal braces in every frame buckled or yielded. However, no legs or piles exhibited any hinging throughout the analysis, and no nonlinear events in the deck structure were recorded.

The maximum axial loads experienced by the piles are also shown in Figure 11. Eleven of the twelve piles experienced a loading that exceeded the pile's calculated static axial capacity at some point during the analysis, causing soil degradation in the range of 15 to 35 percent. This foundation response is driven by the vertical mode of the platform, which was excited by the relatively high vertical pseudo-accelerations associated with the spectrum near the platform's vertical period.

The free vibration response of the structure was measured after the application of the seismic time history, and showed a 20% increase in the primary sway periods of the structure. This softening of the structure is most likely attributed to the response of the foundation to the seismic loading. With this shift in period, less load is transmitted into the structure, as the spectral content of this particular input motion decreases with increasing sway period in the range of interest for this platform. At the conclusion of the seismic analysis, no collapse mechanism was shown to have been formed; therefore, the platform has been judged to survive the 1,000-year seismic event.

A.6 Consideration of Mitigations (Section 17.8)

The primary mitigation considered for Platform Q was the marine growth management program discussed in Section A.2. Besides marine growth management, removal of unnecessary conductors, risers, caissons, and other appurtenances are viable mitigation options. Not only does this reduce platform total wave load, it can also reduce platform structural and hydrodynamic added mass, which aids in reducing seismic loading on the platform.

Other mitigations that can be considered for platforms in seismic regions primarily concern the topsides. Removal of large, unnecessary pieces of equipment help to reduce the total topsides weight, and thus the total seismic load.

A.7 Summary Note

Platform Q was subjected to two assessment procedures -- one for storm waves, and one for earthquakes.

Storm Waves

Under storm wave analysis, the platform would not pass the assessment for design-level analysis because seven of twelve piles did not have adequate factors of safety. However, Table 17.5.2b states that 85% of the lateral loading caused by the 100-year environmental conditions can be used for this assessment process. In that case, the platform would pass the storm wave condition at the design-level analysis stage.

The ultimate strength analysis performed on Platform Q yielded reserve strength ratios of 2.43 and 2.48, far greater than the recommended RSR of 1.6 to satisfy the assessment requirements.

Earthquakes

As this platform did not pass the design basis check, an ultimate strength analysis was required. The method chosen was global inelastic analysis through the application of a seismic time history acceleration record to the platform. The results of this assessment demonstrated the structure withstood collapse while experiencing local overstress. Therefore, the platform passed the assessment at the ultimate strength analysis level.

While response spectrum analysis is not required for this assessment process, reserve strength ratios are calculated here for interest only. For the North-South direction, the RSR would be 5,600/4,100 = 1.37, and for the East-West direction, the RSR would be 5,700/4,300 = 1.33. The ultimate strength values were based on the storm wave pushover profile, and may not exactly represent the actual collapse load for the platform under seismic loading. Note that the maximum base shear under seismic loading of approximately 5,600 kips, while just at the pushover load previously calculated, did not cause the formation of a collapse mechanism within the structure. This is most likely due to the dynamic nature of the seismic loading.

PART B: -- REVIEW AND FEEDBACK TO THE API TG 92-5

Section 17.2.5 -- The word "justified" is better replaced with the following language for the last two sentences:

> *Minor structural damage may be judged acceptable by appropriate structural analysis without performing a detailed assessment. However, the cumulative effects of damage must be documented and, if not determined to be insignificant, be accounted for in the detailed assessment.*

Section 17.3.1 -- Life Safety. Are bridge-connected structures considered "manned"? Could we add some kind of definition to this section regarding bridge-connected structures, or does an adequate definition exist somewhere else in RP 2A?

Section 17.5.1, #2. Assessment through the use of explicit probabilities of failure. Are there any target criteria to satisfy this assessment? Is there a defined scope for all failure probabilities to include (i.e. hurricanes, ship impact, fire, explosions, helicopter crash, etc.)? The language in the commentary is vague.

17.6.2b, #2. Deck Height Check should be as prescribed in 17.6.2a.2, not 17.6.2a.5, which doesn't exist. Concerning lowering of the ultimate strength storm tide from that in Table 17.6.2-2, what can you lower it to? Why not just prescribe an adequate tide to use with the defined wave height?

	PREPARED BY	CHECKED BY	DATE
TRIAL PARTICIPANT "Q"			August 1, 1994
SUBJECT			A.F.E. NO.
PLATFORM "Q" – COMPUTER MODEL 3-D VIEW			645-1 1008

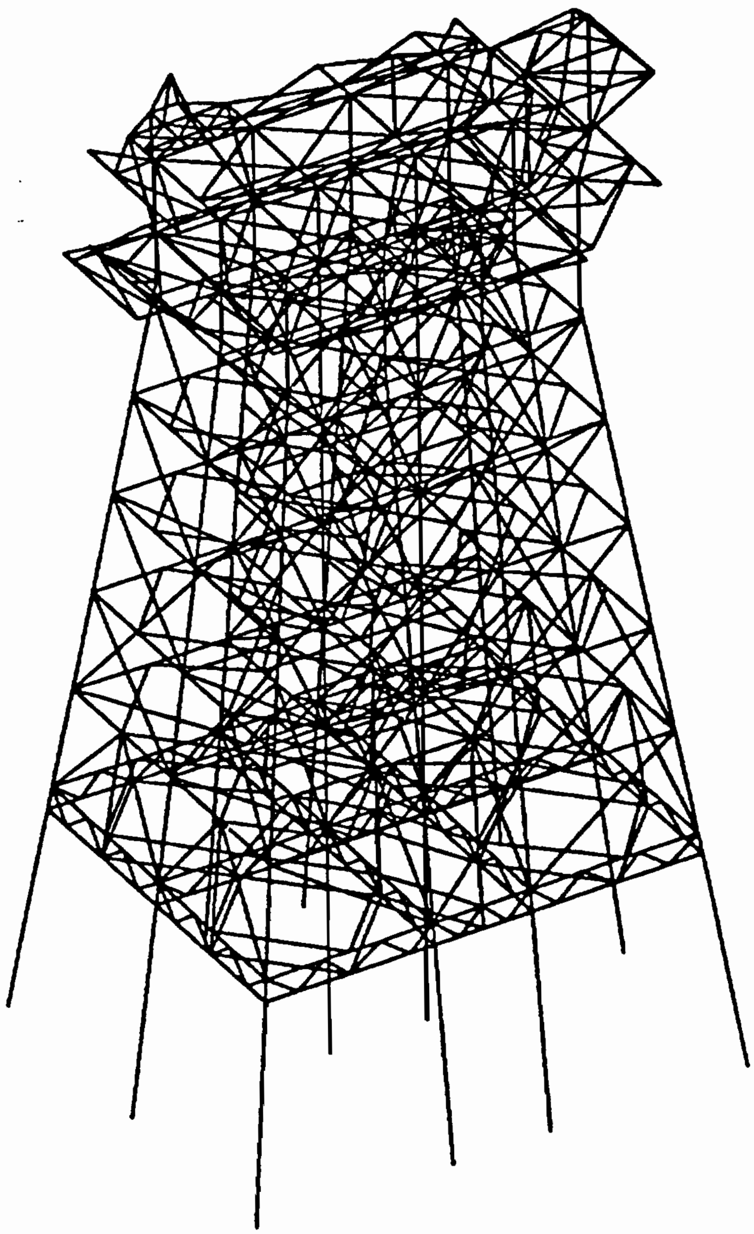
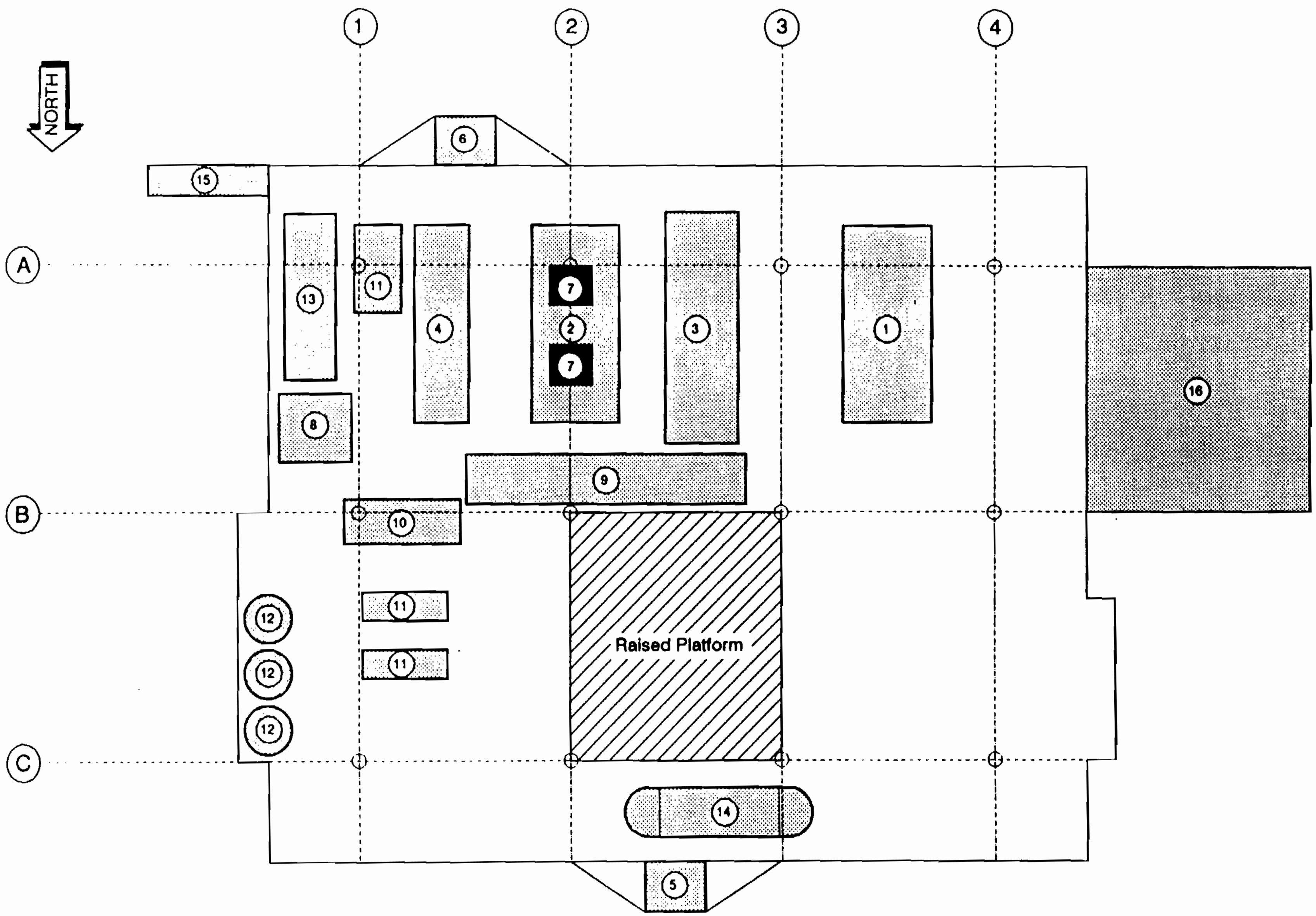


FIGURE 1

#	ITEM -- DRILLING DECK	EQUIPMENT LOAD (KIPS)	OPERATING LOAD (KIPS)	TOTAL LOAD (KIPS)
1	Drilling Rig + Setback	400	75	475
2	Pipe Rack	120	150	270
3	Main Mud System	50	200	250
4	S.C.R.	55	0	55
5	70 Ton Crane	50	0	50
6	50 Ton Crane	50	0	50
7	Mud Pumps	75	0	75
8	Fuel Tank	5	40	45
9	Reserve Mud Tank	25	100	125
10	Bay Rite	15	135	150
11	Generators	60	0	60
12	Cement Pods	15	125	140
13	Wemco Flotation Cell	20	15	35
14	Free Water Knockout	20	100	120
15	Flare Boom	30	0	30
16	Helipad & Crew Quarters	90	30	120
17	Drill Deck Structure Weight	850	0	850
18	Area Live Load (15,400 @ 30 psf)	0	462	462
	TOTALS	1,930	1,432	3,362

PLATFORM Q -- DRILL DECK LOADS

FIGURE 2



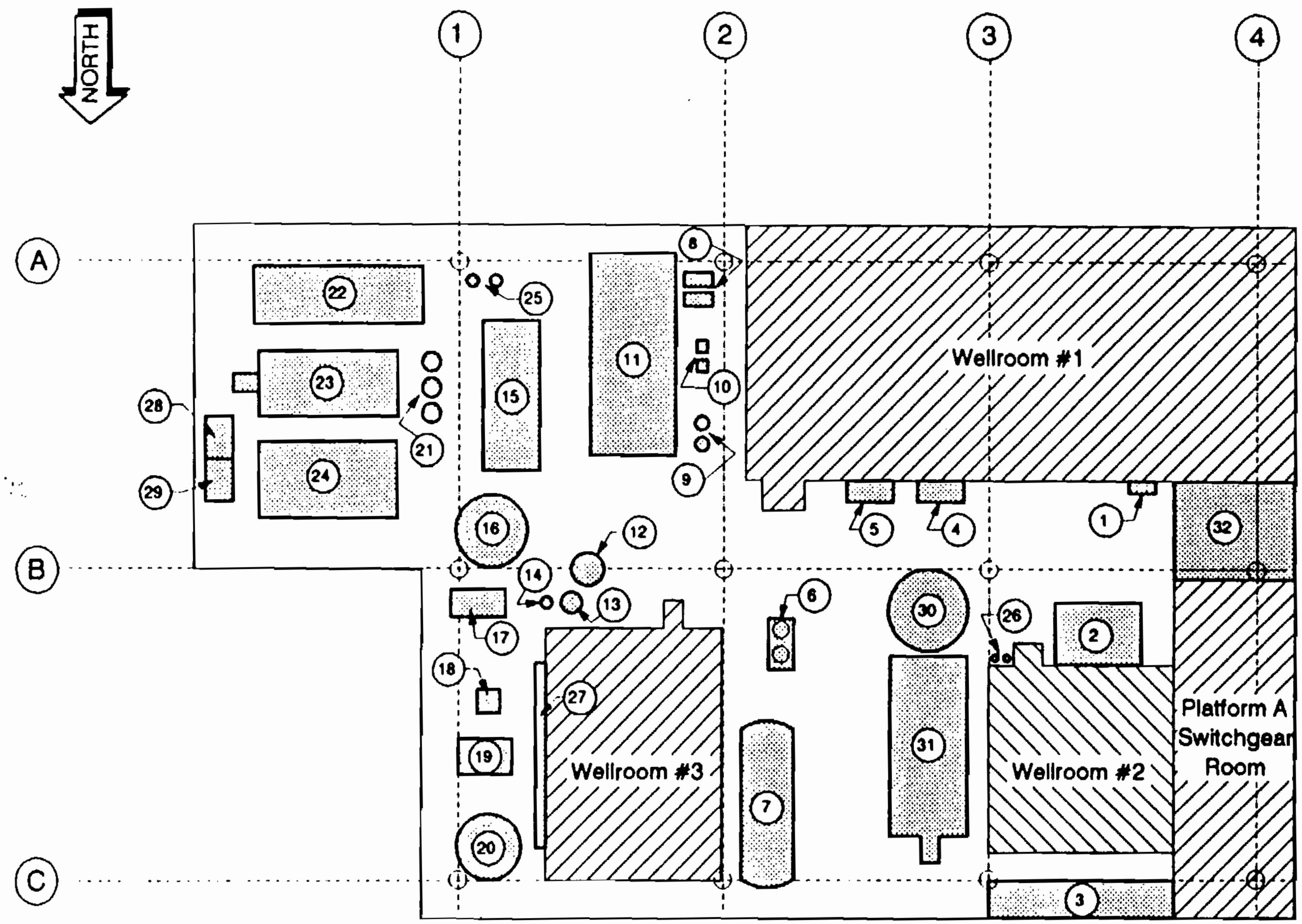
PLATFORM Q - DRILL DECK LAYOUT

FIGURE 3

#	ITEM -- PRODUCTION DECK	EQUIPMENT LOAD (KIPS)	OPERATING LOAD (KIPS)	TOTAL LOAD (KIPS)
1	Hydraulic Unit	1	0	1
2	Lab	1	0	1
3	Generator Switch Gears	3	0	3
4	Chemical Tanks	4	18	22
5	Chemical Pumps	1	0	1
6	Test Separators	9	26	35
7	Gross Separators	24	78	102
8	Charging Pumps	1	0	1
9	Injection Filter	2	0	2
10	Injection Pumps	4	0	4
11	Habitat Switchgear Room	0	0	0
12	Main Gas Scrubber	3	8	11
13	Vent Scrubber	3	17	20
14	Final Gas Scrubber	2	3	5
15	Cooper Compressor	50	0	50
16	Polymer Storage Tank	9	131	140
17	Air Compressor	1	0	1
18	Vapor Compressor	2	0	2
19	Union Shipping Pump	12	0	12
20	Shipping Surge Tank	13	110	123
21	Reda Shipping Pumps	37	0	37
22	Glygol Unit	20	0	20
23	AC Compressor	22	0	22
24	Polymer Injection System	12	0	12
25	Pig Launcher	2	0	2
26	Generator Fuel Filters	1	0	1
27	Chemical Tanks	7	8	15
28	Fin Fan	3	0	3
29	Scrubber	1	2	3
30	Well Clean Tank	19	40	59
31	Solar Turbine Generator	60	0	60
32	Control Room	35	0	35
33	Sub Deck	20	10	30
34	Stairs & Walkways	45	0	45
35	Production Deck Structure Weight	650	0	650
36	Area Live Load (10,350 @ 30 psf)	0	311	311
	TOTAL	1,079	762	1,841

PLATFORM Q -- PRODUCTION DECK LOADS

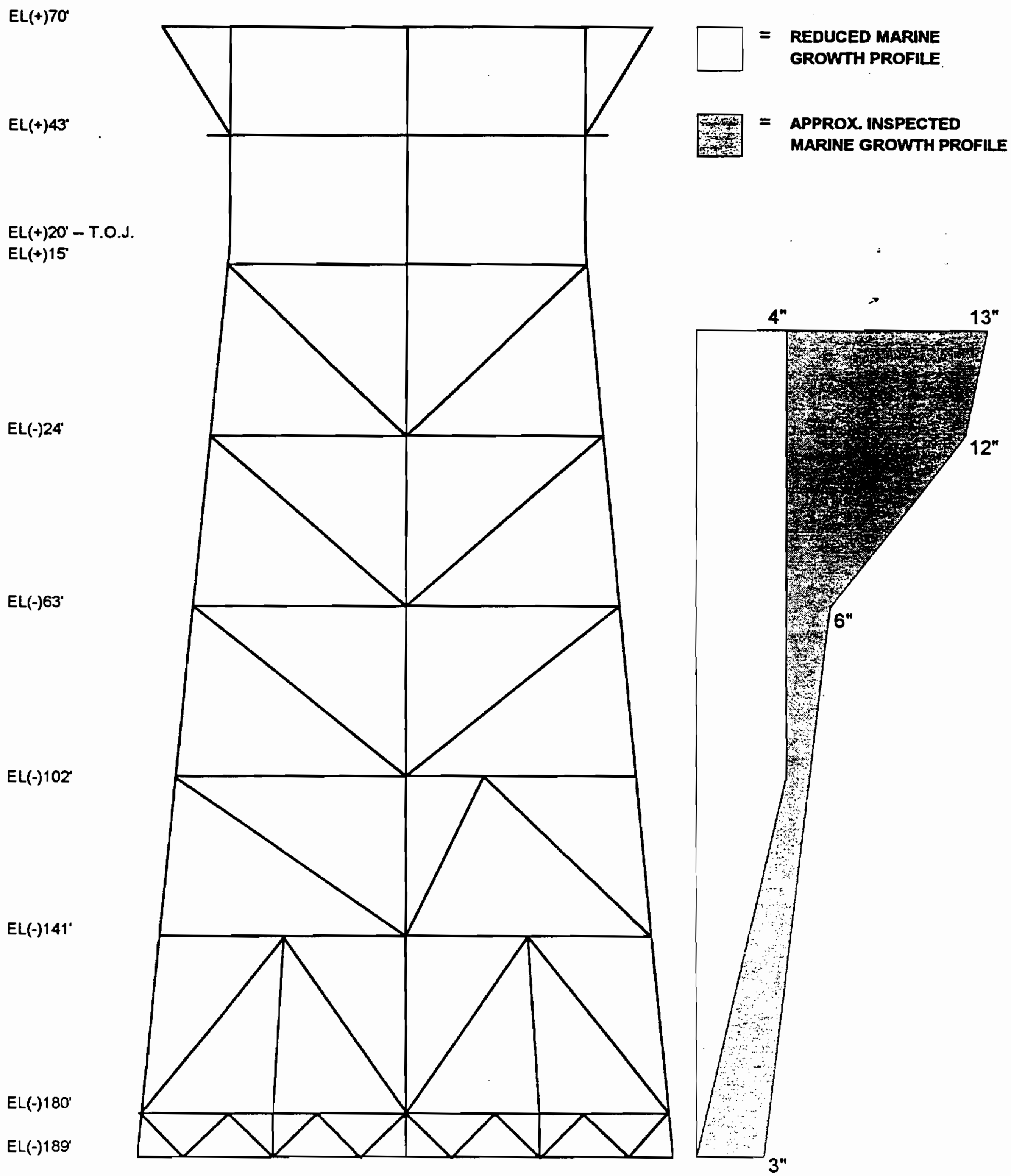
FIGURE 4



PLATFORM Q - PRODUCTION DECK LAYOUT

FIGURE 5

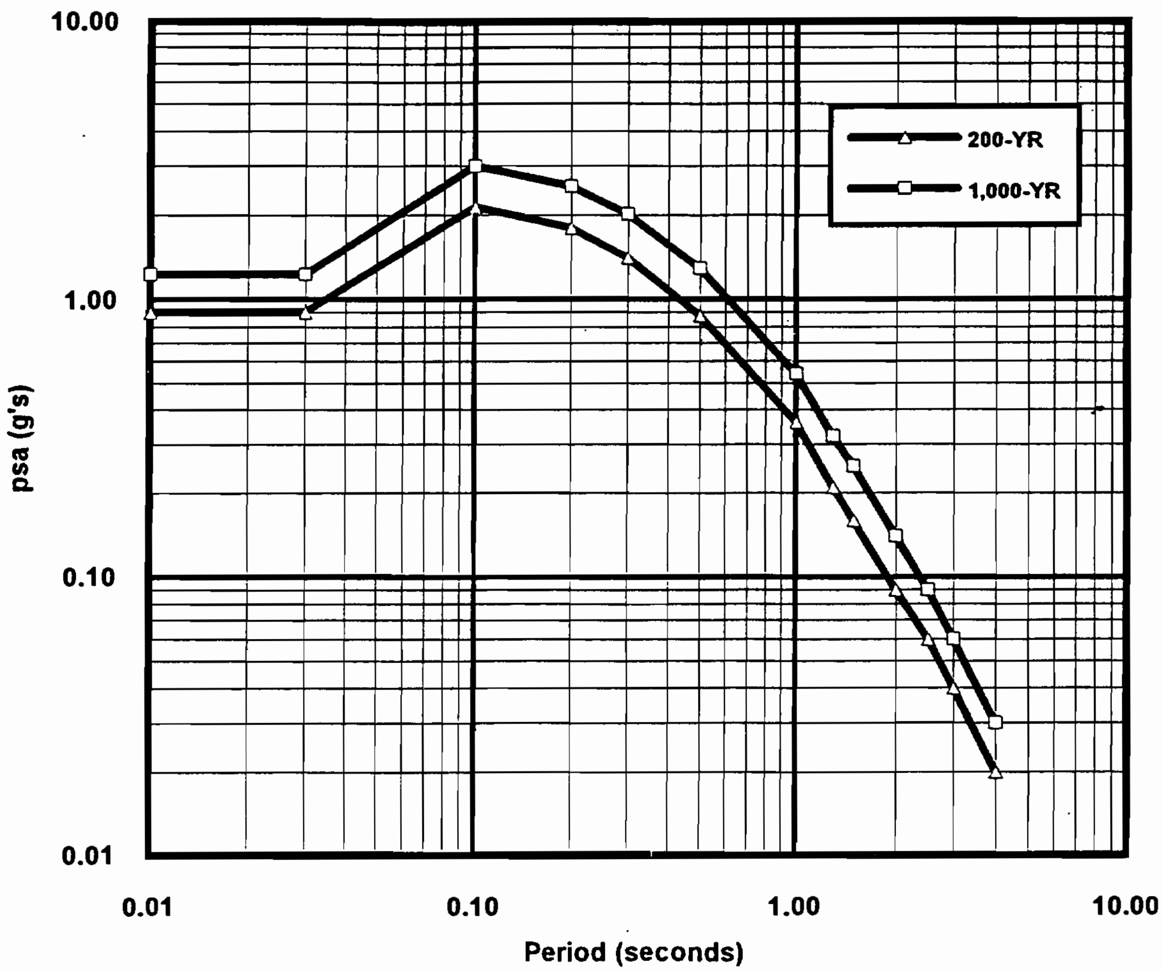
TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" – Marine Growth Profiles			A.F.E. NO. 645-1 1008



MARINE GROWTH PROFILE

FIGURE 6

PLATFORM "Q" -- Response Spectra



T	200-YR	1,000-YR
0.01	0.90	1.23
0.03	0.90	1.23
0.10	2.14	3.00
0.20	1.80	2.54
0.30	1.41	2.02
0.50	0.88	1.29
1.00	0.36	0.54
1.30	0.21	0.32
1.50	0.16	0.25
2.00	0.09	0.14
2.50	0.06	0.09
3.00	0.04	0.06
4.00	0.02	0.03

FIGURE 7

MAXIMUM PILE LOADS, PILE CAPACITIES, AND FACTORS OF SAFETY

Maximum Pile Loads

Maximum Pile Gravity Loads, kips

	1	2	3	4
A	740	850	770	680
B	830	980	880	770
C	760	920	800	680

Maximum Wave Loads (N-S), kips

	1	2	3	4
A	1,320	980	900	1,260
B	1,410	1,110	1,010	1,350
C	1,340	1,050	930	1,260

Maximum Wave Loads (E-W), kips

	1	2	3	4
A	1,280	1,390	1,310	1,220
B	830	980	880	770
C	1,300	1,460	1,340	1,220

Maximum Wave Loads (Diag.), kips

	1	2	3	4
A	1,540	1,290	1,210	1,480
B	1,310	1,100	1,000	1,250
C	1,560	1,360	1,240	1,480

Maximum Response Spectrum Loads, kips

	1	2	3	4
A	1,450	1,820	1,990	2,520
B	1,780	1,380	1,270	1,830
C	2,610	2,170	1,700	1,400

Pile Capacities

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

Pile Factors of Safety

	1	2	3	4
A	2.81	2.44	2.69	3.06
B	2.49	**2.09**	2.33	2.69
C	2.74	2.25	2.59	3.06

	1	2	3	4
A	1.58	2.11	2.30	1.65
B	**1.47**	1.85	2.03	1.53
C	1.55	1.97	2.23	1.65

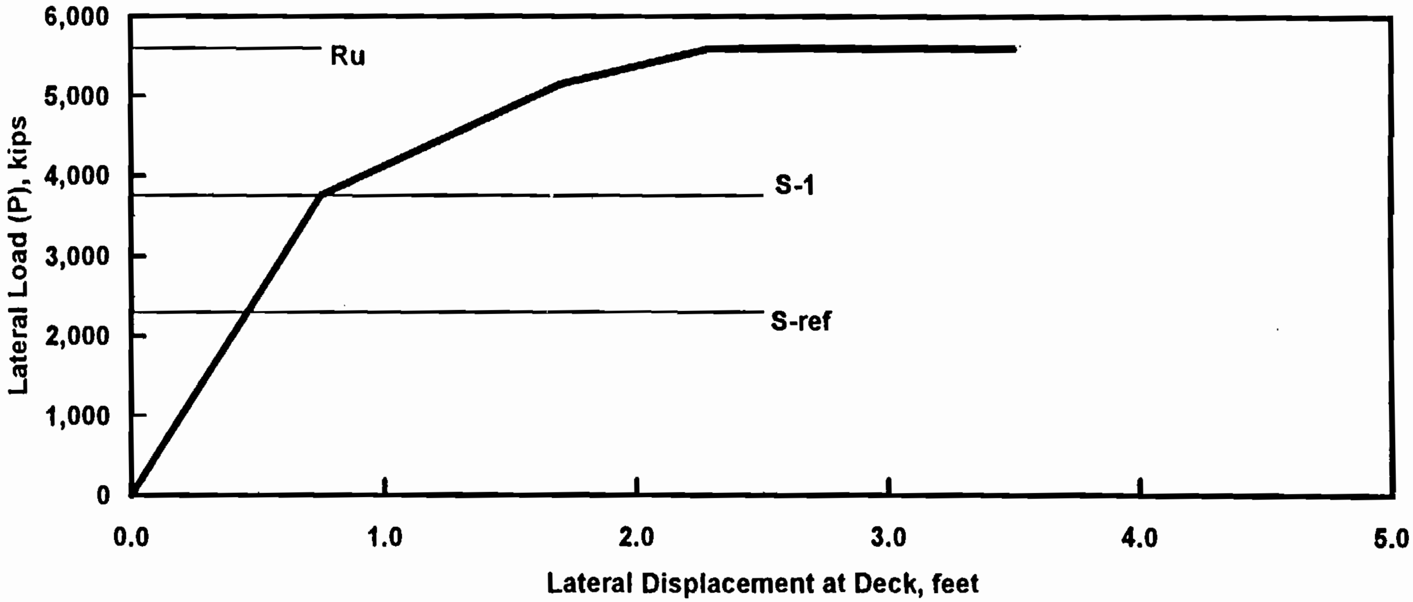
	1	2	3	4
A	1.63	**1.49**	1.58	1.70
B	2.49	2.09	2.33	2.69
C	1.60	**1.42**	1.54	1.70

	1	2	3	4
A	**1.35**	1.60	1.71	**1.41**
B	1.58	1.86	2.05	1.66
C	**1.33**	1.52	1.67	**1.41**

	1	2	3	4
A	1.43	**1.14**	**1.04**	**0.83**
B	**1.16**	1.49	1.61	**1.13**
C	**0.80**	**0.95**	**1.22**	1.49

FIGURE 8

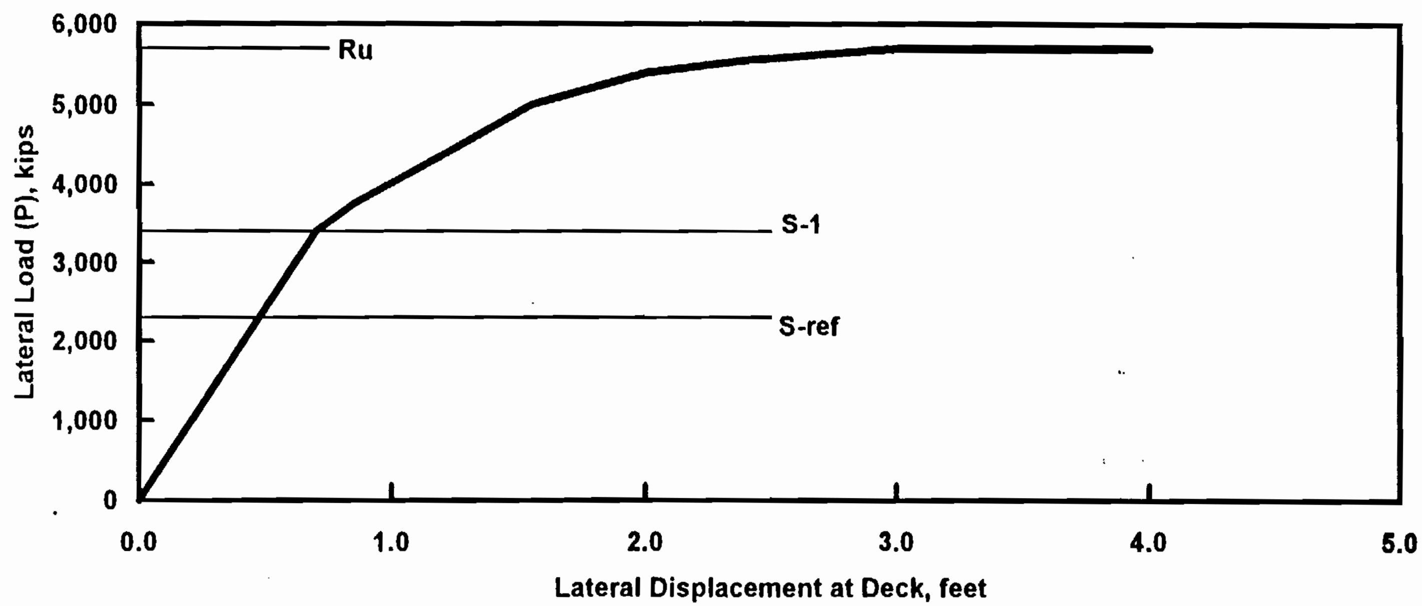
ULTIMATE STRENGTH ANALYSIS
PLATFORM "Q" -- NORTH-SOUTH DIRECTION



Load Level at which first component reaches utilization ratio of 1.0 (S-1)	=	3,750 kips
Reference Level Load (S-ref)	=	2,300 kips
Ultimate Capacity (Ru)	=	5,600 kips
Reserve Strength Ratio (RSR)	=	2.43
Platform Failure Mode	=	Soils

FIGURE 9

ULTIMATE STRENGTH ANALYSIS
PLATFORM "Q" -- EAST-WEST DIRECTION



Load Level at which first component reaches utilization ratio of 1.0 (S-1)	=	3,400 kips
Reference Level Load (S-ref)	=	2,300 kips
Ultimate Capacity (Ru)	=	5,700 kips
Reserve Strength Ratio (RSR)	=	2.48
Platform Failure Mode	=	Soils

FIGURE 10

SEISMIC TIME HISTORY ANALYSIS RESULTS SUMMARY

ITEM	VALUE
North-South Base Shear	3,600 kips
East-West Base Shear	5,300 kips
Combined Base Shear	5,600 kips
Vertical Reaction	25,800 kips
OTM -- East-West Axis	393,000 kip-feet
OTM -- North-South Axis	465,000 kip-feet
Torsional Reaction	36,000 kip-feet
N-S Deck Deflection	13.9 inches
E-W Deck Deflection	8.3 inches
Vert. Deck Deflection	-9.6 inches
N-S Mudline Deflection	1.1 inches
E-W Mudline Deflection	0.7 inches
Vert. Mudline Deflection	-10.2 inches

NON-LINEAR EVENTS

Row	Vertical Digonals	Main Horiz.
A	8 of 15	3 of 18
B	9 of 16	8 of 18
C	5 of 15	9 of 18
1	6 of 13	2 of 12
2	6 of 13	1 of 12
3	4 of 12	1 of 12
4	7 of 13	2 of 12

PILE LOAD RESULTS

Max. Seismic Load kips

	1	2	3	4
A	2,370	2,240	2,230	1,940
B	2,360	2,370	2,220	2,180
C	2,380	2,400	2,250	2,240

Pile Capacity kips

	1	2	3	4
A	2,080	2,070	2,070	2,080
B	2,070	2,050	2,050	2,070
C	2,080	2,070	2,070	2,080

Capacity / Applied Load

	1	2	3	4
A	0.88	0.92	0.93	1.07
B	0.88	0.86	0.92	0.95
C	0.87	0.86	0.92	0.93

FIGURE 11

APPENDIX

PLATFORM Q MODEL SKETCHES

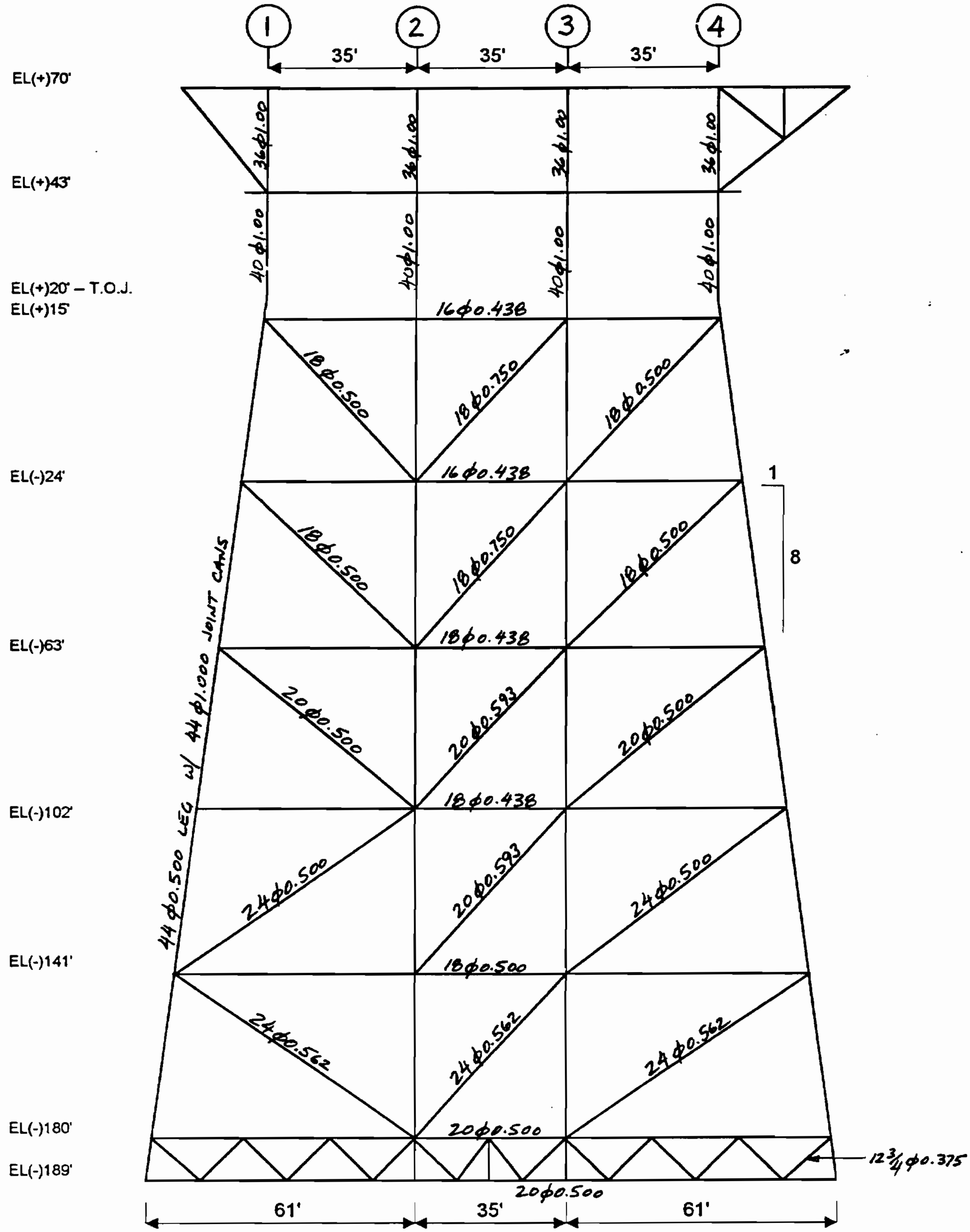
ROW A	EL(+)70'
ROW B	EL(+)43'
ROW C	EL(+)15'
ROW 1	EL(-)24'
ROW 2	EL(-)63'
ROW 3	EL(-)102'
ROW 4	EL(-)141'
	EL(-)180'
	EL(-)189'

PILE & FOUNDATION INFORMATION

P-Y CURVE DATA

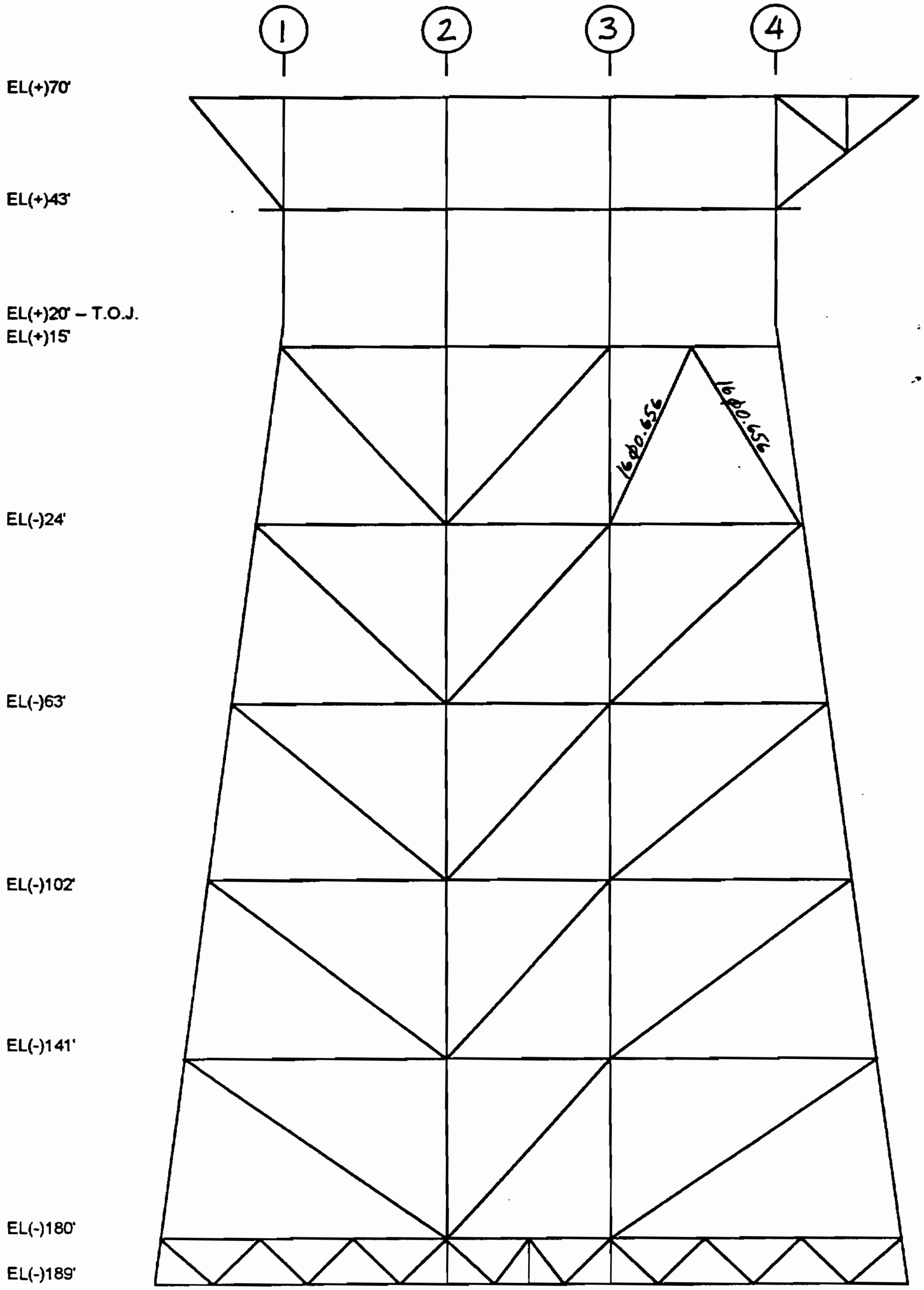
ACCELERATION TIME HISTORY INFORMATION

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" – Model Sketches			A.F.E. NO. 645-1 1008



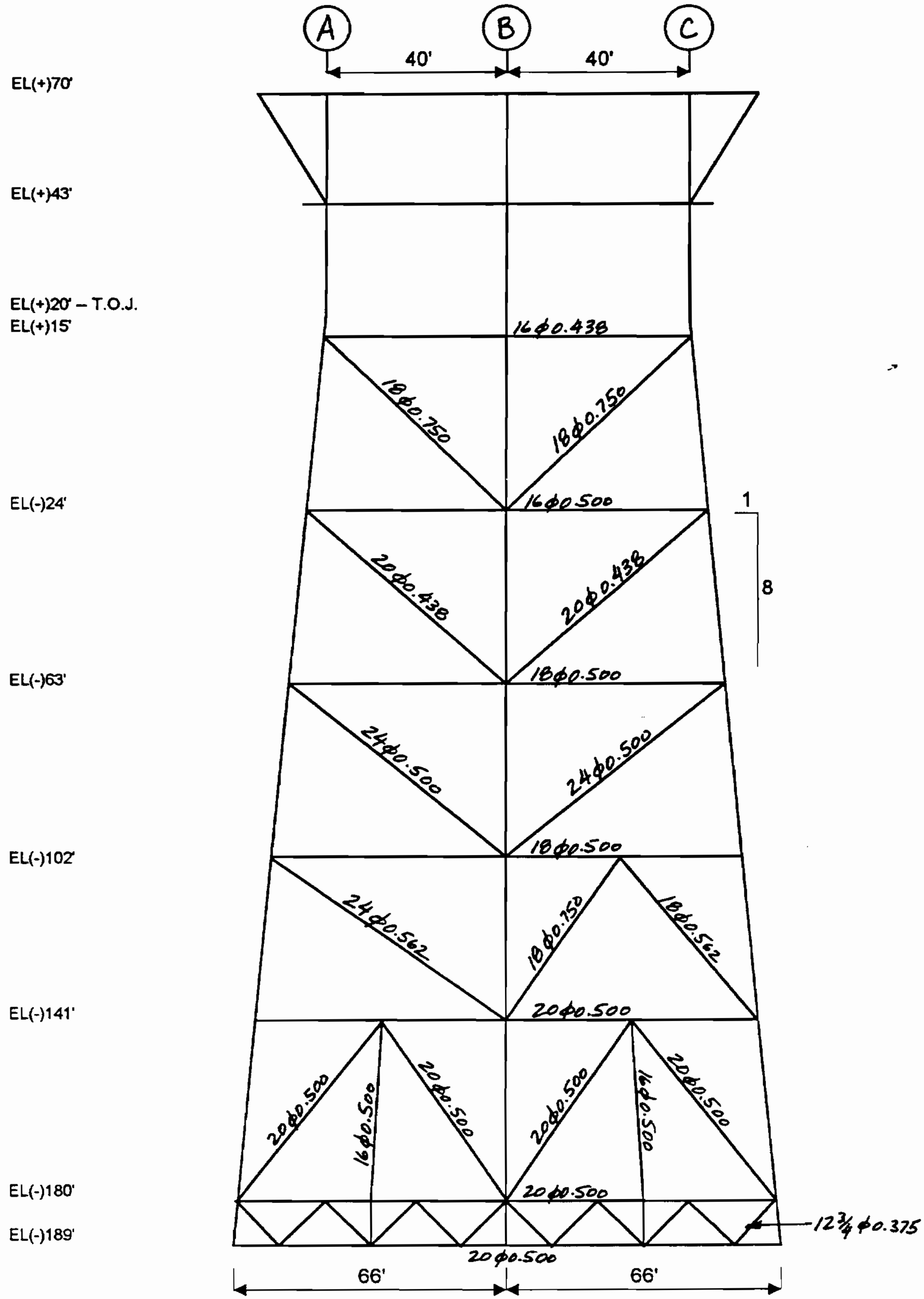
ROW "A"

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" – Model Sketches			A.F.E. NO. 645-1 1008



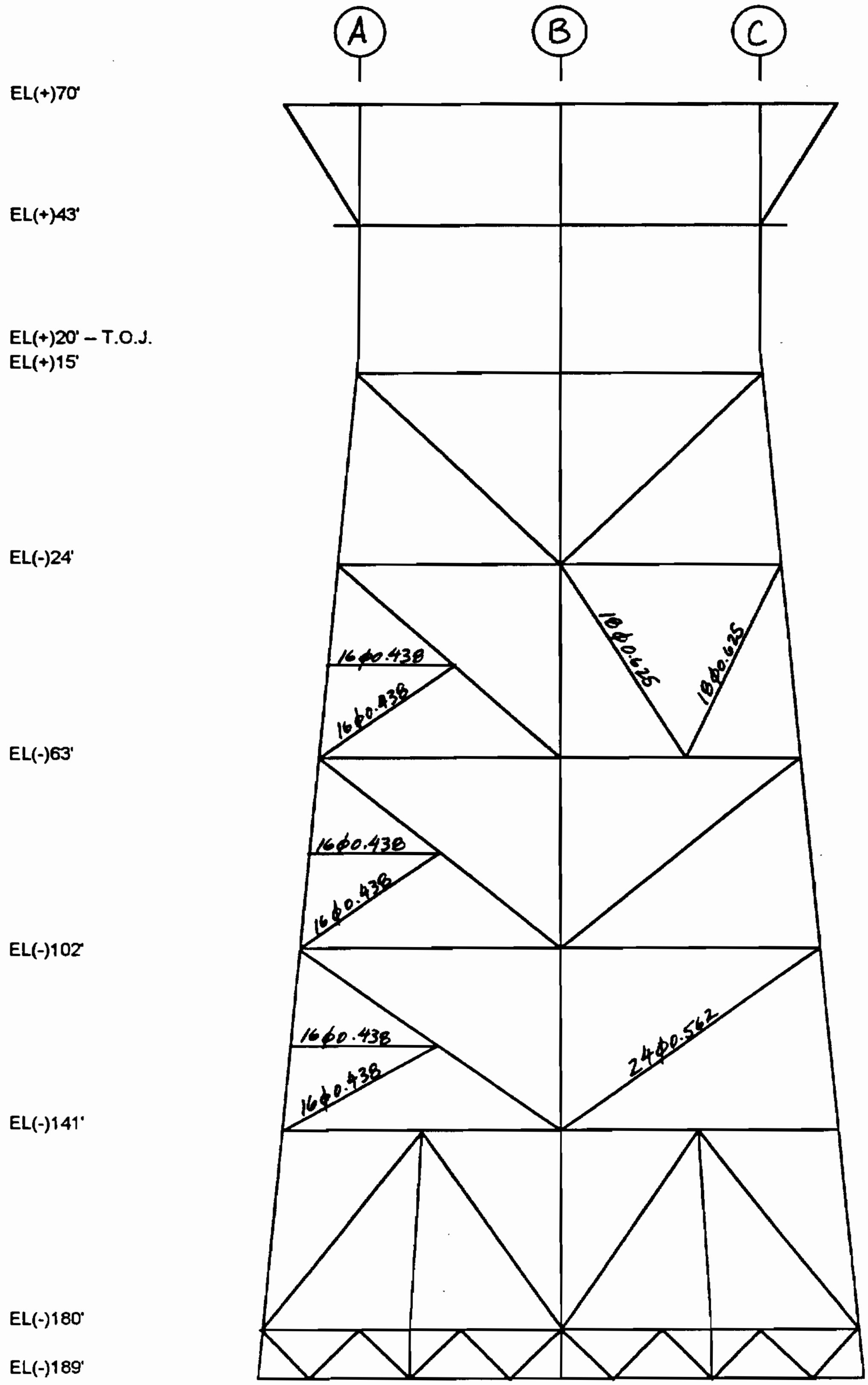
ROW "B"

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" – Model Sketches			A.F.E. NO. 645-1 1008



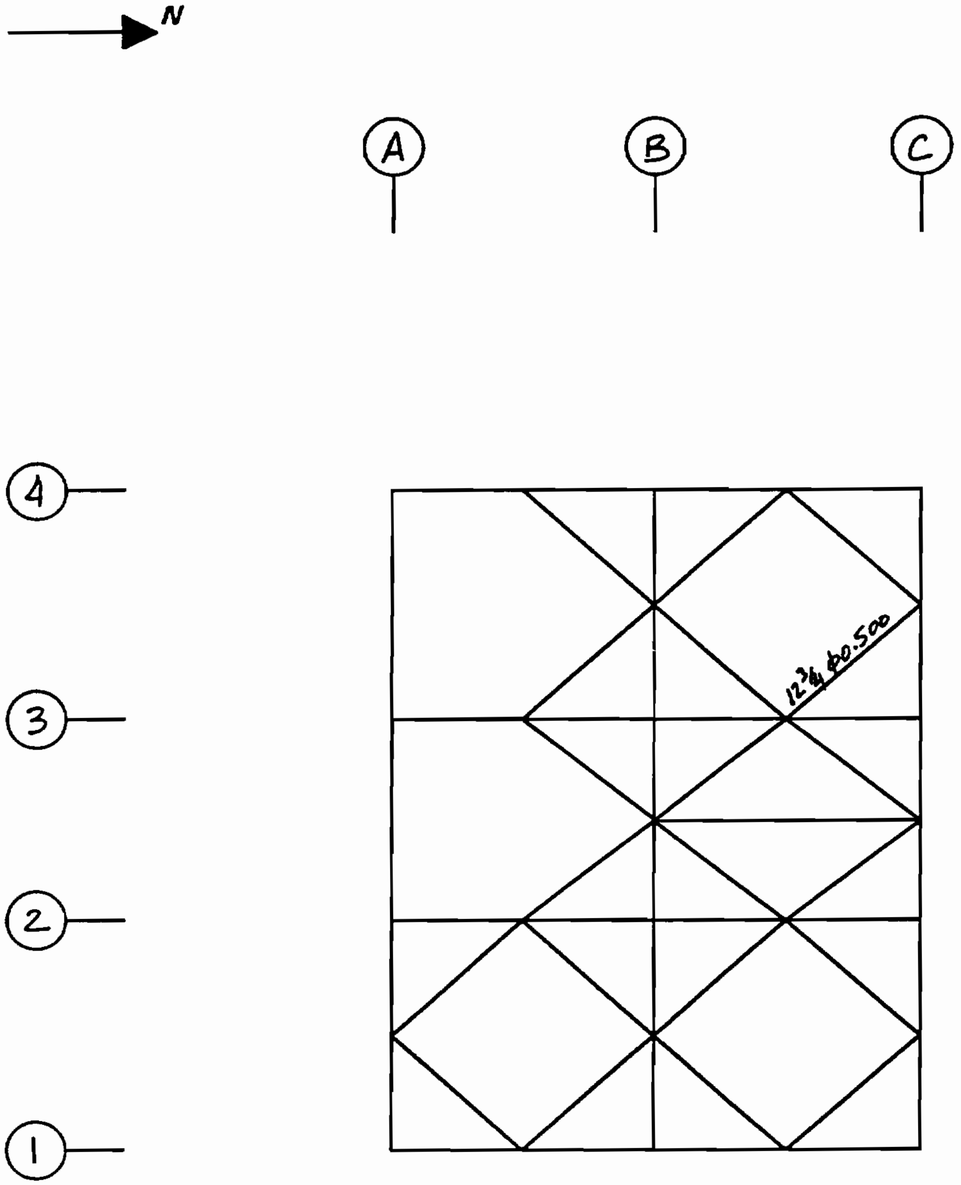
ROW "1"

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" -- Model Sketches			A.F.E. NO. 645-1 1008



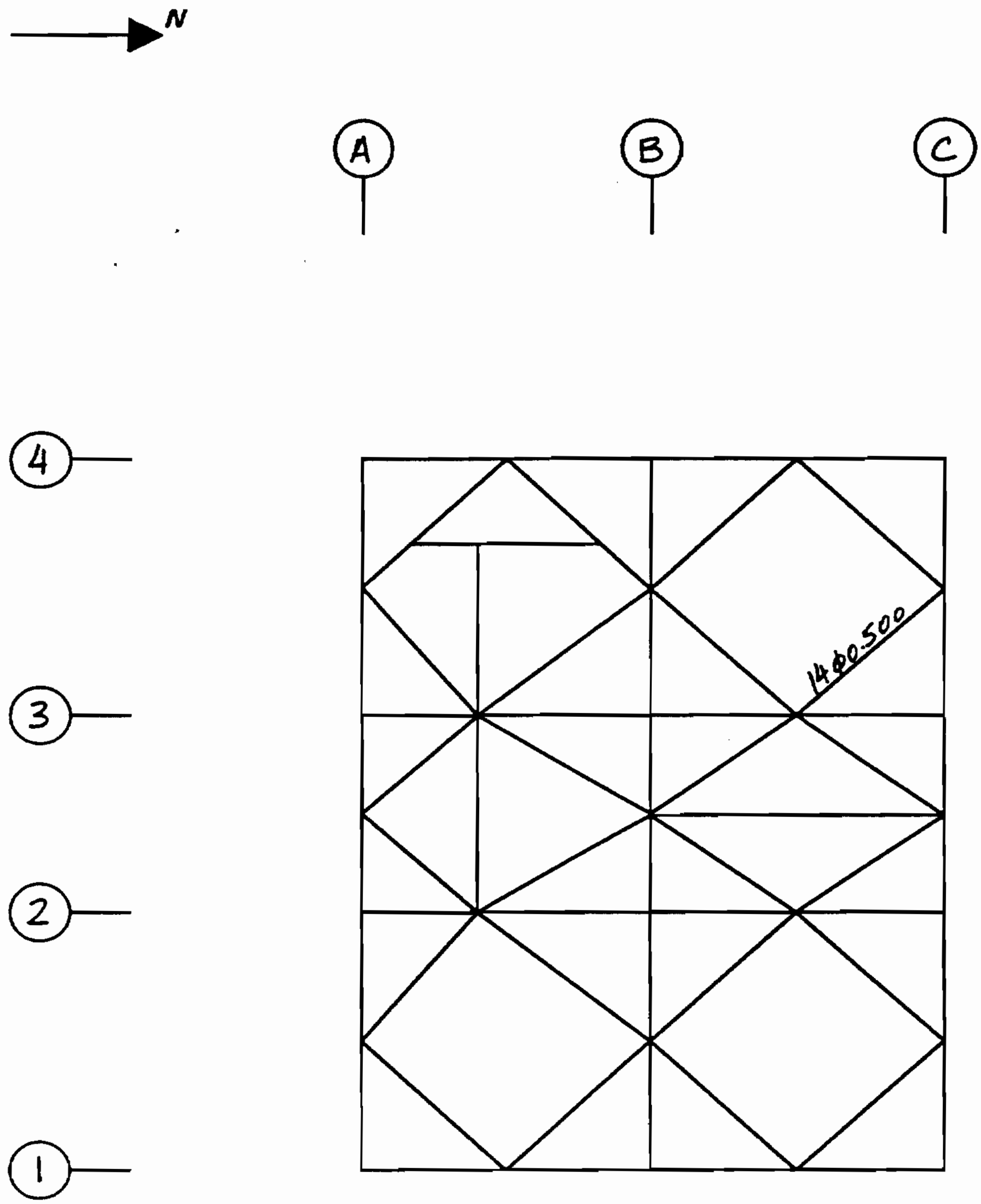
ROW "2"

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" – Model Sketches			A.F.E. NO. 645-1 1008



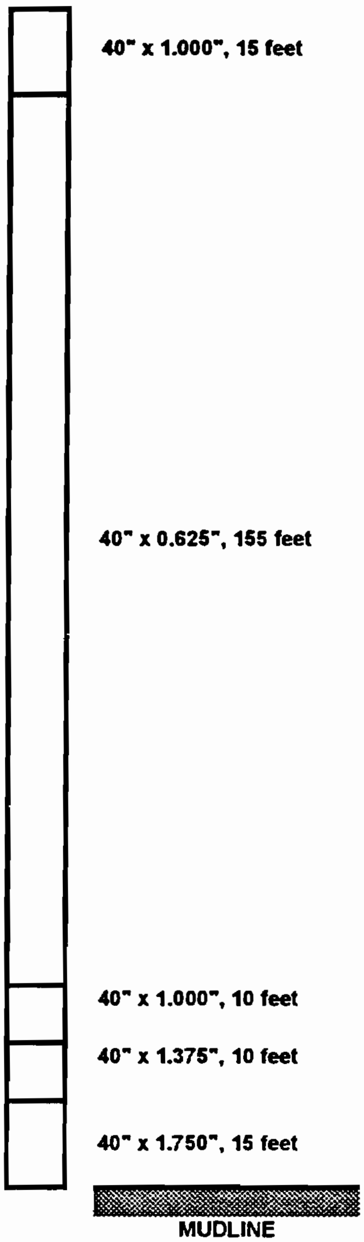
EL(-)24'

	PREPARED BY	CHECKED BY	DATE
TRIAL PARTICIPANT "Q"			August 1, 1994
SUBJECT: PLATFORM "Q" – Model Sketches			A.F.E. NO. 645-1 1008

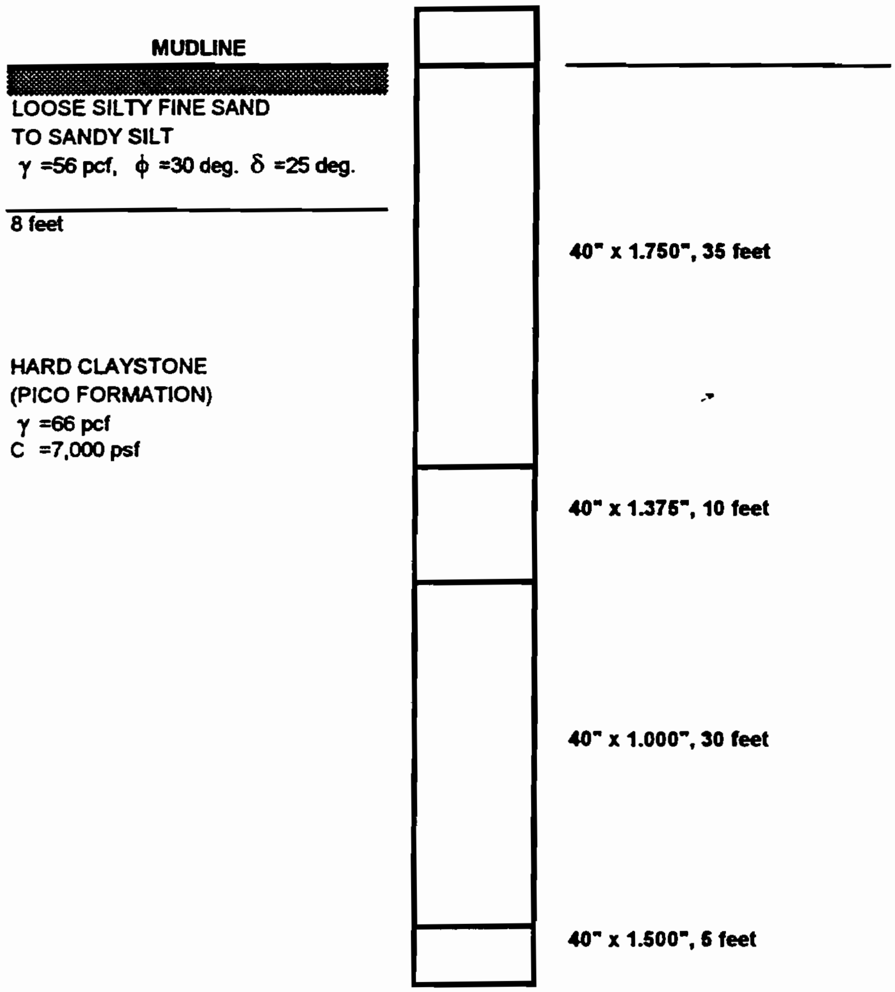


EL(-)63'

TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" -- PILE & FOUNDATION INFORMATION			A.F.E. NO. 645-1 1008



NOT TO SCALE



NOT TO SCALE

PILE CAPACITY = 2,050 kips (vertical piles)

2,080 kips (battered piles)

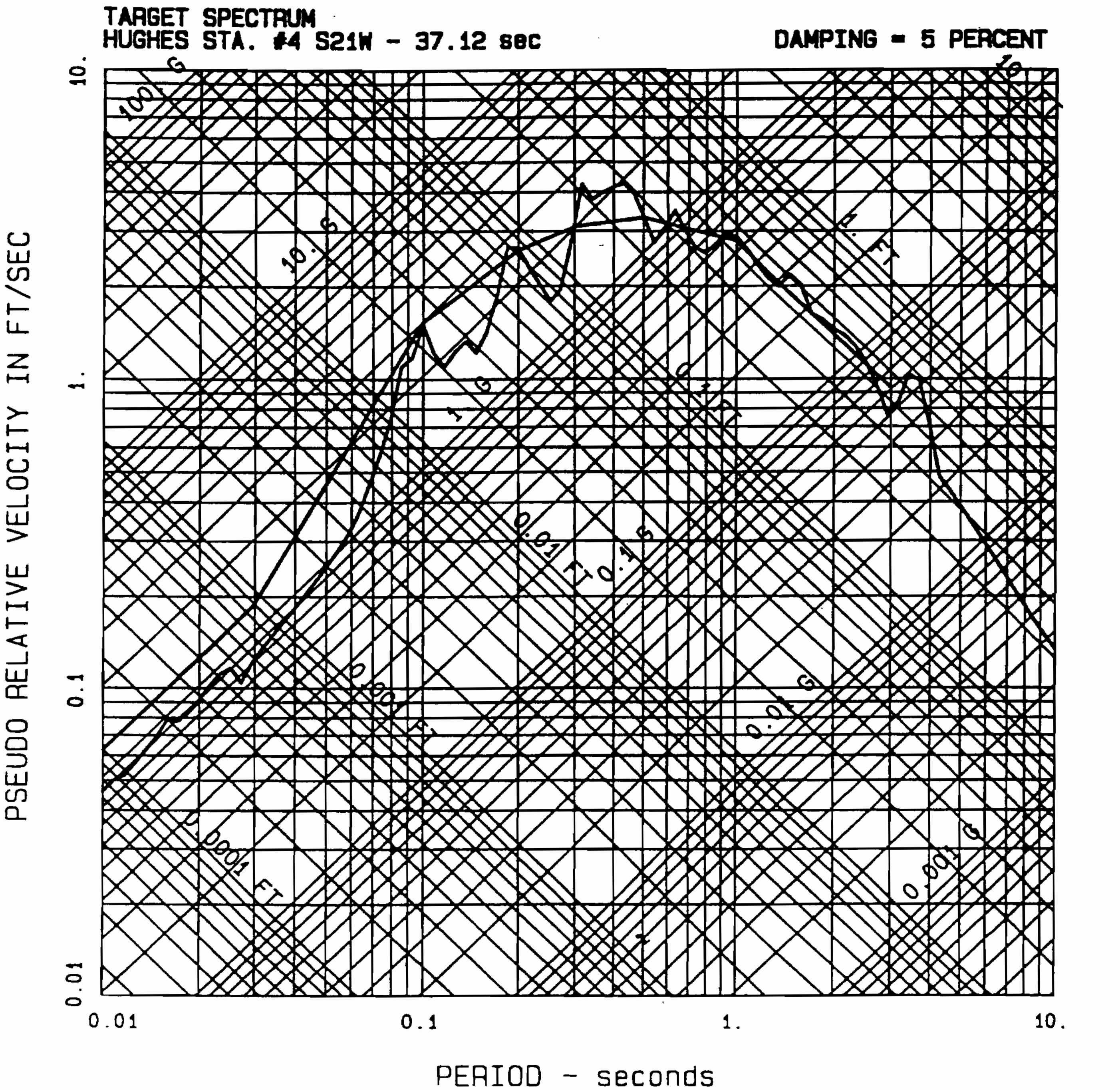
TRIAL PARTICIPANT "Q"	PREPARED BY	CHECKED BY	DATE August 1, 1994
SUBJECT PLATFORM "Q" -- P-Y CURVE DATA			A.F.E. NO. 645-1 1008

STATIC p-y CURVE DATA

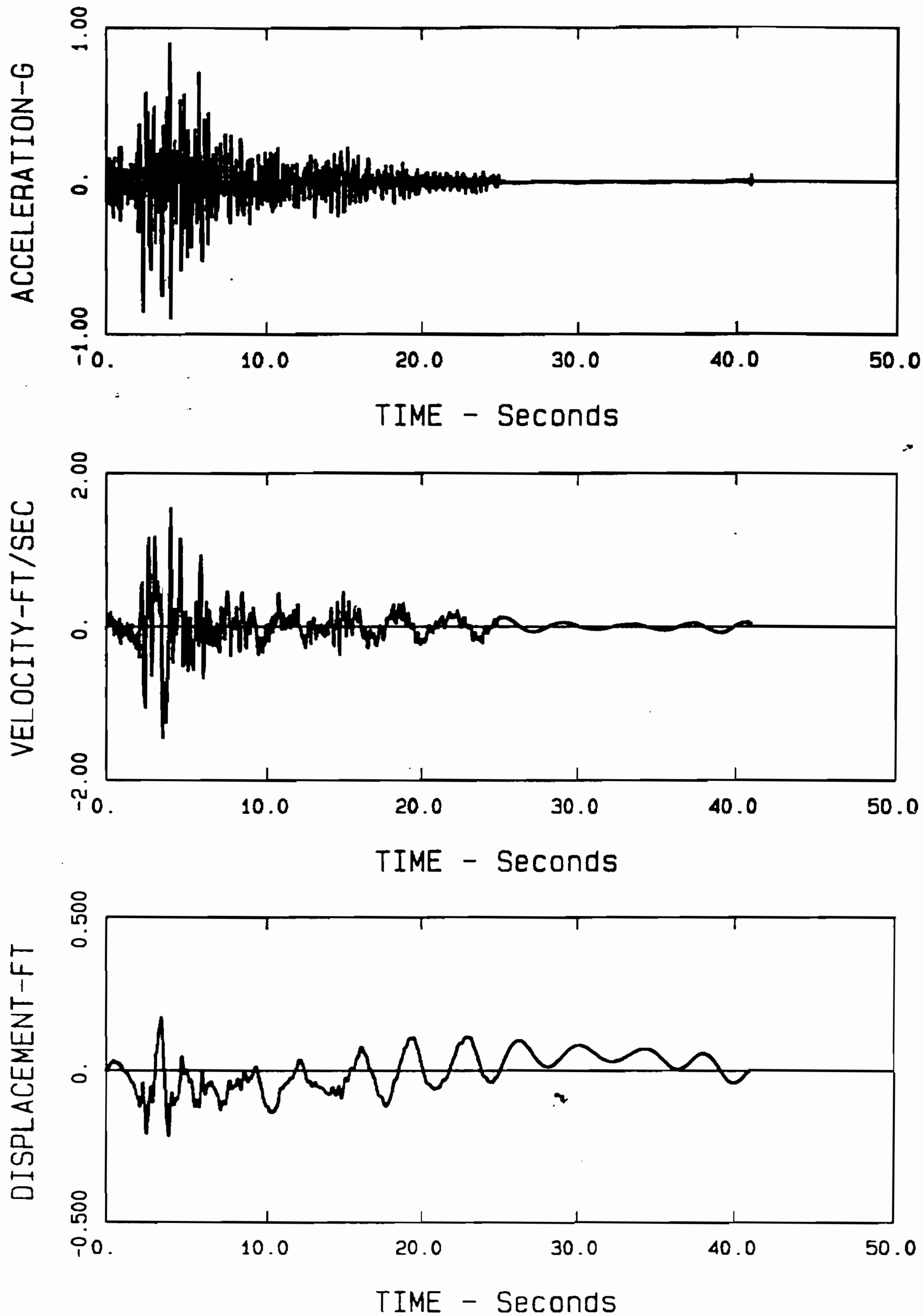
Coordinate of Curve Point	Penetration Below Mudline (feet) 0	3.5	5	7	9	16 to 78
y(1)	0.0	0.0	0.0	0.0	0.0	0.0
p(1)	0.0	0.0	0.0	0.0	0.0	0.0
y(2)	0.06	0.06	0.06	0.06	0.12	0.12
p(2)	0.0	44.0	67.0	93.0	5369.0	5856.0
y(3)	0.11	0.11	0.11	0.11	0.24	0.24
p(3)	0.0	89.0	133.0	187.0	7592.0	8282.0
y(4)	0.17	0.17	0.17	0.17	0.47	0.48
p(4)	0.0	133.0	200.0	280.0	9645.0	10536.0
y(5)	0.22	0.22	0.33	0.33	0.70	0.72
p(5)	0.0	178.0	267.0	373.0	10553.0	11547.0
y(6)	0.33	0.33	0.33	0.33	0.94	0.96
p(6)	0.0	267.0	400.0	560.0	10873.0	11920.0
y(7)	0.50	0.50	0.50	0.50	1.40	1.44
p(7)	0.0	331.0	493.0	661.0	10437.0	11493.0
y(8)	0.67	0.67	0.67	0.67	3.28	3.36
p(8)	0.0	360.0	540.0	727.0	4630.0	5082.0
y(9)	1.5	1.5	1.5	1.5	4.21	4.32
p(9)	0.0	495.0	748.0	1027.0	1727.0	1875.0
y(10)	10.0	10.0	10.0	10.0	10.0	10.0
p(10)	0.0	495.0	748.0	1027.0	1727.0	1875.0

CYCLIC p-y CURVE DATA

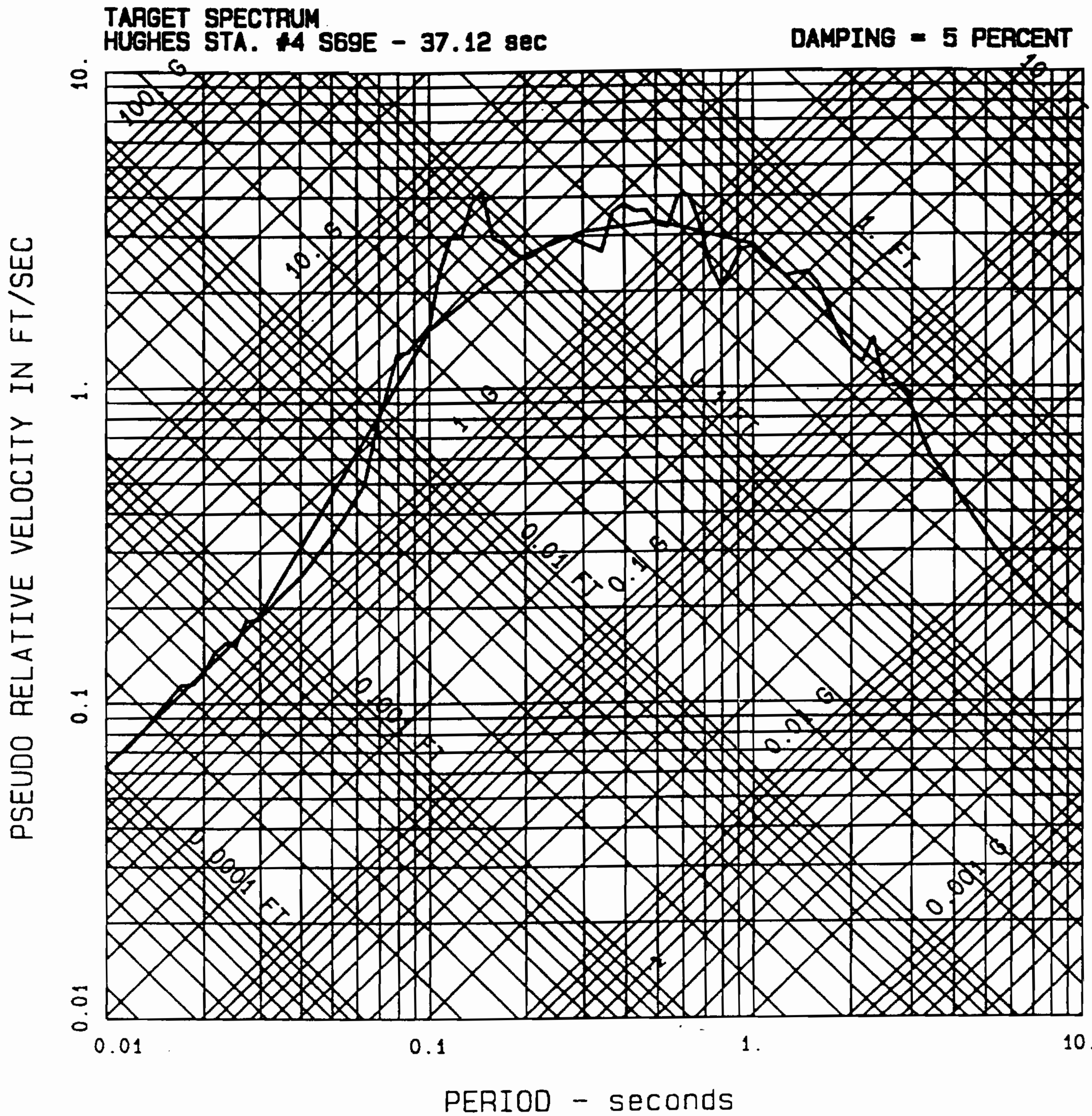
Coordinate of Curve Point	Penetration Below Mudline (feet) 0	3.5	5	7	9	16 to 78
y(1)	0.0	0.0	0.0	0.0	0.0	0.0
p(1)	0.0	0.0	0.0	0.0	0.0	0.0
y(2)	0.06	0.06	0.06	0.06	0.085	0.098
p(2)	0.0	44.0	67.0	93.0	1793.0	2993.0
y(3)	0.11	0.11	0.11	0.11	0.13	0.20
p(3)	0.0	89.0	133.0	187.0	2449.0	4939.0
y(4)	0.17	0.17	0.17	0.17	0.21	0.30
p(4)	0.0	133.0	200.0	280.0	3338.0	6003.0
y(5)	0.22	0.22	0.33	0.33	0.30	0.34
p(5)	0.0	178.0	267.0	373.0	3755.0	6265.0
y(6)	0.33	0.33	0.33	0.33	0.38	0.44
p(6)	0.0	267.0	400.0	560.0	3844.0	6415.0
y(7)	0.50	0.50	0.50	0.50	0.47	0.54
p(7)	0.0	331.0	493.0	661.0	3755.0	6265.0
y(8)	0.67	0.67	0.67	0.67	1.19	1.38
p(8)	0.0	360.0	540.0	727.0	1728.0	2427.0
y(9)	1.5	1.5	1.5	1.5	1.53	1.77
p(9)	0.0	495.0	748.0	1027.0	792.0	639.0
y(10)	10.0	10.0	10.0	10.0	10.0	10.0
p(10)	0.0	495.0	748.0	1027.0	792.0	639.0



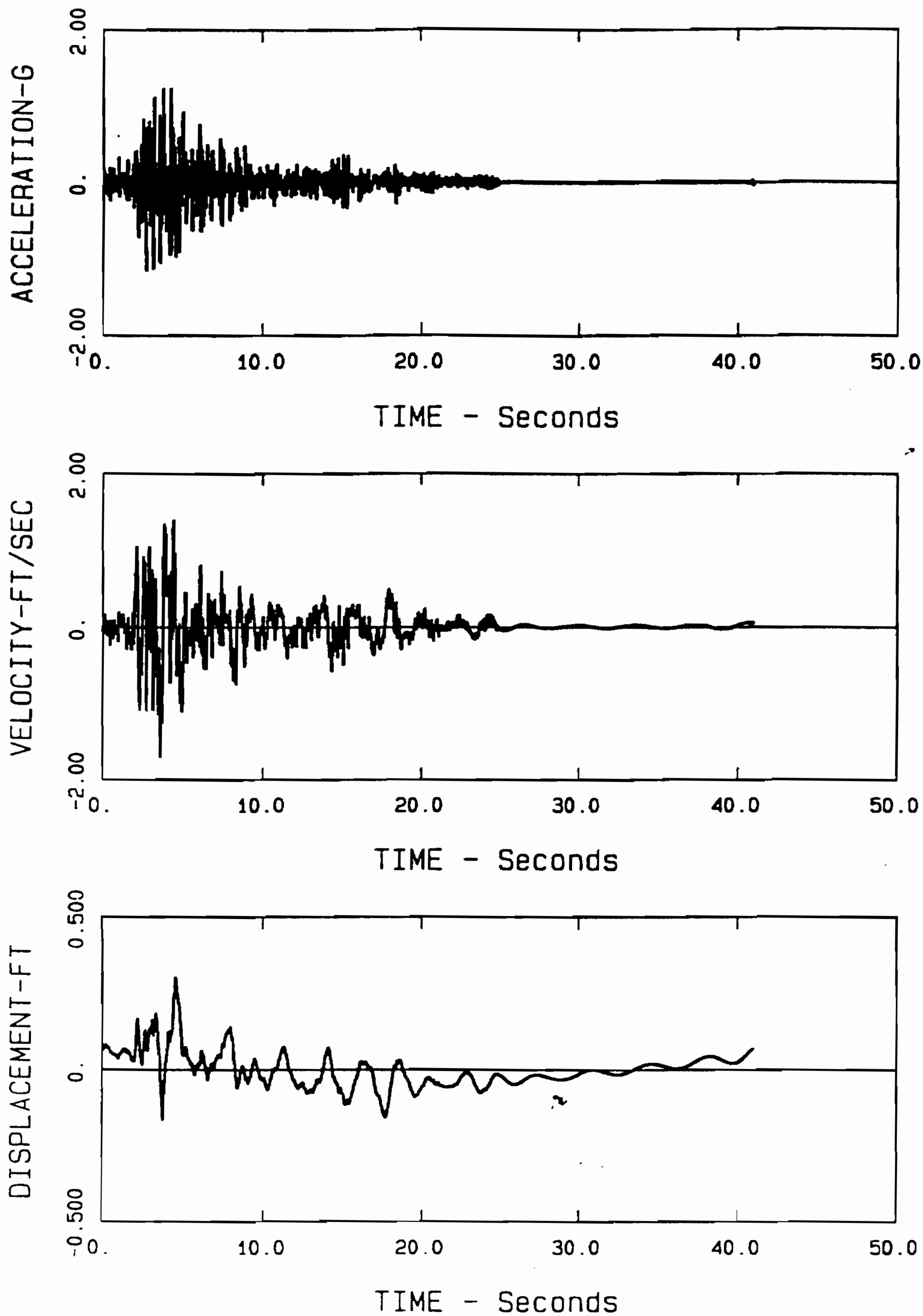
PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, S21W, 2/9/71, 1000 YR RP



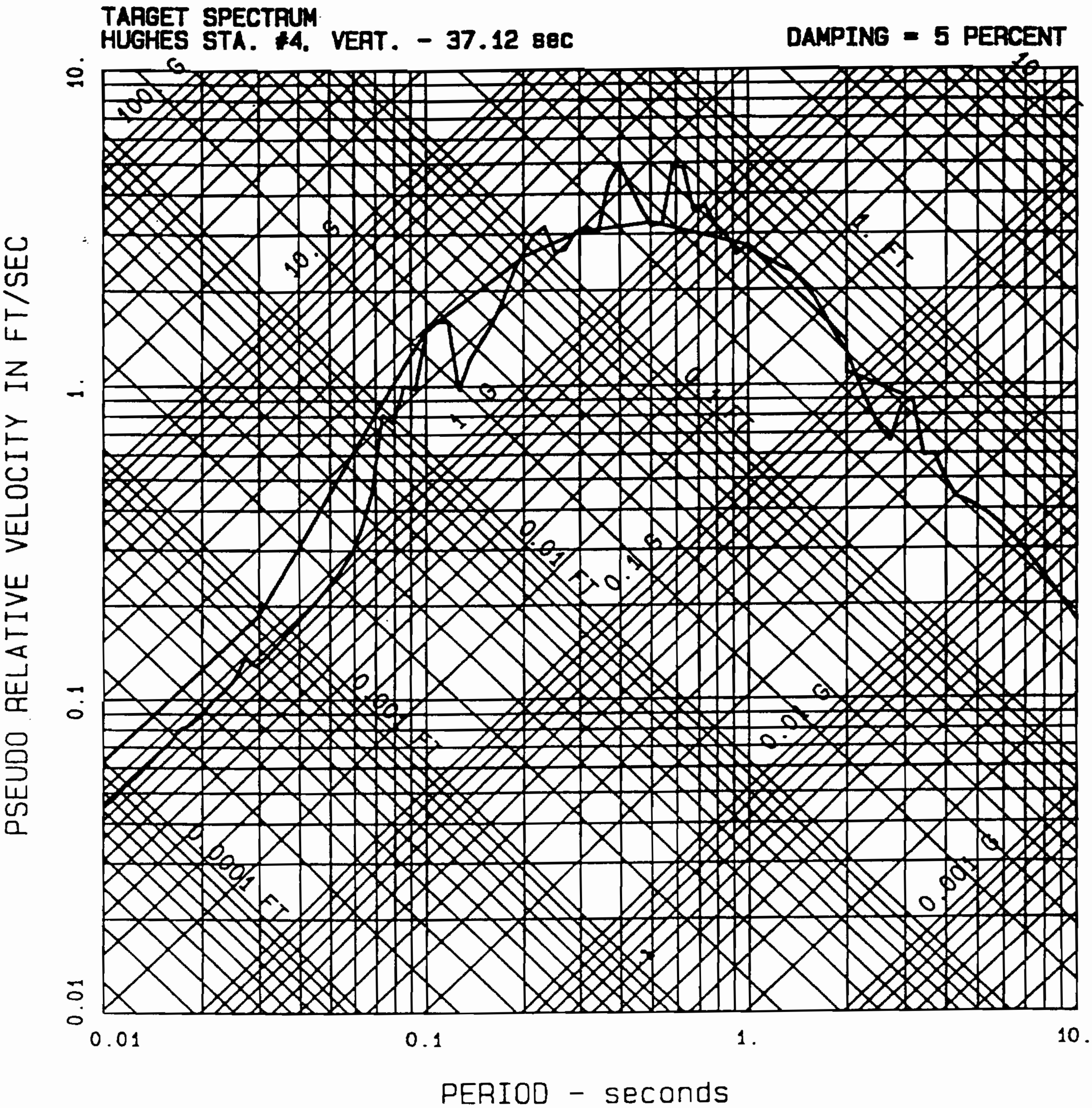
PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, S21W, 2/9/71, 1000 YR RP



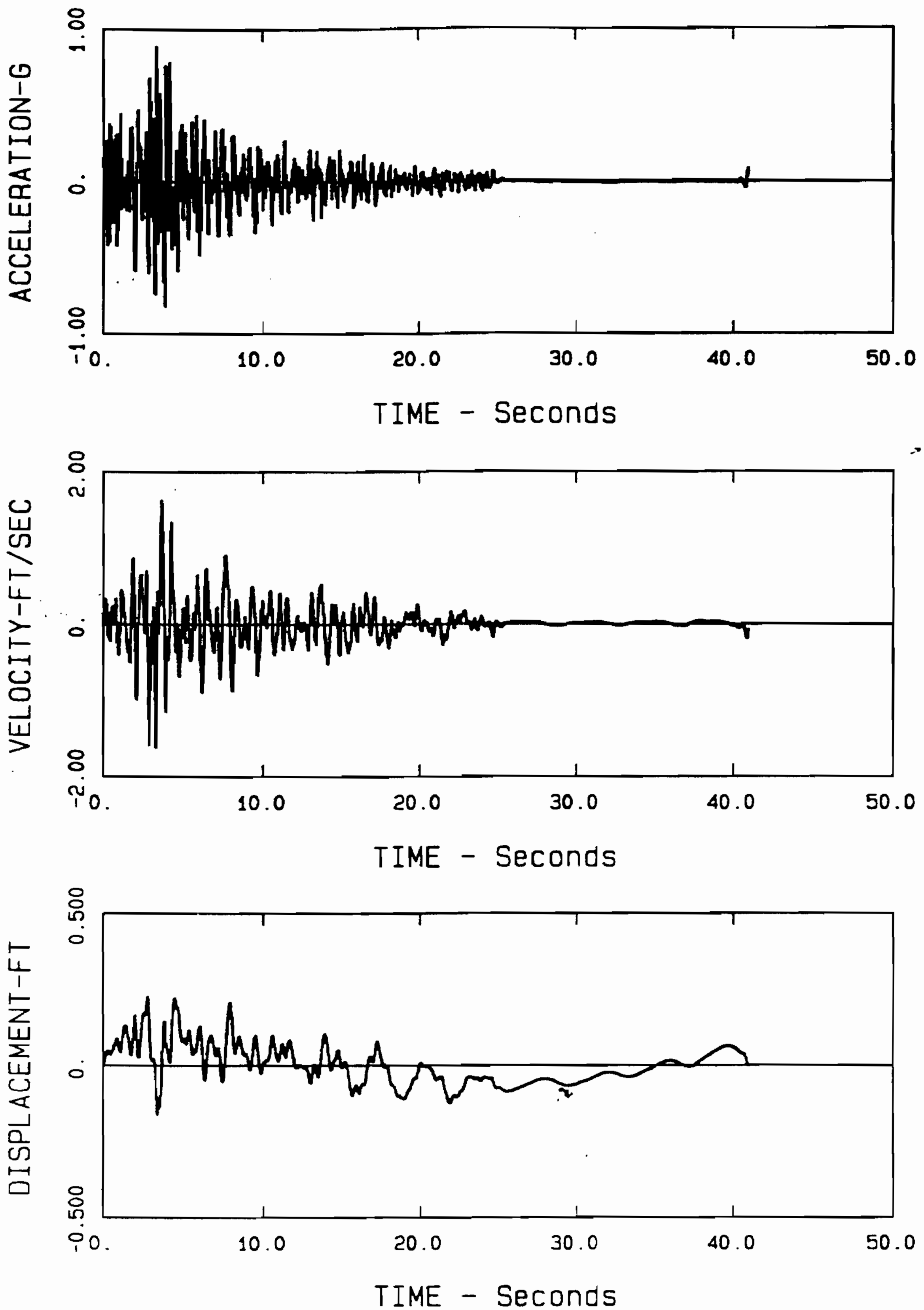
PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, S69E, 2/9/71, 1000 YR RP



PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, S69E, 2/9/71, 1000 YR RP



PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, VERT, 2/9/71, 1000 YR RP



PLATFORM A - ACCELERATION TIME HISTORY
LK. HUGHES #4, VERT, 2/9/71, 1000 YR RP

PLATFORM "R"

Platform R
Platform Information

1. General Information

a. The platform is still owned by the original owner

b. The platform is in its original location. The use of the platform has not changed since originally installed. The platform produces an average of 125 MMSCF per day of natural gas, no oil.

c. The drilling and production platform is located in 100 feet of water (MLLW) in a high current, ice, and earthquake loading area. Platform North is approximately 50 degrees West of True North. The long axis of the platform is located parallel to the predominate ice and current loading which runs true Northeast to Southwest (platform East to West). The current and ice loads reverse direction with each tide change.

d. The platform consist of four 14 foot diameter vertical grout reinforced legs with X bracing in each vertical face of the jacket and horizontals at the minus 30 foot elevation, Figure 1.

e. There are 8 well slots located inside three of the four legs for a total of 24 slots. The leg next to the quarters does not have any well slots. Two gas pipelines to shore run inside the legs and X bracing members to protect them from the ice loads.

f. In normal operations, the platform is manned with a staff of five.

g. The platform has withstood over 20 years of ice loading and has not been loaded by any major earthquake vibrations.

2. Original Design

a. The platform was designed by McDermott in 1967.

b. A full set of structural drawings of the jacket are available. The original material specification called for ASTM A516 Gr. 70 Modification A steel for the jacket and exposed portions of the deck structure. The piles were made from 50 ksi material, ASTM A441 with A516 at the mudline areas where the piles could be exposed to cold temperatures.

c. The platform was designed to the Uniform Building Code (UBC) that was current in 1967.

d. The platform was designed for the following criteria:

Wave Height	27.5 ft
Wave Period	8.5 sec
Wind Velocity	80 mph (1 hour average)
Current Velocity	13.5 fps at surface 10.4 fps at -80 feet, 6.5 fps at seafloor
Tidal Range	0 to 30 feet
Earthquake Loading	0.1g (10% of the vertical dead and live loads applied as a lateral static load)
Ice Loading	120 kips/ft dia. on two front legs, 50 kips/ft dia on two back legs, total of 4,760 kips

e. The bottom of the 14 foot horizontal cylindrical girders is at elevation +43. No structures are below this level.

f. The deck design loading conditions is shown in Figures 2 to 6. An original drill rig load of 2000 kips was also included. All equipment loads on the decks are well below the original area deck load criteria.

g. The foundation soils consist of:

Stratum	From	To	Description
I	0	55	Dark Gray Coarse to Silty Fine Sand w/gravel
II	55	85	Hard Dark Gray Silty Clay w/ Gravel
III	85	219	Gray Coarse to Silty Fine Sand w/ Gravel: predominately Gravel 100 to 180 ft.

as reported by McClelland Engineers in a site specific investigation in May, 1967. The design soil parameters used in this assessment are:

Stratum	Unit Weight	Phi Angle	Shear Strength
I	60 pcf	35	
II(top)	50 pcf		3000 psf
II(bottom)	50 pcf		5000 psf
III	60 pcf	40	

h. The structure is supported in 32 30 inch diameter piles, 8 per leg, driven to a penetration of 175 feet. The piles are used as the conductors for the wells in three of the four legs.

i. One escape capsule is located on the Northwest corner of the girder deck level and is the only appurtenance. All pump risers and other caissons are run inside the legs. There are no boat bumpers or landings on the structure.

3. Construction

a. The structure was fabricated in Korea by Mitsubishi Heavy Industries and installed by McDermott in 1968.

b. No changes to the structure were made during installation requiring any "as-built" drawings.

c. The original fabrication, welding and construction specifications are available in Company files.

d. No material traceability records are available

e. Pile driving records are available as copies of daily reports during installation. Piles were installed by driving to refusal, drilling pilot hole, then driving to required depth. Detailed pile driving records are not available.

f. The annulus between a 7 foot diameter inner and 14 foot diameter outer column where the 8 foundation piles/well conductors are located was completely grouted from the seabed to the top of the jacket at +42 feet as noted in the daily reports.

4. Platform History

a. As noted in 1.g. above, the platform has withstood over 26 years of lateral ice loading and has not experienced any major earthquakes in that time.

b. There have been no accidental loads or collisions with the structure. The only change in loading is that the original 2000 kips drilling rig load has been replaced by a new 2600 kips rig load. Details of the structural consequences of this change in loading will be discussed later.

c. Detailed API Level II and III underwater condition surveys were made in the summers of 1983, 1986, 1990, and 1993. The Level III surveys have inspected the major joint nodes at the minus 30 foot elevation and the center nodes in the face x-bracing. A swim through inspection of the entire jacket and a seafloor scour survey is included in each inspection. Seafloor scour surveys were make yearly for the first few years after installation to determine that scour is not a problem. Minor dents in the horizontal interior bracing members were discovered in 1990 and 1993. Recent structural analyses have shown that these dents are in lightly loaded members and were deemed to be structurally insignificant.

d. There have been no repairs to the jacket portion of the structure. The original cathodic protection system was replaced by an upgraded system consisting of ten seabed impressed current anode sleds and impressed current anodes in each inner leg. No corrosion has been noted during the underwater inspections.

e. Because of the higher drilling rig loads, in the Northeast corner of the deck, one skid truss member was strengthened by adding cover plates welded to the flanges on the open side of the I-beam. The operating loads (setback and hook load combination) of the rig are limited on the Southeast corner of the deck so that the original rig loads will not be exceeded.

5. **Present Condition**

a. The deck size, location, and elevations are shown in Figures 2 to 6. No changes to original design conditions.

b. The design area deck loadings are shown in Figures 2 to 6. No changes to the area loadings have been made. The original drill rig loading of 2000 kips has been replaced with a higher drill rig loading of 2600 kips.

c. Field measured deck clearance is +43 feet.

d. The platform produces an average of 125 MMSCF per day of dry gas, no oil. Any produced liquids are stored in girder beam tanks. The storage capacities on board are shown in Figures 5 and 6.

e. Two additional escape capsules have been added to the girder deck to supplement the single existing capsule. The new capsules are located on the Southeast corner and the West side of the girder deck.

f. There are 24 well slots, 8 in each of the Southeast, Southwest, and Northwest legs. The quarters sit next to the Northeast leg that has no well slots. All conductors are run inside the legs through the foundation piles to protect them from the ice loading.

g. The deck and jacket above water is inspected yearly.

h. The jacket was inspected to Level II and III in 1993. Other than the dents previously discovered in the 2 foot diameter interior horizontal bracing members at minus 30 feet, no anomalies were discovered.

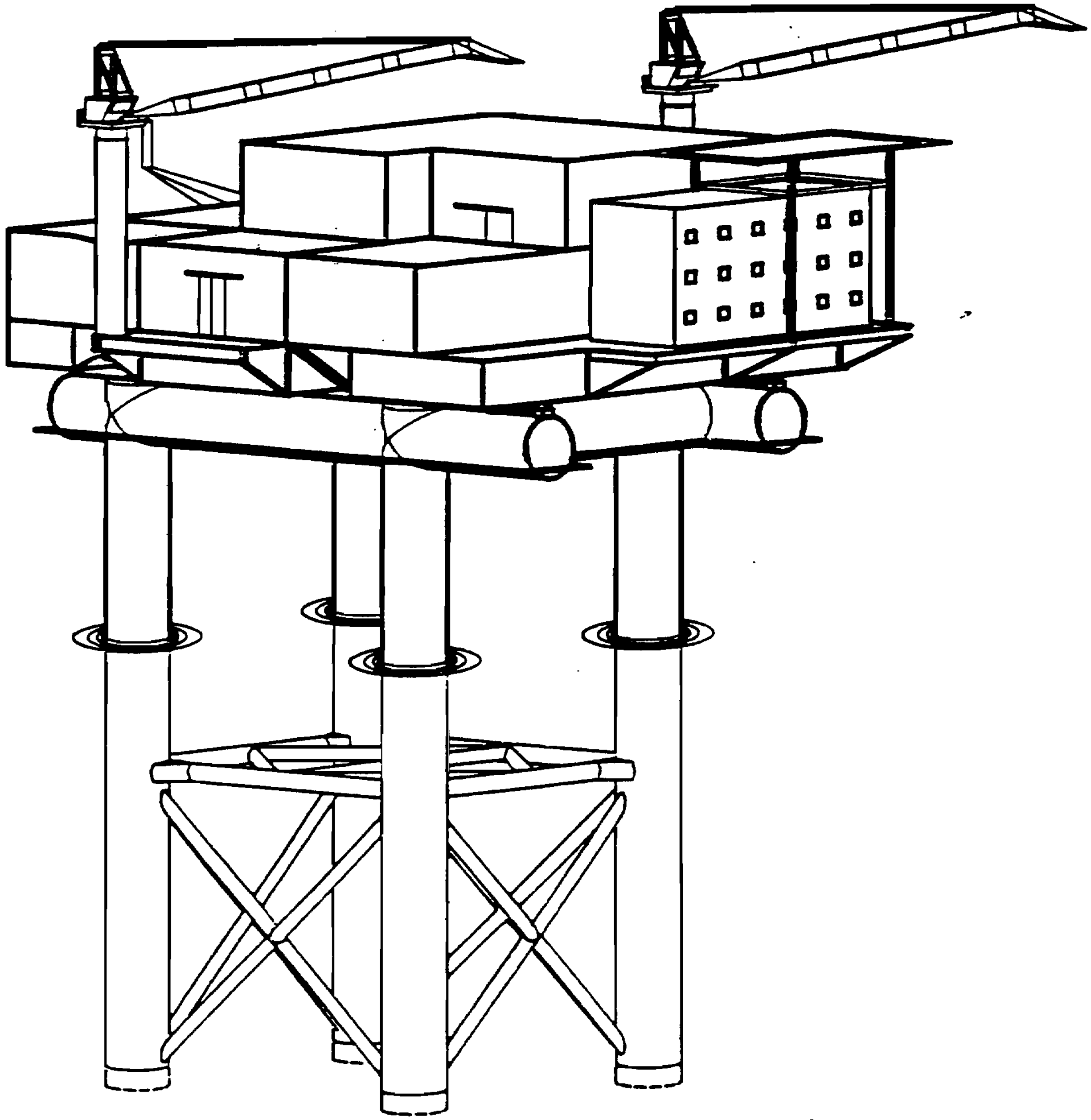
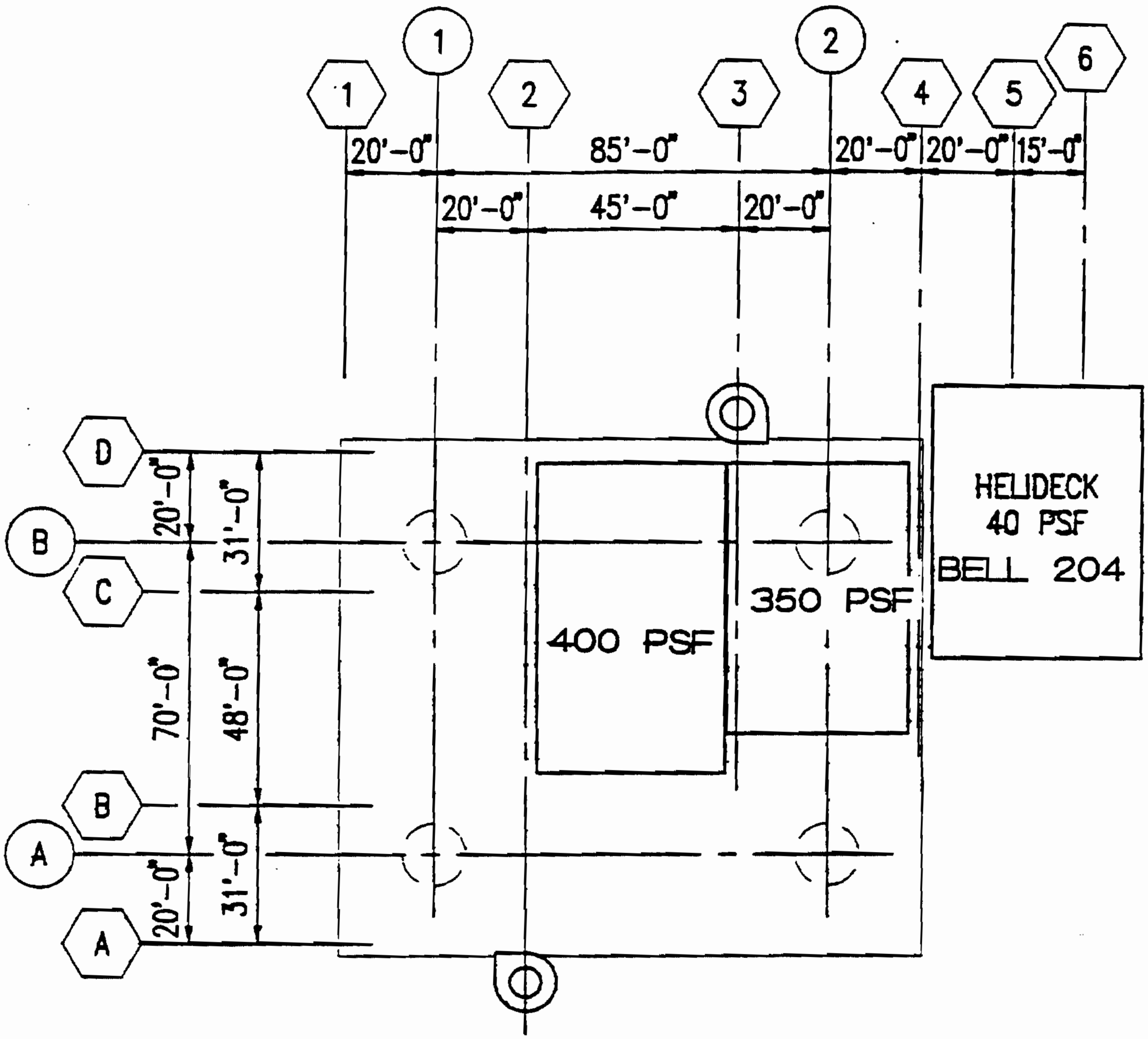


Figure 1



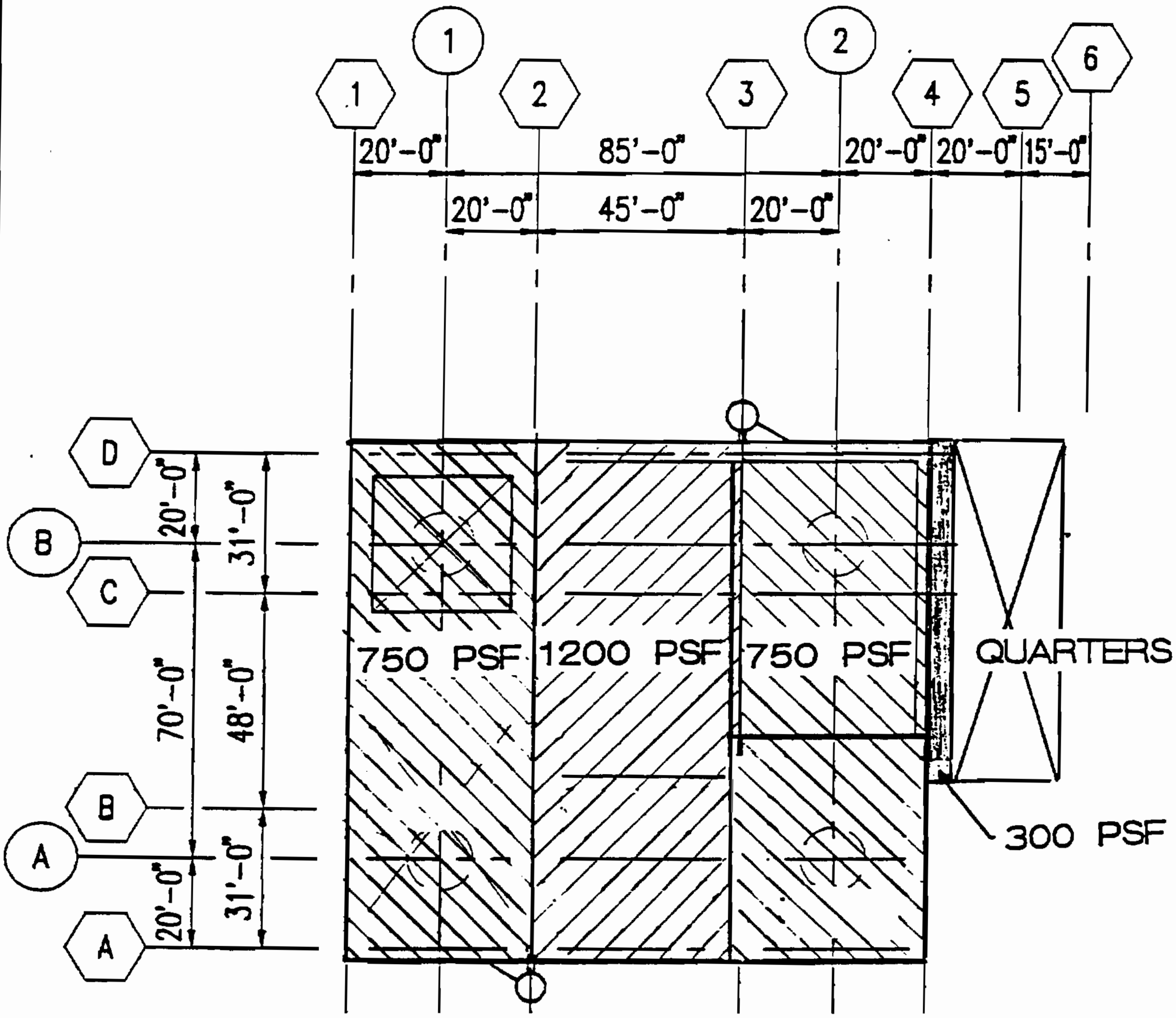
Note: Support members designed for 0.75 times the area load

AREA LIVE LOAD – KEY PLAN

PIPE RACK DECK

ORIGINAL MCDERMOTT DESIGN LOADS – 1968

Figure 2

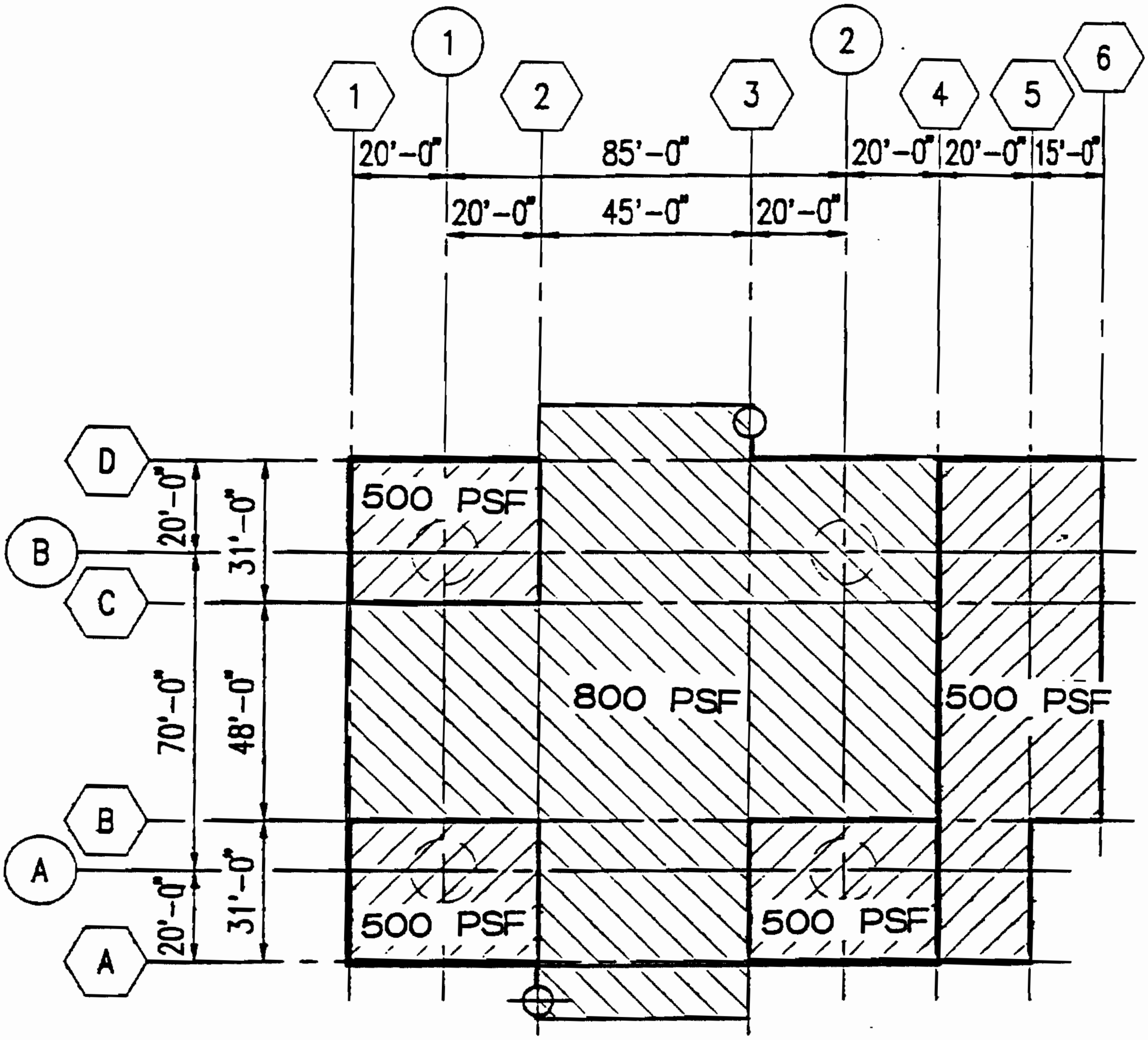


Note: Support members designed for 0.75 times the area load

AREA LIVE LOAD – KEY PLAN
DRILLING DECK

ORIGINAL MCDERMOTT
DESIGN LOADS – 1968

Figure 3

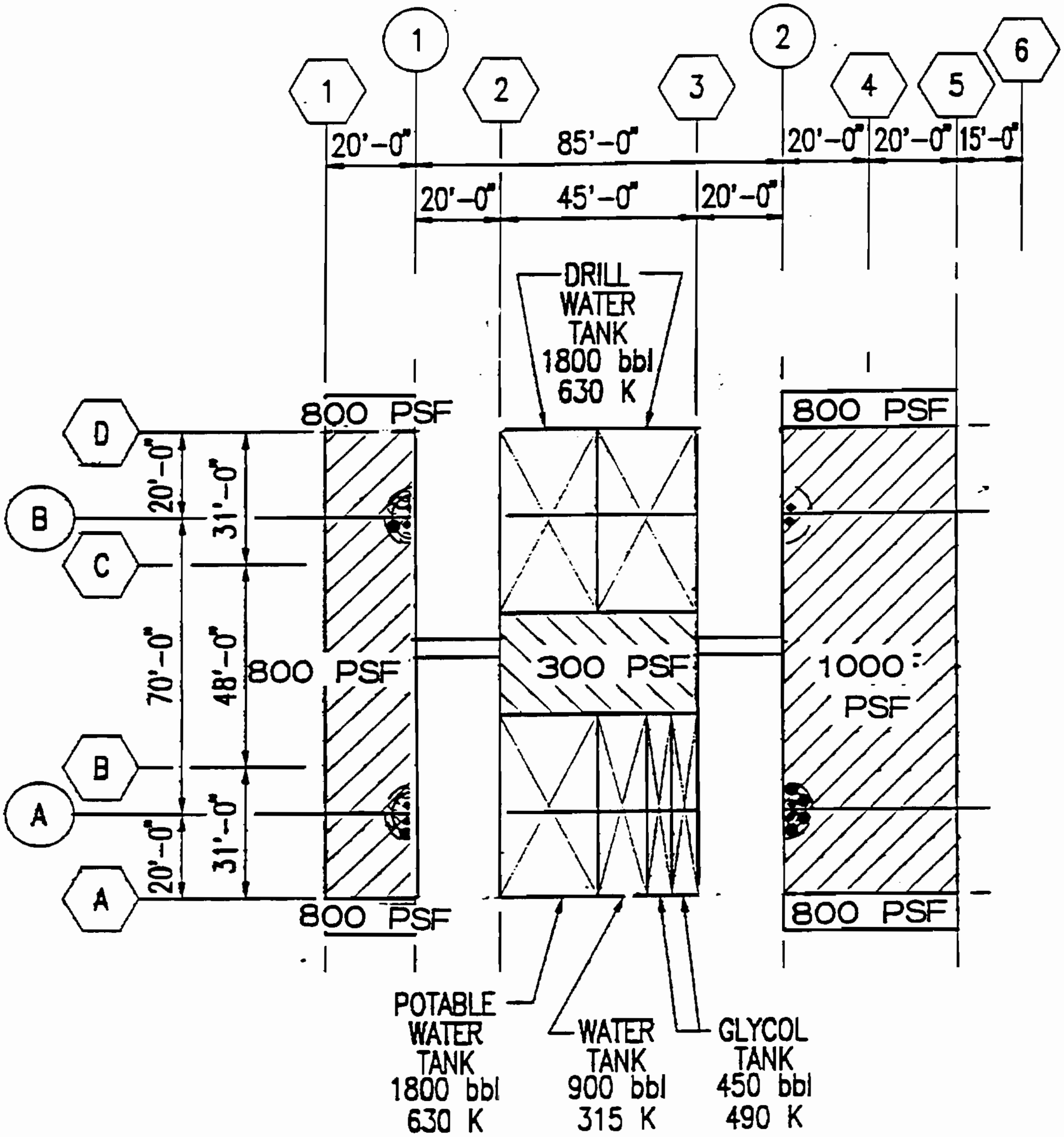


Note: Support members designed for 0.75 times the area load

AREA LIVE LOAD – KEY PLAN
PRODUCTION DECK

ORIGINAL MCDERMOTT
DESIGN LOADS – 1968

Figure 4



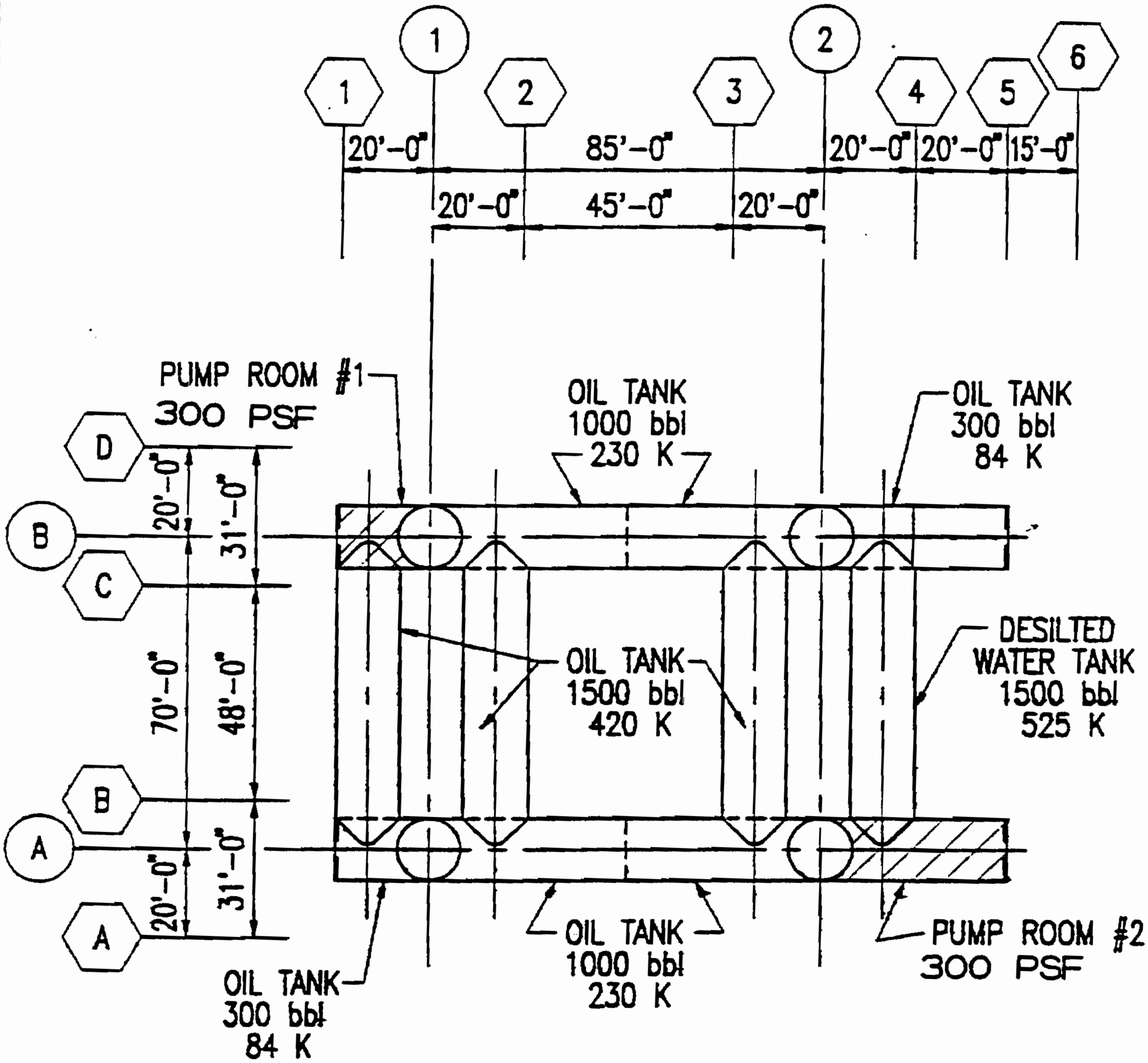
AREA LIVE LOAD – KEY PLAN

SUB-DECK

Note: Support members designed for 0.75 times the area load

ORIGINAL MCDERMOTT DESIGN LOADS – 1968

Figure 5



Note: Support members designed for 0.75 times the area load

AREA LIVE LOAD – KEY PLAN
CYLINDRICAL GIRDERS (+)50 ft ELEV.

ORIGINAL MCDERMOTT
DESIGN LOADS – 1968

Figure 6

Platform R
Platform Assessment

A.1 Platform Selection

This platform was selected for assessment based on addition of a heavier drilling rig (additional 600 kips), extension of the life of the structure to 40 years, changes in the earthquake loading and analysis required, and change in ice loading requirements based on API RP2N, 1st Edition.

A.2 Platform Categorization

The platform is continuously manned and environmentally sensitive.

A.3 Condition Assessment

As noted in Section 1, Platform Information, the platform is in excellent shape. Subsea inspections Level II and III inspections have not discovered any significant damage to the platform.

A.4 Design Basis Check

The platform was designed and constructed to the 1967 UBC code because API RP2A and RP2N did not exist at that time. It does not pass the design basis checks.

A.5 Analysis Checks

A.5.1 - Metocean, seismic, and ice criteria loads

The major loadings on this structure occur in response to the ice and earthquake loads. The only metocean data that applies is the wind (80 mph, 1 hour average) and the current (13.5 fps at the surface) in combination with the ice loading. The ice loads applied to the structure for the analysis is 166 kips/foot leg diameter as calculated according to API RP2N, 1st Edition (original ice load of 120 kips/foot). The seismic loads were calculated using a site specific spectrum. The site specific spectrum curve lies between API Zone 4 and 5 curves. This area is designated as API Zone 4.

<u>A.5.2 - Screening</u>

No screening procedures applied to this structure.

<u>A.5.3 - Design Level - Ice</u>

The complete (deck and jacket) computer model of this structure consisted of approximately 740 members and 330 nodes without the foundation modeled (Figure 7). The deck portions of the structure above the girder tank level was condensed and the deck dead and live loadings were applied to the top of the legs at the girder level resulting in a computer model with 115 members and 90 nodes including the foundation model (Figure 8 and 9). The pile groups were modeled using an equivalent single pile model with p-y and t-z curves modified for group action. The loads on the pile group were checked using a separate model of the pile group where the maximum foundation reactions were applied to the pile group top. Loads and deflections from the pile group model were compared to the structural model to insure that the group model and the structural pile model had similar behaviors.

Stress interaction ratios for the static 100 year ice loading from -X, -XY, and -Y directions are shown in Figures 10, 11, and 12. No interaction ratios exceeded 0.93. The -X direction is down inlet (maximum loading condition), the -Y direction is across inlet. The same ice loading was used for all directions. The maximum current was used for the -X and -XY directions. No current was included in the -Y direction because the current cannot flow across the inlet. The structure passes the ice design level criteria. Note that Section 17, Figure 17.5.2b only requires 85% of the 100 year loading to be applied. For this analysis, 100% of the loading was applied.

Design Level - Earthquake

The 200 year site specific earthquake spectrum was applied to the structure and no interaction equations exceeded 0.98 as shown in Figure 13. The second highest IR was 0.96. Both of these members are 14 foot diameter members. The internal diaphragms and stiffeners were not included in the computer model. If these are added, the IRs would be lower. The highest IR in the bracing members was 0.93. This analysis check is not required by Section 17, but the structure passes current API RP2A-WSD, 20th Edition earthquake linear criteria.

A.5.4 - Ultimate Strength - Ice

To run a non-linear ultimate analysis, the X bracing in the computer model was changed to run from leg node to leg node with no center node because no nodes can have only non-linear members framing into it. The X braces were modeled as Marshall Struts, the leg members as Beam Columns. The revised member numbers are shown in Figure 14.

The load deflection curves, load level at first IR=1.0, reference level loads, ultimate capacity loads, and the RSR for the -X, -XY, and -Y directions are shown in Figures 15 to 20. The RSR was determined based on the combined ice and current loads. Both loads were increased to determine the ultimate capacity of the jacket. Full dead and live deck loads, included maximum drilling rig setback and hook loads, were included in the model.

The minimum RSR was 2.25 in the -XY direction. As shown by the figures, this is not the RSR for platform collapse, but the RSR for failure of the first main structural member. The required RSR for ice loading, Section 17, Figure 17.5.2b, is 1.6. This structure passes the ultimate strength check.

Ultimate Strength Check - Earthquake

A previous study by another contractor was completed in 1992. This study is the basis for all earthquake checks. A seismic ductility level analysis using the 1000 year return period earthquake ductility level spectrum was applied to the structure. The ductility representation factors reflecting the fraction of yield to which jacket members are loaded under the ductility level spectrum is shown in Figure 21. The worst case loading resulted in a maximum ductility factor of 2 in the cross bracing. This ductility factor is acceptable.

The platform's foundation was checked to insure satisfactory performance. The original design capacities were used to determine the allowable foundation capacities. The foundation capacity, with API safety factors, was not exceeded in any analyses.

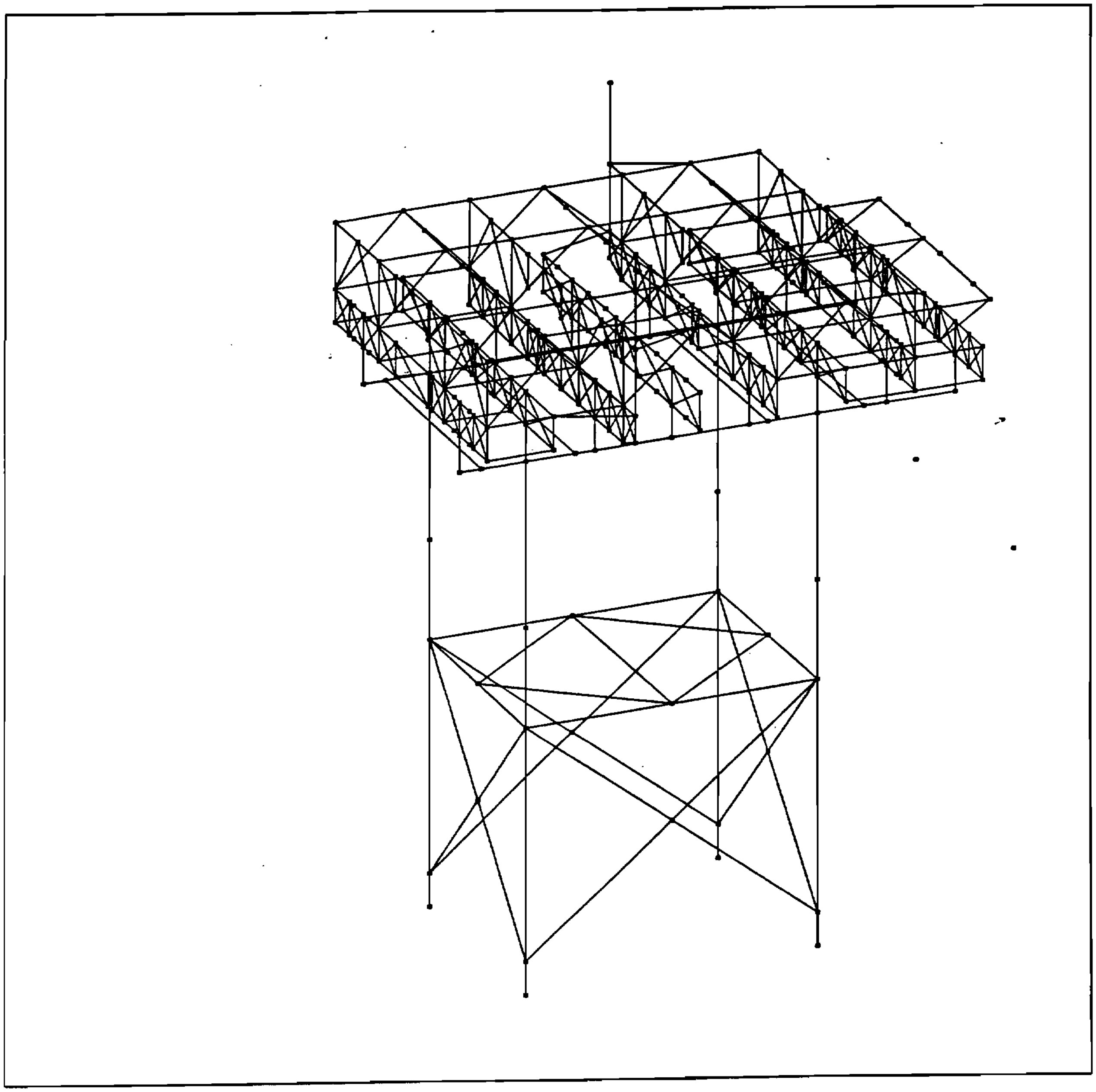
A.6 Mitigation Alternatives

Since this structure passed all Section 17 requirements, no mitigation alternatives are necessary.

A.7 Summary

Platform R passes all Section 17 requirements, both ice design and ultimate, and earthquake, design and ultimate.

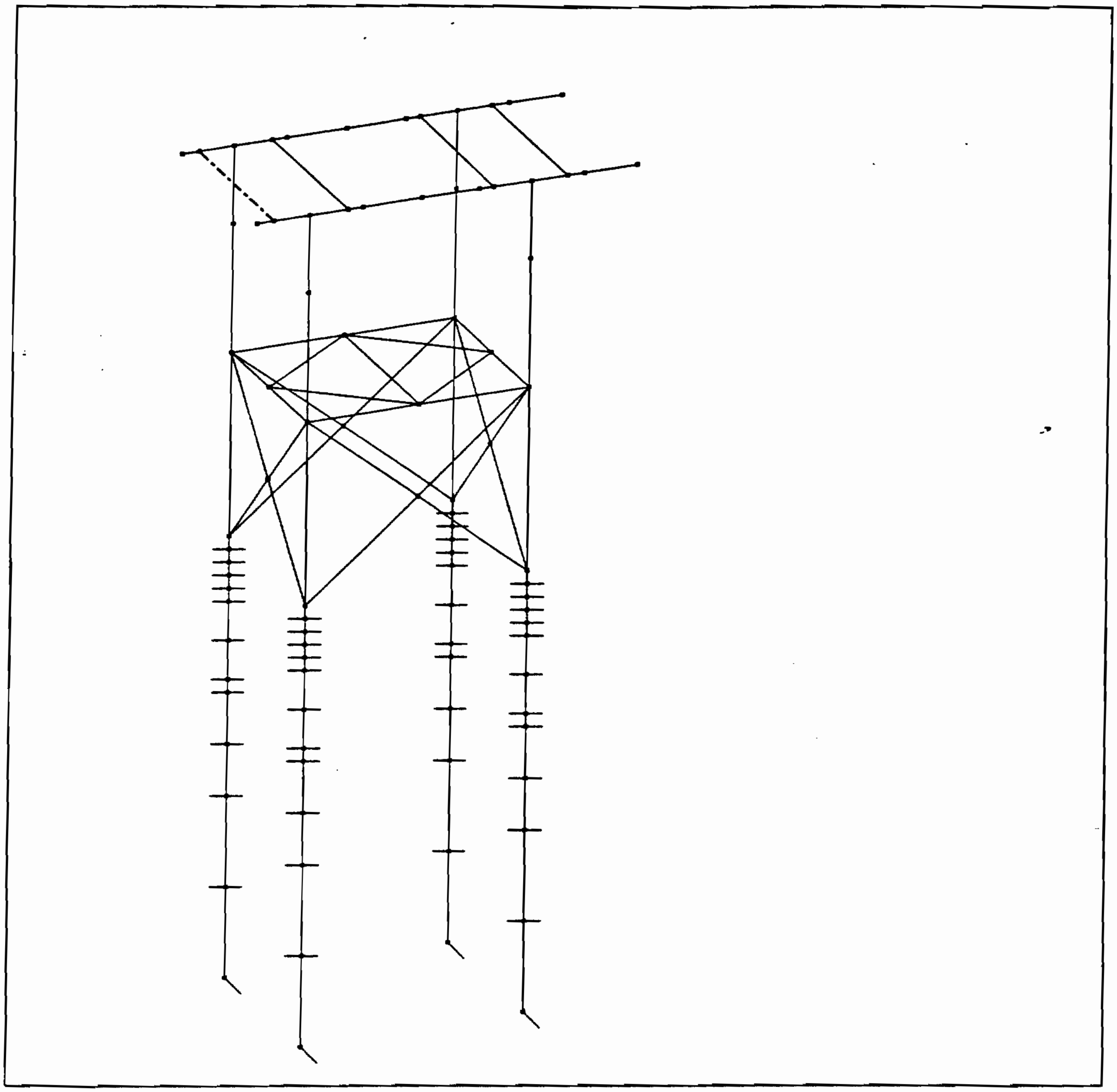
Project: Platform-R Model: Geometry Version: 1 Fri Apr 29 10:47:09 1994 *



CAP Complete Structure

Figure 7

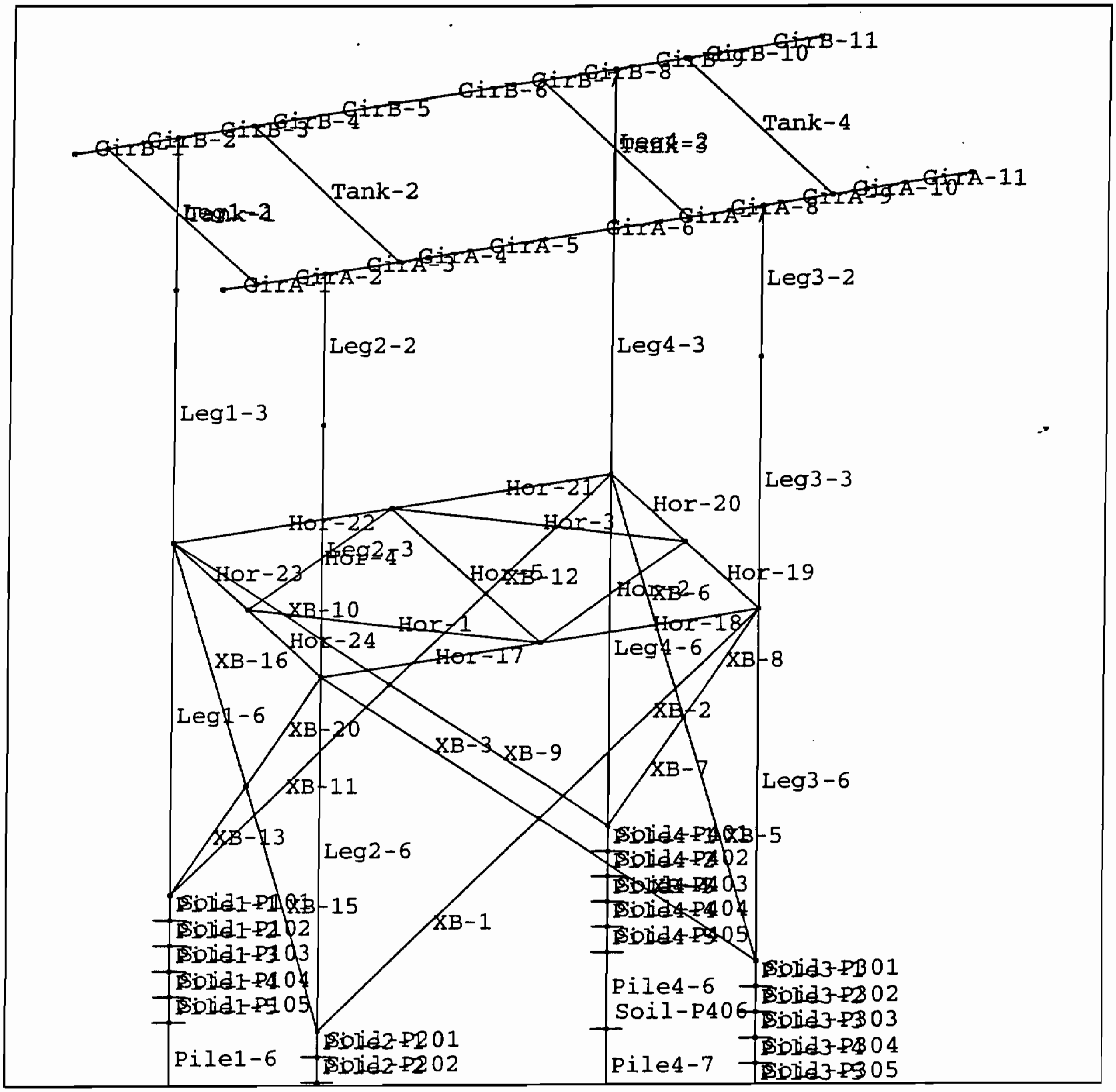
Project: Platform-R Model: Linear Version: 1   Fri Jul 22 13:41:12 1994 *



CAP   Platform R - Linear Computer Model

Figure 8

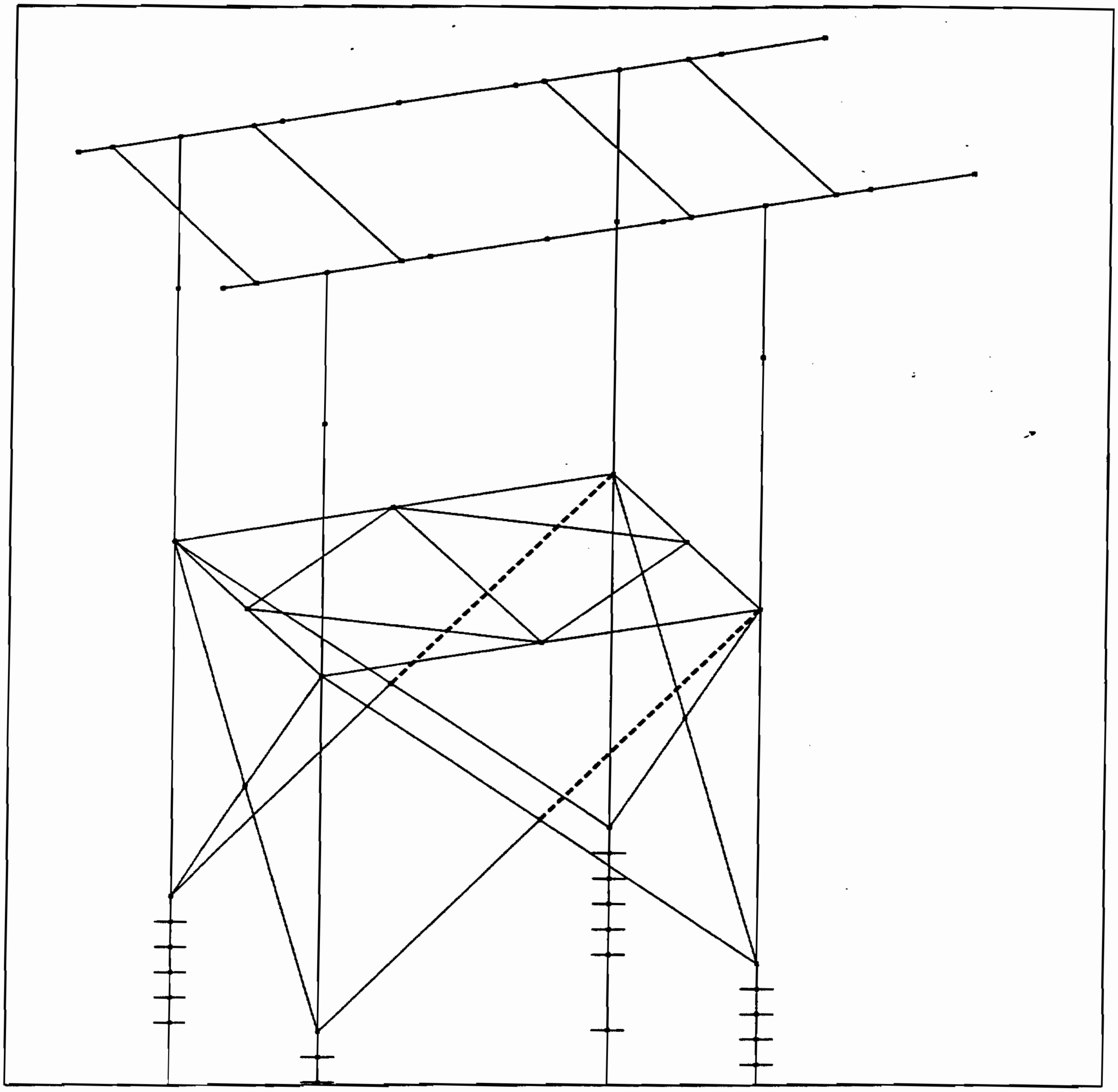
Project: Platform-R Model: Linear Version: 1   Fri Jul 22 13:40:00 1994 *



C∧P z x   Platform R - Linear Computer Model, Members

Figure 9

Project: Platform-R Model: Linear Version: 1   Thu Jul 21 11:23:13 1994 *



CAP z x

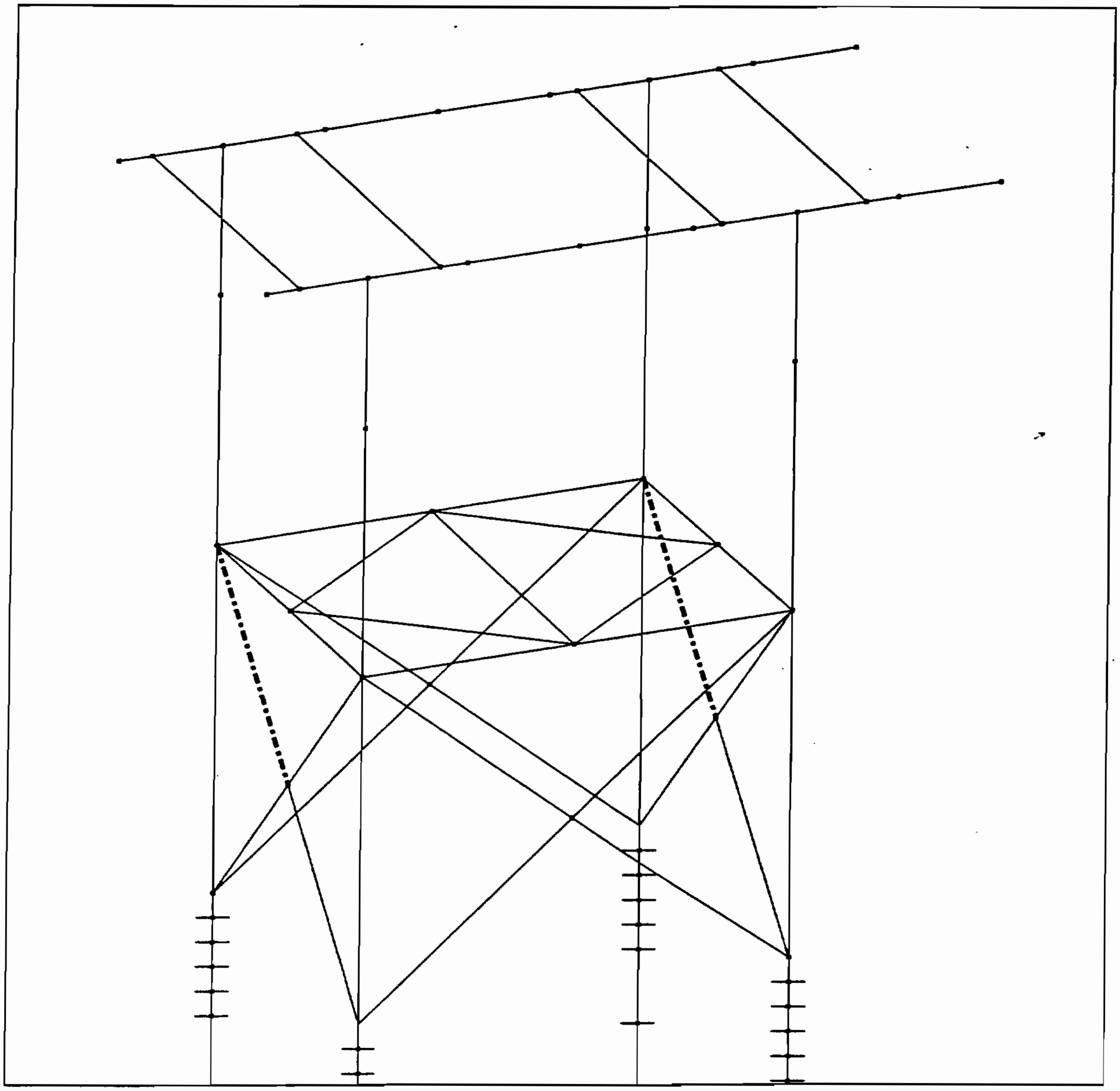
Loads from the East (-X) Direction

IR Legend

0.0000000 -	0.2000000	0.2000000 -	0.8000000
0.8000000 -	0.8500000	0.8500000 -	0.9000000
0.9000000 -	1.0000000	1.0000000 -	Infinity

Figure 10

Project: Platform-R Model: Linear Version: 2 Thu Jul 21 11:24:34 1994 *



CAP

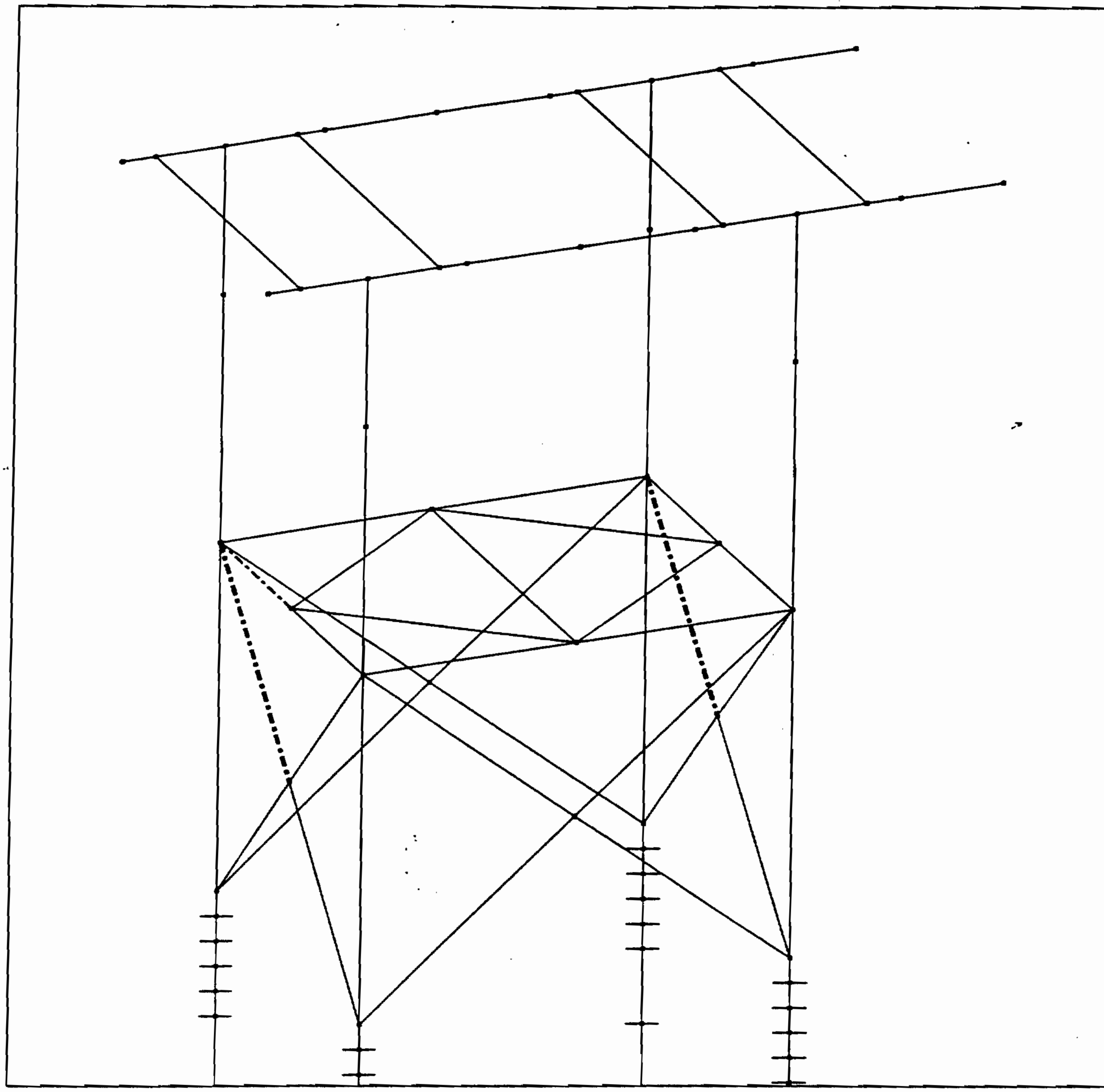
Loads from the Northeast (-XY) Direction

IR Legend

0.0000000 - 0.2000000		0.2000000 - 0.8000000	
0.8000000 - 0.8500000		0.8500000 - 0.9000000	
0.9000000 - 1.0000000		1.0000000 - Infinity	

Figure 11

Project: Platform-R Model: Linear Version: 3   Thu Jul 21 11:25:59 1994 *

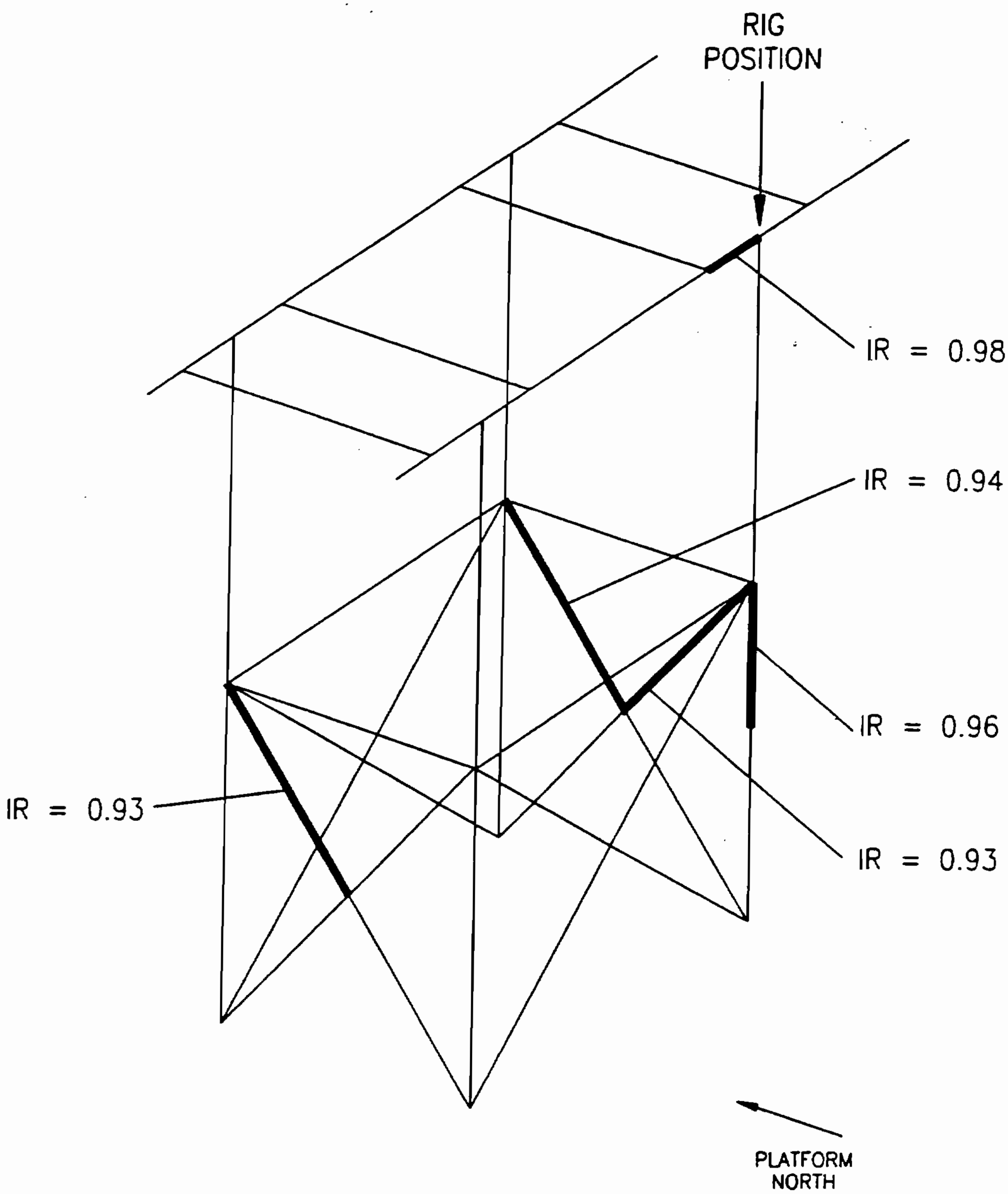


CAP Loads from the North (-Y) Direction

IR Legend

————	0.0000000 - 0.2000000	————	0.2000000 - 0.8000000
-.-.-.-.-	0.8000000 - 0.8500000	---------	0.8500000 - 0.9000000
-..-..-..-	0.9000000 - 1.0000000	━━━━	1.0000000 - Infinity

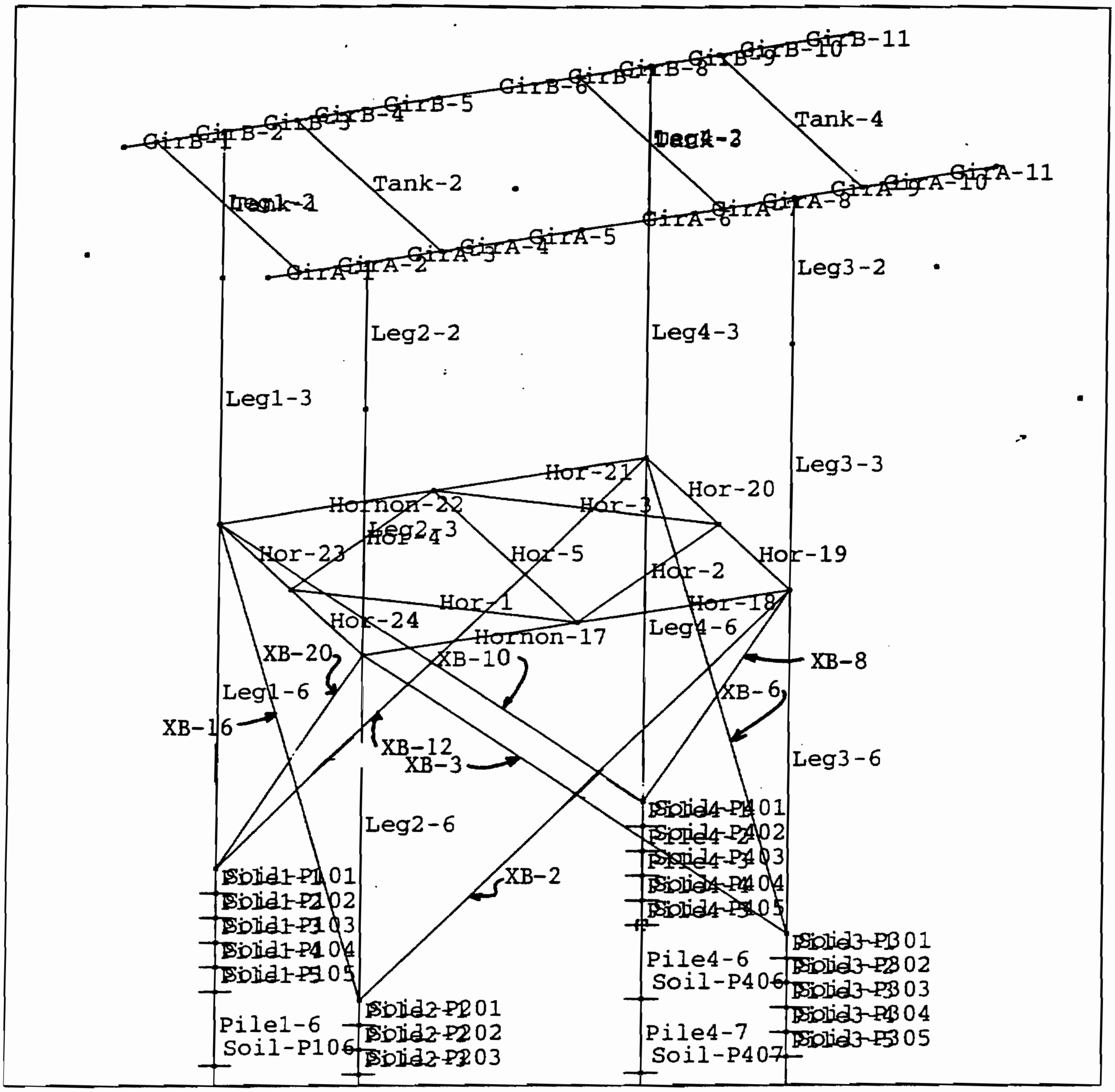
Figure 12



Schematic of Jacket Model Showing
Location of Maximum Interaction Ratios
Strength Level Response Spectrum Loading

Figure 13

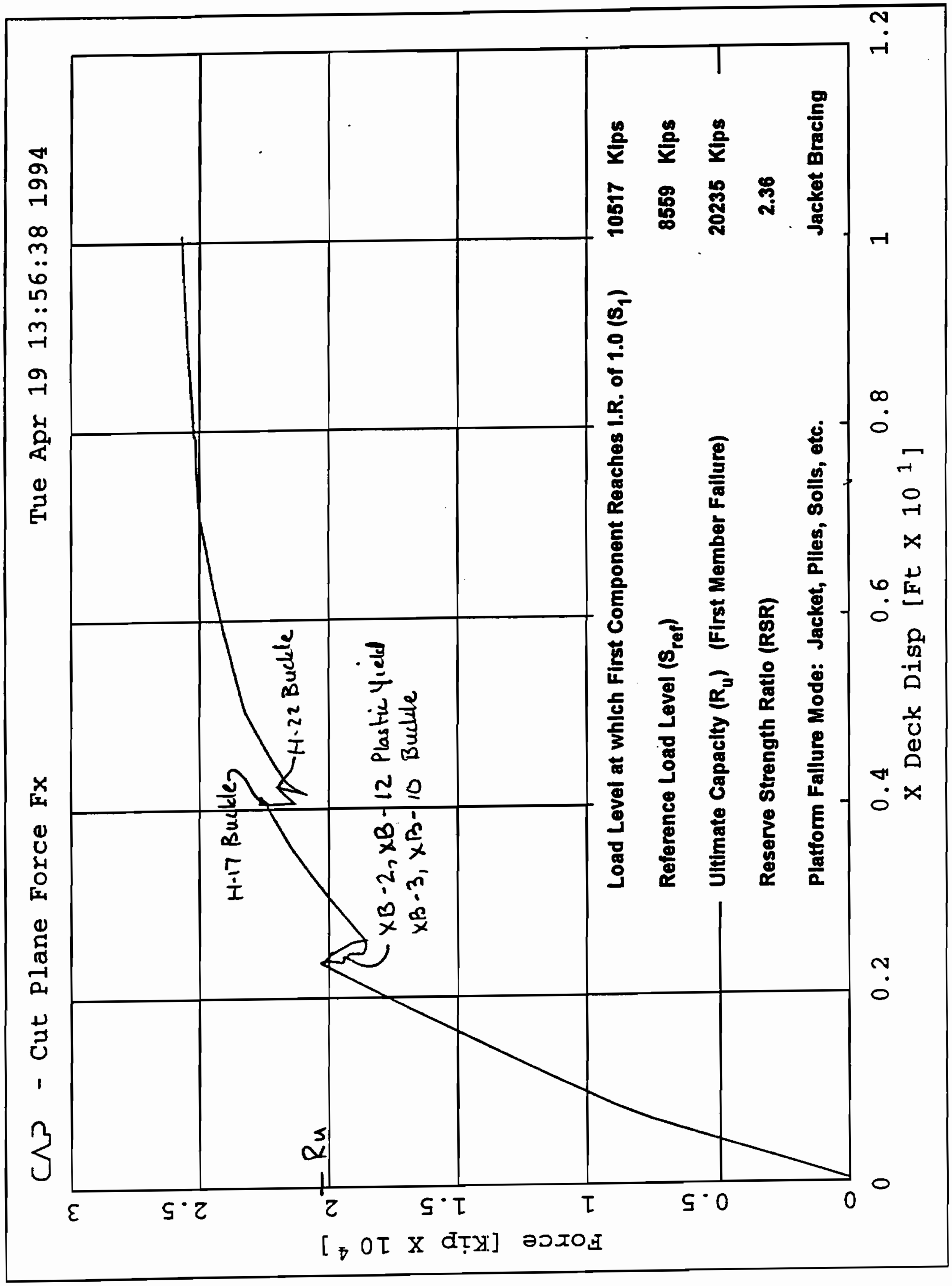
Project: Platform-R Model: Pushover Version: 1 Fri Jul 22 13:37:18 1994 *



CAP

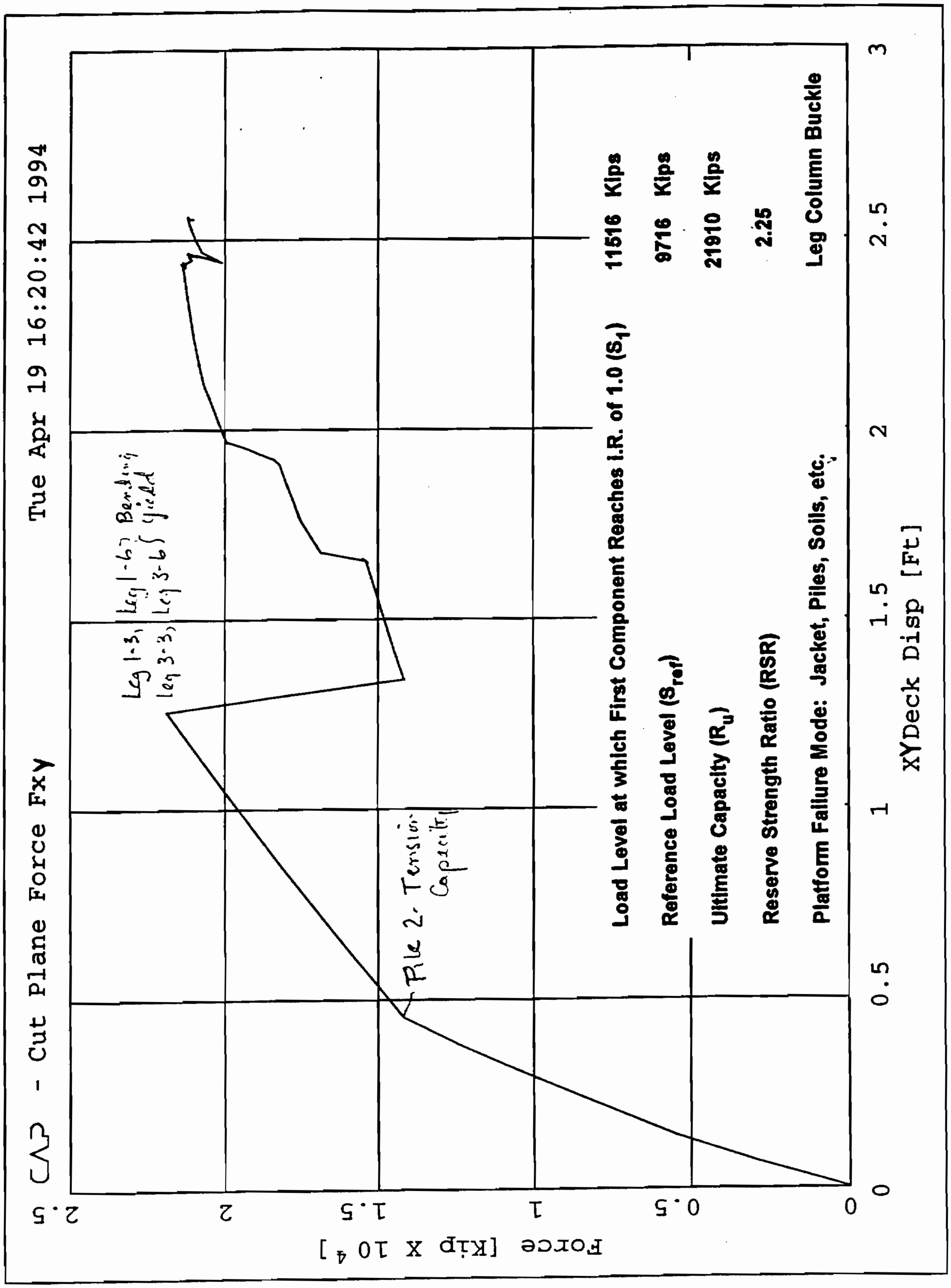
Platform R - Non-Linear Computer Model, Members

Figure 14



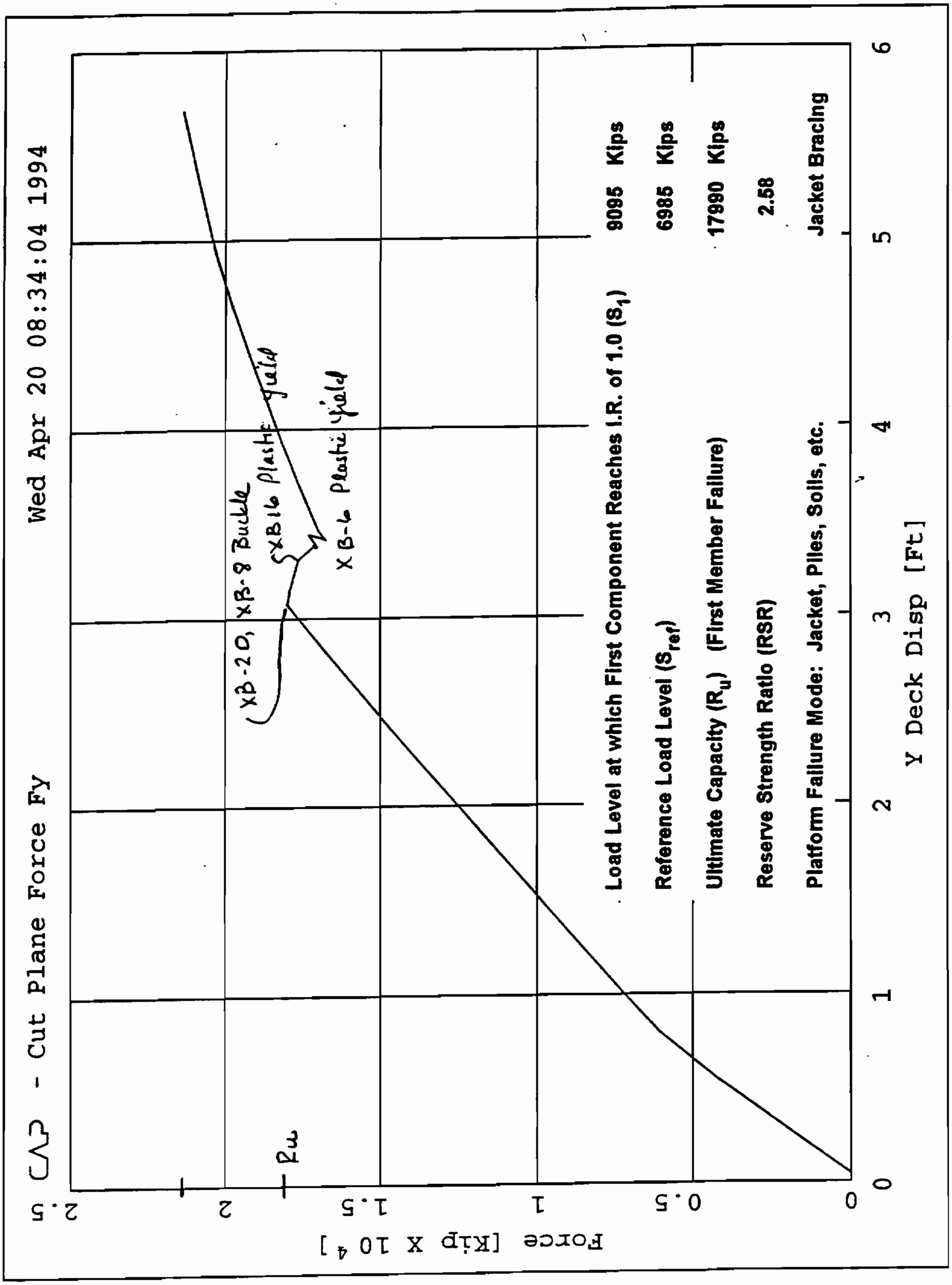
Project: Platform-R Model: Pushover Version: 1
CAP - Cut Plane Force Fx
Tue Apr 19 13:56:38 1994
Force [Kip X 10 4]
X Deck Disp [Ft X 10 1]
Ultimate Capacity in X Direction
0
0.5
1
1.5
2
2.5
3
0
0.2
0.4
0.6
0.8
1
1.2
Ru
XB-2, XB-12 Plastic Yield
XB-3, XB-10 Buckle
H-17 Buckle
H-22 Buckle
Load Level at which First Component Reaches I.R. of 1.0 (S1) 10517 Kips
Reference Load Level (Sref) 8559 Kips
Ultimate Capacity (Ru) (First Member Failure) 20235 Kips
Reserve Strength Ratio (RSR) 2.36
Platform Failure Mode: Jacket, Piles, Soils, etc. Jacket Bracing

Figure 15



Project: Platform-R Model: Pushover Version: 2
CAP - Cut Plane Force Fxy
Tue Apr 19 16:20:42 1994
Force [Kip X 10^4]
XYDeck Disp [Ft]
Ice & Current from Northeast (-XY)
0
0.5
1
1.5
2
2.5
3
Load Level at which First Component Reaches I.R. of 1.0 (S1) 11516 Kips
Reference Load Level (Sref) 9716 Kips
Ultimate Capacity (Ru) 21910 Kips
Reserve Strength Ratio (RSR) 2.25
Platform Failure Mode: Jacket, Piles, Soils, etc. Leg Column Buckle
Pile 2. Tension Capacity
Leg 1-3, Leg 3-3,
Leg 1-6, Leg 3-6
Bending yield

Figure 16



Project: Platform-R Model: Pushover Version: 3
CAP - Cut Plane Force Fy
Wed Apr 20 08:34:04 1994
Force [Kip X 10 4]
Y Deck Disp [Ft]
Load in Y Direction
Load Level at which First Component Reaches I.R. of 1.0 (S1) 9095 Kips
Reference Load Level (Sref) 6985 Kips
Ultimate Capacity (Ru) (First Member Failure) 17990 Kips
Reserve Strength Ratio (RSR) 2.58
Platform Failure Mode: Jacket, Piles, Soils, etc. Jacket Bracing
XB-20, XB-8 Buckle
XB16 Plastic Yield
XB-6 Plastic Yield
Ru

Figure 17

-X Direction

Analysis case: 3-D Model

Lateral load level for first member with unity check = 1.0 # 10517 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures * (voluntary)	Component Failure Mode ** (voluntary)	Remarks
16	2.35	20235	XB-3	Strut Buckling	
25	2.45		XB-10	Strut Buckling	
47	2.59		XB-2	Tension Strut	Plastic
56	2.63		XB-12	Tension Strut	Plastic
76	4.05	22415	HORNON-17	Strut Buckling	
89	4.22		HORNON-22	Strut Buckling	

* Indicate name (e.g., element no. in computer model or on sketch of platform) of component(s) which failed during the load step

** Identify failure mode of the component: buckling, yielding, double hinge, etc.

\# Brace, leg, joint, pile or foundation

Figure 18

XY Direction

Analysis case: 3-D Model

Lateral load level for first member with unity check = 1.0 # 11516 **Kips**

Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures * (voluntary)	Component Failure Mode ** (voluntary)	Remarks
9	2.07	14900	Pile 2	Tension Capacity	
18	4.52	21910	Pile 1-3	Bending Yield	
			Pile 1-6	Bending Yield	
			Pile 3-3	Bending Yield	
			Pile 3-6	Bending Yield	

* Indicate name (e.g., element no. in computer model or on sketch of platform) of component(s) which failed during the load step

** Identify failure mode of the component: buckling, yielding, double hinge, etc.

Brace, leg, joint, pile or foundation

Figure 19

-Y Direction

Analysis case: 3-D Model

Lateral load level for first member with unity check = 1.0 # ......9095...... Kips

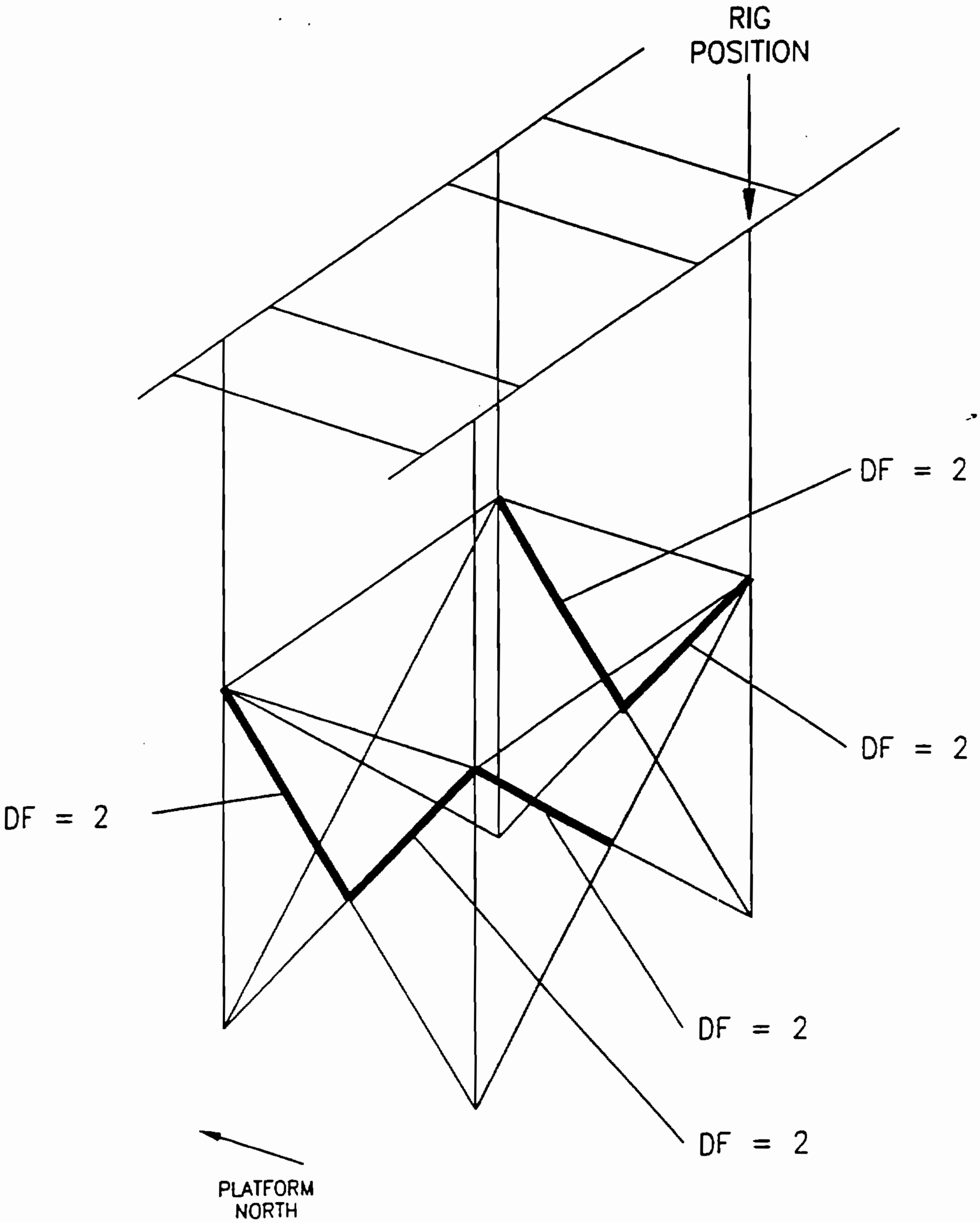
Load Step	Lateral Displacement at Deck Level ft.	Lateral Load Kips	Element Failures * (voluntary)	Component Failure Mode ** (voluntary)	Remarks
17	3.08	17990	XB 20	Strut Buckle	
17	3.08	17990	XB 8	Strut Buckle	
28	3.31		XB 16	Tension Strut	Plastic
38	3.44		XB 6	Tension Strut	Plastic

* Indicate name (e.g., element no. in computer model or on sketch of platform) of component(s) which failed during the load step

** Identify failure mode of the component: buckling, yielding, double hinge, etc.

\# Brace, leg, joint, pile or foundation

Figure 20



Schematic of Jacket Model Showing Location of Maximum Ductility Factors Ductility Level Response Spectrum

Figure 21

PLATFORM "S"

1.0 PLATFORM INFORMATION

This section provides a brief description of ' general and specific characteristics.

1.1 GENERAL CHARACTERISTICS

Hogan Platform is located about 3.5 miles offshore Carpinteria, California, in Federal Lease OCS P-0166. This conventional twelve-legged platform was installed in a 155 ft water depth site in 1968. While the integrated drilling and production facilities were originally designed to accommodate sixty-six wells, at present only 39 of the conductors are in place and the drilling rig is not on the platform.

The platform has successfully resisted operating cyclic and extreme environmental loads since 1968. However, any future revisions to the platform will require requalification of the platform in accordance with the draft Section 17 of API RP 2A.

Some of the pertinent general characteristics of the platform are summarized on Table 1.1-1. As indicated on the Table, the drilling rig was removed following the completion of the drilling program. Platform inspections also resulted in some modification of the platform in the 1980s.

1.2 SPECIFIC CHARACTERISTICS

The platform deck structure is currently supporting about 2,720 kips of equipment and variable loads. Although a drilling rig is not on the structure, a drilling rig and associated items (i.e., setback, hook load) was considered in-place to account for a potential future drilling program.

The structural steel weight of the deck and the jacket structure was estimated to be about 5,100 kips. This total includes the weight of the piles within each of the 12 legs. The platform legs are typically 40 inches in diameter with a wall thickness of 0.50 inches. The leg joint chords are typically one inch thick and provide just adequate room for 36 inch diameter piles within.

The diagonal braces on each elevation typically vary from 24 inches in diameter at the jacket base to 18 inches in diameter near the water surface. The plan level braces typically vary from 18 inches in diameter at the jacket base to 12.75 inches in diameter near the water surface. While most brace members have a wall thickness of 0.375 inches, 0.50 inch thickness is provided to all members in the wave splash zone to provide sacrificial steel.

Some of the other specific information directly applicable to the trial application of Section 17 of API RP 2A, including original design basis and applied loading, is summarized on Table 1.2-1.

TABLE 1.1-1 GENERAL CHARACTERISTICS

CHARACTERISTIC	DESCRIPTION	COMMENTS
1. GENERAL INFORMATION		
Owner - Original		
- Current		
Function	Drilling/Production	
Location	Southern California	In 155 feet water
Configuration	12-legged conventional	Piles grouted to legs
No. of Wells	66 slots	39 wells
Manning Levels	?	
Performance History	Good	
2. ORIGINAL DESIGN		
Design Contractor	McDermott	
Design Drwgs/Specs.	Yes	
Design Code	Unknown	
Design Criteria - Wave	?	Only a 43 feet wave at
- Wind	?	max. tide will hit the
- Seismic	Yes	lower chord bottom
Deck Clearance Elev.	+32.0 feet	
Operational Criteria	?	
Soil Data	Yes	
Pile Size & Penetration	12 - 36"	
Conductor Size & Penetration	39 - 18"	
Appurtenances	Fenders (4)	
3. CONSTRUCTION		
Fabr/Install Contractors/Date	McDermott	
"As Built" Drawings	?	
Construction Specifications	?	
Material Traceability Records	?	
Pile & Cond. Driving Records	?	
Pile Grouting Records	?	
4. PLATFORM HISTORY		
Environmental Loads		
Operating Loads	Reduction > 700 kips	
Survey/Maint. Records	Yes	
Repair Descriptions & Dates	Yes	
Modification Descr. & Dates	Yes, 1981/1982	
5. PRESENT CONDITION		
Deck Configuration	As-designed	
Deck Loads	Reduced to 2720 kips	
Deck Clearance Measurements	?	
Prod. & Storage Inventory	?	
Appurtenances		
Wells - No., Size etc.	39 - 18-inch	
Level I Surveys (Above Water)	Yes	
Level II Surveys (Below Water)	Yes	

NOTE: See Section 1.2 for specific data

TABLE 1.2-1 SPECIFIC CHARACTERISTICS

CHARACTERISTIC	DESCRIPTION	COMMENTS
1. ENVIRONMENTAL LOADING		
Wind Loading - Unknown	?	Glenn report (1981) lists:
Wave Loading - Unknown	?	H_{max} 35.0' @ 12.0 sec. &
Seismic Loading -		Crest elev. as +27.7'
Original Base Shear - X dir (kips)	1,695	ORIGINAL DESIGN
Associated O.M. (kip-ft)	268,063	LOADS FOR PLATFORM
Original Base Shear - Z dir (kips)	1,761	FROM 1967
Associated O.M. (kip-ft)	278,567	
1981 Base Shear - X dir (kips)	1,567	1981 McDermott Data
Associated O.M. (kip-ft)	243,608	prior to revisions
1981 Base Shear - Z dir (kips)	1,593	
Associated O.M. (kip-ft)	248,823	
2. OTHER LOADING		
Weight of Deck Equipment		
and Steel - 1967 Estimate	7,320	With two drilling rigs
- 1981 Estimate	4,580	With no drilling rigs
Weight of Jacket and		
and Appurtenances	3,122	Including piles within
Weight of Piles - above ML	included above	legs
- below ML		
Net Buoyancy - Buoyancy minus		
ballast/grout	1,400	
3. MEMBER SIZES		
Legs with Pin Piles	40.0" OD x 0.50"	
Pin Piles	+ 36.0" OD x 1.500"	
Diagonal Braces - Max.	24.0" OD x 0.375"	
Diagonal Braces- Min.	24.0" OD x 0.375"	
Plan Level Braces - Max.	18.0" OD x 0.375"	
Plan Level Braces - Min.	12.75" OD x 0.375"	

PART A: PLATFORM ASSESSMENT

A.1 PLATFORM SELECTION

An existing platform is to undergo the assessment process if any of the following conditions listed occurs:

- Addition of personnel
- Addition of facilities
- Increased loading on the structure
- Damage found during inspections

Several platforms were reviewed to identify one for the trial application API RP 2A, Section 17. Hogan platform, located offshore Carpinteria, Southern California was chosen for two reasons:

(1) The review of available data has shown that none of the platform assessment initiators listed above has occurred. However, applicable platform design criteria, methodology for the computation of applied loads and the formulations for determining component member and joint capacities have changed substantially. Thus, Hogan platform is likely to be subjected to an earthquake loading much more severe than that envisioned only fifteen years ago. Thus, Hogan platform was considered suitable for trial application of Section 17.

(2) Hogan platform may be used to drill additional wells in the near future, requiring careful assessment of the platform reserve capacity to resist both increased deck loading and a more severe seismic loading. Thus, Hogan platform is well suited for trial application.

A.2 CONDITION ASSESSMENT

A.2.1 General

Assessment of the platform was made to document all pertinent parameters that were grouped under the categories of:

- General information
- Original design
- Construction
- Platform history
- Present condition

The summary findings on the above categories are presented on Table 1.1-1. While the structure appears to be in good condition for a unit to have been in operation for over 25 years, however, it has been subjected to modifications over the years.

A.2.2 Surveys

Surveys carried out over the years and the current assessment indicate the following:

- Pin piles within each leg were grouted, thus increasing platform load carrying capacity.
- Drilling rig and some of the associated equipment were removed. Thus, the overall design functional loads are in excess of current deck loading.
- Platform did sustain some damage over the years, including cracked joints and dented/lost tubulars. Corrective measures were taken in the 1980s.

A.3 CATEGORIZATION

The exposure categories applicable to Hogan platform are as follows:

A.3.1 Life Safety

Of the three categories for life safety, the one applicable "**manned, non-evacuated**" is underlined:

- **Manned, non-evacuated**
- Manned, evacuated
- Unmanned

A.3.2 Environmental Impact

Environmental impact is defined either as "significant" or "insignificant." This platform is identified to have "**significant**" impact due to its operational characteristics.

This categorization results in the selection of the following assessment criteria:

For Design Level (DL) Analysis:	Extratropical storm is considered for platform site. Maximum applicable wave height is 34.0 feet.
	Strength Level Earthquake (SLE) spectra was developed.
For Ultimate Strength (US) Analysis:	Storm wave height for ultimate strength analysis should yield an acceptable RSR. A 46.0 feet wave crest reaching lower deck chord may be considered.
	Ductility Level Earthquake (DLE) spectra was defined to yield a base shear equal to twice the SLE base shear.

A.4 DESIGN BASIS CHECKS

The first query on determining the design basis checks is the location of the platform. Hogan platform is Offshore Southern California, subjected to much higher seismic loading than loading associated with extreme extratropical storms. It was designed to meet a reasonable seismic loading. However, seismic criteria applicable to both onshore and offshore design work in California has become more severe in the last two decades.

Thus, the platform requires sequential analysis checks as discussed in the following section.

A.5 ANALYSIS CHECKS

A.5.1 Design Criteria and Loads

Metocean criteria used during the original design are not known. Glenn report of 1981 recommends the use of 35.0 feet maximum wave height for this location. Current API requirements for this site require the use of 34.0 feet with an associated wave period of 12.0 seconds.

- Maximum wave and wind loading on the platform, based on 20th Ed. of API RP 2A, is 625 kips along platform x-axis.

- Maximum wave and wind loading on the platform, based on 20th Ed. of API RP 2A, is 601 kips along platform y-axis.

The original design is based on a 1,695 kip base shear acting along the platform x-axis while a 1,763 kip base shear is acting along the platform y-axis. The original base shears are much higher than storm environment loads computed now. The same design base shears are likely to be lower than seismic loads based on current criteria.

Application of draft Section 17 requirements resulted in the selection of wave heights, wave periods, current velocities and the computation of a series of parameters. A summary of omnidirectional metocean criteria is presented in Appendix A.

A complete computer model of the platform was developed and the environmental loads generated based on the metocean criteria summarized in Appendix A. Resulting base shear and overturning moments are summarized on Table A.5.3-2.

A.5.2 Screening

During the development of draft Section 17 an approach defined as "screening" to allow passing of the platform based on the assessment of applied loading was considered. Establishing the adequacy of a platform based on such a simplified procedure requires a thorough understanding of applied loads on the platform and the response of the platform to these applied loads.

Since substantial uncertainties exist as to the design basis, applied loading, stress distribution within the platform as well as the condition of the platform components, the "screening" procedure is considered not applicable.

A.5.3 Design Level Analysis

Deck Height Criteria:

Based on Section 17, applicable extratropical storm wave height of 34 feet can have a crest height of about +28 feet during high tide. Since the lower deck chord is at +32 feet elevation the crest clears the deck with ample freeboard.

For the ductility level extreme storm the wave crest may reach the lower deck of the platform. ~~However, this is acade~~mic as the storm environment loads do not control the overall platform design or its survivability.

Other Parameters:

Other parameters used in the determination of the applied environmental loads are summarized on Table A.5.3-1.

TABLE A.5.3-1

DESCRIPTION	MAGNITUDE	COMMENTS
Storm Tide (ft)	6.0	
Marine Growth from		
Inspections h > 5'	0.0	Inches on diameter
h = 0'	6.0	
h = -155'	1.0	
Drag Coefficient		
C_d @ h > 5'	0.65	
C_d @ h < 5'	1.05	
Inertia Coefficient		
C_m @ h > 5'	1.6	
C_m @ h < 5'	1.2	
Wave Spreading		
Factor	1.00	

The SLE response spectrum acceleration used in this study is defined with the following formulas for Pseudo Acceleration PSA (defined in "g"):

$PSA = 0.25$ for period $T < 0.05$ seconds

$PSA = 100\ T^{2.0}$ for $0.05 < T < 0.10$ seconds

$PSA = 1.00$ for $0.10 < T < 0.50$ seconds

$PSA = 0.392\ T^{-1.352}$ for $0.50 < T < 4.00$ seconds

PART A: PLATFORM ASSESSMENT

The appropriate design response acceleration spectra for Hogan platform was not available. Based on an input from Fugro West, Inc. (See Appendix AAA9. References) a response spectrum (i.e., acceleration versus period) given above was prepared for this study. Although this SLE spectra is assumed to be conservative and appropriate (i.e., for this study), any future work would require an SLE spectra developed based on seismic hazard analysis.

Applied Loads:

Storm environmental loads generated for the DL analysis range from a low of 601 kips to a high of 625 kips (See Table A5.3-2). Application of SLE response spectra yields base shears of 2,474 kips along the x-axis and 2,283 kips along the y-axis, respectively. Applying 100% SLE loading along both x- and y-axes result in an overall diagonal resultant load of over 3300 kips, the actual magnitude depending on whether a SRSS or a CQC method is chosen.

Table A5.3-2 provides a summary of the applied loads on the platform.

TABLE A.5.3-2 ENVIRONMENTAL LOADING BASED ON DESIGN LEVEL ANALYSIS

DIRECTION DESCTIPTION	PLATFORM X-AXIS	PLATFORM Y-AXIS	COMMENTS
WAVE DATA			
Height (ft)	34.0	34.0	Omnidirectional
Period (sec)	12.4	12.4	
SLE Criteria	See Appendices		
BASE SHEAR (kips)			
Wind, Wave and Current	625.0	601.0	
SLE	2,474.0	2,283.0	SLE Controls Design Resultant diagonal 3,355 kips

Analysis Validation:

The computer model was first validated by applying symmetrical quality control loads at the deck level in the x-, y-, z-axes and verifying the load path by reviewing the reactions. The applied functional and environmental loads were validated and the platform reactions reviewed.

Platform distortions at the jacket/pile interface and the deck level were then reviewed for their compatibility with the applied loads and the range of viable foundation response.

Member Utilizations:

Platform response to the Design Level loading meets the design criteria. As-designed jacket member utilizations are generally within the allowable limits. However, application of a more severe seismic loading (i.e., an average of 40% increase over original design base shears) results in more than a dozen members exceeding the allowable 1.0 utilization ratio. However, half of these members are within 10% of the limiting ratio and can be shown to meet the criteria by computing member slenderness ratios as a function of supporting joint rigidities.

Based on the SLE spectra used, only four members have excessive combined stress levels. Should this spectra prove to be appropriate, these four members will require further review to determine appropriate corrective measures.

Joint Utilizations

The joint utilization checks reveal that many joint cans do not meet the 20th edition of API RP 2A. A majority of the overutilizations noted fail to meet Section 2.3.6e of the API RP 2A. This requirement, not in existence at the time of the platform design, stipulates that all joint cans are to be designed to the full capacity of the member connecting to the joint. The objective of Section 2.3.6e is to prevent premature collapsing of a platform due to the unzipping effect of joints failing in advance of the members connected to them. However, it should be noted that:

- All primary brace members are connected to leg cans. The leg cans, typically one inch thick, are grouted to the 36 inch pile within the legs. Thus, provided grouting was performed properly, the primary joint cans have the capacity to resist full component member loads.

- Some of the joints not meeting the objective of Section 2.3.6e are internal plan level brace-to-brace connections. Stress levels are generally low and the joints are not likely to develop their full capacities.

The joint cans adjacent to the conductor framing area at each level require further scrutiny. Actual condition of such joints (i.e., defects, cracks, etc.) may have a greater impact on the load path at each plan level.

A.5.4 Ultimate Strength Level Analysis

A Ductility Level Earthquake (DLE) Push Over analysis (i.e., Ultimate Strength) was performed at a load level equal to twice the SLE loading. Thus, the base shears associated with loading along the x- and y-axes were 4,950 and 4,565 kips, respectively. Since the recommended loading combination is to apply 100%, 100%, and 50% of the loading along platform X-, Y- and Z-directions, the resultant lateral DLE loading was equal to 6,710 kips. Push Over analysis was performed up to 140% of the DLE (i.e., 2.8 times the SLE) loading, where the analysis solution failed to converge.

PART A: PLATFORM ASSESSMENT

Deck Height Criteria:

The deck height criteria will be met for extratropical storm waves more than 25% higher than the design level 34.0 feet wave. However, seismic loading controls Hogan platform assessment and no additional effort was expended in evaluating an ultimate strength level wave height.

Environmental Loads:

Ductility Level Earthquake loading was taken equal to be twice the design (i.e., strength) level loading applied on the platform. As stated above, the base shears associated with the DLE loading were 4,950 kips along the x-axis and 4,565 kips along the y-axis, yielding a resultant base shear equal to 6,710 kips along the diagonal axes.

Analysis Validation:

The ultimate strength level analysis model used for the Push Over study is essentially the same as the strength level analysis model and further discussed in Section 7. Applied load paths and platform deformations were reviewed and found to be valid.

Platform Response and Member Capacities:

A total of 29 load increments were applied to reach to the 9,394 kip base shear level, which is 2.8 times the SLE base shear of 3,355 kips. The first tubular to reach its capacity is the 36-inch diameter stab-on leg extending from one platform corner to the deck directly above. This event is associated with load increment no. 11, when the applied lateral load along the platform diagonal axis is equal to 3,690 kips and the deck level lateral displacement is 4.6 inches. It should be noted that the first member to reach capacity does so at a base shear level 10% above the SLE base shear of 3,355 kips. As expected, the next component to reach capacity is the adjacent stab-on leg at load increment no.12. Both legs reach the capacity, forming single hinges.

As illustrated on Table A7.2-1, the next series of components reaching their capacity are the tubular braces in buckling mode, forming double hinges. The components reaching their capacities are also identified on Figures in the Appendices. As illustrated on these figures, the components reaching their capacity are either tubular leg members or brace members connecting the legs.

The upper bound collapse load for Hogan platform may be taken as 2.8 times the SLE load (i.e., Reserve Strength Ratio, RSR = 2.8) when all of the joint cans have the capacity to carry loading associated with full member capacities. This conclusion is valid even if some of the joint can capacities are degraded due to possible grouting imperfections. However, if a large number of joint cans are grouted improperly and may not be developing the connecting member capacities, platform response to the applied loads will change and the RSR will be reduced.

Considering that component members to reach capacity first were the stab-on legs and the number of diagonal braces reaching capacity did not occur until load step 18 (i.e., RSR = 1.8) a 2.0 RSR should be achieved even with joint cans not having full capacity. A more comprehensive Push Over analysis, accounting for elasto-plastic capacity of each joint based on "as-is" condition, should confirm the validity of adequacy of RSR. One such approach, accounting for joint can deformations, was attempted during this study. However, time constraints of the study and lack of information on "as-is" condition of the joint cans did not allow incorporation of joint deformation/capacity parameter into this study.

Further discussion on the findings is presented in Section 7.

A.6 MITIGATION ALTERNATIVES

The structure is able to resist environmental loads more than twice the strength level loading.

Thus, while no mitigation alternatives are considered at present, key diagonal members and joint cans are identified for a closer review during next planned platform inspection. When such a review is performed, an up-to-date SLE response spectra based on a seismic hazard analysis should be developed.

A.7 SUMMARY OF FINDINGS AND CONCLUSIONS

A.7.1 Design Level Analysis

Platform Deformations

The deck displacements are linear with increase in the applied loads for the design level analysis. At full design level loading the deck lateral displacements average about 7 inches.

Member Utilizations

Application of a more severe seismic loading (i.e., an average of 40% increase over original design base shears) results in more than a dozen members exceeding the allowable 1.0 utilization ratio. However, half of these members are within 10% of the limiting ratio and can be shown to meet the criteria by computing member slenderness ratios as a function of supporting joint rigidities.

Implication of Findings - Conclusions

Based on the SLE spectra used, only four members have excessive combined stress levels. Should this spectra prove to be appropriate, these four members will require further review to determine appropriate corrective measures.

A.7.2 Ultimate Strength Level Analysis

An ultimate strength level analysis consisted of pushover analyses of the platform by applying 100% of the DLE loading in both platform X- and Y-axes and applying 50% of the DLE loading along the vertical Z-axis of the platform. Hogan platform has an RSR of over 2.8. At the 29th load increment, with the resultant lateral load (i.e., base shear) of 9,394 kips, the analysis solution failed to converge.

Hogan structure is over 25 years old and some of the components (both tubulars and joints) will not have their "as-designed" capacities. However, even accounting for some degradation, platform appears to have adequate reserve capacity.

Tracking of Components Reaching Capacity

The pushover analysis was performed until the structure collapsed. Each member component's "capacity ratio" was tracked for each incremental load step. Members reaching capacity are noted on figures with the member number, incremental load step number and whether a single or a double hinge was formed. These figures are presented in the Appendices. A tabular summary of these findings is presented on Table A7.2-1.

PART A: PLATFORM ASSESSMENT

Platform Response and Reseve Capacity

The platform response was linear up to load increment (i.e., step) 18 when the resultant base shear reached 6,039 kips (i.e, RSR of 1.8). Platform nonlinearity increased beyond this load step until a converging solution was not obtained on step 29 at RSR of 2.8.

Conclusions

For the purposes of this trial application only, the ultimate strength level analysis indicates that this platform would pass the requalification assessment based on the Draft API RP2A Section 17 requirements.

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
2	0.91	671			
4	1.82	1,420			
6	2.74	2,013			
8	3.67	2,684			
10	4.59	3,355	None		
11		3,690	XVD35	Single Hinge	DH Step 12
12	5.52	4,026	XVC35	Single Hinge	DH Step 13
14	6.53	4,697	DAB11	Buckling-DH	
			DAB14	Buckling-DH	
			DAB34	Buckling-DH	
			XVB15	Double Hinge	
15		5,032	DD211	Buckling-SH	DH Step 16
			DD214	Buckling-DH	
			DAB21	Buckling-DH	
			DAB24	Buckling-SH	DH Step 16
16	7.56	5,368	213	Buckling-DH	
			DAB13	Buckling-SH	DH Step 17
			DBC12	Buckling-DH	
			DC211	Buckling-SH	DH Step 17
17		5,703	DB211	Buckling-SH	DH Step 18
			DD213	Buckling-DH	
			DBC13	Buckling-DH	
			DBC14	Buckling-DH	
			DBC22	Buckling-DH	
18	8.60	6,039	D104	Tension-DH	
			313	Buckling-DH	
			413	Buckling-DH	
			D404	Buckling-DH	
			DD212	Buckling-DH	
			DAB12	Buckling-DH	
			DAB22	Buckling-DH	
			DAB23	Buckling-SH	DH Step 19
			DAB33	Buckling-DH	
			DBC11	Buckling-SH	DH Step 20
			DBC21	Buckling-SH	DH Step 21
			DBC32	Buckling-DH	
			DBC34	Buckling-SH	DH Step 19
			XVB25	Buckling-SH	DH Step 22
			XVD25	Buckling-DH	
19		6,274	D504	Buckling-DH	
20	10.23	6,710	113	Buckling-DH	
			D204	Buckling-DH	
			DA211	Buckling-SH	DH Step 22
			DAB31	Buckling-DH	
			DAB32	Buckling-DH	
			DAB33	Buckling-DH	
21		7,045	DBC23	Buckling-DH	
			DBC31	Buckling-SH	DH Step 23
			XVC25	Buckling-DH	

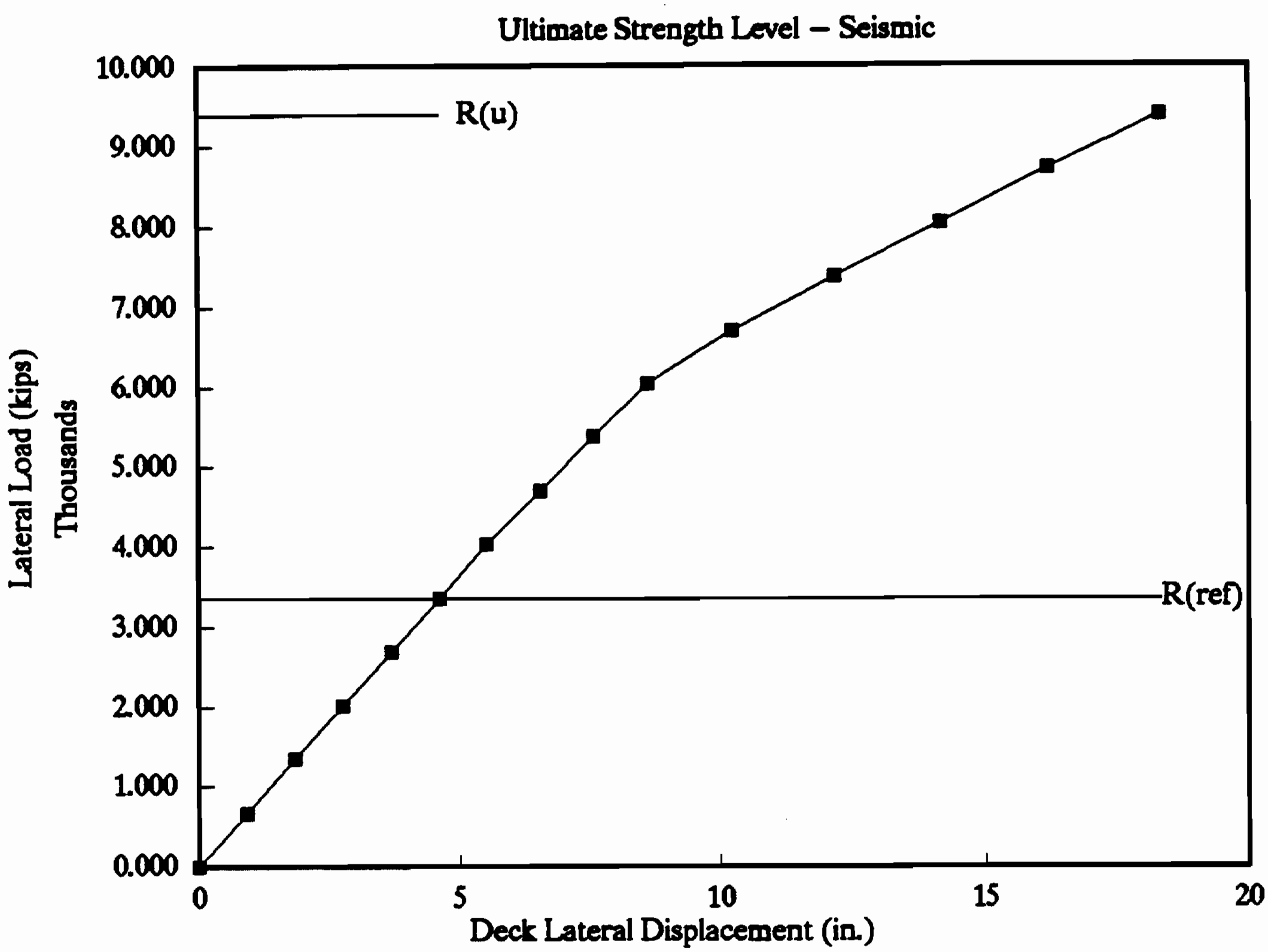
SH = Single Hinge, DH = Double Hinge

Table A7.2-1 (Sheet 1 of 2)
Ultimate Strength Analysis Results - Ductility Level Earthquake

Load Step	Lateral Displ. at Deck (in.)	Lateral Load (kips)	Elements at Capacity	Component Capacity Mode	Remarks
22	12.17	7,381	DC214	Buckling-DH	DH Step 27
			B501	Buckling-SH	DH Step 24
23		7,716	D304	Buckling-DH	
			DB214	Buckling-SH	
			T116	Single Hinge	
24	14.17	8,052	B104	Buckling-DH	
			323	Buckling-DH	
			A404	Buckling-DH	
			DBC24	Buckling-DH	
25		8,387	A104	Buckling-DH	
			B204	Buckling-DH	
			333	Buckling-DH	
			B404	Buckling-DH	
			DA212	Buckling-DH	
			DA213	Buckling-DH	
			XVD15	Single Hinge	DH Step 29
26	16.21	8,723	133	Buckling-SH	DH Step 27
			233	Buckling-DH	
			433	Buckling-DH	
			533	Buckling-SH	DH Step 27
			A204	Buckling-DH	
			C304	Buckling-DH	
			DA214	Buckling-SH	DH Step 27
			DCD14	Tension-DH	
			DCD21	Tension-SH	DH Step 29
			DC231	Tension-SH	DH Step 29
			XVB35	Single Hinge	
			T117	Single Hinge	
27		9,058	B304	Buckling-DH	
			C104	Buckling-SH	
			D501	Single Hinge	
			DCD11	Tension-SH	
			TA17	Single Hinge	
			TB17	Single Hinge	
28	18.31	9,394	423	Buckling-DH	
			513	Buckling-DH	
			T111	Single Hinge	
			T112	Single Hinge	
29	Collapse	9,394 +/-			

SH = Single Hinge, DH = Double Hinge

Table A7.2-1 (Sheet 2 of 2)
Ultimate Strength Analysis Results - Ductility Level Earthquake



Reference Level Load, (S_{ref})	n/a
Strength Level Earthquake Load (SLE)	3,355 kips
Ultimate Strength Level Load (USL)	6,710 kips
Ultimate Capacity (R_u)	9,394 kips
Reserve Strength Ratio (RSR) - to SLE	2.8

Platform Failure Mode: Jacket, Pile, Soils, etc.

Figure 7.2-1
Ultimate Strength Level Load-Displacement Results - Platform Diagonal Axis

PART A: PLATFORM ASSESSMENT

APPENDIX AA

METOCEAN CRITERIA SUMMARY

RESPONSE SPECTRUM ACCELERATION

PART A: PLATFORM ASSESSMENT

METOCEAN CRITERIA SUMMARY

METOCEAN CRITERIA FOR EACH DIRECTION	DRAFT DLA	SEC. 17 DLE	API RP 20TH	COMMENTS
NORTH				API RP 2A 20TH
Wave Height, H(ft)	34.0		45.0	Range 35-55
Wave Period, T (sec)	12.0		12.6	1/18 slope taken
Current				
Blockage Factor	0.80		0.80	Assumed same
Velocity (knots)	1.00		1.00	
Velocity (fps)	1.67		1.67	
Direction	North		North	
Wave dir V (fps)	1.67		1.67	
Computed Parameters				
V / gT	0.004		0.004	
d / gT^2	0.035		0.035	
T_a / T	1.035		1.035	
Apparent Period, T_a	12.4		13.0	
H / gT_a^2	0.007		0.008	
d / gT_a^2	0.033		0.030	
Wind (1 hr at 10m)				
velocity (knots)	45		50	
velocity (mph)	52		58	
SOUTH				API RP 2A 20TH
Wave Height, H(ft)	34.0		45.0	Range 35-55
Wave Period, T (sec)	12.0		12.6	1/18 slope taken
Current				
Blockage Factor	0.80		0.80	Assumed same
Velocity (knots)	1.00		1.00	
Velocity (fps)	1.67		1.67	
Direction	South		South	
Wave dir V (fps)	1.67		1.67	
Computed Parameters				
V / gT	0.004		0.004	
d / gT^2	0.035		0.035	
T_a / T	1.035		1.035	
Apparent Period, T_a	12.4		13.0	
H / gT_a^2	0.007		0.008	
d / gT_a^2	0.033		0.030	
Wind (1 hr at 10m)				
velocity (knots)	45		50	
velocity (mph)	52		58	

RESPONSE SPECTRA ACCELERATION

PERIOD (SEC)	PSEUDO ACCELERATION (% g)	COMMENTS
0.00	0.25	Constant PSA = 0.25 to T = 0.05 sec
0.05	0.25	
0.07	0.49	T > 0.05 sec
0.09	0.64	PSA = 100 $T^{2.0}$
0.10	0.81	T < 0.10 sec
0.30	1.00	T > 0.10 sec
0.50	1.00	PSA = 1.0 to T < 0.50 sec
0.75	0.578	T > 0.50 sec
1.00	0.392	PSA = 0.392 $T^{-1.392}$
1.50	0.227	to T < 4.0 sec
2.00	0.154	
2.50	0.114	
3.00	0.089	
4.00	0.060	
1000.0	0.001	

PART A: PLATFORM ASSESSMENT

APPENDIX AAA

ANALYSIS DETAILS

PART A: PLATFORM ASSESSMENT

AAA1. INTRODUCTION

This section presents the background information applicable to the assessment of Hogan platform. The analysis details discussed in this section relate to the computer modeling, applied loads, analytical procedure and the specifics of some of the findings.

AAA2. COMPUTER MODEL

The platform jacket, deck and foundation structure and applicable appurtenances, including conductors, caissons and risers, were modelled using the structure analysis and design software ASADS (Advanced Structures Analysis and Design System).

TOPOLOGY

A three-dimensional space frame computer model generated for the DL analyses was revised to meet the US level "Push Over" analysis objectives. Conductor, caisson and riser members were modelled only for the purpose of load generation. The conductors offer substantial resistance to applied environmental loading and were included in the analysis. Member sizes were taken from available platform sketches and work undertaken by others.

PROPERTIES

Tubular members were typically input as one-segment prismatic tubulars with specified outside diameter and thickness. Built-up plate or wide flange girder members were input as one-segment prismatic girder members unless specific size variations are indicated on the drawings. Input will be further revised to reflect on-going design effort.

Equivalent pile-soil matrices were determined for a range of nonlinear foundation response to DL and US analysis loadings. The validity of the assumed matrices was confirmed prior to the initiation of "Push Over" analysis. At each step of the US analysis, platform stiffness matrices were recomputed.

Jacket appurtenances consist of conductors, caissons, and risers. These are all included in the model for load generation. Only the structural members remain active for the Push Over analyses.

AAA3. FUNCTIONAL AND ENVIRONMENTAL LOADS

FUNCTIONAL LOADS

Deck equipment and variables load were computed based on available information. The following table provides a summary of functional loads.

All equipment and variable loads were input as concentrated loads at the upper and lower deck levels. Steel selfweight, buoyancy and ballast were internally computed.

LOADING	DESCRIPTION	MAGNITUDE	COMMENTS
1	All Deck Equipment and Variables	2,521.5	
2	Drilling Rig & Variables	1,425.0	Planned future utilization of rig
'DEAD'	Steel Selfweight	5,102.0	
'BUOYANCY'	Net Buoyancy	-2,053.0	Buoyancy minus ballast

ENVIRONMENTAL LOODS

Although environmental loads do not control platform assessment, based on API RP 2A recommendations wind, wave and current loads acting on the Hogan platform were generated. A resultant load of 625 kips along the platform x-axis and 601 kips along the platform y-axis are substantially smaller than the Strength Level Earthquake loads.

AAA4. MASS MODEL

The mass model of the platform was generated to accurately represent Hogan platform. The mass model incorporated the deck mass, jacket mass, appurtenance mass, added mass, mass due to marine growth and contained mass. Added mass is the mass of water assumed to move in unison with the member as it displaces through the water and is dependent on the direction of movement of the member. For tubulars moving perpendicular to its axis, a value of mass numerically equal to the water mass displaced by the submerged member will be used. contained mass is the fluid contained or enclosed by the members.

For this analysis, the water depth was taken to be mean sea level (MSL). All members below MSL will therefore have added mass. All jacket leg members below MSL are assumed to be flooded; all other members will be assumed to be unflooded.

All platform mass were appropriately lumped at modelled nodes in a manner that ensures the overall center of mass is maintained. The total platform mass output was confirmed during the execution of analyses.

COMPUTER GENERATED MASS

The program ASADS calculates the added mass, entrained water mass, marine growth mass and structure mass for every active tubular and conical structure member in the model. Structure mass is determined for all member types excepting equivalent stiffness matrices.

The added mass is assumed equal to the displaced water mass of all tubulars below MSL. The contained water mass is generated for all flooded members. Marine growth is accounted for on all applicable members when evaluating added mass. The marine growth mass were generated based on the design marine growth profile specified and the outside diameter of the jacket members.

The structure mass of all modelled members was generated based on the member properties in the model. An allowance was made to account for additional node and stiffening steel. Sacrificial steel thickness provided to all members in the splash zone were neglected for stress determination but its mass were included for mass and load generation.

The pile mass above mudline and attributable hydrodynamic mass were internally generated by ASADS. Total foundation mass was output and confirmed prior to analysis.

EXPLICITLY INPUT MASS

The mass that cannot be automatically generated by ASADS, such as deck equipment and flare structure, were hand-calculated prior to analysis and allocated to the appropriate center of gravity positions on the model.

Masses on the deck were lumped at primary truss intersection nodes according to the overall center of mass. Weights for consumable or variable items were included in the deck mass while live loads such as the hook load were not.

Mass due to non-modelled components were explicitly input; all other mass was internally generated by ASADS based on input member properties. Conductors, caissons and risers were modelled for the purposes of mass generation. Total deck mass was output and confirmed prior to the analysis.

AAA5. PLATFORM CHARACTERISTICS AND BASIC LOAD CASES

The mass model generated is summarized in the following table:

LOADING	DESCRIPTION	MASS-X	MASS-Y	MASS-Z
A	INERTIAL	172.9	172.9	172.9
B	ADDED	31.0	32.0	19.9
C	FLOODED	34.0	34.0	34.0
D	MARINE GRO	87.8	87.8	87.8
E	ADDED JOINT	205.3	205.3	110.0
TOTAL		531.0	532.0	424.5

PART A: PLATFORM ASSESSMENT

The platform primary natural periods computed were 1.98 and 1.92 seconds along platform x- and y-axes. Overall dynamic participation factors were 0.96, 1.06, and 1.05 along the platform x-, y-, and z-axes. Platform principal mode deformations are illustrated on a series of figures at the end of Appendix AAA.

AAA6. BASIC LOAD CASES

STILLWATER LOAD CASES

This section summarizes dead, live, buoyancy and ballast load conditions implemented on completed **DL** analysis and the revisions introduced for **US** level analysis.

Dead loading was generated by ASADS for all beam-type members based on input cross-sectional areas and input or assumed weight densities. Dead load for facilities, equipment, non-modelled appurtenances and other significant structure (e.g. mudmats) was hand-calculated and explicitly input at the appropriate location.

Generalized area loads were developed based on equipment weights applied to the deck structure.

Deck area live loads were modelled per the Design Basis with appropriate reductions for the primary platform structure in recognition of the fact that not all areas will be loaded simultaneously to their design live load. Deck area live loads were modelled as linear live loads on adjacent modelled girders as a function of deck beam framing orientation. Deck area live load conditions were developed for each deck level and may be further subdivided to allow subsequent load combinations utilizing varying combinations of live load.

Buoyancy loads were generated by ASADS for all tubular members below MSL.

DL AND US LEVEL "PUSH OVER" LOAD COMBINATIONS

Stillwater condition loads were combined with the seismic loads applied along all three orthogonal axes of the platform.

Load Factors and Stillwater Condition

A stillwater loading combination developed consisting of the following load conditions for both **SLE** and **DLE** analyses were used for the analysis:

- Deck facilities and equipment and other load
- Deck live load
- Jacket and deck dead (self weight) load
- Jacket buoyancy load

DL and US Level Load Combination

The loading combinations consist of the stillwater load condition and seismic loading. For the Design Level (DL or SLE) analysis loads generated were combined accordingly:

- Stillwater condition + 100% of DL level in X direction
  + 100% of DL level in Y direction + 50% of DL in Z direction

PART A: PLATFORM ASSESSMENT

For the Ultimate Strength (US or DLE) level incrementally increasing lateral load condition was used:

- Stillwater condition + 100% of US level in X direction +100% of US level in Y direction + 50% of US level in Z direction

AAA7. ANALYSIS METHOD AND VALIDATION

GENERAL METHODOLOGY

DL analysis was performed following standard procedures and require no further discussion. The US Level analyses require further discussion. It may be performed by implementing either one of the following methods:

- Push-Over Method
- Time-Domain Method
- Equivalent Method

The Time-Domain Method will accurately capture structural dynamic response due to excitational loads. An alternate Equivalent Method (such as Serrahn's FOURDYN, Reference 4), utilizing a Frequency-Domain method that incorporates dynamic modes (each with its own participating factor through the use of Fourier Series) can be effectively used to capture dynamic response of the structure to excitational loads. Since the Grand Isle platform dynamic response to excitational loads will be negligible, both of the methods described are not applicable for such platforms.

A Push-Over Method was used in the project to effectively track the performance of platform by incrementally increasing the applied loads. The US analysis results are directly applicable for Push Over analysis. A fraction of this loading was applied on the platform and incrementally increased after each step. The following generalized steps were taken:

- Apply stillwater load condition; reformulate component stiffness and iterate if any component reaches capacity.
- Incrementally apply the pushover loading, reformulate the component stiffnesses for components reaching capacity.
- Iterate to find the solution for each increment of load application.
- Conclude the analysis at structure collapse or at the application of predefined loading level (**US or higher**), whichever occurs first.

To conservatively assess the overall reserve capacity of the platform an initial Push Over analysis was first performed for the actual **US** loading. This effort indicated total number of components reaching capacity and overall load absorption capacity of the platform.

LINEAR VERSUS NONLINEAR BEHAVIOR

All jacket tubular members were modelled as nonlinear beam-column members. Deck non-tubular members were not expected to reach capacity; thus for the purposes of analysis efficiency, all non-tubular deck members were modelled as linear truss (strut) members. Piles were modelled as non-linear beam-columns with additional orthogonal linear strut elements to properly model the pile head loads and resultant pile response during the analysis.

Understressed component members and members contributing little to overall resistance of the platform to increased Push-Over loading could have been defined as linear members since such members are expected to perform elastically throughout the ultimate strength level Push Over analysis. However, all jacket members were defined as non-linear beam-columns to validate the assumption. Deck girders and beams, secondary members, and non-structural members were considered to be good candidates for such a definition and some of these were modeled as linear struts. Analysis validation included review of all members to ensure that the elastic behavior assumption for these members were maintained.

Those members defined to have nonlinear behavior can be defined by material stress-strain relationship in resisting applied axial compression or tension and bending. When a member reaches its yield load capacity, its post-yield capacity is determined. Initially, post failure may be defined to be zero capacity. However, since such an approach is conservative and will lead to erroneous tracking of any potential collapse mechanism, components reaching their capacities were defined as a function of their properties and deformations. Two basic types of linear/nonlinear elements can be used to define member behavior:

- Strut Elements: Platform braces expected to fail primarily in axial tension/yielding or axial compression/buckling may be modeled with strut-type elements which account for reductions in strength and stiffness after yielding/buckling. Typically, member slenderness and D/t ratios influence performance of such members.

  The assumed post-yield/buckling capacity is the yield/buckling capacity. This is equivalent to a strain-hardening ratio of 0.0.

- Beam-Column Elements: Platform legs, piles and other members with low slenderness ratios and high bending stresses will be defined as Beam-Column Elements. These members are primarily expected to fail due to high bending stresses with increases in the applied loading. Beam-Column elements will effectively account for axial and bending interaction and facilitate definition of reduction in capacity with progressive elasto-plastic hinge formation.

ANALYSIS VALIDATION

A linear analysis is not considered to be valid unless it is compatible with the nonlinear foundation system. At the beginning of DL analysis, pile top reactions and displacements were obtained for the highest utilized pile and compared with the nonlinear pile response. Typically, if adequate compatibility is not obtained, revised equivalent pile stiffnesses are determined. The global stiffness analysis will then be rerun and another compatibility check performed until adequate pile compatibility is obtained.

Same approach is implemented in the Push-Over analysis. Since static equivalent loading is applied, the pile-structure interaction can be automatically accounted for and compatibility achieved at each step of the analysis. However, an automated pile-structure interaction option was not used during the analysis of the platform and instead, compatibilities checked at predefined load increments manually.

Other validation efforts include, but are not limited to the following:

- The assumption of elastic behavior for selected members was verified for these members at the end of the US level analysis.

- The effect of member post-failure capacities on the overall platform behavior and reserve strength was reviewed to assess their impact on failure path.

AAA8. MEMBER AND JOINT DESIGN

Member forces and moments obtained from the DL loading combinations previously discussed was used for member strength and stability checks. In general, all tubular members and joint chords were checked against the requirements of API RP 2A, 20th Edition (Reference 2). Wide flange shapes and truss connections were generally checked against the requirements of AISC, 9th Edition (Reference 3).

LIMITING STRESSES

The DL design loading combinations were checked against 1.70 times basic allowable stresses; i.e. 70 percent increase in basic allowable stresses. The stillwater loading was also checked at basic allowable stresses.

JACKET MEMBERS AND JOINTS

All tubular members were checked for adequacy against the following failure modes:

- yield
- local buckling
- column buckling
- bending alone
- shear
- external hydrostatic pressure
- combined axial and bending
- combined axial, bending and external hydrostatic pressure

Column buckling effective length factors (K) for all members were based on API RP 2A recommendations.

For code checking, unbraced lengths were taken as the joint-to-joint (work point to work point) length for all members provided that they are adequately braced in orthogonal directions. For bracing members which may be supported in only one direction, two member unbraced lengths were computed, each length being the actual unbraced length in that direction.

Bending moment reduction factors (Cm) for all members were based on the recommendations of API RP 2A. In general, utilization ratios were determined at the two ends and at midspan. The target utilization ratio for members reaching capacity was defined as 1.0.

Member utilization ratios were summarized by member and by utilization ratio for all utilizations at each US step to facilitate review of potential failure paths.

Unstiffened simple tubular joints were checked for punching shear in accordance with API recommendations. The target joint utilization for failure definition was 1.00.

The punching shear check utilized API RP 2A's "nominal loads method" to determine punching shear utilizations. It should be noted that all joints were checked to meet the requirements of API RP 2A, Section 4.1.1-1.

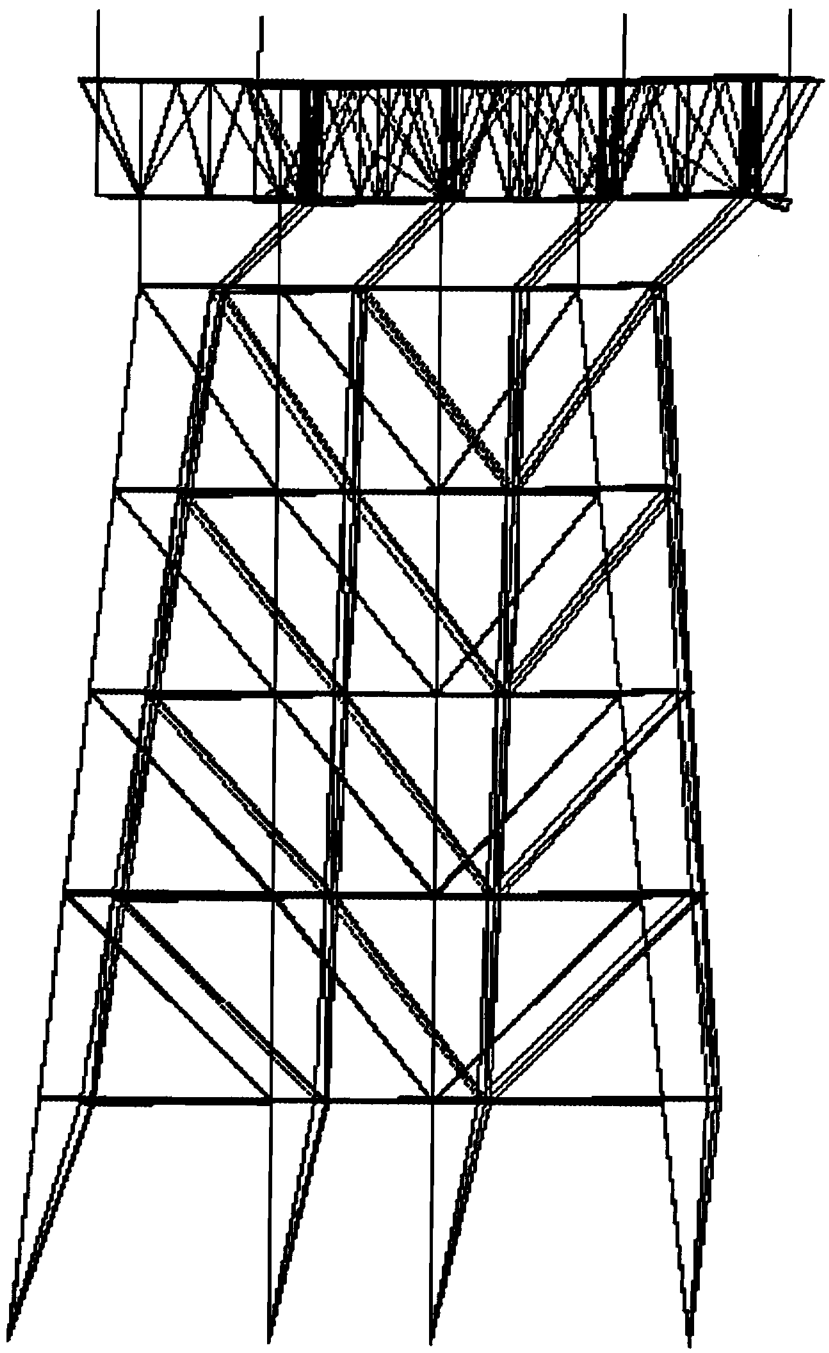
FOUNDATIONS

Foundation members consist of piles and the pile/jacket interfaces. The piles were modeled as non-linear "Beam-Column" elements and checked against degradation of their load carrying capacity due to elasto-plastic hinge action.

Since the plastic hinge formation occurs due to combined axial compression and bending effects, elasto-plastic capacity of piles were determined for a range of load combinations and deformations.

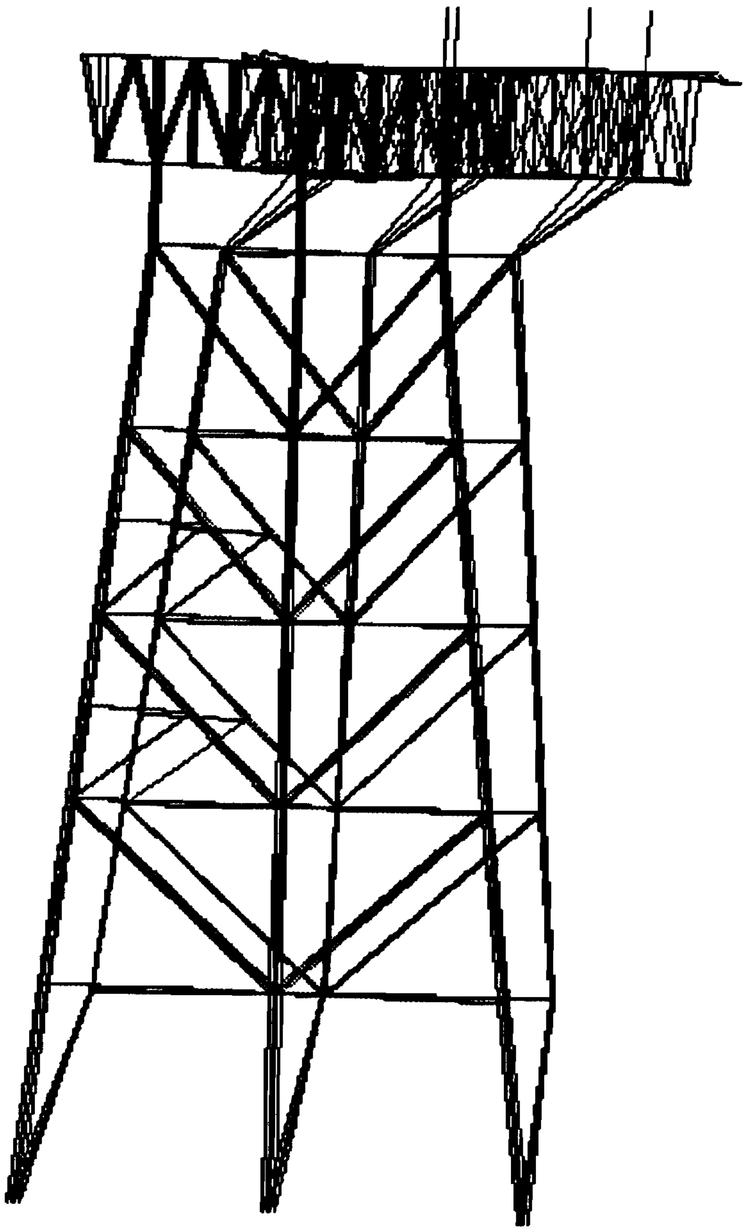
AAA9. REFERENCES

1. Letter entitled: "Cursory Preliminary Review of Seismic Hazard, Platforms ı, Federal OCS P-0166, Carpinteria Offshore Field", transmitted by Fugro West, Inc. to , dated May 20, 1994.
2. American Petroleum Institute, "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," API RP 2A, 20th Edition, July 1993.
3. American Institute of Steel Construction, "Manual of Steel Construction - Allowable Stress Design," Ninth Edition, 1989.
4. Moses, Fred, API PRAC PROJECT 83-22, "Implementation of a Reliability Based API RP 2A Format, Appendix E by C.S. Serrahn: Dynamic Response Using Fourier Series Loadings", American Petroleum Institute, January 1985.



FIRST LATERAL MODE ALONG X-AXIS

T = 1.98 Sec



FIRST LATERAL MODE ALONG Y-AXIS

T = 1.92 Sec

PART B: FEEDBACK TO THE API TG 92-5

COMMENTS ON JOINT DESIGN AND ANALYSIS

Most of the comments on Draft Section 17 were discussed during the execution of this JIP and corrective measures were taken. One very important comment affecting not only Section 17 but also Sections 2 and 4 is provided to ensure that a corrective measure is considered by the Subcommittee on Fixed Structures.

- Section 4.1 of API RP 2A states that the "joints develop the strength required by design loads, but not less than 50% of the effective strength of the member".
- Section 2.3.6e of API RP 2A provides additional guidelines, stating that if the horizontal ground motion is 0.05g or greater, the joints for the primary structural members should be sized for the capacity of the member connected to the joint.

The approach taken by API has worked well for the Gulf of Mexico where the storm loading controls the design and seismic design is not considered, and for offshore Southern California where the seismic loading controls the design.

For an area such as the South China Sea or offshore Trinidad, the controlling design condition is the typhoon/hurricane event. However, the structure also needs to be analyzed for seismic loads. While the seismic loading may require that a brace be sized 24-inch diameter with 0.5 inch wall thickness, the storm event may require a 1.0 inch wall thickness. Thus, while the correct procedure is to determine the seismic capacity required for strength level seismic design and ensure that the joint is capable of resisting loads associated with full member capacity (i.e., to achieve API's objective; namely prevention of an unzipping effect), Section 2.3.6e may be misinterpreted and the designer/analyst may provide joint resistance for full capacity of the member designed for the extreme storm environment.

We do not necessarily endorse the 50% rule. We also understand the basis for the two contradictory requirements to be due to both the differences in the recurrence intervals considered for storm (100 versus 500 years) and the earthquake (200 versus 2,000 years) and the characteristics of the applied loading and structure response to the applied loads.

Please consider inserting a statement in Section 2.3.6e, indicating that the recommendation is applicable to members capacities controlled by seismic design.

PLATFORM "T"

1. SUMMARY

This report presents the results of the trial application of the API RP 2A draft Section 17.0 for the assessment of existing platforms (1) to a production platform in the Southern sector of the North Sea. The six-legged X-braced jacket was installed in 1968 and therefore designed prior to the introduction of formal guidance for offshore structures. The platform is an essential thoroughfare for life safety, being bridge linked from the drilling platform and to the quarters platform. Furthermore with three input risers and one export riser to the gas terminal onshore, the platform is placed in the most stringent exposure category.

The assessment trigger is the addition of new riser guards increasing base shears by some 17%. There is no damage and no planned increase in manning levels or additional facilities to initiate the assessment otherwise, and deck height is adequate.

The platform does not pass the design basis check by virtue of age and location. The absence of experience for similar structures and the specific conditions for the platform, preclude a screening check. The assessment therefore focused on analysis checks.

Detailed metocean data for the site were not available for the assessment to enable relations such as base shear vs return period, wave height vs return period etc., to be derived as required for Section 17.0. However, the Operator has previously developed metocean data for assessment which give less severe and more realistic loading than required for new design. These were adopted for the Design Level Analysis (using STRUDL) but the structure did not pass at this stage of the assessment. Even for the existing structure before the additional of new riser guards, utilisations of up to 3.2 were calculated to API RP 2A (2) at primary β=1.0 X joints (without cans). Reference was made to recent tubular joint databases to better quantify the joint capacities and this information was used to help determine the most onerous directions for which to perform ultimate strength analyses (using SAFJAC).

The flexibility and capacity of primary joints was modelled enabling tension and compression load paths through the bracing to be fully mobilised and demonstrating a significant reserve strength within the structural system. The nonlinear analyses using SAFJAC demonstrated that the section capacity of the piles became the limiting factor and sensitivity studies were performed regarding the degree of soil restraint, to give rigorous assessment of the ultimate capacity. The platform was deemed to pass the assessment at this Ultimate Strength Level Analysis stage but further work is necessary to quantify the capacity in terms of survival probabilities and to provide correlation with the Section 17.0 philosophy.

The assessment was initiated prior to the drafting of Section 17.0 but the sequence in passing from simplified/conservative to complex/realistic levels is endorsed. Further correlation between North Sea practice and Section 17.0 is required in terms of factors and implicit safety levels, and this must involve the HSE and Certifying Authorities as well as Operators. Nevertheless, a number of key points have been raised in the trial assessment:

- In the absence of significant change (ie. additional riser guards) no assessment of the platform need have been initiated in accordance with Section 17.0, yet the early platform design has exceptionally high component utilisations to present day standards. The X bracing enabled redistribution and system reserves to be demonstrated through nonlinear analysis. A K braced configuration would have been less redundant. Should design era (eg. pre formal guidance) be an assessment indicator?

- The ultimate strength of the jacket was shown to rely on the facility for plastic deformation at the joints and therefore on the ability for these to be modelled in the analysis.

- The attachment of risers to the bracing diagonals and the associated member plasticity under extreme loads demanded that both members and joints need to be separately modelled within each bracing panel. This is the standard case in SAFJAC but is not necessarily accommodated in all software.

- The load case (direction) generating the highest utilisation in the elastic Design Level Analysis did not correspond to the most critical case in the Ultimate Strength Level Analysis, demonstrating the care required if selective ultimate strength checks are to be performed.

- Although jacket members had significantly higher utilisations than the piles in the Design Level Analysis, realistic modelling of component capacities in the Ultimate Strength Analyses revealed that the system response was limited by plastic hinge failures in the piles. This demonstrates the importance of a fully integrated analysis of the system including both structure and foundation and the appropriate interactions.

This report presents a summary of the analyses undertaken, both elastic (Design Level) and inelastic (Ultimate Strength Level). Detailed reports have been provided to the Operator (3,4,5) giving more extensive information and discussion than required by the JIP. This report is intended to bring out the issues, uncertainties and lessons learned in relation to the future applications of Section 17.0, with particular reference to its adaptation for non-US waters.

A summary of the trial assessment is given in Table 3-8#*.

Notes:

1* Tables noted with a # suffix have identical numbering and format as those in the DRAFT of "Trial Applications", prepared by PMB Engineering Inc., September 1994.

2. The views expressed in the report are solely those of the authors and do not necessarily reflect the opinions of the platform Operating Company or its personnel.

2. PLATFORM INFORMATION

The 'trial' platform is a production facility and is located in the Southern sector of the North Sea in 118 feet water depth. Figure 2.1 indicates the block location. The platform is supported by a six-legged X-braced jacket shown in Figure 2.2. The jacket was designed and installed by de Groot Zwijndrecht in 1968 (ie. pre API RP 2A). Details of the structural design premise are not available to the assessment. On Rows A and C the lower bay X bracing comprises 0.5" thick 24" diameter members but on Rows 1, 2 and C the specified wall thicknesses are only 0.375" with no X joint cans. The legs do however have cans at the K joint intersections.

Figure 2.3 provides the certification data for the platform. The jacket was founded with six 36" diameter insert piles penetrating to about 105 feet (slant distance) and welded at the top of the jacket at +17.5ft LAT and cut off at +19.0 ft LAT. Centraliser plates at the lower plan bracing level of -117.96ft LAT reduce the annular gap from 1" to 0.625".

A 1983 investigation of the soil and piled foundations is available to this assessment. The major soil type comprises fine sand and slightly silty to silty sand with clay seams. The density of the sands is taken as medium-dense in the near surface layers becoming dense with depth. Inspections for this and similar locations indicate scour to a depth of 5 feet should be considered local to the platform.

Although the jacket leg-pile annuli were ungrouted at installation, the Leg C1 annulus was filled with grout in 1982 as a repair measure following damage. The deck legs extend vertically from the pile cut-off level to a trussed superstructure containing two deck levels.

As indicated in Figure 2.3, the production platform is bridge linked to adjacent structures. Men from the drilling platform within the complex also reach the quarters facilities via the production platform and bridge links. Three important risers bring gas from nearby complexes to the platform which forms the hub for transmission of the gas to shore via a 30" export riser.

The riser locations can be seen in Figure 2.3 with one riser protection frame in place on Row 2 between transverse Rows B and C. As part of a comprehensive programme to reduce the risk of fires and explosions the operator installed emergency shutdown valves and planned additional riser protection guards at the locations shown in Figure 2.4. The riser guard configuration is shown in Figure 2.5. The planned installation of these riser guards and the additional loads they would attract provided impetus to the assessment.

The risk addressed is with regard to static overload under extreme storm conditions. Detailed metocean statistics were not available to the assessment, however parameters for assessment were developed by a consultant for the operator in 1986. These 50 year return period values are shown together in Figure 2.6 and may be compared with recommended omnidirectional values for use in design of 76.2 miles/hours for hourly mean wind speed, 49.5ft wave height and 5.6 ft/sec surface current.

The physical and operational characteristics are summarised for the JIP in Table 1-1# and 1-2#.

3. PLATFORM ASSESSMENT PRELIMINARIES

3.1 Platform Selection

Of the six assessment initiators in Section 17.2 of the draft guideline for assessment (1), 'increased loading on structure' provides the trigger for the assessment of the Southern North Sea structure. Additional riser guards have been specified to protect risers from vessel impact and thereby to reduce the subsequent fire/explosion risk. The riser guard locations are shown in Figure 2.4 with the typical configuration in Figure 2.5. The riser guards attract significant additional loads to the jacket giving some 17% increase in base shear. This is sufficient to trigger an assessment. The assessment initiators are summarised in Table 3-1#.

3.2 Categorisation

The platform supports gas production activity and may be argued not to give rise to significant environmental impact in the event of collapse. However the structure is manned with quarters reached by a bridge link to another platform. Furthermore the platform also provides a staging in bridge links between the adjacent drilling platform within the complex and the quarters. Extreme storms can be generated very rapidly in the North Sea without adequate warning to fully evacuate the platform. Given these considerations the platform is categorised as manned, non-evacuated.

Furthermore the platform imports gas from three other complexes (3 risers) exporting the combined products to the onshore terminal (one 30" riser/pipeline). It therefore performs a key function in the Company's operations and commercial/business considerations may be expected to demand the highest levels of integrity.

Hurricanes do not feature in the North Sea environmental conditions and the extreme event is driven by winter storms.

The categorisation is summarised in Table 3-2#.

3.3 Condition Assessment

The platform was installed in 1968 and full design details were not available to this assessment. However, during the 1980s the Operator undertook extensive survey work to confirm structural configuration details, topsides loading and layout, foundation properties and environmental conditions. Furthermore the structure is inspected annually in accordance with HSE Guidelines and as approved by the Certification Authority. This information is therefore comprehensive and formed the basis of the assessment.

The condition assessment is presented in Table 3-3#.

3.4 Design Basis Check

The increase in load due to the proposed riser guards means that the Design Basis Check does not apply. Furthermore the significance of the platform, its locations, outside the Gulf of Mexico and that fact that it was designed before formal guidance existed, would mean that it would fail the check even if the loading were not to increase. This is reported in Table 3-4#.

The assessment process is therefore progressed to the Analysis Checks.

4. PLATFORM ASSESSMENT

4.1 Screening

Insufficient data or experience exists for the assessment of this platform on the basis of screening. By inspection, the old design without joint cans at primary X joint nodes suggested limited capacity in light of current design practices. Specific analysis checks were therefore undertaken.

4.2 Section 17.0 Metocean Loading Philosophy

The basis of Section 17.0 is that less stringent criteria need to be imposed in assessment than new design. This allows for the risk and cost of implementing strengthening measures and for the survival experience of older structures. Section 17.0 also requires that relationships between base shear and return period, wave height and return period etc., be established so that the realistic metocean parameter combinations (wave height, current etc) are determined corresponding to the required resistance base shear for the specified survival probability. This is depicted in Figure 4.1 for a Gulf of Mexico case (6).

For the Gulf of Mexico, experience and comprehensive metocean data have enabled specific factors to be derived for each exposure category for the levels of base shear required to be sustained in Design Level (BS_{des}) and Ultimate Strength Level (BS_{ult}) analyses as proportions of the 100 year 'design' values (7). Reference 6, presenting the background to the metocean criteria in the draft Section 17.0, states that for other areas "the analyst will have to come up with realistic combinations of wave height, wave period, current and wind speed that correspond to BS_{des} and BS_{ult}".

For the present study data were not available for the underlying relationships to be derived and future work will attempt to calculate the survival probabilities associated with the approach adopted below with further comparison with a rigorous Section 17.0 approach.

However, as noted in Section 2 of this report, the Operator had directional metocean criteria developed for assessment purposes in 1986 as shown in Figure 2.6. These are less onerous than the omnidirectional design values recommended in the same report (ie. 49.5ft wave height, 76.2 mph wind speed and 5.6ft surface current). Each value corresponds to the 50 year return period and the directional combinations are not therefore 'associated' as assumed in Section 17.0. This conservative superposition of maxima has until recently been standard North Sea practice. For the purposes of the platform assessment, these conditions were therefore adopted as the criteria for the Design Level Analysis in the spirit for which they were originally intended.

For the Ultimate Strength Analysis of this 'trial' platform, in which the nonlinear system response was to be modelled realistically, the criteria needed to be separately established.

for the Ultimate Strength Level Analysis, require that 1.8 times the Design Level base shears be sustained without collapse. This is summarised in Table 3-5#.

4.3 Design Level Analysis

The Design Level Analysis was performed using STRUDL for the eight wave attack directions indicated in Figure 2.6 with 12 steps per wave and for two water depths (ML + depth @ LAT and MHWS + 50 year surge + depth at LAT). The current velocity profile was adjusted for horizontal mass continuity. Wave particle velocities and accelerations were calculated using Dean's Fifth Order Steam Function Theory. Marine growth was taken as 2" on the member radii between -40ft and +10ft elevations and zero elsewhere. Deck loads were based on the inventory and photographs to indicate the location of key equipment. The loads attracted by the risers, caisson and other non-structural appurtenances were transmitted to the structure. Soil structure interaction was included with soil parameters derived for the 1980s survey (see Section 3.3). The environmental loads were calculated in line with HSE Guidance (8), not API RP 2A 20th Edition (9).

From the series of analyses the maximum base shear in each direction was determined and code checks to API RP 2A (2) performed. For joint checks the classification was by load sharing and geometry. The allowable stresses were increased by 1/3 in the code checks for these extreme conditions.

The first analyses were performed for the platform in its current condition without consideration of the proposed riser guards. [It is recognised that this is not required by Section 17.0 for which the structure with riser guards could be assessed directly.] The minimum safety factor on pile capacity with regard to the foundation is 1.55. The maximum utilisation in the legs and piles (both above and below the mudline) is 0.74, in the X braces is 0.67 and in the plan and horizontal bracing ranges between 0.45 and 0.60. Similarly the leg K joints and top bay X joints with a thickened wall can have interaction ratios (IRs) below unity. However for the lower bay β=1.0 X joints, the IRs exceed unity. Figures 4.3 and 4.4 show the joint IRs for the end-on and diagonal (Loadcases 2 and 5 in Figure 2.6) for which the corresponding base shears are 1163 kips and 1386 kips, respectively.

As permitted by Section 17.0, reference was made to more recent compilations of test data than underlying API RP 2A tubular joint strength provisions, as well as to HSE Design Guidance (8) which admits greater strengths for inclined brace X joints than API, particularly for tension loadcases where capacity is linked to ultimate strength rather than first cracking. This enabled the apparently more critical directions from an Ultimate Strength Analysis (realistic modelling) viewpoint to be determined. These corresponded to Loadcases 2 and 5 for which Figures 4.3 and 4.4 reveal the maximum API RP 2A joint IRs to be 3.24 and 2.20, respectively, even for the analysis without the new riser guards.

Clearly the platform would not pass the Design Level Analysis in the existing condition as summarised in Table 3-6# and it was necessary to consider realistic modelling of the structure within an Ultimate Strength Level Analysis both to assess the current condition and the future performance with additional riser guards.

4.4 Ultimate Strength Level Analysis

The platform was subject to a series of nonlinear ultimate strength analyses using SAFJAC (11). The program includes quartic elements, to model geometric (P-Δ) nonlinearity effects, which can automatically subdivide on the detection of yield and introduce a plastic hinge to model material nonlinearity. An alternative approach uses cubic elements to model the gradual spread of plasticity through a section and along its length. Joint elements comprise piecewise linear springs to model P-δ and M-ϕ deformations and capacity limits at tubular joints. Pile-soil interaction is modelled in terms of T-Z and end bearing resistances, with coupled translations in the horizontal plane to model the P-Y lateral restraint. The program has been extensively calibrated against test results and classic solutions in the open literature.

Specific modelling of the 'trial' platform is described below and Figures 4.5 through 4.11 show examples of the SAFJAC modelling for the Ultimate Strength Analysis.

Element selection

Quartic elements with the ability to subdivide forming plastic hinges (Type 34) were adopted for jacket members. Extra nodes and elements were introduced on the legs to model the leg cans. The simplified deck was modelled using quartic elements (Type 33), which allow for elastic large deflection behaviour of the deck but not for material nonlinearity (plasticity). This approximation for the deck was considered reasonable because the deck is only supported on six vertical columns, the deck trusses are much stiffer than the columns and the wave loading attracted by the jacket is unlikely to have any significant effect on the forces in the deck.

The piles inside the legs were modelled using quartic plastic hinge elements (Type 34). Spring elements were introduced between the piles and legs to restrain the lateral movement of the pile to that of the leg. Below mudline the piles were modelled either with cubic elements (Type 31), to account for the gradual spread of plasticity, or alternatively using the quartic plastic hinge elements. The axial response (T-Z curves) and the lateral response (P-Y curves) of the soil, obtained by linear regression of the data, were modelled using spring elements (Type 41).

The four risers, one caisson and their supports were modelled using quartic plastic hinge elements (Type 34). The existing riser protection frame on Row 2 was also modelled using plastic hinge elements.

Material modelling

The yield stress levels was retained conservatively at the minimum steel specifications because of uncertainty in the as-delivered properties, although Section 17.0 indicates that a mean value should be adopted to better represent the true system response.

Load application

Loads were applied as concentrated nodal forces in the SAFJAC analyses. Table 4.1 indicates the wind, wave and current, and topsides loading for the SAFJAC analyses which are in good agreement with those of the Design Level Analysis following the conversion to nodal loads.

The loading in the pushover analyses was applied in two stages. The first stage (initial loads) corresponds to the still water condition and the second stage (proportional loads) corresponds to the environmental loading due to the relevant 50 year extreme storm wave and current derived for assessment purposes. Wind loading on the topsides was included in the initial loading stage. In the SAFJAC analyses, after the initial loads had been applied (load factor $\lambda_p = 1.0$), the proportional loads were increased to determine the ultimate strength of the platform.

The load factors in this assessment therefore apply just to the proportional loads about which there is uncertainty and which may increase significantly in an extreme event. The comparison of λ_{pmax} with the target 1.8 on base shear, discussed in Section 4.2, may therefore be slightly different from the total base shear factors in References 1 and 6. However without better data and a rigorous reliability assessment the current approach is considered to be adequate.

Analyses

Several alternative analyses were performed to assess the sensitivity of the ultimate strength predictions.

Eight 2D analyses were undertaken for the broadside framing with extreme storms from both diagonal and end-on directions to assess the influence of joint capacity assumptions and foundation modelling. These demonstrated the significant facility for redistribution at X joints (supported by work published subsequently from the Frames Project (12)) and provided impetus to full 3D analyses as indicated in Table 4.2, firstly in the existing condition and finally with the proposed riser guards in place. Example results are presented in Table 3-7b# in the format required by the JIP.

The immediate impression from Table 4.2 is the high load factors obtained such that the structure, both in its existing condition and for the real assessment condition with additional riser guards, would pass at the Ultimate Strength Level Analysis stage. The load factor λ_p exceeds the target 1.8 in all cases even without account of the additional resistance from steel having a higher yield than the minimum specification. However, given the inability to pass the platform at the Design Level Analysis stage, further discussion of the sources of reserve strength is presented below.

Influence of joint flexibility and limiting capacity

The Design Level Analysis and Figures 4.3 and 4.4 confirm that the lower bay X joints are the first components not to satisfy API RP 2A resistance criteria in the elastic analysis. As a base case, nonlinear analyses [1] and [5], reported in Table 4.2, were undertaken for diagonal and end on-wave attack directions respectively, ignoring foundation effects and assuming rigid tubular joints. Very high load factors were achieved (7.34 and 6.53) confirming the under-utilisation of other structural members in the jacket. However for analyses [3] and [7] the flexibility and limiting capacity of tubular joints in terms of P-δ and M-ϕ characteristic were taken into account. Mean capacity equations underlying HSE Guidance were adopted, in conjunction with flexibility coefficients derived from examination of raw test results and regression of the data with respect to geometric factors. The P-δ, M-ϕ relations now form part of the SAFJAC library.

Table 4.2 indicates the significant reduction in capacity compared with the rigid joint cases when proper account of the joint strength is taken (eg. λ_p = 3.54 compared with 7.34). However the maximum load factor still exceeds the target 1.8 level. The reason for this is that the X joint response softens in response to brace loads, well before the maximum capacity is attained such that a higher proportion of the applied loads is transmitted along the chord load path. This gradual softening and redistribution of loads therefore leads to efficient use of the X-braced panels and increases the capacity. Figure 4.12 indicates the extent of plasticity in the members even when X joint capacity limits are accounted for, with the first hinge forming at a load factor of 3.1. Figure 4.13 compares the overall structural responses depending on the extent of joint modelling and level of detail. An additional feature of the structural collapse revealed by the SAFJAC analyses is the formation, closure and reopening of hinges as redistribution progresses.

Influence of foundations compared with fixed base

Considering the system as a whole the foundation is reintroduced to the model as described previously. It can be seen from Table 4.2 and analyses [2] & [6] and [4] & [8], that the foundation compliance now governs the ultimate capacity. However, at the time the peak load factor of λ_p = 2.72 is attained in analysis [8], all eight X joints in the bottom bays of the broadside frames Rows 1 and 2 had reached their capacity. The redistribution leads to hinges forming in the chord members as shown in Figure 4.14. The extent of plasticity is far less than for the rigid foundation case in Figure 4.12, but not shown are the plastic hinges that formed in the piles below the mudline. The pile utilisations in Figure 4.15 confirm that all six piles had effectively failed at a location some 8 to 10 diameters below the mudline.

Figure 4.16 shows the load-displacement plot for a node at the top of leg B2 but with reference to the deflected shapes in Figure 4.17, it can be seen that the deformations are dominated by the pile head movement.

With realistic modelling of the tubular joint behaviour, the jacket is no longer the critical component in the ultimate system response. This serves to underline the need to comprehensively model the system in ultimate strength analysis and to be cautious in the use of Design Level Analysis code checks to steer the ultimate strength evaluations.

Foundation sensitivity studies

The additional analyses [8A] to [8E] in Table 4.2 were undertaken to examine the influence of foundation modelling assumptions on the ultimate strength evaluations.

For a 100% P-Y stiffness and no axial T-Z stiffness 0-15m below mudline, the jacket response is almost identical to that using the full T-Z stiffness. Increasing the lateral P-Y stiffness by 50% produces a 3% increase in the reserve strength of the structure (jacket and foundation), whilst reducing the P-Y stiffness by 25% decreases the reserve strength of the structure by 3.7%. For this latter analysis (Analysis [8C]) the first plastic hinge forms at a load factor of 2.29 and the maximum load factor is 2.62. In all the analyses plastic hinges form in the diagonal compression bracing emanating from the bottom of Legs A1 and A2, and either eleven or twelve hinges form before the maximum load factor is reached. For the weakest soil conditions [8C], all except one of the last seven plastic hinges to form, form in the piles just above the mudline. The reduction in lateral soil resistance increases the bending moments in the piles.

More accurate modelling of the soil using five-part piecewise linear curves [8D] reduces the maximum load factor by approximately 6% from $\lambda_p = 2.72$ to 2.56 and the number of plastic hinges from twelve to five, the reduction in the number of plastic hinges matching the number of hinges that formed between load factors of $\lambda_p = 2.56$ and 2.72 in analysis [8]. Extending the use of the five-part curves to model the tension-compression behaviour of the X joints in the jacket (Analysis [8E]) increases the maximum load factor marginally to $\lambda_p = 2.58$ but no additional plastic hinges form. The piles again fail approximately 8m below the mudline.

Despite the significant changes in modelling assumptions the maximum load factor appears to be reasonably robust around 2.6. Underlying this, the order and sequence of pile failures differs as shown in Table 4.3.

In a separate series of analyses for a different structural model, analyses included variations in the T-Z resistance. Reducing the values by 25% led to failures in the foundation whereas in the base case hinges formed in the piles as for the trial case. For that structure, changing the lateral resistance (P-Y) also changed the failure mode with fewer piles forming hinges and more extensive plasticity in the jacket limiting the system capacity with maximum load factors varying by as much as 29%. These experiences serve to underline the need for sensitivity studies when evaluating ultimate capacity to reflect the uncertainty in the realistic characteristics of certain key parameters.

Additional riser guards

Having demonstrated the adequacy of the platform in its existing condition the system was fully analysed for both wave attack directions with the riser guards in place and the modelling philosophy of analysis [8E] (see Table 4.2).

For the end-on wave the effect of the new riser protection system is to reduce the reserve strength from λ_p = 2.56 to 2.11. The reduction of 17.6% is very similar to the 16.6% increase in base shear generated by the new riser system. For the diagonal wave a direct comparison of results is not possible because an analysis has only been performed for the existing riser protection system using three-part curves.

The deflected shape from the end on wave for the platform with riser guards is shown in Figure 4.18 and Figure 4.19 indicates the softening in the global load displacement response. For the diagonal wave it can be seen in Figure 4.20 that following the hinge formation in the first pile at the maximum load factor of 1.81, the global load cannot be sustained and, in falling to a load factor of 1.7 under displacement control, hinges form in the other five piles some 8-10m below the mudline.

Interestingly because of the change in critical component the diagonal wave is associated with the lowest resistance load factor, whereas it was the end on wave case which exhibited the highest component utilisations in the elastic code check. On this care is required in selecting and reviewing the choice of load case for ultimate strength analyses if only selective collapse analyses are to be undertaken.

Summary

Despite the high component utilisations indicated in the Design Level Analysis, the Ultimate Strength Analysis has indicated that the platform would pass the assessment with a minimum load factor of 1.8. Furthermore additional strength would be derived directly from an increase in the yield modelling in the analysis reflecting the mean steel yield compared with the minimum specification conservatively adopted in the present analyses.

5. REFERENCES

1. American Petroleum Institute. 'Assessment of existing platform', Draft Section 17.0 to API RP2A, June 28 1994.

2. American Petroleum Institute. 'Recommended practice for planning, designing and constructing fixed offshore platforms', API RP2A, Eighteenth Edition, 1989.

3. 'Structural assessment of .... jacket', Reference 179/90. (Confidential to operator client).

4. Pushover analysis of jacket', Reference C568R001.02 (Confidential to operator client).

5. Pushover analysis of jacket - Part II', Reference C568R001.05 (Confidential to operator client).

6. Krieger W F et al. 'Process for assessment of existing platforms to determine their fitness for purpose', Paper No 7482, Offshore Technology Conference, 1994.

7. Petrauskas C et al. 'Metocean criteria/loads for use in assessment of existing offshore platforms', Paper No 7484, Offshore Technology Conference, 1994.

8. HSE. 'Offshore Installations: Guidance on design, construction and certification', Fourth Edition, 1990.

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10. Efthymiou, M E et al. 'Reliability analysis of offshore platforms', Workshop on Requalification and Reassessment of Offshore Installations, Institution of Mechanical Engineers, London, 1994.

11. SAFJAC. 'Strength Analysis of Frames and JACkets', BOMEL.

12. Bolt, H M et al. 'Results from large-scale ultimate load tests on tubular jacket frame structures', Paper No 7453, Offshore Technology Conference, 1994.

Table 1-1#: Summary of Basic Platform Information - Physical Characteristics

Platform	Location	Water Depth (ft)	Year of Installation	Year Designed or API Edition	Physical Characteristics: Number of Legs	Number of Piles	Pile Diameter (inch)	Leg/Pile Annulus	Pile Penetration (ft)	Brace Type in Vertical Frames	Lowest Deck B.O.S. Elev. (ft)	Damages to Jacket Primary Members	Platform Modifications Since Original Design
North Sea													
T	Southern N Sea	118	1968	? pre-API	6	6	36	Ungrouted	105	X	42.1	None	Repair/grouting to leg C1

Table 1-2#: Summary of Basic Platform Information - Operational Characteristics

Platform	Location	Water Depth (ft)	Year of Installation	Operational Characteristics: Type of Facility	Manned / Unmanned Operation & Evacuation During Storm	Number of Wells / Conductors	Future Modifications Planned or Proposed
North Sea							
T	Southern N Sea	118	1968	Production	Manned	4 gas risers	Additional riser guards

Table 3-1#: Summary of Assessment Initiator Triggers

Platform	Addition of Personnel	Addition of Facilities	Increased Loading on Structures > 10%	Inadequate Deck Height	Damage Found During Inspections	Assessment Intiator Triggers
North Sea						
T	No	No	Yes	No	No	Increased loading

Table 3-2#: Summary of Platform Categorization

Platform	Type of Facility	Manning Evaluation: Number of Men	Manned / Unmanned Operation & Evacuation During Storm.	Environmental Impact Evaluation: Number of Wells/ Conductors	Oil Storage on Deck (bbl)	Proximity to Shore (miles)	Environmental Impact Category	Metocean Criteria Category
North Sea								
T	Production	? #	Manned	4 risers with ESVs	None	38	Significant	Extreme Storm

\# Bridge linked to quarters platform, but also forms part of bridge link from drilling platform to quarters.

Table 3-3#: Summary of Condition Assessment

Platform	Survey Level	Last Survey (year)	Is Platform Damaged ?	Is Deck Height Inadequate ?	Has the Loading Inceased ?	Is Platform Unmanned ?	Does It Have Insignificant Environmental Impact ?	Does Platform Passes Assessment at This Stage ?
North Sea								
T	HSE requirement	1994	No	No	Yes	No	No	No

Table 3-4#: Summary of Design Basis Checks - Gulf of Mexico Platforms

Platform	Water Depth (ft)	Year Designed (Original)	API RP 2A Edition Used	Is Platform Designed to 9th Edition or Later?	Original Design Details Available	Does It Pass At This Stage
North Sea						
T	118	pre 1968	pre API RP 2A	No	No	No

Table 3-5#: Summary of Metocean Criteria

Metocean Criteria Category	Platform	Water Depth (ft)	Platform Orientation w.r.t. True North	Section 17 Design Level *1 Wave Height (ft)	Section 17 Design Level *1 Current Speed (knots)	Section 17 - Ultimate Strength Wave Height (ft)	Section 17 - Ultimate Strength Current Speed (knots)	Section 17 - Ultimate Strength Required Deck Height (ft)	API RP 2A, 20th Edition *2 Wave Height (ft)	API RP 2A, 20th Edition *2 Current Speed (knots)
North Sea										
Extreme Storms	T	118	N45E	46.3	3.3	Factored wave profile (ULR=>1.8)			49.5	3.3

* Operator's values *1 Elastic/code check assessment - directional *2 New design - omnidirectional

Table 3-6#: Summary of Design Level Analysis Results

Number of Legs	Platform	Water Depth (ft)	Number of Conductors / J-Tubes	Wave Height (ft)	Maximum Base Shear (kips)	Maximum I.R. (Primary Members)	Member Types with I. R. > 1.0	Pile Axial Capacity (F.O.S.)	Assessment Pass / Fail at Design Level Analysis
North Sea									
6	T	118	4 risers	46.3	1163	3.24*	12 No. X joints	1.55	Fail

* Longitudinal wave attack direction

Table 3-7b#: Summary of Ultimate Strength Analysis Results

Platform	Water Depth (ft)	Base Shear: Section 17 Ult. Load S-17 (kips)	Base Shear: 20th Ed. * Ref. Level S-20 (kips)	Analysis Results: Load at 1st Member with Lin. IR.=1.0 S-1 (kips)	Analysis Results: Load at 1st Member with NLin Event R-1 (kips)	Analysis Results: Ultimate Capacity Ru (kips)	Analysis Results: Platform Failure Mode	Ultimate Capacity / Ultimate Load = Ru/S-17	Assessment Pass / Fail at Ultimate Strength Analsysis	Information to the API TG: Reserve Strength Ratio RSR = Ru/S-20	Information to the API TG: Ultimate Linear Ratio ULR = Ru/S-1 #	Information to the API TG: Load Reduction Factor LRF = S-1/S-20	Information to the API TG: Redundancy Factor RF = Ru/R-1
North Sea													
T*1	118	2093	N/A	359	1414	3116	Joints/Piles	1.55	Pass	N/A	8.68	N/A	2.20
T*2	118	2495	N/A	630	1716	3693	Joints/Piles	1.55	Pass	N/A	5.86	N/A	2.15

*1: Longitudinal wave attack direction *2: Diagonal Wave attack direction

\# ULRs are very high because critical jacket components in elastic code check are beta=1.0 X joints for which API capacity is conservative.
Ultimate strength model includes mean strength flexible joints which give even load distribution through X-braced framing such that the piles then limit the overall capacity.

Table 3-8#: Summary of Trial Assessments

Platform	Assessment Intiator Triggers	Metocean Criteria Category	Does Platform Pass at Condition Assessment Stage ?	Does Platform Pass at Design Basis Check Stage ?	Does It Pass at Design Level Analysis Stage ?	Does It Pass at Ultimate Strength Analysis Stage ?
North Sea						
T	Increase in loads	Extreme storms	No	No	No	Yes

Loading	Ultimate Strength Analysis				Design Level Analysis			
	Fx	Fy	Fz	Resultant	Fx	Fy	Fz	Resultant
<u>Wave & current: Max Base shear</u>								
LC 5 Diagonal	-904	-888	-	1267	-886	-869	-	1241
LC 2 End-on wave	1120	-	-	1120	1093	0	-	1093
<u>Wind</u>								
LC 5 Diagonal	-82	-120	-	145	-82	-120	-	145
LC 2 End-on wave	70	-	-	70	70	0	-	70
<u>Topside</u>								
Deck steelwork & equipment	-	-	-4210	-	-	-	-4210	-

Table 4.1 Applied loads (kips)

Modelling		[Analysis Reference] Load factor, λ_p				
Foundation	Joint	LC2 End-on wave $\lambda_p=1 \equiv 1190$kips BS		LC5 Diagonal wave $\lambda_p=1 \equiv 1412$kips BS		Notes
x	x	[5]	7.34	[1]	6.53	
✓	x	[6]	2.81	[2]	2.82	
x	✓	[7]	3.54	[3]	5.08	
x	✓ X and K joints	[7A]	3.79		-	
✓	✓	[8]	2.72	[4]	2.80	
✓	✓	[8A]	2.72		-	Zero T-Z top 15m to maximise P-Δ
✓	✓	[8B]	2.80		-	As 8A with 150% P-Y lateral resistance
✓	✓	[8C]	2.62		-	As 8A with 75% P-Y lateral resistance
✓	✓	[8D]	2.56		-	As 8 with 5 part linear 'curves' representing foundations
✓	✓	[8E]	2.58		-	As 8D with 5 part linear 'curves' representing joint behaviour
		$\lambda_p=1 \equiv 1429$kips BS		$\lambda_p=1 \equiv 1164$kips BS		
✓	✓	[8F]	2.11	[8G]	1.81	As 8D with riser guards

X means piles fixed at mudline and/or tubular joints modelled as rigid

✓ means full account of soil-structure interaction along piles with 3 part linear representation of stiffnesses
uno
and/or full modelling of tubular X joint flexibility and limiting capacity with 3 part linear curves

Table 4.2 Summary of Ultimate Strength Analyses · Modelling and Results

Pile	Load factor at which pile fails					
	AM8IT	AM8A	AM8B	AM8C	AM8D	AM8E
A2	2.38	2.37	2.44	2.25	2.32	2.42
A1	2.39	2.38	2.46	2.26	2.34	2.44
B2	2.57	2.57	2.73	2.45	2.47	2.54
B1	2.58	2.57	2.74	2.46	2.49	2.56
C2	2.65	2.64	2.62	2.54	2.51	2.57
C1	2.65	2.65	2.62	2.54	2.52	2.58

Table 4.3 Order of pile failure for different levels of soil stiffness - end-on wave

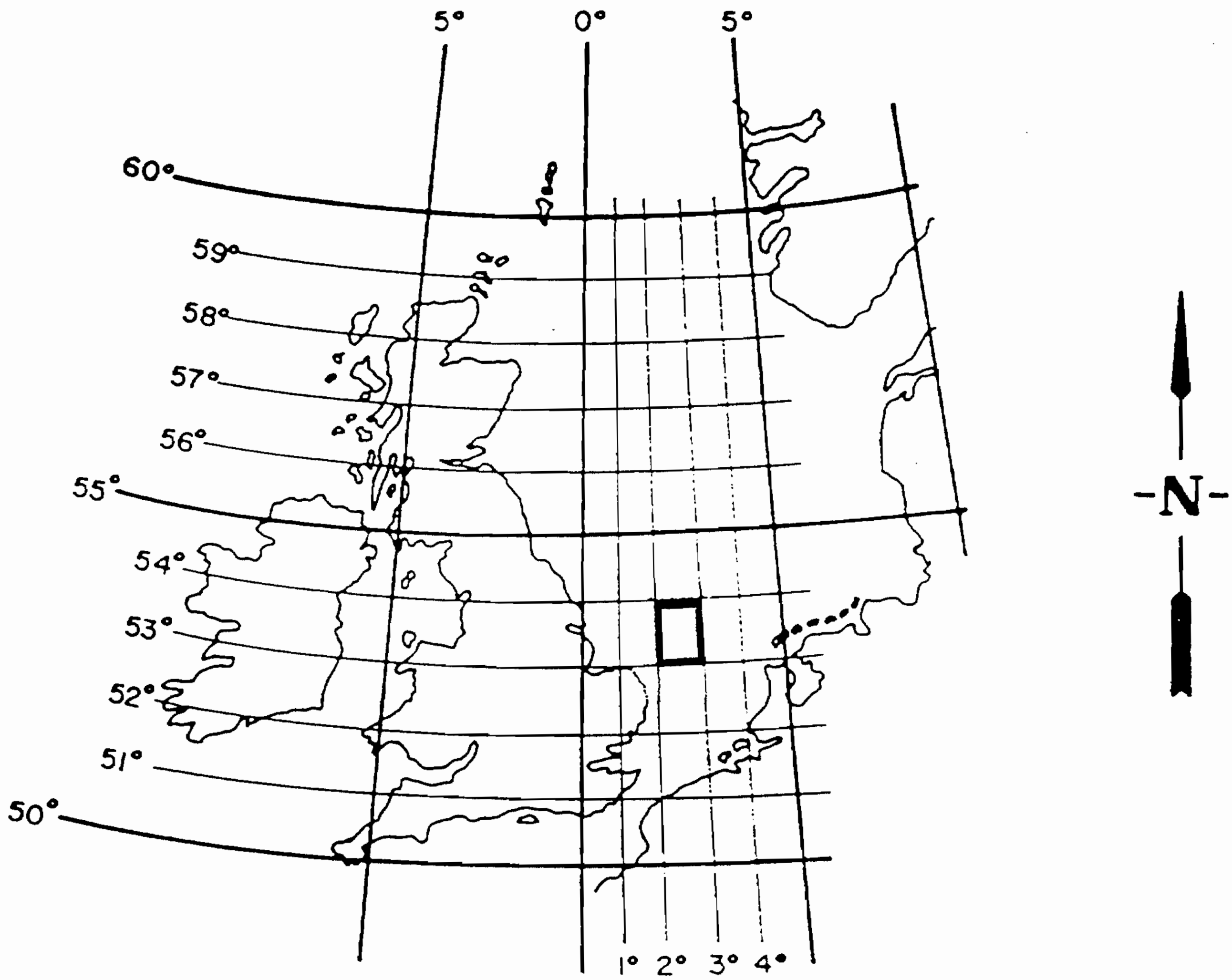


Figure 2.1 Location of Southern North Sea platform

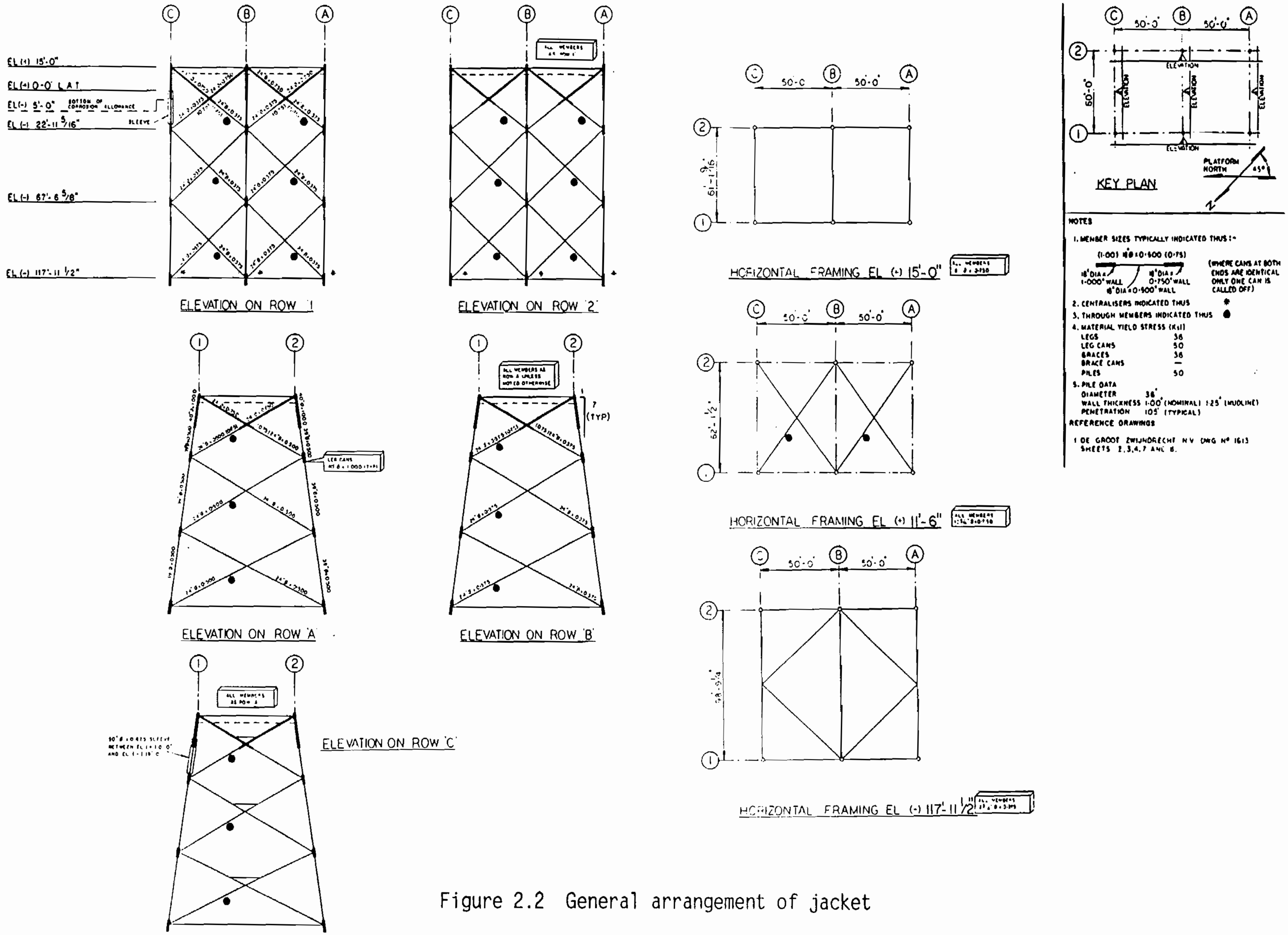


Figure 2.2 General arrangement of jacket

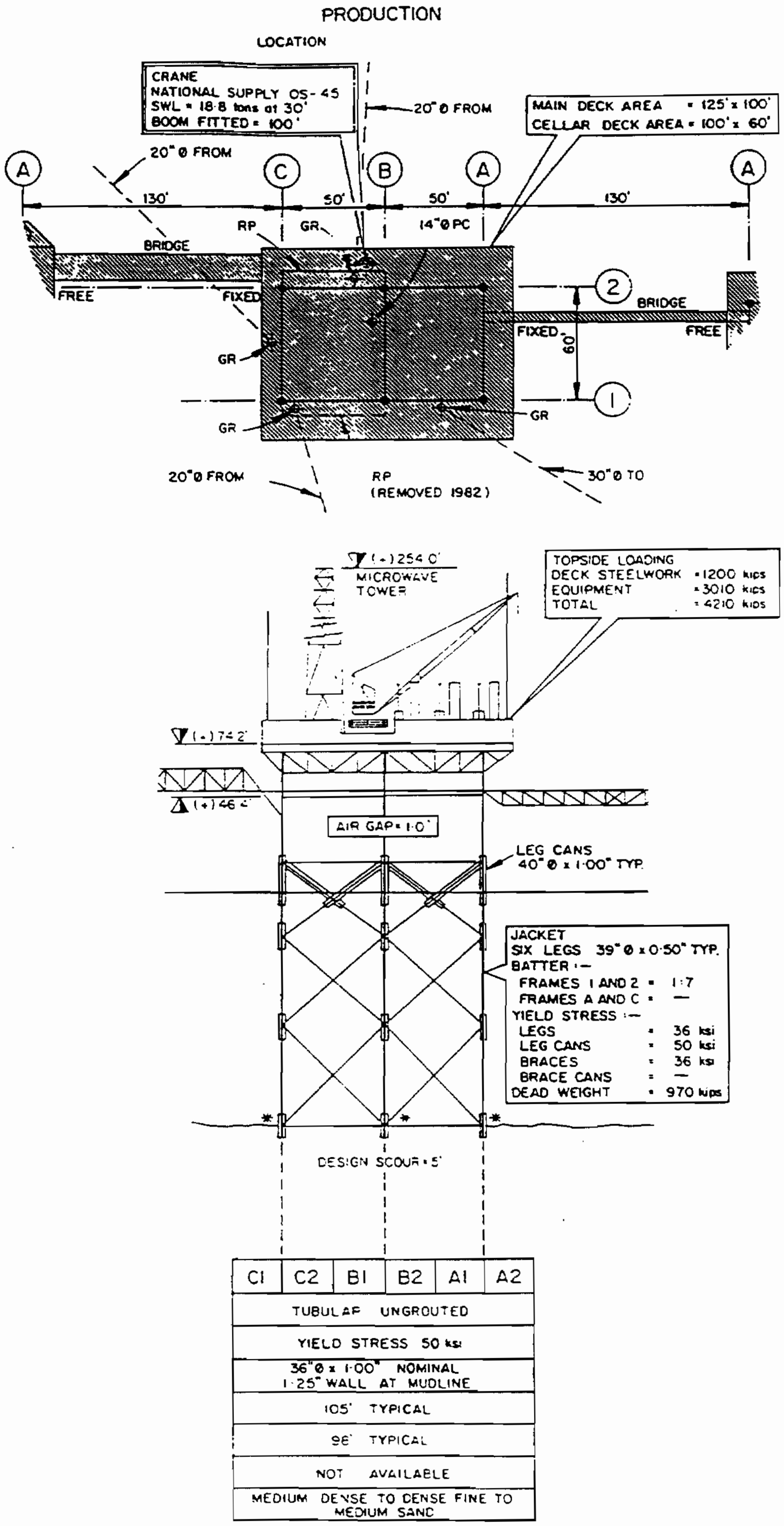
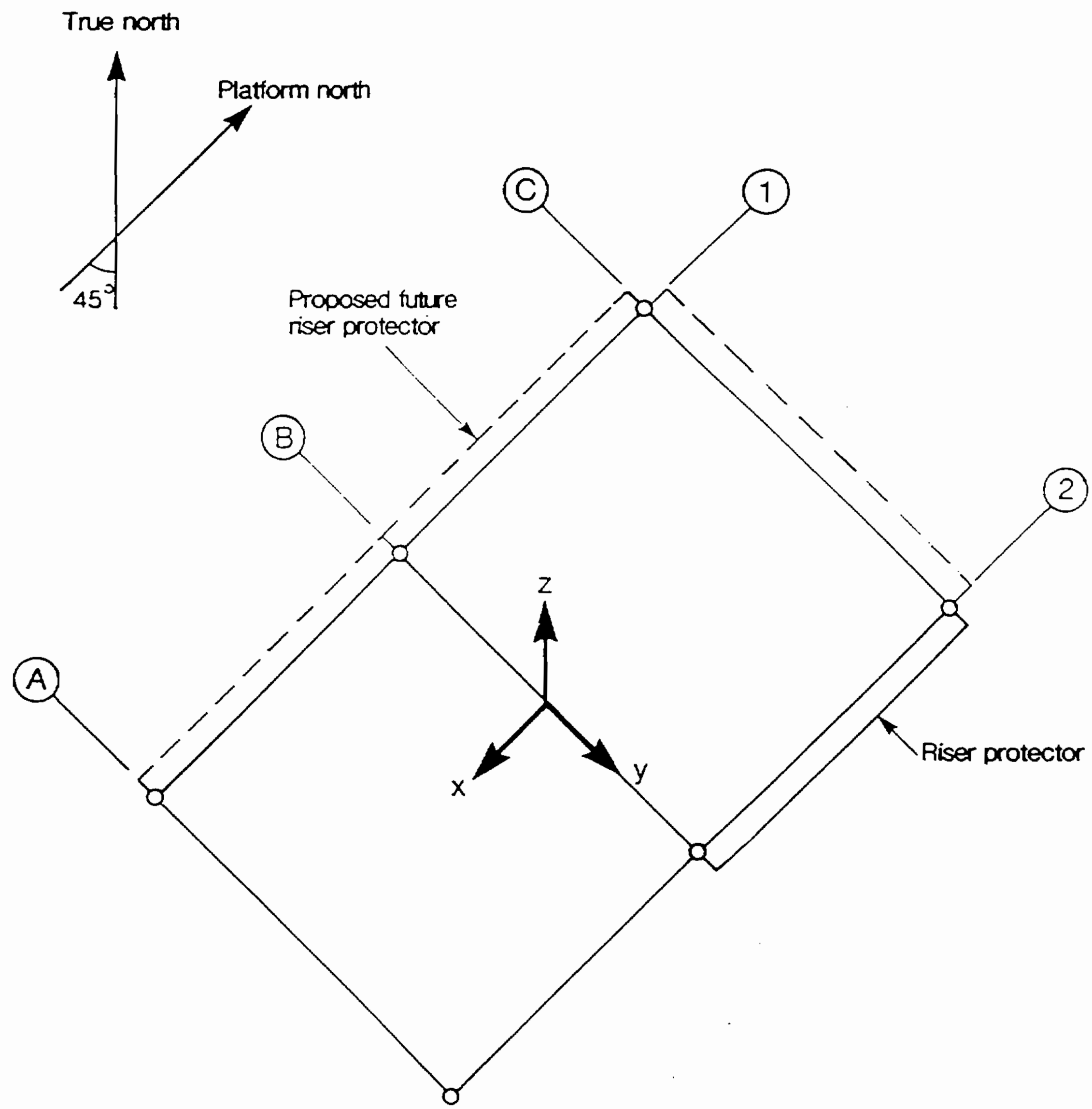


Figure 2.3 Platform details - Certification data



Notes: x, y, and z axes shown, refer to the Global Coordinate System.
z is vertically upwards, with origin at mudline.

Figure 2.4 Location of proposed riser guards

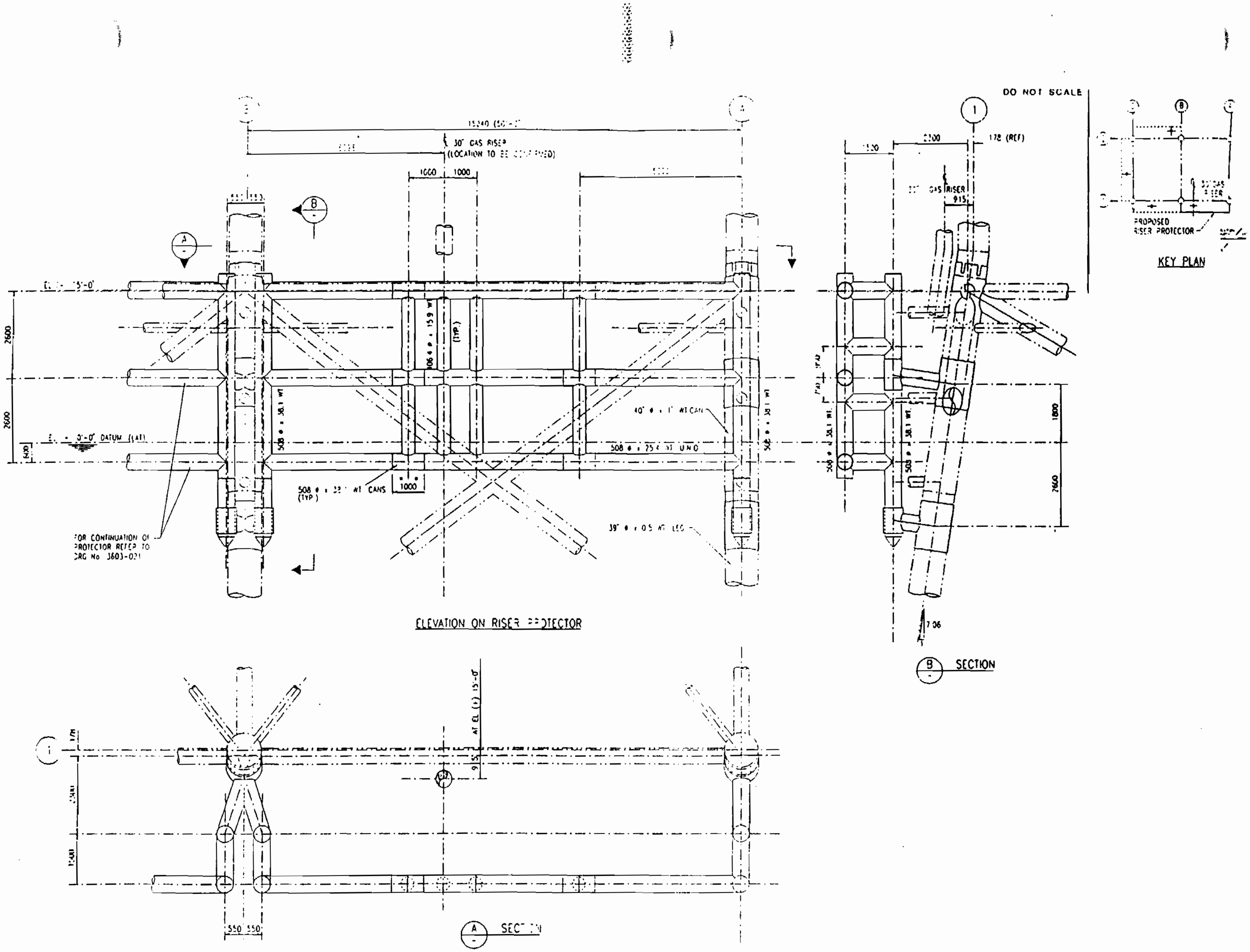
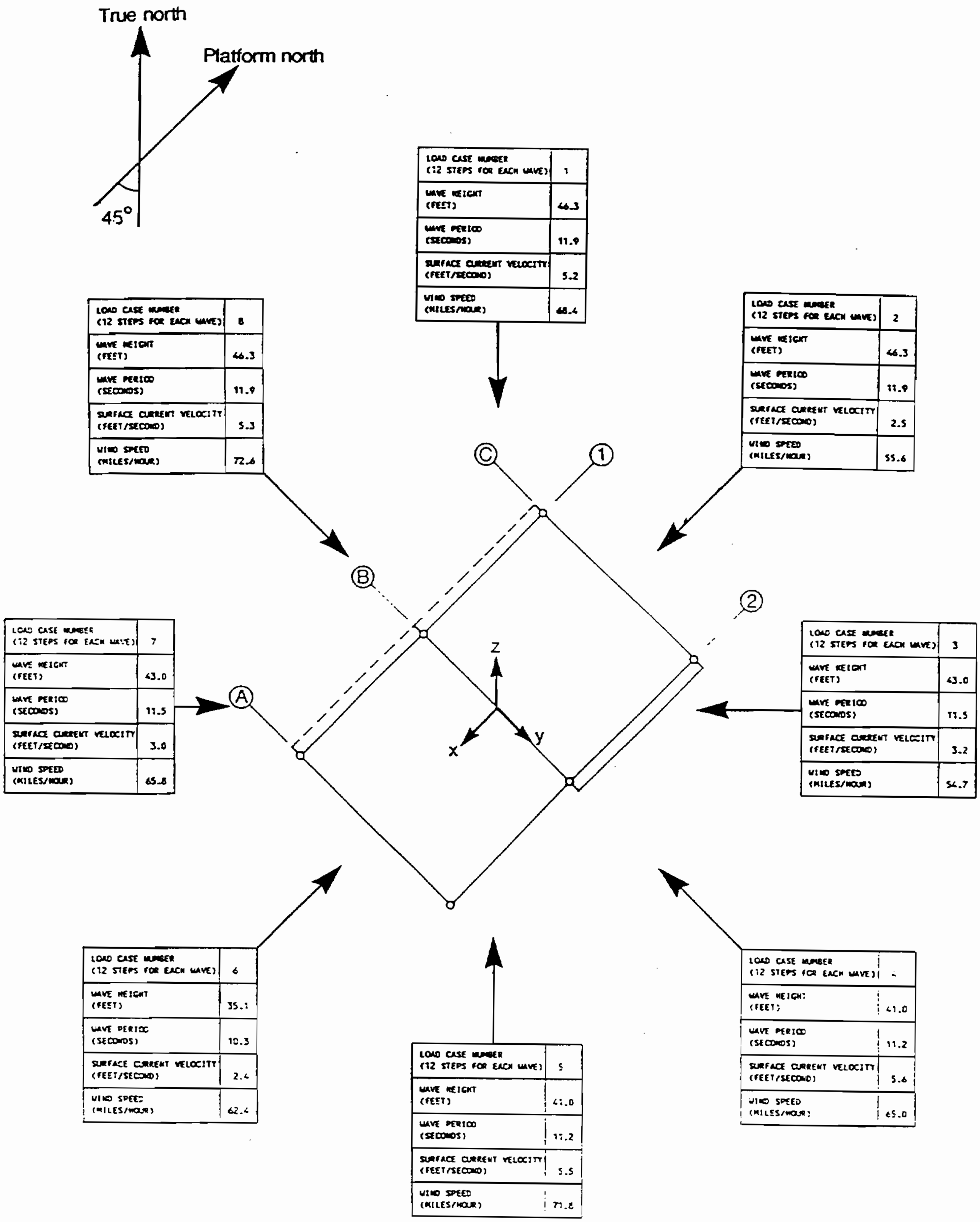


Figure 2.5 Typical configuration of proposed riser guards



Note: 2 water depths are investigated (ML and MHWS plus surge).

Figure 2.6 Directional metocean parameters for Design Level Analysis

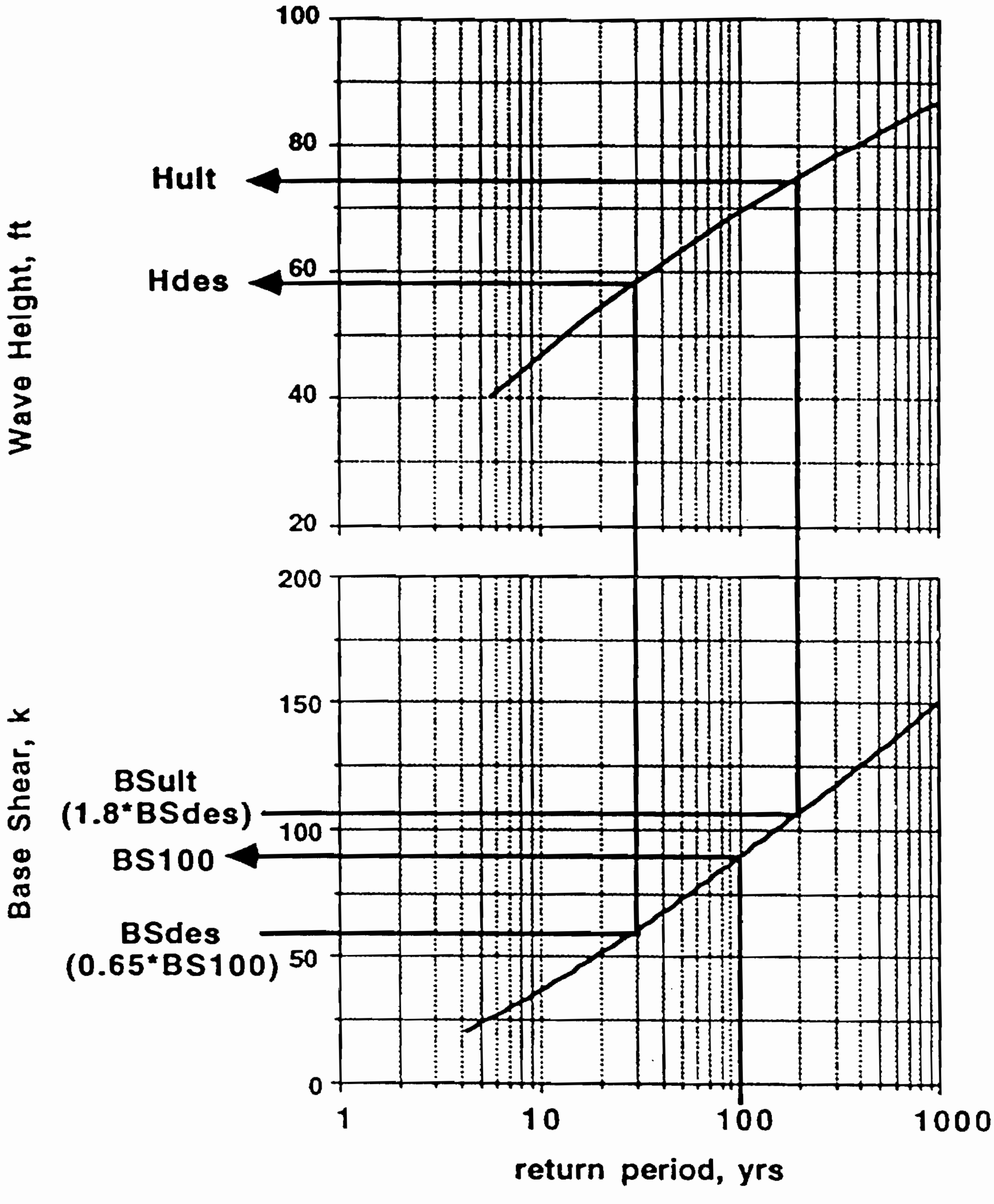


Figure 4.1 Basis of metocean criteria of assessment (6)

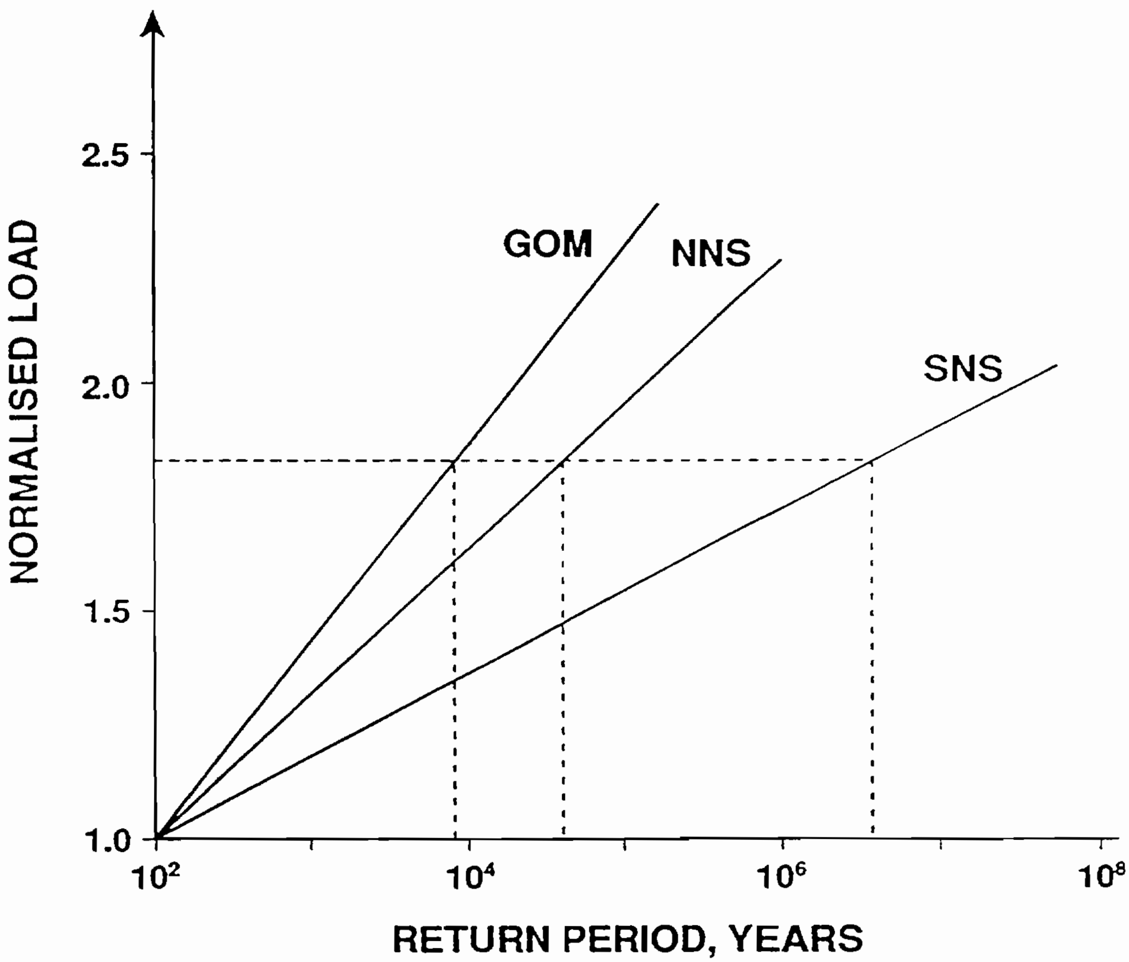
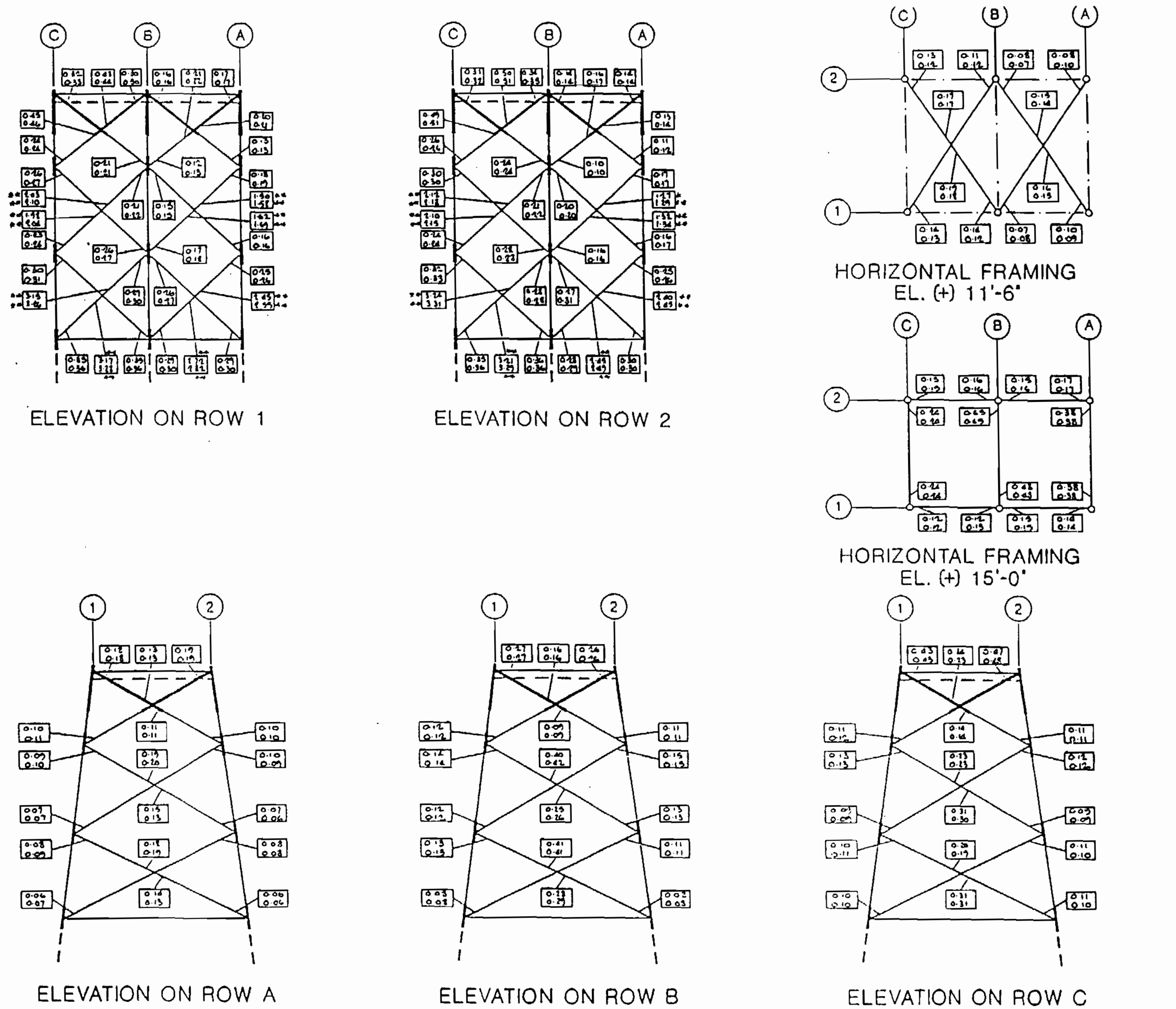
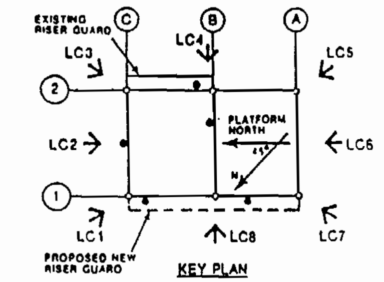


Figure 4.2 Relation between base shear and return period with environment (10)





GENERAL NOTES

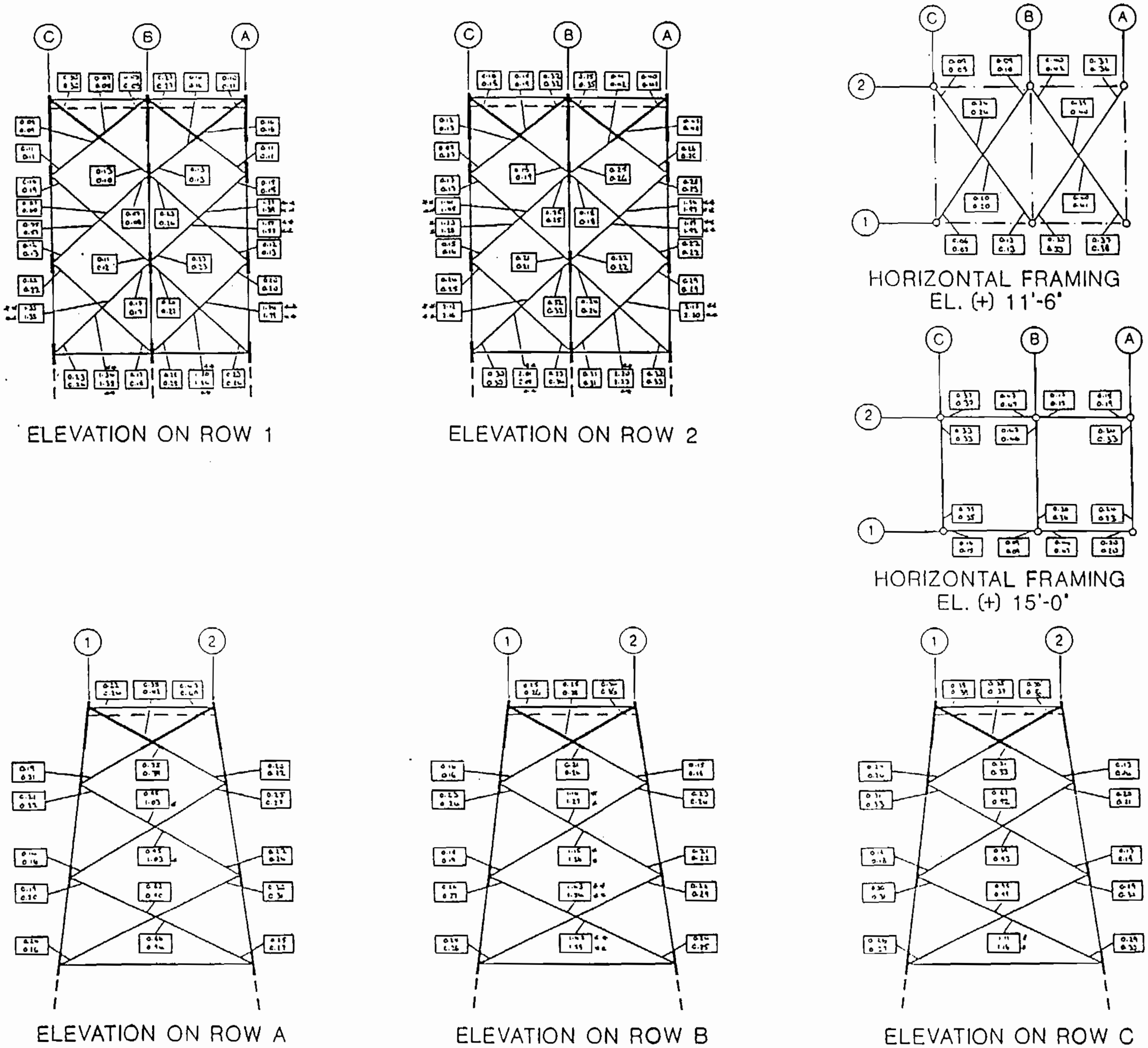
1. Analysis carried out by BOMEL in January 1990.
2. For environmental data, see section 3.4 of report number BOMEL 179/90 Rev.0, March 1990.
3. The unity ratios shown are based on API RP2A 18th Edition.
   A 1.33 allowable stress increase is applied.
4. Legend

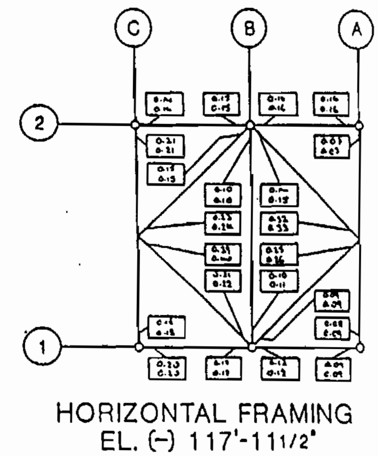
Joint UF Structure A
Joint UF Structure B

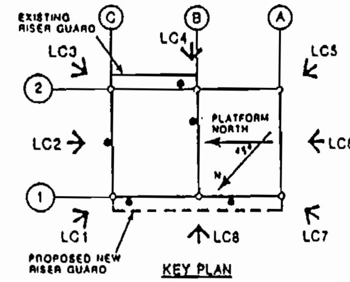
where Structure A is the jacket structure as at January 1990,
and Structure B is the jacket structure as at January 1990
plus additional riser protectors on Row 1.

5. Values shown thus * exceed the permissable utilisation factors. Values shown thus ** exceed the API lower bound capacity of the joint.

Figure 4.3 Tubular joint code check results for design level analysis - End-on wave loadcase







GENERAL NOTES

1. Analysis carried out by BOMEL in January 1990.
2. For environmental data, see section 3.4 of report number BOMEL 179/90 Rev.0, March 1990.
3. The unity ratios shown are based on API RP2A 18th Edition. A 1.33 allowable stress increase is applied.
4. Legend

Joint UF Structure A
Joint UF Structure B

where Structure A is the jacket structure as at January 1990, and Structure B is the jacket structure as at January 1990 plus additional riser protectors on Row 1.

5. Values shown thus * exceed the permissible utilisation factors. Values shown thus ** exceed the API lower bound capacity of the joint.

Figure 4.4 Tubular joint code check results for design level analysis - Diagonal wave loadcase

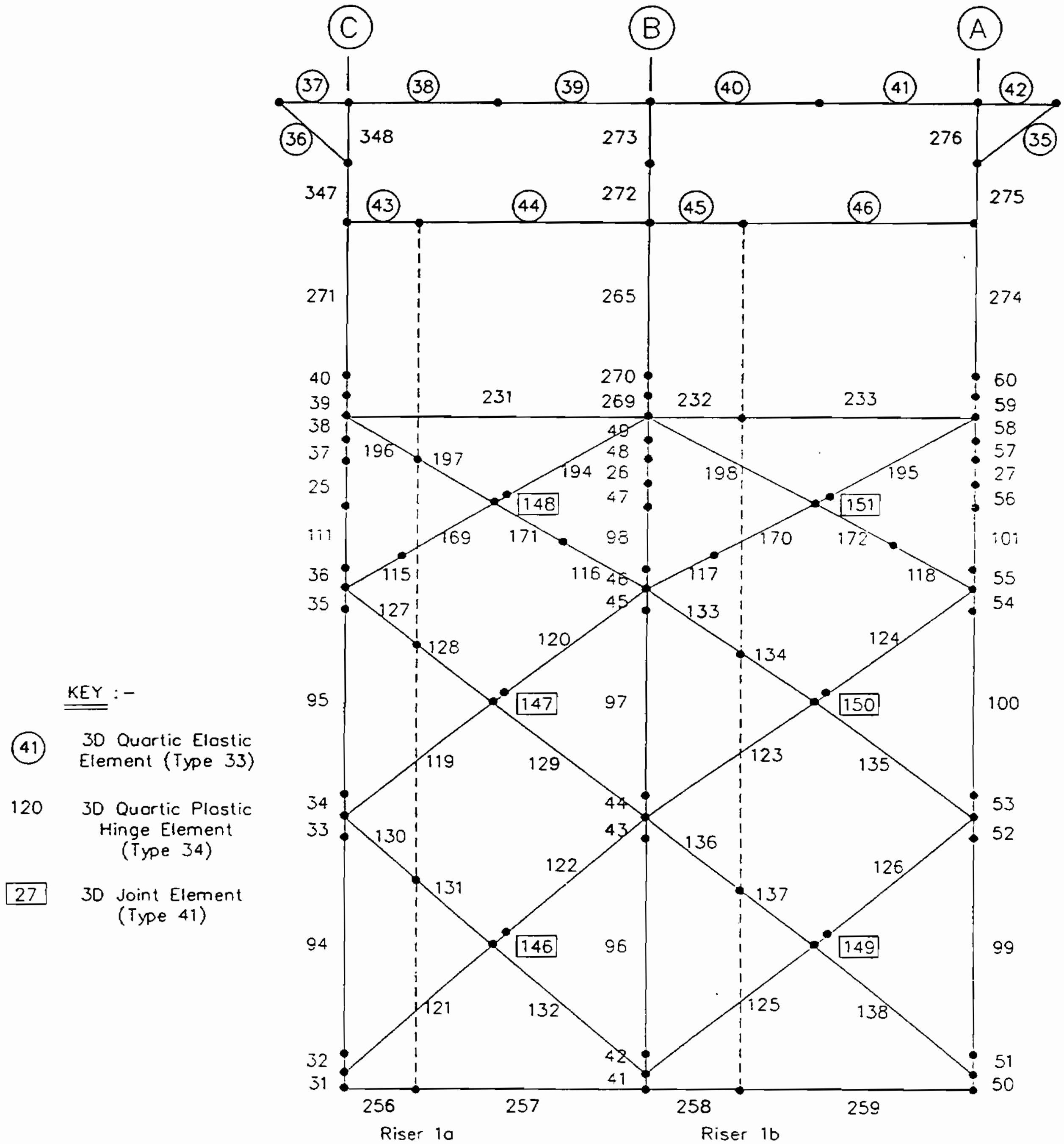


Figure 4.5 Row 1 elements in SAFJAC model

KEY :-

(9)	3D Quartic Elastic Element (Type 33)
182	3D Quartic Plastic Hinge Element (Type 34)
[43]	3D Joint Element (Type 41)

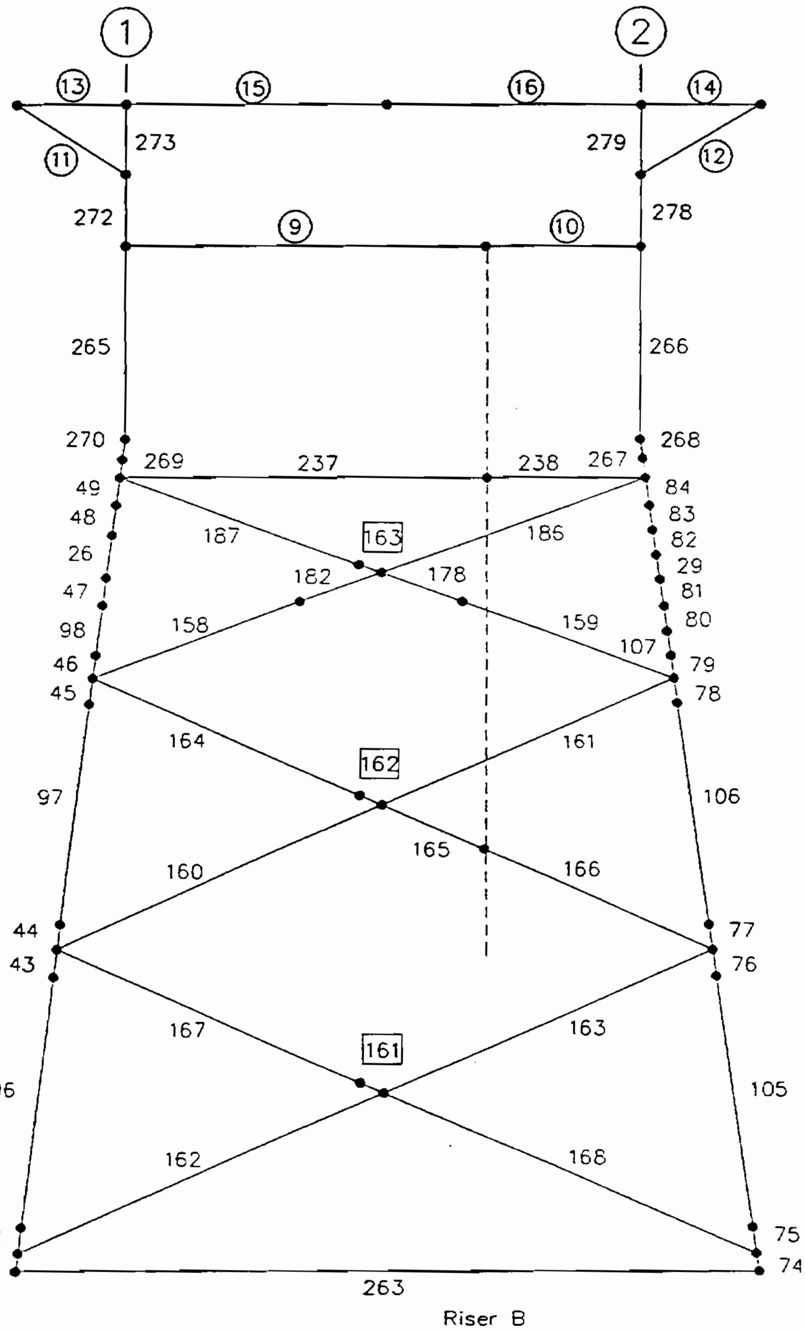


Figure 4.6 Row A elements in SAFJAC model

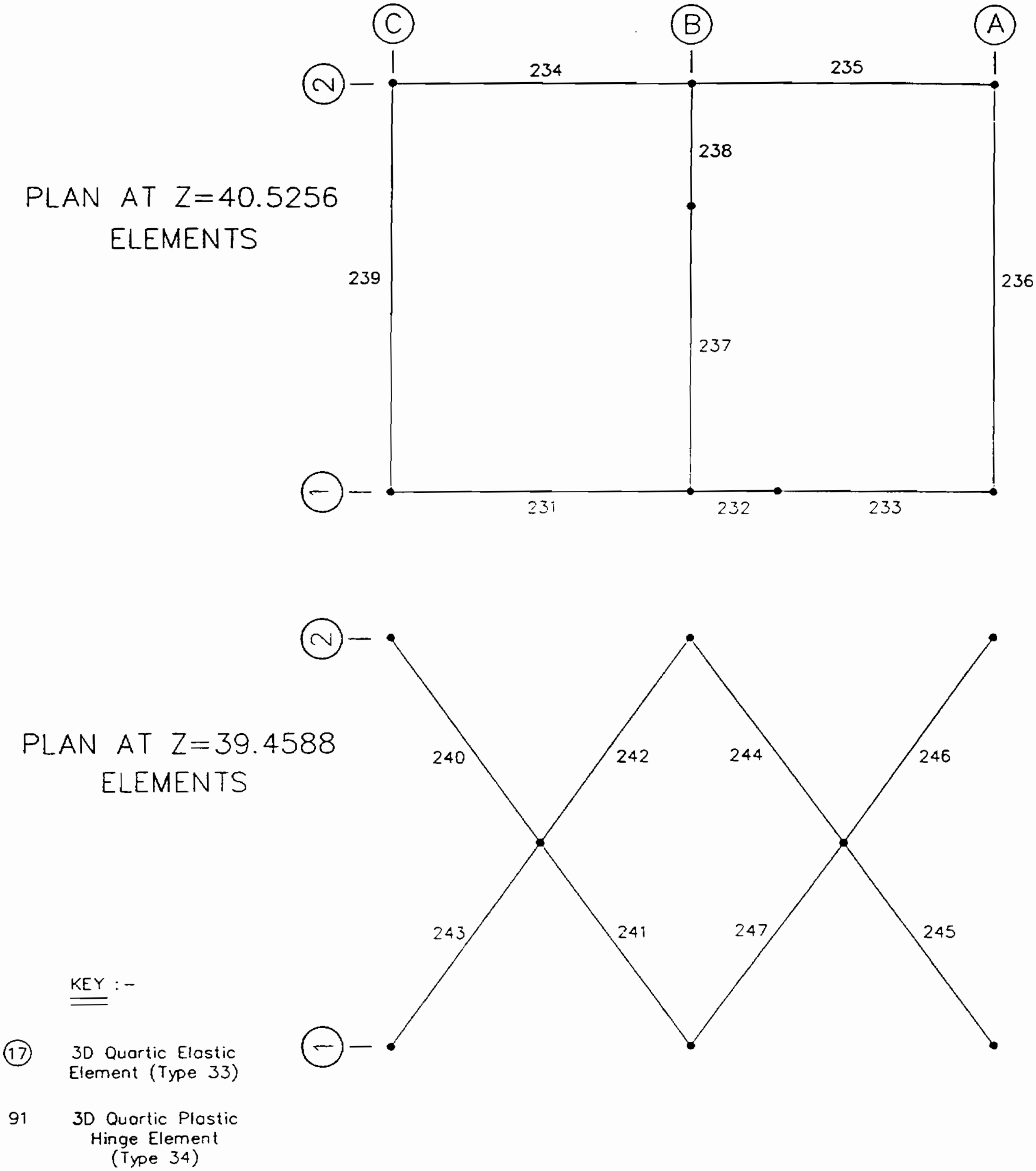


Figure 4.7 Plan elements in SAFJAC model

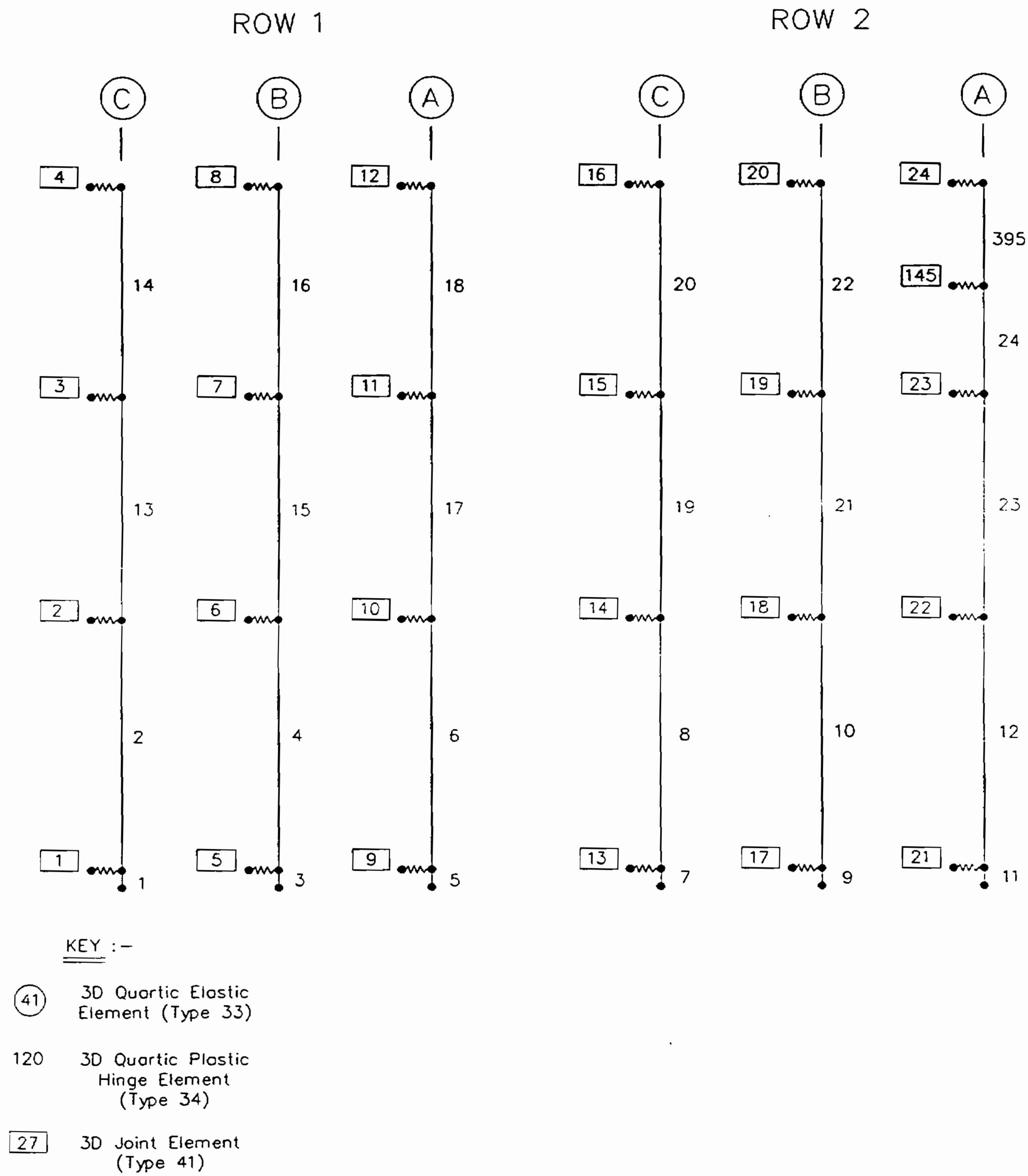


Figure 4.8 Leg pile elements in SAFJAC model

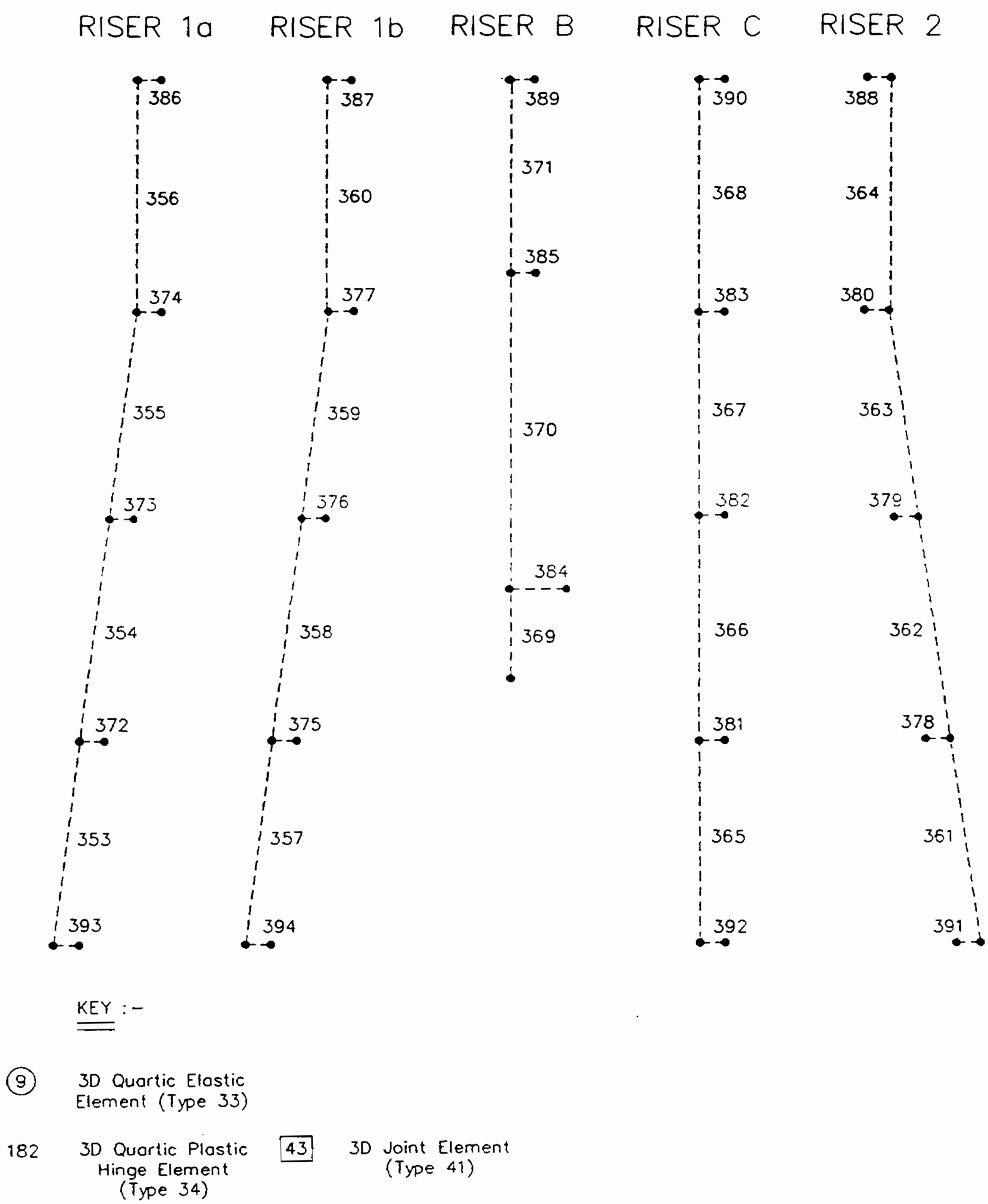


Figure 4.9 Riser/caisson elements in SAFJAC model

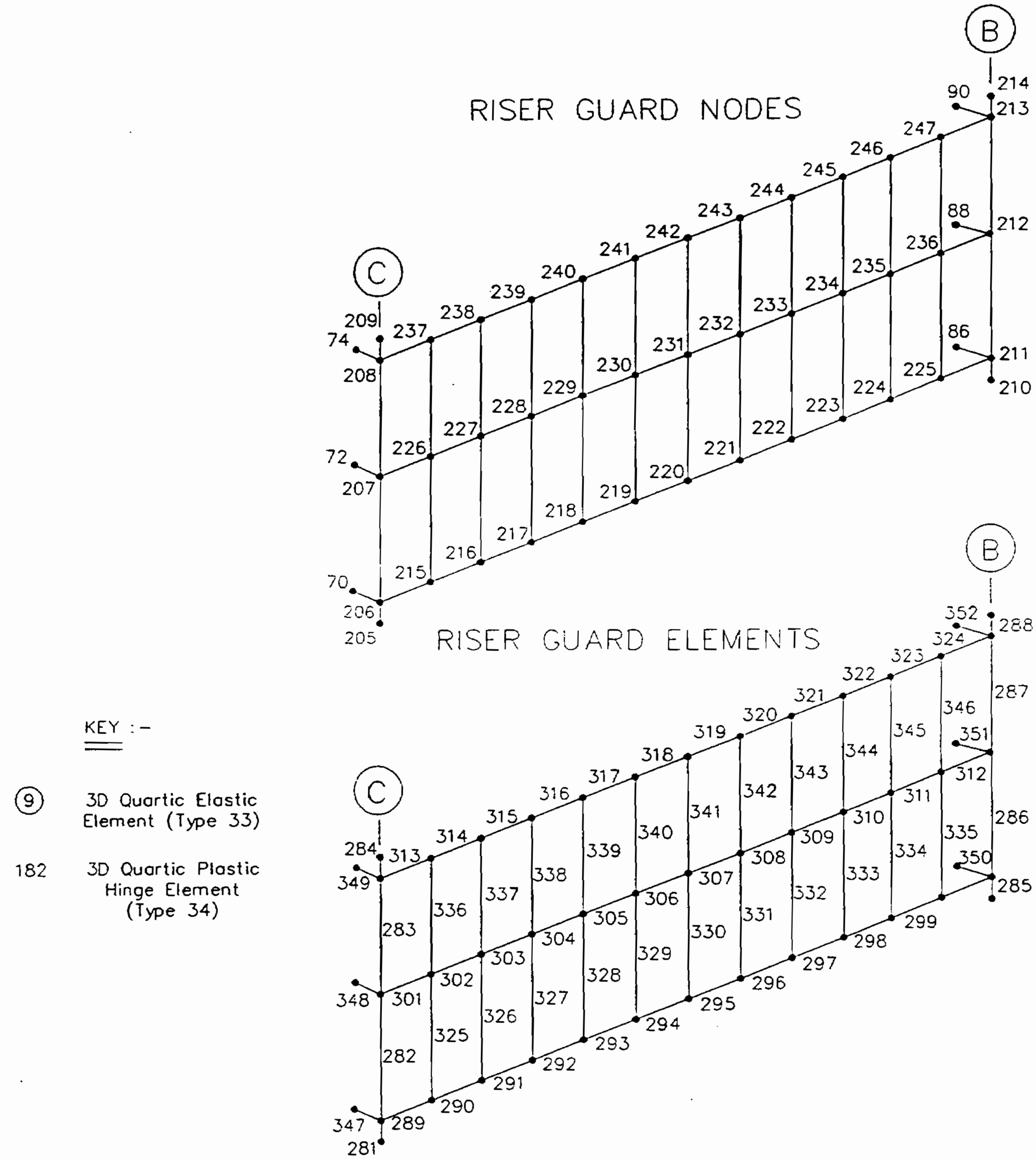


Figure 4.10 Riser guard representation in SAFJAC model

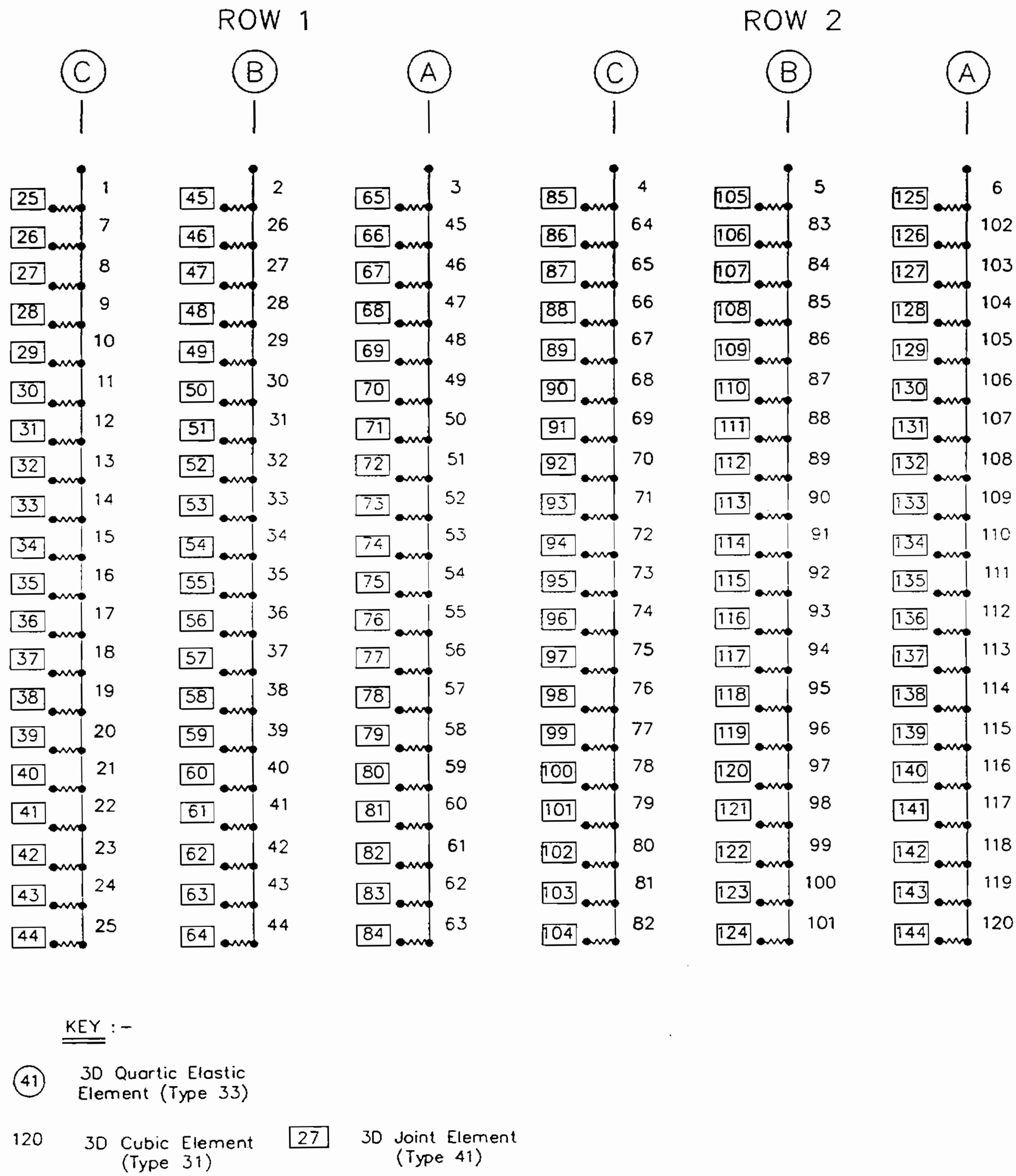


Figure 4.11 Pile elements in SAFJAC model

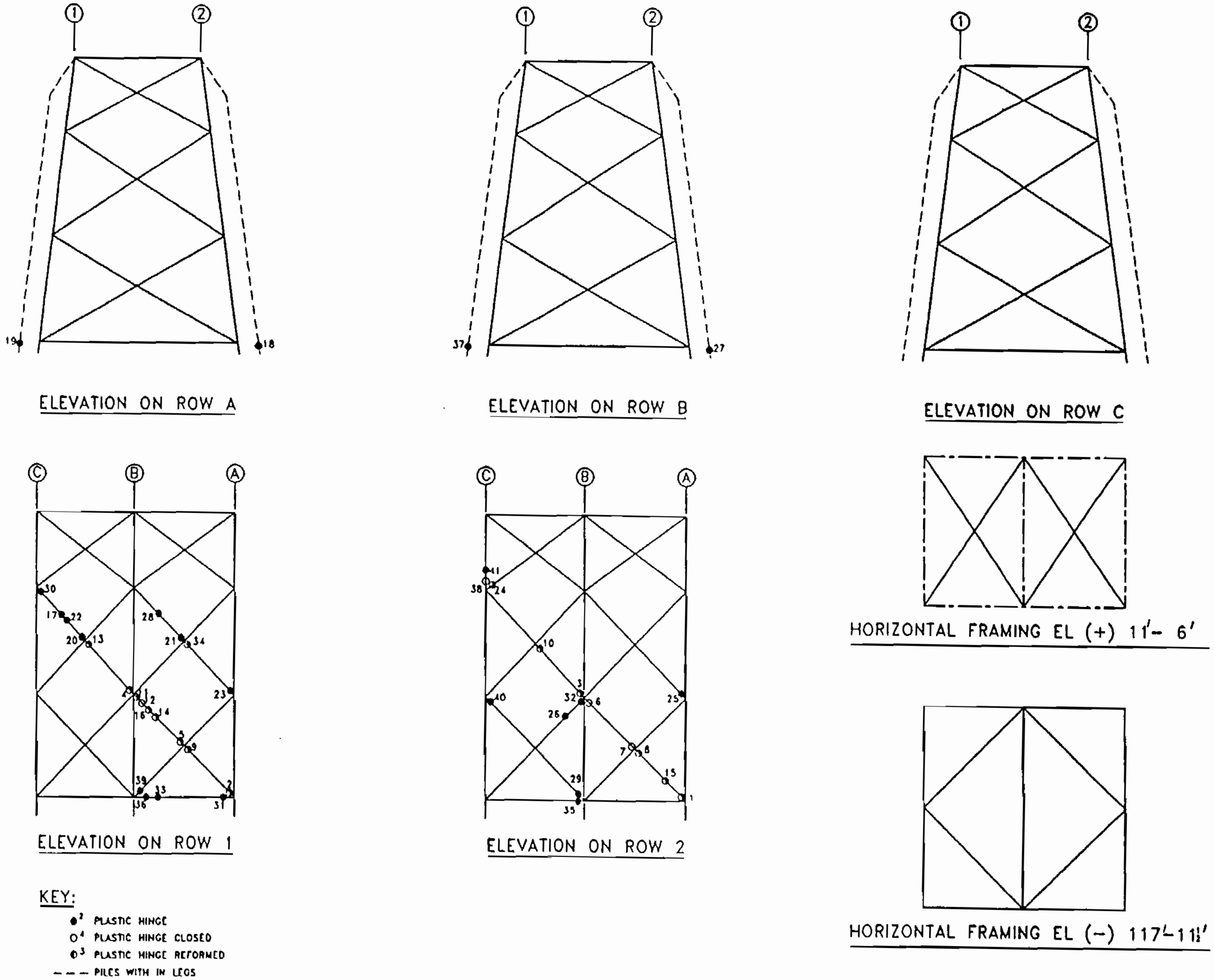


Figure 4.12 Plastic hinge formation for fixed based jacket, end-on wave with X joint modelling included (Analysis reference 7, see Table 4.2)

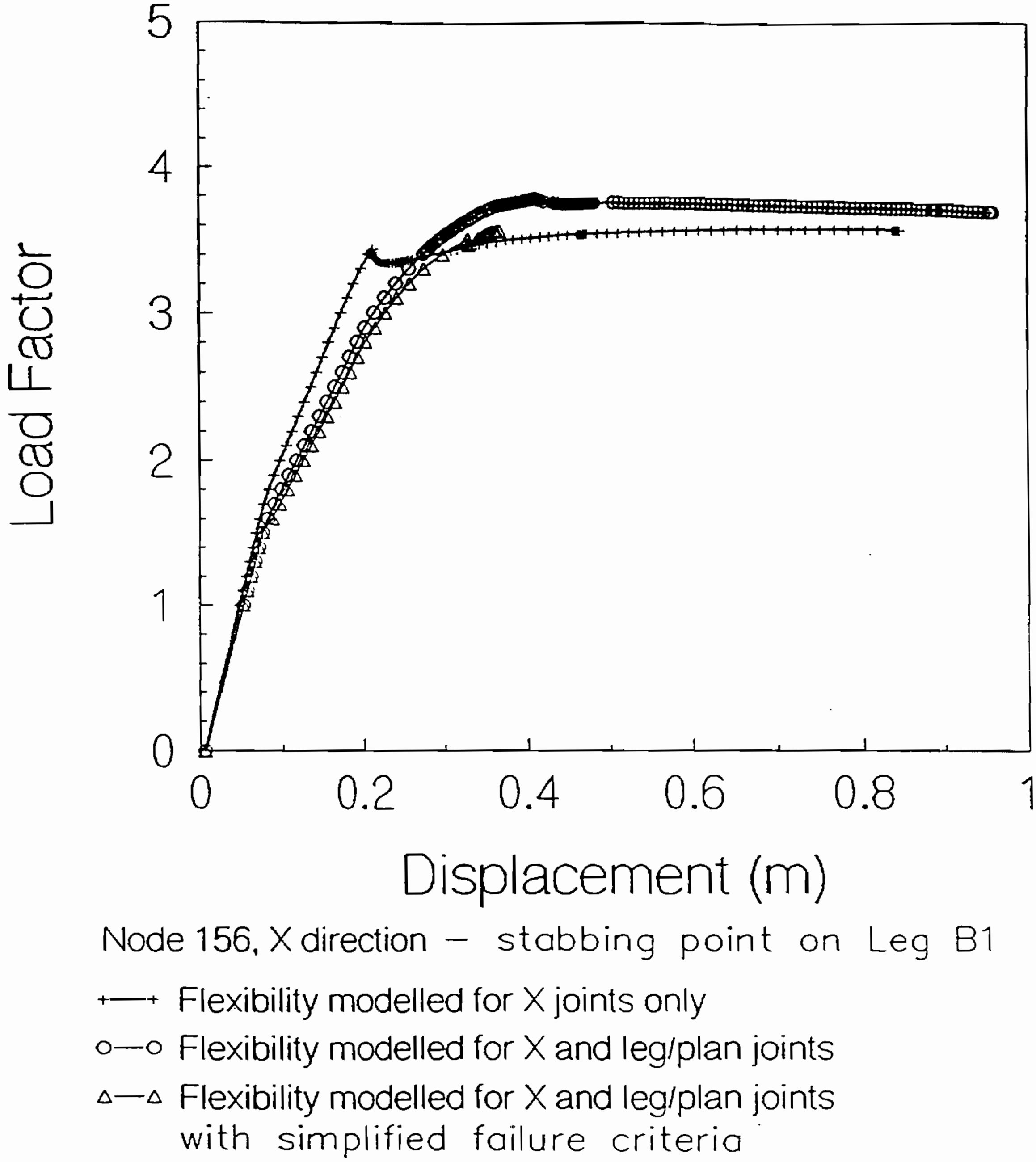


Figure 4.13 Comparison of global responses for fixed base jacket and end-on wave, with different joint modelling assumptions

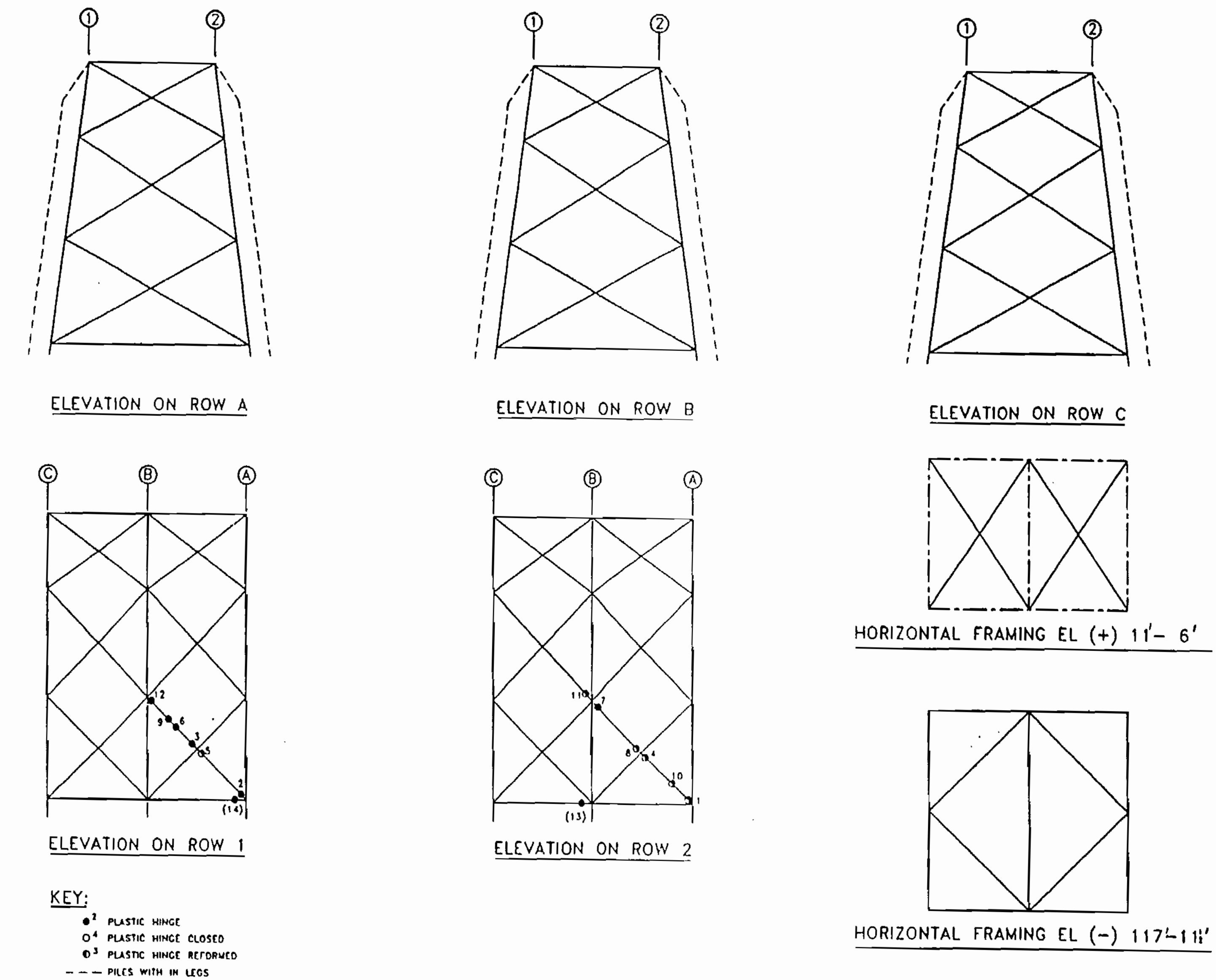


Figure 4.14 Plastic hinge formation for piled jacket, end-on wave with X joint modelling included (Analysis reference 8, see Table 4.2)

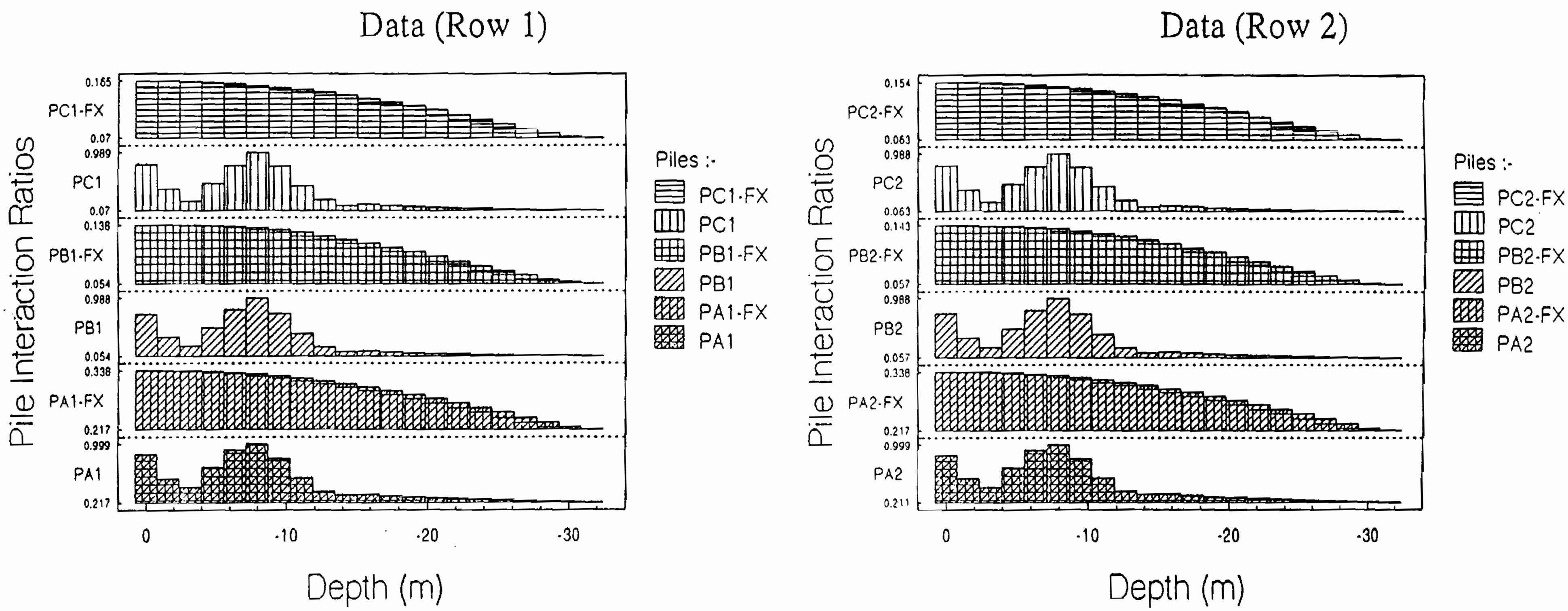


Figure 4.15 Pile utilisation at maximum load factor for piled jacket, end-on wave with X joint modelling included
(Analysis reference 8, see Table 4.2)

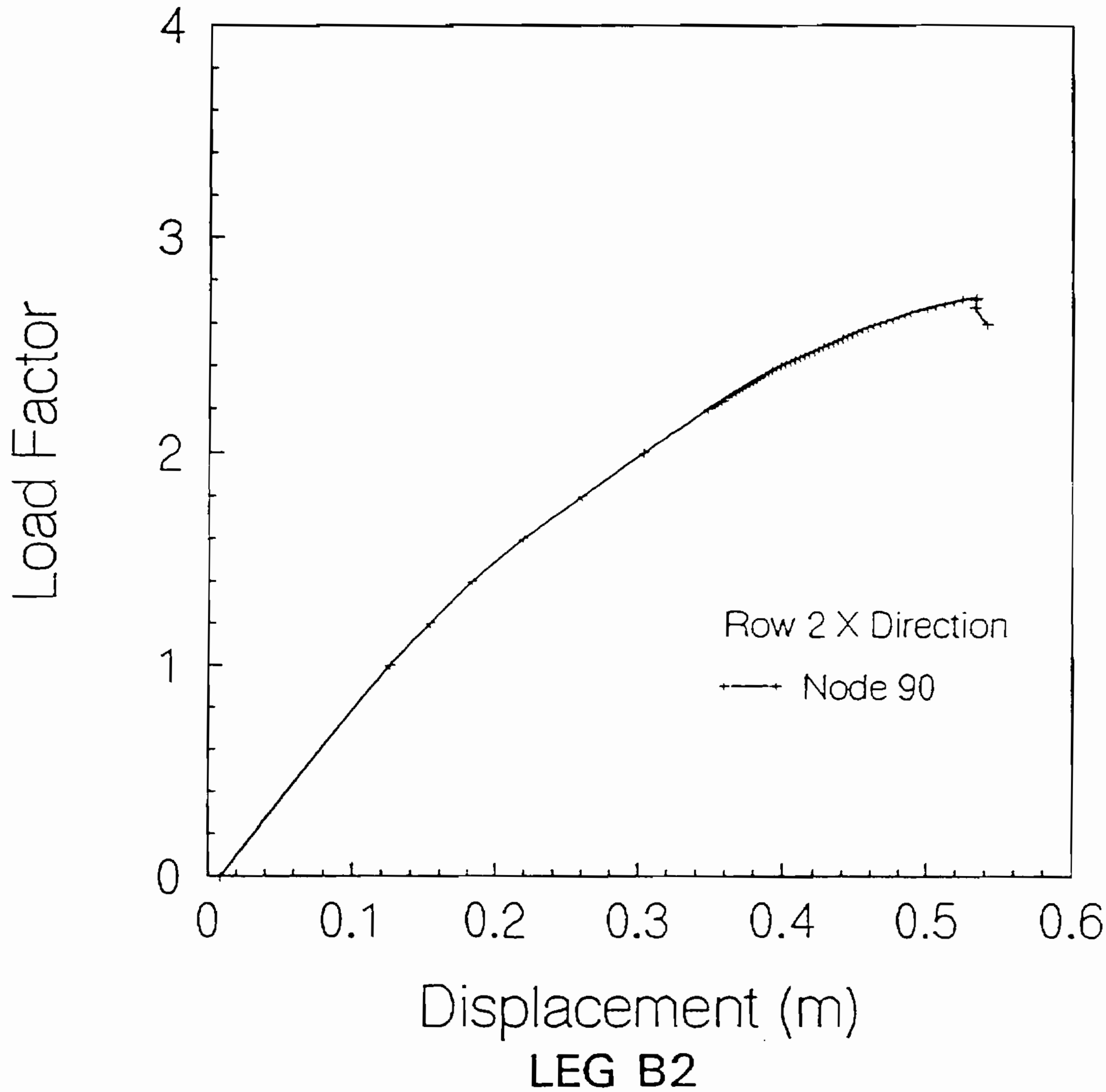


Figure 4.16 Global load-deflection responses determined for leg nodes for piled jacket, end-on wave with X joint modelling included (Analysis reference 8, see Table 4.2)

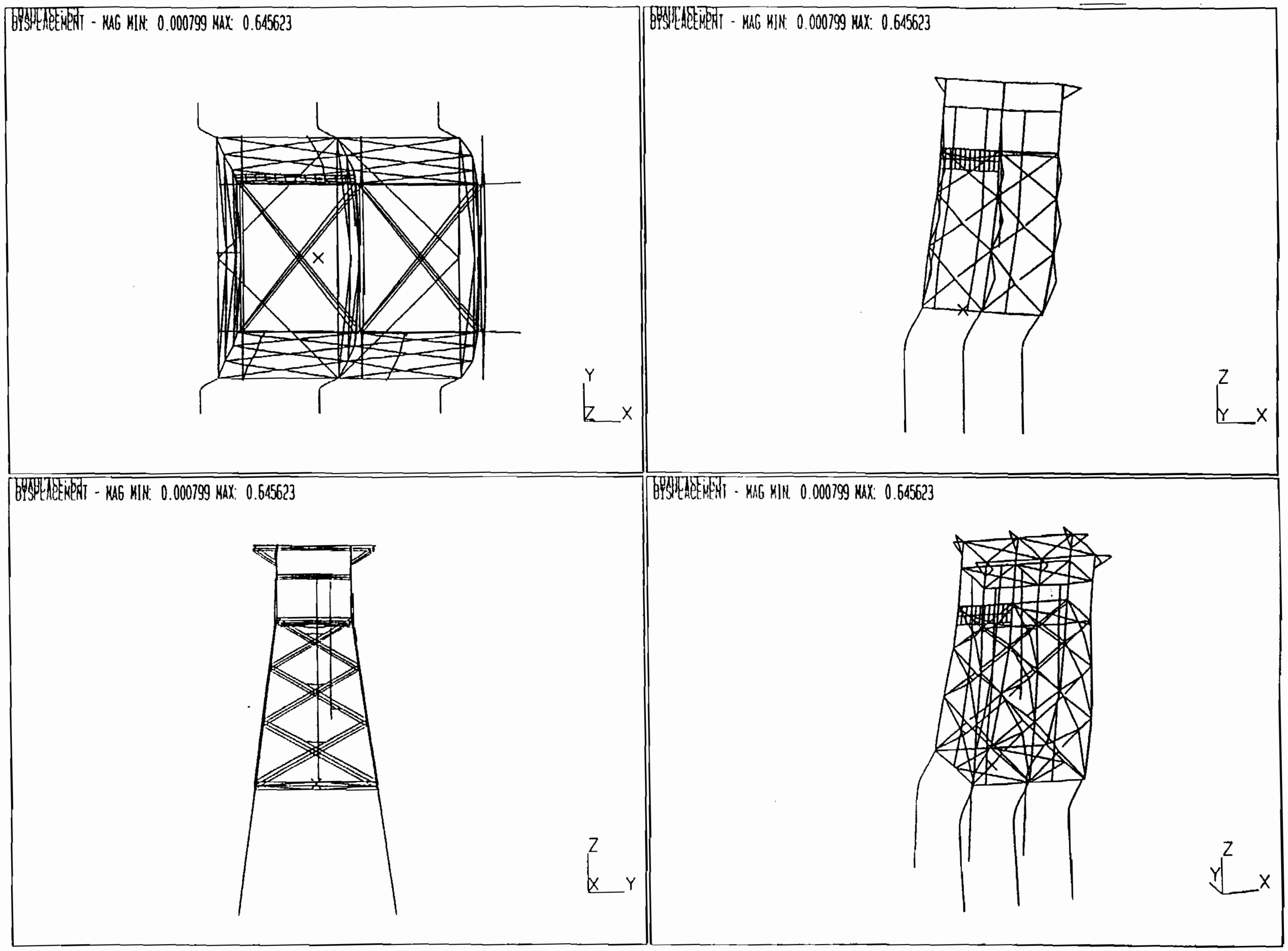


Figure 4.17 Deflected shape at maximum load factor for piled jacket, end-on wave with X joint modelling included (Analysis reference 8, see Table 4.2)

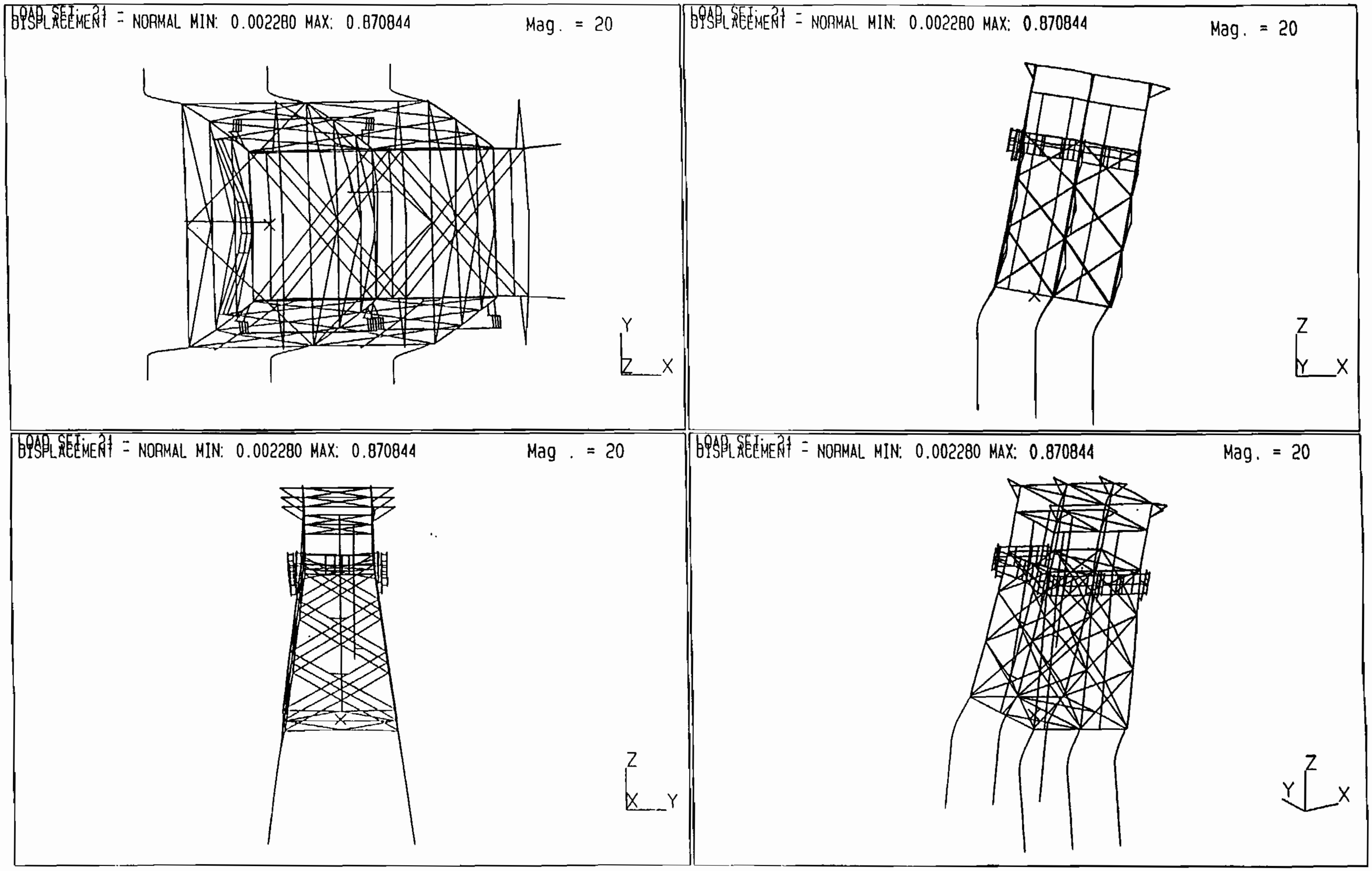


Figure 4.18 Deflected shape at maximum deflection for platform with additional riser guards, end-on wave, and piled foundation and X joint modelling with 5 part linear springs (Analysis reference 8F, see Table 4.2)

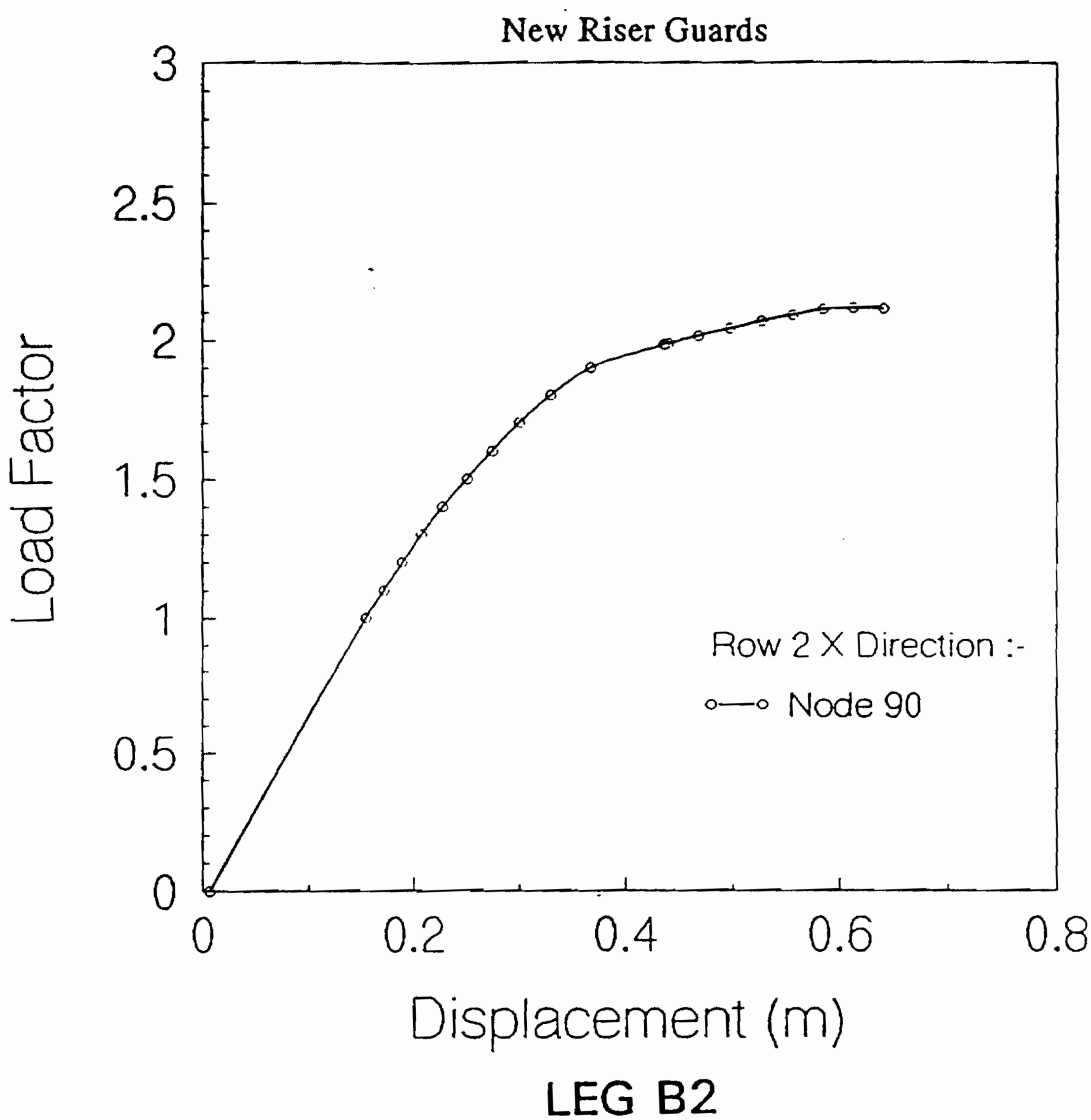


Figure 4.19 Global load-deflection response for platform with additional riser guards, end-on wave, and piled foundation and X joint modelling with 5 part linear springs (Analysis reference 8F, see Table 4.2)

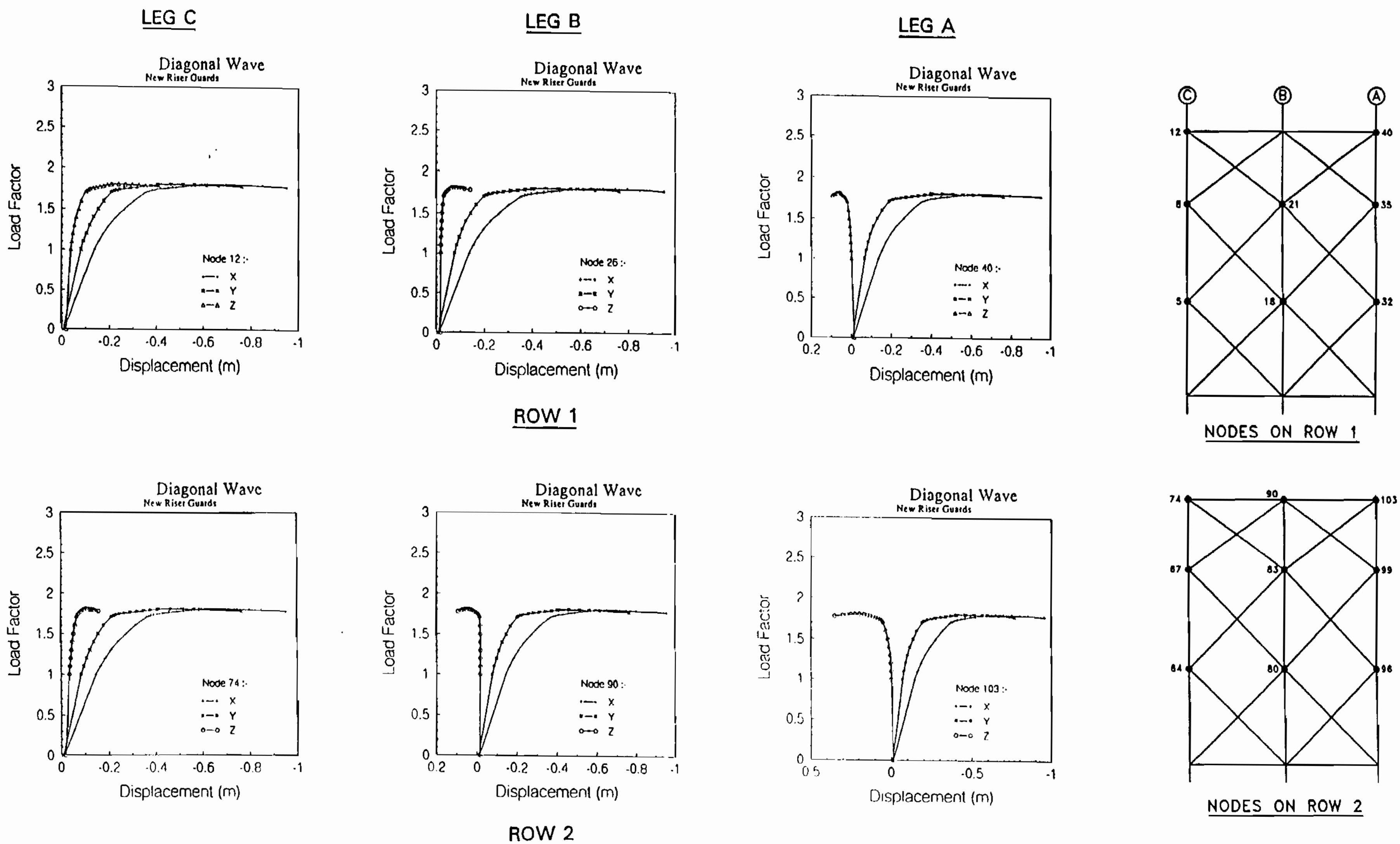


Figure 4.20 Global load-deflection response for platform with additional riser guards, diagonal wave, and piled foundation and X joint modelling with 5 part linear springs (Analysis reference 8G, see Table 4.2)

PLATFORM "U"

PLATFORM 'U' – REVISED PAGES (REV. 1).

Table of Contents

1.0 SUMMARY

This report presents the results of the trial application the API RP 2A draft Section 17.0 for the assessment of existing platforms (Ref. 1) to the Nohoch B drilling platform in the Bay of Campeche in the Mexican Gulf. The platform is an 8 leg jacket in 125ft water depth. It was designed and installed in 1979, presumably to the 9th Edition of API RP 2A. The platform has significant damage (dents, bowed members and cracks) in the splash zone due to vessel impacts. Furthermore a cantilever deck, for storage and associated facilities, has been added during the service life**. Damage was the principal assessment initiator. The platform was classified as manned-evacuated with significant environmental impact.

As the platform falls outside the US Gulf of Mexico, specific metocean assessment criteria had to be devised. Furthermore new data were available indicating significantly lower 100 year return period loads than assumed in the original design. Given the lack of an experience base for these metocean parameters it was deemed prudent to adopt the 100 year base shear (BSC100) as the criterion for evaluating the Design Level Analysis and 1.8*BSC100 for the Ultimate Strength Level Analysis in place of the corresponding 0.65*BSC100 and 1.2*BSC100 factors for the US Gulf of Mexico.

Given the extent of damage the design basis check was not appropriate, however the platform passed the assessment at the Design Level Analysis stage. The analysis models included specific modelling of the damage. The subsequent ultimate strength analyses demonstrated the system reserves available. The new loading data contributed to the low probability of failure that could be demonstrated for this particular structure.

The assessment has confirmed that the performance of the platform is satisfactory and has served to validate the Section 17.0 process whilst demonstrating its appropriate adaptation for non-US waters.

A summary of the trial assessment is given in Table 3-8#*.

**This information is preliminary and was not verified in the field, but was considered convenient to be taken in account for assessment purposes.
* Tables noted with a # suffix have identical numbering and format as those in the DRAFT of "Trial Applications" prepared by PMB Engineering Inc., September 1994.

2.0 PLATFORM INFORMATION

2.1 Physical Features

Nohoch B is a drilling platform, located in a 38.100m (125 ft) water depth, in the Bay of Campeche offshore in the Gulf of Mexico (Figures 2.6a and b) and installed in 1979. It has four plans at elevations +3.658, -8.534, -22.250 and -38.100m; 2 longitudinal and 4 cross vertical frames. The jacket has eight legs of 52" diameter with eight foundation piles, each running the full length of each leg with the annulus ungrouted. The pile penetrations are 210m at the middle and 240m at the corners of the jacket. The bracing types are K, X and Y (diagonal). The deck has two plans, cellar deck at elevation +15.850 and the main deck at elevation +20.749 m. Overall, the platform appears similar to the conventional type of drilling, quarters and production structure designed for the US Gulf of Mexico.

2.2 Operational Information

Nohoch B has 12 wells (8 in service). At cellar deck there is a separator tank for 70,000 barrels per day. At some stage the platform may have living quarters.

This information is preliminary and was not verified in the field, but was considered convenient to be taken in account for assessment purposes.

2.3 Pertinent Inspection Information

The platform was periodically inspected and evaluated:

Year 1983	Inspection Report No. NHBF2 610-1-U
Year 1985	Inspection Report No. NHBF2 610-2-U
Year 1988	Inspection Report No. NHBF2 610-3-P/S
Year 1990	Inspection Report No. NHBF2 610-4-U
Year 1991	Inspection Report No. NHBF2 610-5-U
Year 1993	Inspection Report No. NHBF2 610-6-U

At the time of the assessment, the structure has 1 cracked joint (partially repaired) and 4 dented members in the splash zone of which 3 are bent, and there are 3 cracks around a barge bumper connection to the leg.

3.0 PLATFORM ASSESSMENT

3.1 Platform Selection

Nohoch B is an eight-leg jacket drilling platform located in Campeche Bay in the Gulf of Mexico, installed in February 1979 in 38.100m (125ft) water depth. It was designed to support 2,500 tons of drilling equipment and to withstand a 100-year return period storm, fixed on the seabed through an eight pile foundation.

The API RP 2A Section 17.0 Draft Guidelines provide the following six initiators for an assessment:

1. Addition of Personnel
2. Addition of Facilities
3. Increased Loading of the Structure
4. Inadequate Deck Height
5. Damage Found During Inspections
6. Regulatory Requirement

It will be demonstrated that according to Section 17.2, this platform could be considered to require assessment due to the addition of facilities and therefore increase of loading if these were shown to be significant. However, several dented members and cracked joints have been found during inspections and Item 5. in the above list was taken as the principal initiator. The summary of assessment initiators is presented in Table 3-1#.

In order to follow the methodology indicated in "Platform Selection" of the Draft Guideline, additional information is presented below.

3.1.1 Addition of personnel

The platform is classified as "Manned-Evacuated" for purposes of the current assessment work, as described in Section 3.2 of this report. The platform has been operating in the same condition since it was installed, and no changes in the manning conditions have occurred that could be considered as important for performing an assessment.

3.1.2 Addition of facilities

Continuous changes in production strategies for the oil field in which the platform is located have led to a change in its original category. The platform changed from being just a "Drilling" platform to being a "Drilling and Production" platform, and as a result, a separation tank and pipe lines were added to the original facilities in order to achieve the production capacity for new requirements. This information is preliminary and was not verified in the field, but was considered convenient to be taken in account for assessment purposes. This leads to consideration of additional loads as discussed in Section 3.1.3 below.

3.1.3 Increased loading of the structure

The structure was originally installed in February 1979 and served as a "Drilling Platform" without any important structural or services modification until 1992. In this year an oil/gas separation tank was added to the platform services and therefore it was necessary to build a "cantilever" for supporting the tank. The cantilever and tank were located at row number one on the "Cellar Deck" level of the platform (Elev. + 52' - 0"). Figures 3.1.3.a and 3.1.3.b show a general view of the tank location and added area to original structure. Loadings were increased by 195 metric tons (6.3% in relation to the original design), mainly due to tank weight, and cantilever dead and live loadings.

The following table shows the changes to the original design in terms of increased loading:

	WEIGHT (TON)	
LOADINGS	ORIGINAL	UPDATED
1. Main structural members (including buoyancy)	1092	1092
2. Dead Load (Deck)	148	175
3. Live Load (Deck)	197	253
4. Equipment	2741	2853
Total 2 to 4	3086	3281

The increase in loading itself (at 6.3% of the total) would not be considered significant or sufficient to trigger an assessment in accordance with the definition of "significant" presented in Section 17.2.6 of the draft Section 17.0 API RP 2A Guideline.

Because of the tank addition a static "in-situ" structural analysis was initiated in order to perform a "working stress" level check of main structural members and foundation. From the static structural environmental/operational analysis including new loadings and structural modifications, it was found that several principal members showed working stresses to allowable stress ratios greater than 1.0, as well as some tubular joints with acting punching shears to allowable punching shear ratios greater than the maximum allowable values. Most of the critical components were associated with vertical braces and principal legs as well as foundation piles. Both structural members and joints were checked according to API RP 2A 19th Edition recommendations in has containment equipment available for collecting any oil spilling in Campeche Bay. The containment equipment is located in Ciudad del Carmen dock.

3.3 Condition Assessment

In accordance with Section C17.4 in the commentary to the Draft Section 17.0 guideline, the following information is provided for the condition assessment of the platform. A summary of the condition assessment, as required by the JIP, is listed in Table 3-3#.

3.4 Design Basis Check

This step is the last of the four steps required to complete the screening, after this step the platform has to be assessed by applying analysis checks such as:

- Design Level Analysis
- Ultimate Strength Analysis.

As stated in Section 17.0, the check is only valid for structures located in the US Gulf of Mexico and designed to the 9th Edition of API RP 2A or later. The Nohoch B platform was installed in 1979. Thus, it may be assumed that it was designed to the 9th edition of API RP2A, but with the site specific data supplied by A H Glenn (Ref.7). Although the Bay of Campeche is a part of the Gulf of Mexico, it does not have the hurricane survival history on which the Gulf of Mexico Design Basis criterion is based. For that reason the Design Basis check is not valid. Nevertheless, the process is considered here for Mexican waters to demonstrate the process.

Further requirements for the design basis check for metocean loading to apply are that:

1. No significant damage is present.
2. The platform has adequate deck height for its category.
3. There have been no significant changes from the design premise.

As explained in the condition assessment described in Section 3.2 of this report, Nohoch B has damage to primary members and additional facilities increasing loads by some 6.3%. The load increase is less than the 10% level of significance specified in Section 17.2.6 of the draft guideline. The level of damage together with the

The explanation for the significant change in the 100 year return period base shears is in the better representation of the mild conditions local to the Nohoch B site. The sheltered location of the platform is seen in Figure 2.6a. The passage of hurricanes across the surrounding landmass dissipates significant energies and furthermore the hurricane circulation opposes the hurricane forward speed for storm tracks north of the Nohoch B region thereby diminishing the incident loads further. The data supplied (Ref. 5) indicate that the conditions are local to the Nohoch B site and cannot be extrapolated to surrounding regions. Separate evaluation of the data (Ref. 5) is required to quantify the variations on a regional basis, nevertheless the potential benefits are indicated.

Insufficient data are available for the development of directional criteria and the winds for the dominant hurricane direction are applied omnidirectionally. The data in Tables 3.5.1a and b therefore relate solely to Direction 1.

3.5.1.4 Section 17.0 assessment analysis loads

For the assessment of existing Gulf of Mexico platforms, Section 17.0 recognizes the satisfactory performance of platforms designed to the 9th to 19th Editions of API RP 2A and deems this level to be acceptable for assessment as opposed to the more stringent requirements of new design to the 20th Edition. It was shown (Ref. 11) that loads generating a base shear of around 65% of the 20th Edition level was `equivalent' to a 9th to 19th Edition design requirement. The Section 17.0 Design Analysis check was therefore set at 0.65*BSC100 (see Figure 3.5.1a). Given a typical `ultimate to linear' ratio of 1.8 (ie. the ratio between ultimate capacity and the load level at which the first IR exceeds unity) this set the Ultimate Strength Level requirement at 1.8*0.65≃1.2 times the 100 year base shear (1.2*BSC100).

Given the situation of the Nohoch B platform in the wider Gulf of Mexico and the parallel relation between base shear and return period in both locations (Figures 3.5.1c and 3.5.1g), it was considered that factors of 0.65 and 1.2 on the 100 year base shear calculated to the 20th Edition could be justified. The analyses in Sections 3.5.3 and 3.5.4 are based on Oceanweather wave heights and associated parameters corresponding with these factored levels of base shear. The Columns 3 and 4 of Table 3.5.1a presents the input metocean parameters to the Design Level and Ultimate Strength Level analyses on the basis of this `Gulf of Mexico' analogy. Columns 5 and 6 of Table 3.5.1a contain the factored base shears and wave heights which can be seen to represent a very significant reduction below the original AH Glenn design loads.

However, the premise for adopting the 0.65 factor needs to be examined more closely. With reference to Table 3.5.1b it can be seen that moving from a 9th to 20th Edition methodology for evaluating the base shear using the original AH Glenn data (Refs. 4 and 7) gives only a 1.5% increase (4807 from 4562kips) in the design requirement (compared with the 50% (1/0.65) increase for US Gulf of Mexico structures (Ref. 11)). [Whereas the inclusion of currents has contributed to increases in US Gulf of Mexico design loads, currents were already accounted for in design requirements at Nohoch B. The net reduction in design loads at Nohoch B in moving from the 9th to the 20th Edition loading recipes, can be seen for the same wave height of 54.8ft in columns 1 and 2 of Table 3.5.1b. This is attributable to new blockage and shielding factors, doppler effects etc, offset in part by changes in recommended drag and inertia coefficients.] A further point in reviewing the applicability of the 0.65 Design Level Analysis factor as recommended for the US Gulf of Mexico is that in adopting the new Oceanweather data for the Bay of Campeche, the environmental loading for the 100 year event is further reduced by a significant amount (1843 kips compared with 4807 kips, see Table 3.5.1b). Such low levels have not previously been used in design or proven through survival statistics in practice. It must therefore be recognized that unlike the US Gulf of Mexico there is no experience base to justify a 65% reduction on the Oceanweather loads to be used as the acceptance criterion for a Design Level Analysis. On that basis it is recommended that the Design Level Assessment criterion be set at the full 100 year environmental return period load for the new data. The ultimate to linear ratio of 1.8 can be justifiably retained (Ref. 11) leading to an Ultimate Strength Level analysis criterion based on a base shear with a factor of 1.8 on the 100 year value.

These revised Bay of Campeche criteria are presented alongside the Gulf of Mexico approach in Table 3.5.1a and 3.5.1b. Although new analyses have not been undertaken for these revised criteria, it can be confirmed that the higher waves will not inundate the deck and therefore the different approaches can reasonably be compared on the basis of load factors (see Sections 3.5.3 and 3.5.4).

3.5.2 Screening

No screening level analysis was considered appropriate for this platform for the study. Extrapolation of the results obtained in this study for a very similar platform in a similar environment could be judged as an appropriate screening analysis in subsequent investigations.

3.5.3 Design Level Analysis

The design level analysis was performed using the SACS Program (Ref. 10) along with the modelling guidelines in Ref. 8. The metocean parameters in Table 3.5.1a for the `Gulf of Mexico Approach' Assessment, Column 3, Design Level was used. Three directions of wave approach shown in Figure 2.6b were considered. The dominant hurricane loads encompass a broad sector. The three analysis directions selected fall within this sector and relate to the three principal directions for the jacket, ie. longitudinal, transverse and diagonal.

The modelling used was identical to that used for new designs, except for the damaged members. The pile-soil behavior was modelled with the Pile/Soil capability within the SACS program. For the damaged elements shown in Figures 3.3a, b and c, the stiffness of the members was modelled with a reduced stiffness over the length of the dent or the crack. For the bent members, an extra node was placed at the location of the maximum lateral deflection of the member.

No unity checks exceeded 0.85 for the `Gulf of Mexico' Design Level Analysis Assessment. The pile axial loads and the base shears are shown in Table 3.5.4.a. Had the alternative approach using the full Oceanweather Bay of Campeche 100 year loads as the basis for the Design Level assessment been adopted, additional SACS analyses demonstrate that again no unity checks would exceed 0.85. This is to be expected given the significant reduction in loads moving from the AH Glenn parameters (Refs 4 and 7) to the new values from Oceanweather (Ref. 5) as shown in Table 3.5.1a.

The platform would therefore be deemed to pass at the Design Level Analysis stage of the Section 17.0 assessment process.

A summary of design level analysis results is given in the format for the JIP in Table 3-6#.

3.5.4 Ultimate Strength

Although the platform has been demonstrated to have passed the Section 17.0 assessment process at the Design Level Analysis stage, an Ultimate Strength Level Analysis was performed to satisfy the requirements of the JIP and to indicate the level of reserve strength. The results are summarized for the JIP in Table 3.7b#.

Three ultimate strength analyses were undertaken for the same incident wave directions as for the Design Level analyses. However, the metocean loads were

The table below indicates the load factors and base shears attained in each direction. It is clear from this table that, if the Gulf of Mexico approach were adopted in conjunction with the lesser Oceanweather data, the platform would easily pass the assessment.

	LOADCASE Direction 1	Direction 2	Direction 3
100 year Oceanweather loads: Base shear (kips)	1843	1785	1669
Ultimate strength resistance* Base shear (kips)	9237	9225	7007
λ_p	4.2	4.3	3.5
Ultimate strength resistance*/ 100 year Oceanweather loads	5.0	5.2	4.2
100 year AH Glenn loads (20thEdn) Base shear (kips)	4630	4484 #	4193 #
Ultimate strength resistance* Base shear (kips)	9237	9225	7007
λ_p	1.66	1.71	1.39
Ultimate strength resistance*/ 100 year A H Glenn loads	2.0	2.1	1.7

* Note - minimum specified yield adopted whereas some 20% additional capacity may be available given the likely higher yield of steel as-delivered.

\# Note - base shears for Direction 2 and 3 using A H Glenn metocean parameters calculated from Direction 1 assuming same base shear ratio as from Ocean Weather data,
eg. $BS_{(Direction\ 2\ AH\ Glenn)}$ = 4630 * 1785 / 1843 = 4484

Comparison with Oceanweather loads

As discussed in Section 3.5.1.4 the Oceanweather data (Ref. 5) are newly available and there is not the experience base to justify a lesser requirement in assessment than the 100 year parameters. It was demonstrated in Section 3.5.1.4 that an Ultimate Strength Level criterion at 1.8*BSC100 may be more appropriate. The condition of the structure at this load level can be examined as follows:

$\lambda_p = 1.0$ corresponds to 1.2*BSC100

therefore, $\lambda_p = 1.5 = (1.8/1.2)$ corresponds to 1.8*BSC100.

Again it can be seen that for all wave attack directions the structure more than satisfies this requirement.

in the piles, detailed examination of the SAFJAC output indicates that the maximum T-Z shear capacity is being mobilized along the length of the four corner piles indicating little additional system capacity. On that basis it is not justified to increase the resistance to account for higher steel properties and the ratio should remain at 1.7 as shown in the table. However, this comparison with the AH Glenn data is preliminary for a number of reasons:

- The 100 year base shear is presented on an approximate basis (see note #).
- For a full Section 17.0 assessment it would be required to back-calculate the metocean parameters associated with the target Ultimate Strength Analysis level base shear as shown in Figure 3.5.1a.
- The target base shear should be established on an assessment of the survival probabilities taking proper account of incident loads and resistances by direction.
- Alternatively, it may be considered that the 100 year parameters formed the basis of the original design, with metocean loads calculated using a slightly less onerous methodology than given by the API RP 2A 20th Edition loading recipe. The Structures have survived without collapse under extreme metocean loads. It may be deemed appropriate to adopt a lesser resistance for the Design Level Analysis (β), i.e. β.x*BSC100, leading to an Ultimate Strength Analysis criterion of β.x*1.8*BSC100 in line with the philosophy for the US Gulf of Mexico (Ref. 11). However the factor 0.x would have to be developed on the basis of specific extreme events and survival experience for Mexican waters.

On that basis it seems probable that the platform could be demonstrated to pass the assessment process from a metocean overload standpoint. A more rigorous evaluation beyond the scope of the current project would be required although this discussion serves to demonstrate the need and benefit of developing the philosophy fully for the region.

The individual responses determined by the SAFJAC Ultimate Strength Analyses in each of the wave attack directions can be described as follows:

Direction 1

PLATFORM 'U' - ORIGINAL DOCUMENT (REV. 0, Oct. 1994)

Table of Contents

1.0 SUMMARY

This report presents the results of the trial application the API RP 2A draft Section 17.0 for the assessment of existing platforms (Ref. 1) to the Nohoch B drilling platform in the Bay of Campeche in the Mexican Gulf. The platform is an 8 leg jacket in 125ft water depth. It was designed and installed in 1979, presumably to the 9th Edition of API RP 2A. The platform has significant damage (dents, bowed members and cracks) in the splash zone due to vessel impacts. Furthermore a cantilever deck, for storage and associated facilities, has been added during the service life. Damage was the principal assessment initiator. The platform was classified as manned-evacuated with significant environmental impact.

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Given the extent of damage the design basis check was not appropriate, however the platform passed the assessment at the Design Level Analysis stage. The analysis models included specific modelling of the damage. The subsequent ultimate strength analyses demonstrated the system reserves available. The new loading data contributed to the low probability of failure that could be demonstrated for this particular structure.

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A summary of the trial assessment is given in Table 3-8#*.

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* Tables noted with a # suffix have identical numbering and format as those in the DRAFT of "Trial Applications" prepared by PMB Engineering Inc., September 1994.

2.0 PLATFORM INFORMATION

2.1 Physical Features

Nohoch B is a drilling platform, located in a 38.100m (125 ft) water depth, in the Bay of Campeche offshore in the Gulf of Mexico (Figures 2.6a and b) and installed in 1979. It has four plans at elevations +3.658, -8.534, -22.250 and -38.100m; 2 longitudinal and 4 cross vertical frames. The jacket has eight legs of 52" diameter with eight foundation piles, each running the full length of each leg with the annulus ungrouted. The pile penetrations are 210m at the middle and 240m at the corners of the jacket. The bracing types are K, X and Y (diagonal). The deck has two plans, cellar deck at elevation +15.850 and the main deck at elevation +20.749 m. Overall, the platform appears similar to the conventional type of drilling, quarters and production structure designed for the US Gulf of Mexico.

2.2 Operational Information

Nohoch B has 12 wells (8 in service). At cellar deck there is a separator tank for 70,000 barrels per day. At some stage the platform may have living quarters.

2.3 Pertinent Inspection Information

The platform was periodically inspected and evaluated:

Year 1983	Inspection Report No. NHBF2 610-1-U
Year 1985	Inspection Report No. NHBF2 610-2-U
Year 1988	Inspection Report No. NHBF2 610-3-P/S
Year 1990	Inspection Report No. NHBF2 610-4-U
Year 1991	Inspection Report No. NHBF2 610-5-U
Year 1993	Inspection Report No. NHBF2 610-6-U

At the time of the assessment, the structure has 1 cracked joint (partially repaired) and 4 dented members in the splash zone of which 3 are bent, and there are 3 cracks around a barge bumper connection to the leg.

2.4 Structural Assessment Data

Due to the damage found in the inspection report (610-3-U), this platform was analyzed in October 1989 and the members 118-214, 105-203, 214-221, 212-314, 203-216, 216-302, 301-402, 304-403, 101-206, 201-307, 102-207, 202-308, 302-409, 313-416, 314-319, 101-202, 201-302 and the joints 221, 319, 206, 216, 317, 307, 410, 309 presented high stress ratios according to API RP 2A 18th Edition guidelines (see Figure 3.3.d for inspection node numbers).

In September 1992, Nohoch-B platform was analyzed due to the addition of a separator tank on the cellar deck; this is described in Section 3.1.3. In July 1993 the platform was analyzed again due to an impact which produced a 43cm long crack in an important joint at splash zone (report 610-5-U). The elements 115-120, 319-417 and the members and joints reported in the previous analysis, presented high stress ratios according to API RP 2A 18th Edition guidelines.

2.5 Soil Boring Logs and Shear Strength Profile

The boring logs revealed that the seafloor soil at the Nohoch B field consists of very soft to soft clay in a range of penetration to about 47 ft. Alternating layers of dense calcareous and carbonate clays were generally encountered below 47 ft penetration up to the terminal penetration of the boring.

2.6 Platform sketches

Figures showing the location, orientation, vertical and plan bracing, deck framing, appurtenances, piles and equipment layout are presented in the figures (Figures 2.6a - z). The as-is condition is summarised for the purposes of the JIP in the following two tables:

Table 1-1# : Summary of Basic Platform Information-Physical Characteristics
Table 1-2# : Summary of Basic Platform Information-Operational Characteristics

3.0 PLATFORM ASSESSMENT

3.1 Platform Selection

Nohoch B is an eight-leg jacket drilling platform located in Campeche Bay in the Gulf of Mexico, installed in February 1979 in 38.100m (125ft) water depth. It was designed to support 2,500 tons of drilling equipment and to withstand a 100-year return period storm, fixed on the seabed through an eight pile foundation.

The API RP 2A Section 17.0 Draft Guidelines provide the following six initiators for an assessment:

1. Addition of Personnel
2. Addition of Facilities
3. Increased Loading of the Structure
4. Inadequate Deck Height
5. Damage Found During Inspections
6. Regulatory Requirement

It will be demonstrated that according to Section 17.2, this platform could be considered to require assessment due to the addition of facilities and therefore increase of loading if these were shown to be significant. However, several dented members and cracked joints have been found during inspections and Item 5. in the above list was taken as the principal initiator. The summary of assessment initiators is presented in Table 3-1#.

In order to follow the methodology indicated in "Platform Selection" of the Draft Guideline, additional information is presented below.

3.1.1 Addition of personnel

The platform is classified as "Manned-Evacuated" for purposes of the current assessment work, as described in Section 3.2 of this report. The platform has been operating in the same condition since it was installed, and no changes in the manning conditions have occurred that could be considered as important for performing an assessment.

3.1.2 Addition of facilities

Continuous changes in production strategies for the oil field in which the platform is located have led to a change in its original category. The platform changed from being just a "Drilling" platform to being a "Drilling and Production" platform, and as a result, a separation tank and pipe lines were added to the original facilities in order to achieve the production capacity for new requirements. This leads to consideration of additional loads as discussed in Section 3.1.3 below.

3.1.3 Increased loading of the structure

The structure was originally installed in February 1979 and served as a "Drilling Platform" without any important structural or services modification until 1992. In this year an oil/gas separation tank was added to the platform services and therefore it was necessary to build a "cantilever" for supporting the tank. The cantilever and tank were located at row number one on the "Cellar Deck" level of the platform (Elev. + 52′ - 0"). Figures 3.1.3.a and 3.1.3.b show a general view of the tank location and added area to original structure. Loadings were increased by 195 metric tons (6.3% in relation to the original design), mainly due to tank weight, and cantilever dead and live loadings.

The following table shows the changes to the original design in terms of increased loading:

	WEIGHT (TON)	
LOADINGS	ORIGINAL	UPDATED
1. Main structural members (including buoyancy)	1092	1092
2. Dead Load (Deck)	148	175
3. Live Load (Deck)	197	253
4. Equipment	2741	2853
Total 2 to 4	3086	3281

The increase in loading itself (at 6.3% of the total) would not be considered significant or sufficient to trigger an assessment in accordance with the definition of "significant" presented in Section 17.2.6 of the draft Section 17.0 API RP 2A Guideline.

Because of the tank addition a static "in-situ" structural analysis was initiated in order to perform a "working stress" level check of main structural members and foundation. From the static structural environmental/operational analysis including new loadings and structural modifications, it was found that several principal members showed working stresses to allowable stress ratios greater than 1.0, as well as some tubular joints with acting punching shears to allowable punching shear ratios greater than the maximum allowable values. Most of the critical components were associated with vertical braces and principal legs as well as foundation piles. Both structural members and joints were checked according to API RP 2A 19th Edition recommendations in Sections 3 and 4 respectively. For piles the foundation check in Section 6.3.4 was used which showed that a corner pile had a penetration safety factor lower than the minimum value recommended.

3.1.4 Inadequate deck height

As a first pass a deck height check was performed according to Section 17.6.2a-2. from API RP 2A, with results obtained from Figure 17.6.2-2b. Since the platform was classified as "Manned, Evacuated" and "Significant Environmental Impact" according to Section 17.3 of the draft guideline (see Section 3.2 of this report), a minimum deck height of 14.265m (46.8') was determined. The "Minimum Deck Height" above MLLW was also calculated according to API RP 2A Section 2.3.4g (Ref. 8) giving a 12m (38.0') requirement, and no problem was found since the actual platform deck height of 14.497m (47.6') is greater than both values.

Deck height criteria for the Mexican coast of the Gulf of Mexico were under development at the time of this initial check but it was considered that use of the Northern Gulf of Mexico criterion was conservative for the sheltered Bay of Campeche case.

3.1.5 Damage found during inspections

, has been conducting a continuous inspection, repair and maintenance programme on Mexican Offshore Platforms since 1982. Six inspections of this structure were performed from 1983 to 1990, and several instances of damage to main members were reported from these inspections. Damage such as "dents" and "cracks" due to boat impacts and detached barge bumpers are common.

After two original general visual inspections, Level I and II according to API RP 2A Sections 14.3.1 & 14.3.2, decided to perform a global structural analysis of the platform in 1987 in order to obtain the actual stress state of the structure, the analysis included the damage found in previous inspections. A third general visual inspection of the platform was performed, divided into two phases from 1987 to 1988, and as a result a second structural analysis including accumulated damage was performed in 1989. Results from these first two analyses showed that the behavior was acceptable.

A detailed inspection programme to Level III according to API RP 2A Sections 14.3.3 and 14.3.4 was proposed for the platform in order to obtain an accurate record of the damage already found and any additional damage not identified during the first inspections. The programme was proposed to inspect those members and joints that showed a high working stress level in previous analyses. The inspection was conducted in 1990, and the result was a complete and accurate record of the whole platform. The extent of damage is shown in Figures 3.1.5a, b and c, and the following table summarizes the damage found during the inspections developed to date:

KIND OF DAMAGE AND LOCATION	DAMAGE SITES IN STRUCTURE
Dented members in splash zone	4 members
Bent members in splash zone	3 members
Cracks on legs around the barge bumper connection	3 cracks
Crack in vertical cross joint in splash zone	1 crack

As shown in the figures the damage relates to primary structural members and therefore would provide a sufficient trigger for an assessment in accordance with Section 17.0.

In fact, a structural analysis was performed in 1993 using the original design environmental data and considering all damaged members as well as loading changes in order to determine their combined effect on the global behavior of the structure. The platform was analyzed for storm conditions and environmental/operational loadings based on an average return period of Ts=100 years. Results from the analysis showed that working stresses to allowable stress ratios of several main members were greater than the maximum allowables given in Section 3.2 of API RP 2A 19th Edition. It was calculated that a corner pile has a safety factor (S.F.) for pile penetration of 1.261, smaller than the minimum value S.F.=1.5 recommended in Section 6.3.4 of API RP 2A.

This last analysis might therefore be considered to validate the Section 17.0 requirement to subject the platform to the assessment process because of the cumulative changes that have occurred.

3.1.6 Regulatory requirement

No regulatory requirement is in place in the Mexican sector to otherwise require an assessment of the platform.

3.2 Categorization

The draft guideline Section 17.0 requires that a platform be assessed in accordance with the applicable exposure category based on considerations of life safety and environmental impact. Sections 3.2.1 and 3.2.2 below demonstrate that Nohoch B is considered as a Significant Environmental Impact, Manned-Evacuated platform for the Section 17.0 assessment. A summary of the platform categorization conditions as required by the JIP is presented in Table 3-2#. The specific considerations for the subject platform are as follows.

3.2.1 Life safety

The Nohoch B drilling platform is located in Campeche Bay in the Gulf of Mexico, approximately 80 km to the north of the Ciudad del Carmen shore. Consideration of the exposure category with respect to human life safety leads to the classification as a ***Manned-Evacuated*** platform, because it is normally manned by ¯ personnel, except during an environmental event forecast as a design storm. The personnel are evacuated some hours before the storm arrives at Ciudad del Carmen.

The platform has a set of Emergency Procedures and a methodology of Alarm Transmittal, to alert personnel when fire or an abnormal situation occurs. In addition, there are automatic shutdown valves on all the wells in production stage, to close the wells when fire appears or a storm is coming. According to the last topside survey, the main findings in safety systems are: a fire wall in good condition in the area of Xmas trees, the automatic start system of the fire pumps and the handle extinguishers are in good condition.

3.2.2 Environmental impact

At Nohoch B platform there are eight wells in the production stage, which are monitored from Nohoch A Complex, located 2,600 m away to the north. Also, oil and gas production is sent to Nohoch A through three risers of 16", 24" and 8".

The platform is classified in the ***Significant Environmental Impact*** category, because the collapse of the structure would cause the spilling of oil over the sea and the release of sour gas to the atmosphere which may cause a fire, therefore it could cause considerable damage to the environment, killing a wide variety of marine wildlife. Furthermore, two of the main economical activities of the people of Campeche and Tabasco shores are fishing and tourism and they could be affected as a result.

has containment equipment available for collecting any oil spilling in Campeche Bay. The containment equipment is located in Ciudad del Carmen dock.

3.3 Condition Assessment

In accordance with Section C17.4 in the commentary to the Draft Section 17.0 guideline, the following information is provided for the condition assessment of the platform. A summary of the condition assessment, as required by the JIP, is listed in Table 3-3#.

3.4 Design Basis Check

This step is the last of the four steps required to complete the screening, after this step the platform has to be assessed by applying analysis checks such as:
- Design Level Analysis
- Ultimate Strength Analysis.

As stated in Section 17.0, the check is only valid for structures located in the US Gulf of Mexico and designed to the 9th Edition of API RP 2A or later. The Nohoch B platform was installed in 1979. Thus, it may be assumed that it was designed to the 9th edition of API RP2A, but with the site specific metocean criteria supplied by A H Glenn (Ref.7). Although the Bay of Campeche is a part of the Gulf of Mexico, it does not have the hurricane survival history on which the Gulf of Mexico Design Basis criterion is based. For that reason the Design Basis check is not valid. Nevertheless, the process is considered here for Mexican waters to demonstrate the process.

Further requirements for the design basis check for metocean loading to apply are that:

1. No significant damage is present.
2. The platform has adequate deck height for its category.
3. There have been no significant changes from the design premise.

As explained in the condition assessment described in Section 3.2 of this report, Nohoch B has damage to primary members and additional facilities increasing loads by some 6.3%. The load increase is less than the 10% level of significance specified in Section 17.2.6 of the draft guideline. The level of damage together with the increased load indicates that the structure would fail this Design Basis Check requiring the execution of the Design Level Analysis.

Following the Design Basis check outlined in the draft RP 2A Section 17.0 guideline, the load on a single caisson with the characteristics of a leg member of Nohoch B is 152 kips for the original design metocean criteria (Ref. 7). The reference level load is about 125 kips for a 52 inch member (Figure 17.6.2-1 of Ref. 1). Thus, the Nohoch B platform would pass the assessment if damage had not occurred and if it were considered to be within the population of US Gulf of Mexico platforms.

3.5 Analysis Checks

3.5.1 Metocean and Seismic

The potential for seismic activity exists at the Nohoch B site, but it appears to be less significant than the metocean environment. Thus, only the metocean criteria are addressed in this study. A summary of the metocean criteria adopted in the analyses for the JIP is given in Table 3-5#.

An important consideration in this assessment is the availability of new metocean data from Oceanweather (Ref. 5). The original design and all subsequent code check reanalyses to date have been performed using metocean criteria recommended by A H Glenn (Ref. 7). It is important to note that the new data and metocean criteria developed in this report pertain to the specific location of Nohoch B. Furthermore, the deduction of environmental criteria has been based on limited data provided for the local grid point (Ref. 5) and was undertaken without full sight of the report or any caveats it may contain. It will be demonstrated below that the new data show Nohoch B to be in a protected location enabling less stringent environmental loads to be adopted than given in the A H Glenn recommendations. However, it could be seen from the data sheet containing the meteorological and oceanographic information for gridpoint 420 (Ref. 5) that wave heights, for example, were significantly larger at adjacent grid points further offshore. For that reason the level of load reduction must be viewed as being very site specific and broader conclusions should not be drawn for the fleet of platforms in the region. Nevertheless, the evaluations demonstrate the potential benefit of interpretation of the Oceanweather dataset (Ref. 5) along the lines presented in this report.

The original design criteria (Ref. 7), the reduction of the new Oceanweather data (Ref. 5) and the basis of load levels for assessment in line with the RP 2A Section 17.0 philosophy are discussed in this section. As described by Krieger et al (Ref. 11), Section 17.0 requires the determination of loads which the platform must sustain without overstress in a Design Level Analysis or without collapse in an Ultimate Strength Analysis.

3.5.1.1 Section 17.0 metocean loads philosophy

The philosophy for deriving metocean loads for assessment to Section 17.0 is described in OTC papers (Refs 6 and 11). Figure 3.5.1a demonstrates schematically the underlying relationship between wave heights and return period and between base shears and return period that need to be established.

The curves are developed on the basis of metocean data and associated structural loading analyses. The 100 year return period base shear for the hurricane/winter storm combination (BSC100), is equivalent to the base shear for a longer term hurricane and the corresponding hurricane wave height (HC100) can be read from such a figure. Similarly, if the Design Level Analysis base shear is set at 0.65*BSC100 (as for structures in the manned-evacuated, significant environmental impact category in the US Gulf of Mexico), the corresponding hurricane wave height HASL can be deduced. For an Ultimate Strength level Analysis in the same category the hurricane wave height HAUS is determined from 1.2*BSC100. Further discussion of the appropriate factors for the asssessment at Nohoch B is presented in Section 3.5.1.4.

Figure 3.5.1b reproduces the hurricane and winter storm wave data for 400ft water depth in the US Gulf of Mexico underlying the Section 17.0 guidelines for that region (Ref. 6). Figure 3.5.1c presents the corresponding base shear data (Ref. 6) in which the hurricane and combine hurricane/winter storm responses converge demonstrating the overriding dominance of hurricanes. It will be shown that at Nohoch B, winter storms have a stronger influence on the 100 year base shear.

3.5.1.2 Original design criteria

The original design criteria for the Nohoch B platform (Ref. 7) are given in the first column of Table 3.5.1a. The criteria appear to be based on the 100 year return of the hurricane wave and other associated metocean parameters. Two documents were available relating to the original design data. The first (Ref. 7) had insufficient wave height distributions to develop any parameters other than those shown in Table 3.5.1a. The second (Ref. 4) provided more detailed hurricane and winter storm data from an adjacent site and these distributions were factored by a small amount to correlate with key 100 year hurricane wave height parameters at the Nohoch B site. For example the wave height data at the adjacent gridpoint was factored to give 54.8ft as determined for the specific Nohoch B site (Ref. 7). The wave height distributions are shown in Figure 3.5.1d with corresponding base shear return periods in Figure 3.5.1e. The remaining influence of winter storm near the 100 year return period can be seen for the Nohoch B location in Figure 3.5.1e in comparison with the US Gulf of Mexico (Figure 3.5.1c). The base shears for the structure are based on the original A H Glenn parameters (Refs 4 and 7) but the wave load model adopted the API RP 2A 20th Edition loading recipe in terms of drag and inertia coefficients, blockage factors etc (Ref. 8).

The first columns in Table 3.5.1b use the original AH Glenn data (Ref. 7). In the first column the base shear is based on the API RP 2A 9th Edition loading recipe as in the original design, whereas the 20th Edition is used in the second column. For that reason the base shears for the 54.8 ft wave height differ. Furthermore the account of hurricane and winter storms to the 20th Edition gives an equivalent 100 year hurricane wave height of 55.8ft compared with 54.8ft and a correspondingly higher base shear.

3.5.1.3 Interpretation of Oceanweather data (Ref. 5)

New data have recently been supplied by Oceanweather for the Nohoch B site. Figure 3.5.1f shows the relation between wave height and return period and Figure 3.5.1g relates base shear to return period. The second column in Table 3.5.1a presents the 100 year metocean parameters indicated by these new Oceanweather data (Ref. 5) in comparison with the original AH Glenn parameters (Ref. 7) in the first column. The table below indicates the step change in base shear for each parameter change in turn calculated using a higher order stream function for the Nohoch B platform. It can be seen that changing from the AH Glenn 100 year parameters to the Oceanweather 100 year parameters, presents a significant reduction in loads. These can be compared in the second and third columns of Table 3.5.1b.

Parameter change	Wind speed (knots)	Surge (ft)	Current spd (knots)	Wave ht (ft)	Wave per'd (sec)	Marine gwth (in)	Base shear (kips)	BS ratio
AH Glenn (Refs 4 & 7)	118.0	5.90	1.83	54.8	16.0	2.4	4447	1.00
Wind speed	71.7	5.9	1.83	54.8	16.0	2.4	3998	0.90
Surge	71.7	1.12	1.83	54.8	16.0	2.4	4126	0.93
Current speed	71.7	1.12	0.78	54.8	16.0	2.4	3802	0.85
Wave height	71.7	1.12	0.78	42.1	16.0	2.4	2366	0.51
Wave period	71.7	1.12	0.78	42.1	10.1	2.4	1844	0.41
Marine growth	71.7	1.12	0.78	42.1	10.1	1.5	1729	0.39
Oceanweather (Ref. 5)	71.7	1.12	0.78	42.1	10.1	1.5	1729	0.39

The explanation for the significant change in the 100 year return period base shears is in the better representation of the mild conditions local to the Nohoch B site. The sheltered location of the platform is seen in Figure 2.6a. The passage of hurricanes across the surrounding landmass dissipates significant energies and furthermore the hurricane circulation opposes the hurricane forward speed for storm tracks north of the Nohoch B region thereby diminishing the incident loads further. The data supplied (Ref. 5) indicate that the conditions are local to the Nohoch B site and cannot be extrapolated to surrounding regions. Separate evaluation of the data (Ref. 5) is required to quantify the variations on a regional basis, nevertheless the potential benefits are indicated.

Insufficient data are available for the development of directional criteria are available and the winds for the dominant hurricane direction are applied omnidirectionally. The data in Tables 3.5.1a and b therefore relate solely to Direction 1.

3.5.1.4 Section 17.0 assessment analysis loads

For the assessment of existing Gulf of Mexico platforms, Section 17.0 recognises the satisfactory performance of platforms designed to the 9th to 19th Editions of API RP 2A and deems this level to be acceptable for assessment as opposed to the more stringent requirements of new design to the 20th Edition. It was shown (Ref. 11) that loads generating a base shear of around 65% of the 20th Edition level was 'equivalent' to a 9th to 19th Edition design requirement. The Section 17.0 Design Analysis check was therefore set at 0.65*BSC100 (see Figure 3.3.1a). Given a typical 'ultimate to linear' ratio of 1.8 (ie. the ratio between ultimate capacity and the load level at which the first IR exceeds unity) this set the Ultimate Strength Level requirement at $1.8*0.65 \simeq 1.2$ times the 100 year base shear (1.2*BSC100).

Given the situation of the Nohoch B platform in the wider Gulf of Mexico and the parallel relation between base shear and return period in both locations (Figures 3.5.1c and 3.5.1g), it was considered that factors of 0.65 and 1.2 on the 100 year base shear calculated to the 20th Edition could be justified. The analyses in Sections 3.5.3 and 3.5.4 are based on Oceanweather wave heights and associated parameters corresponding with these factored levels of base shear. The Column 3 of Table 3.5.1a presents the input metocean parameters to the Design Level and Ultimate Strength Level analyses on the basis of this 'Gulf of Mexico' analogy. Columns 5 and 6 of Table 3.5.1b contain the factored base shears and wave heights which can be seen to represent a very significant reduction below the original AH Glenn design loads.

However, the premise for adopting the 0.65 factor needs to be examined more closely. With reference to Table 3.5.1b it can be seen that moving from a 9th to 20th Edition methodology for evaluating the base shear using the original AH Glenn data (Refs. 4 and 7) gives only a 1.5% increase (4630 from 4562kips) in the design requirement (compared with the 50% (1/0.65) increase for US Gulf of Mexico structures (Ref. 11)). [Whereas the inclusion of currents has contributed to increases in US Gulf of Mexico design loads, currents were already accounted for in design requirements at Nohoch B. The net reduction in design loads at Nohoch B in moving from the 9th to the 20th Edition loading recipes, can be seen for the same wave height of 54.8ft in columns 1 and 2 of Table 3.5.1b. This is attributable to new blockage and shielding factors, doppler effects etc, offset in part by changes in recommended drag and inertia coefficients.] A further point in reviewing the applicability of the 0.65 Design Level Analysis factor as recommended for the US Gulf of Mexico is that in adopting the new Oceanweather data for the Bay of Campeche, the environmental loading for the 100 year event is further reduced by a significant amount (1843 kips compared with 4630 kips, see Table 3.5.1b). Such low levels have not previously been used in design or proven through survival statistics in practice. It must therefore be recognised that unlike the US Gulf of Mexico there is no experience base to justify a 65% reduction on the Oceanweather loads to be used as the acceptance criterion for a Design Level Analysis. On that basis it is recommended that the Design Level Assessment criterion be set at the full 100 year environmental return period load for the new data. The ultimate to linear ratio of 1.8 can be justifiably retained (Ref. 11) leading to an Ultimate Strength Level analysis criterion based on a base shear with a factor of 1.8 on the 100 year value.

These revised Bay of Campeche criteria are presented alongside the Gulf of Mexico approach in Table 3.5.1a and 3.5.1b. Although new analyses have not been undertaken for these revised criteria, it can be confirmed that the higher waves will not inundate the deck and therefore the different approaches can reasonably be compared on the basis of load factors (see Sections 3.5.3 and 3.5.4).

3.5.2 Screening

No screening level analysis was considered appropriate for this platform for the study. Extrapolation of the results obtained in this study for a very similar platform in a similar environment could be judged as an appropriate screening analysis in subsequent investigations.

3.5.3 Design Level Analysis

The design level analysis was performed using the SACS Program (Ref. 10) along with the modelling guidelines in Ref. 8. The metocean parameters in Table 3.5.1a for the 'Gulf of Mexico Approach' Assessment, Columns 3 and 4, Design Level were used. Three directions of wave approach shown in Figure 2.6b were considered. The dominant hurricane loads encompass a broad sector. The three analysis directions selected fall within this sector and relate to the three principal directions for the jacket, ie. longitudinal, transverse and diagonal.

The modelling used was identical to that used for new designs, except for the damaged members. The pile-soil behavior was modelled with the Pile/Soil capability within the SACS program. For the damaged elements shown in Figures 3.3a, b and c, the stiffness of the members was modelled with a reduced stiffness over the length of the dent or the crack. For the bent members, an extra node was placed at the location of the maximum lateral deflection of the member.

No unity checks exceeded 0.85 for the'Gulf of Mexico' Design Level Analysis Assessment. The pile axial loads and the base shears are shown in Table 3.5.4.a. Had the alternative approach using the full Oceanweather Bay of Campeche 100 year loads as the basis for the Design Level assessment been adopted, additional SACS analyses demonstrate that again no unity checks would exceed 0.85. This is to be expected given the significant reduction in loads moving from the AH Glenn parameters (Refs 4 and 7) to the new values from Oceanweather (Ref. 5) as shown in Table 3.5.1a.

The platform would therefore be deemed to pass at the Design Level Analysis stage of the Section 17.0 assessment process.

A summary of design level analysis results is given in the format for the JIP in Table 3-6#.

3.5.4 Ultimate Strength

Although the platform has been demonstrated to have passed the Section 17.0 assessment process at the Design Level Analysis stage, an Ultimate Strength Level Analysis was performed to satisfy the requirements of the JIP and to indicate the level of reserve strength. The results are summarised for the JIP in Table 3.7b#.

Three ultimate strength analyses were undertaken for the same incident wave directions as for the Design Level analyses. However, the metocean loads were higher by a factor of 1.8, corresponding to the 'Gulf of Mexico approach' Ultimate Strength check presented in Tables 3.5.1a and b and discussed in Section 3.5.1.4.

The analyses were performed using SAFJAC (Ref. 12). Each jacket member and the piles were modelled with quartic beam-column elements which have the facility to model geometric nonlinearities including P-Δ effects. On detecting yield conditions at any location, the quartic element automatically subdivides introducing a plastic hinge to model the material nonlinearity. The soil-structure interaction was modelled with three-part linear springs. The resistance of the soil to the conductors was modelled similarly. The out-of-straightness of the bent members was incorporated in the model. The cracks associated with the barge bumper were not considered significant in relation to the platform ultimate strength. The wet weld repair to the cracked X joint was reportedly poor so the potential reduced stiffness was accounted for. Initial analyses were run in each direction in conjunction with tubular joint code checks to API RP 2A with the safety factors removed. Where these indicated that tubular joint behavior could influence the ultimate strength, explicit tubular joint elements modelling P-δ, M-ϕ relationships for the specific joint geometries were introduced and the analyses rerun. The joint capacities and flexibilities were obtained from the SAFJAC library derived from mean value test results for the database underlying the UK Health and Safety Executive Guidance Notes (Ref. 13). The joint modelling accounts not only for the capacity limit but also for the elastic flexibility and reduced stiffness as the ultimate joint capacity is reached.

For each incident direction the following information is provided in figures and tables and discussed below:

- Global load-deflection curve
- Tabulated data for load-deflection response
- Deflected shape at $\lambda_{p\,max}$
- Hinge locations at $\lambda_{p\,max}$

The factor λ_p is the incremental factor applied to the 'Gulf of Mexico approach' Ultimate Strength Analysis loads for the platform derived from the Oceanweather data (Ref. 5). Therefore $\lambda_p = 1.0$ corresponds to the Section 17.0 criterion giving a wave height of 46.6 feet and associated parameters given in Table 3.5.1a and b. As discussed in Section 3.5.1.4, this wave height derivation accepts the Gulf of Mexico experience that 0.65 and 1.2 are appropriate factors to apply to 100 year loads for Design Level and Ultimate Strength Level analysis assessments, respectively. For the Campeche Bay, factors of 1.0 and 1.8 have been considered more appropriate and discussion of the derivation of the corresponding λ_p values follows in this section.

The table below indicates the load factors and base shears attained in each direction. It is clear from this table that, if the Gulf of Mexico approach were adopted in conjunction with the lesser Oceanweather data, the platform would easily pass the assessment.

	LOADCASE		
	Direction 1	Direction 2	Direction 3
100 year Oceanweather loads: Base shear (kips)	1843	1785	1669
Ultimate strength resistance* Base shear (kips) λ_p	9237 4.2	9225 4.3	7007 3.5
Ultimate strength resistance*/ 100 year Oceanweather loads	5.0	5.2	4.2
100 year AH Glenn loads (20thEdn) Base shear (kips)	4630	4484 #	4193 #
Ultimate strength resistance* Base shear (kips) λ_p	9237 4.2	9225 4.3	7007 3.5
Ultimate strength resistance*/ 100 year A H Glenn loads	2.0	2.1	1.7

* Note - minimum specified yield adopted whereas some 20% additional capacity may be available given the likely higher yield of steel as-delivered.

\# Note - base shears for Direction 2 and 3 using A H Glenn metocean parameters calculated from Direction 1 assuming same base shear ratio as from Ocean Weather data,
eg. $BS_{(Direction\ 2\ AH\ Glenn)}$ = 4630 * 1785 / 1843 = 4484

Comparison with Oceanweather loads

As discussed in Section 3.5.1.4 the Oceanweather data (Ref. 5) are newly available and there is not the experience base to justify a lesser requirement in assessment than the 100 year parameters. It was demonstrated in Section 3.5.1.4 that an Ultimate Strength Level criterion at 1.8*BSC100 may be more appropriate. The condition of the structure at this load level can be examined as follows:

$\lambda_p = 1.0$ corresponds to 1.2*BSC100

therefore, $\lambda_p = 1.5 = (1.8/1.2)$ corresponds to 1.8*BSC100.

Again it can be seen that for all wave attack directions the structure more than satisfies this requirement.

In fact, in performing Ultimate Strength Analysis, Section 17.0 permits the removal of all bias or conservatism to ensure that a realistic capacity prediction is obtained. The structural modelling of members, and joints in SAFJAC ensures that a 'mean' response is predicted. However, an additional factor is the conservatism in material properties. Design analysis conservatively adopts the minimum specified yield whereas the delivered steel may be some 20% stronger on average (Ref. 11). Had this been allowed for in the ultimate strength analyses here, the critical load factor in terms of structural response would have been $\lambda_p = 1.25$ (ie. 1.5/1.2). This simplified interpretation is valid here because foundation failure only limits the capacity at much higher loads. Hence the structural response and extent of plasticity at $\lambda_p = 1.25$ is considered to be an appropriate pass/fail Ultimate Strength analysis assessment level for this platform in the Bay of Campeche. It should be emphasised that this is a specific interpretation incorporating resistance elements in the criterion and furthermore is only valid where the foundation failure does not play a part as the 20% additional capacity applies only to the steel components.

For future practice it would be recommended that Oceanweather metocean parameters corresponding to 1.8 times the 100 year base shear be derived and applied to a structure in which a 20% increase in steel yield above the minimum specification is accounted for.

In reality what is required from the assessment process is a target survival rate (or conversely probability of failure). With reference to Figure 3.5.1g for the Oceanweather data, it can be seen that an Ultimate Strength level base shear criterion at 1.8*BSC100 corresponds to a return period of 400-500 years. This is commensurate with assessment criteria in other regions (Ref. 11). However, a more rigorous evaluation depends on some directional load data to be assessed in conjunction with the full ultimate strength rosette. Such an approach would be valuable in setting criteria for a family of platforms.

Comparison with A H Glenn loads

As the Oceanweather data have not yet been used in design, the table also provides comparison with the 100 year base shears used in the original design, based on the AH Glenn parameters. Clearly the ratios between the ultimate resistance and 100 year loads are less than for the Oceanweather comparison although for Directions 1 and 2 the values both exceed 1.8. For the end-on longitudinal wave attack, Direction 3, the ratio shown in the lower row, final column is 1.7. Although, as discussed below, the peak capacity is associated with yielding in the bracing and some plasticity in the piles, detailed examination of the SAFJAC output indicates that the maximum T-Z shear capacity is being mobilised along the length of the four corner piles indicating little additional system capacity. On that basis it is not justified to increase the resistance to account for higher steel properties and the ratio should remain at 1.7 as shown in the table. However, this comparison with the AH Glenn data is preliminary for a number of reasons:

- The 100 year base shear is presented on an approximate basis (see note #).

- For a full Section 17.0 assessment it would be required to back-calculate the metocean parameters associated with the target Ultimate Strength Analysis level base shear as shown in Figure 3.5.1a.

- The target base shear should be established on an assessment of the survival probabilities taking proper account of incident loads and resistances by direction.

- Alternatively, it may be considered that the 100 year parameters formed the basis of the original design, with metocean loads calculated using a slightly less onerous methodology than given by the API RP 2A 20th Edition loading recipe. As the structures have survived without collapse under extreme metocean loads, it may therefore be deemed appropriate to adopt a lesser resistance for the Design Level Analysis, ie. 0.x*BSC100, leading to an Ultimate Strength Analysis criterion of 0.x*1.8*BSC100 in line with the philosophy for the US Gulf of Mexico (Ref. 11). However the factor 0.x would have to be developed on the basis of specific extreme events and survival experience for Mexican waters.

On that basis it seems probable that the platform could be demonstrated to pass the assessment process from a metocean overload standpoint. A more rigorous evaluation beyond the scope of the current project would be required although this discussion serves to demonstrate the need and benefit of developing the philosophy fully for the region.

The individual responses determined by the SAFJAC Ultimate Strength Analyses in each of the wave attack directions can be described as follows:

Direction 1

Figure 3.5.5a presents the overall load-deflection response of the platform for Direction 1. This is backed by the data in Table 3.5.5a. Figure 3.5.5b gives the deflected shape for the plane of the structure at the end of the analysis and Figure 3.5.5c indicates the extent of nonlinearity. The loads at which each hinge occurs is indicated in Table 3.5.5a together with information on the load at which sufficient hinges have formed to consitute member failure.

In Table 3.7b# the loads at first member with IR=1.0 is taken from an elastic code check to API RP 2A for the model with the nominal material yield stress and with a 1/3 overstress allowance included (ie. the IR=1.0 corresponds to code non-compliance with a storm factor of safety ~1.25-1.3). The first nonlinear event is not the load at which the first hinge occurs but corresponds to the first member failure, ie. third hinge for a slender compression member or attainment of the mean capacity in a tubular joint component. This approach is adopted to give better correlation with other analysis methods.

For all cases the code check analyses were run and no joints were found to fail or even exceed the API RP 2A lower bound before the target load factor $\lambda_p = 1.25$ is reached.

For the broadside wave attack Direction 1, the ultimate response was governed by yielding in the piles and local plasticity in the K-bracing of the transverse Rows 1 and 2. The K joints have an overlap configuration and an additional analysis indicated that some additional flexibility would be associated with the high load of the joints but the system capacity is little affected.

Direction 2

Figure 3.5.5d presents the global load deflection response for the diagonal wave attack, Direction 2, with underlying data provided in Table 3.5.5b. Figures 3.5.5e and f present the deflected shape and hinge locations for the various stages of the analyses.

Although some local hinges occur in the plan bracing around the conductor area (Figure 3.3.5f3), the system response is limited by the double hinging in the piles and extensive plasticity in the X bracing on Row B. The inflection in the piles can be seen clearly in the deflected shape in Figure 3.3.5e corresponding to the soft upper clay layers of the foundation strata. Nevertheless, as indicated above, significant capacity reserves have been demonstrated beyond the target Ultimate Strength Analysis values.

Direction 3

As for the previous loadcase presentations, Figure 3.5.5g presents the global load deflection response with underlying data provided in Table 3.5.5c. Figures 3.5.5h and i compare the deflected shapes and hinge location for the various stages of the analyses.

For this end-on longitudinal wave attack direction, Figure 3.5.5i reveals the extensive plasticity with tensile yielding and compression buckling in the lower X-braced bay of Rows A and B. In addition the detailed SAFJAC output demonstrates that the maximum T-Z shear resistances has been generated along most of the length of the four corner piles thereby contributing to the limiting system response.

Summary

In all cases it can be seen that the minimum required load factor ($\lambda_p = 1.25$) based on the new Ocean Weather data has been achieved with a significant margin. Furthermore, the damage to bracing due to collisions in the splash zone does not compromise the resistance of the structure to extreme environmental loads.

3.5.5 Fatigue

The performance of a fatigue analysis of the platform is beyond the scope of the JIP.

3.6 Mitigation Alternatives

Given that the platform has been demonstrated to satisfy the assessment procedure, no specific mitigation measures need be considered. However, some discussion of possible mitigation options is given in Appendix B.

3.7 Summary Note

The Nohoch B platform passes the assessment requirement of the draft API RP 2A Section 17.0 (Ref. 1) because of the lower wave heights now expected at the specific site compared with the original design criteria. These conclusions cannot be drawn for other locations and a similar analysis should be repeated using the methods used in this report along with the recent metocean data (Ref.5 or equivalent).

4.0 REVIEW AND FEEDBACK

The development of the metocean criteria for the manned-evacuated/significant environmental impact need to be addressed for areas outside the US Gulf of Mexico

With the exception of comments related to the metocean criteria, all comments have been transmitted in previous studies related to the JIP. These comments are reproduced below:

In this section comments are made on the use of the draft Section 17.0 (April 29 version) as applied to the Gulf of Mexico benchmark structure.

<u>Software</u>

In 17.7.2b and 17.7.3b it is recommended that the clauses read "software developed *and validated* for that purpose".

<u>Conductors</u>

In the final clause of the Commentary, C17.7.3c (g), it is required that the gap between jacket and conductor be modelled. Clearly this is aimed at realism, however there is uncertainty in the initial position of the conductor in the slot. For this reason the added complexity may not necessarily lead to an improved representation of the system behaviour. Perhaps it need not routinely be modelled but if the criteria are only just met this and other factors such as initial member out-of straightness etc should be recommended for inclusion in a sensitivity study.

<u>Ultimate strength modelling</u>

Section 17.7.3c provides instructions on element grouping and this is expanded significantly in the commentary. It is questioned whether the level of guidance in the guideline itself is helpful. It is suggested that the clause should reiterate the intention to use best estimate properties to model components (as stated explicitly for foundations) and indicate that, if required, further guidance on the grouping of similar elements for modelling purposes is contained in the commentary.

The discussion regarding the modelling of structural members in the commentary appears to be written with the concepts of an "INTRA" type analysis in view. Other programs which have been developed and validated for ultimate strength analysis have automatic facilities to accommodate large deflection beam column action including the effects of end fixity without requiring the user to select specific K factors or element types before performing an analysis. It is also unnecessary to scrutinise working stress analysis results to establish which element types should be selected for each location "based on the dominant stresses". These software packages make the single step to ultimate strength check increasingly viable from economic and time standpoints.

Perhaps a more general approach would be to state that the modelling should properly account for beam column effects, the potential onset of plasticity, and the effect of frame restraints on buckling capacity etc. This generality leaves the analyst better able to interpret the guideline and less likely to give inadequate consideration to factors which may cursorily be disregard as irrelevant.

Deck loading

The presentation of deck loading in the commentary C17.6.2 could be open to different interpretation. For example wave loads on the net silhouette area are readily distributed equally to decks above and below. In reality structural members might share the load top to bottom whereas loads incident on equipment/structure standing on the deck will pass loads to the lower level almost exclusively. Should the net area modelling be associated with the net deck area for attracting loads rather than the between deck silhouette. Alternatively the proposed procedure may be adequate but should perhaps be flagged for further investigation in a sensitivity study should the margin beyond the required ultimate strength be small.

Typographical

17.3.1c "platform *is* not"

17.5.2 "environmental" - remove space and hyphen

17.6.2b "Section 17.6.2a.2" ? There is no 17.6.2a.5

17.7.3 " to ensure adequacy"

17.7.3c "deformation"

For clarity

In 17.2.6 suggest changing word "total" to "combined effect".

In 17.5.4 "API RP 2N First Edition" is referenced several times, but never typed the same way.

References

1. "API RP 2A-WSD 20th Edition, Draft Section 17.0, Assessment of Existing Platforms", June 28, 1994.

2. "Analisis de Danos, Plataforma de Perforacion Nohoch-B", Reporte Tecnico, ɔ, Julio de I993.

3. "Geotechnical Investigation, , Boring Nohoch-B, Bay of Campeche", Report No. 0178-090-2, McClelland Engineers, Inc., October 1978.

4. "Oceanographic Study of Cayos Arcas Area", A.H. Glenn and Associates, August 5, 1980.

5. Letter from (.), Transmittal of metocean data developed by Oceanweather Inc. (1994) for Gridpoint 420.

6. "Metocean Criteria/Loads for Use in Assessment of Existing Offshore Platforms", Petrauskas, C., et. al., OTC Paper No. 7484, 1994.

7. "100 Year Storm Wind, Tide, Wave, and Currents Characteristics: 210,131.2, and 42.7 Foot Mean Water Depth Locations, Vicinity 19 Deg. N., 92 Deg W., Gulf of Campeche, Offshore Mexico, A.H. Glenn and Associates, June 14, 1977.

8. API RP2A-WSD, 20th Edition, July, 1993.

9. API RP2A-WSD, 9th Edition, November, 1977.

10. SACS - Structural Analysis Computer System, Engineering Dynamics, Inc.

11. "Process for Assessment of Existing Platforms to determine their Fitness for Purpose", Krieger, W F et al, OTC Paper No. 7482, 1994

12. SAFJAC - Strength Analysis of Frames and JACkets, Billington Osborne-Moss Engineering Limited.

13. HSE 4th Edition Guidance Notes, 1990.

14. Private communication with

Table 1-1#: Summary of Basic Platform Information - Physical Characteristics

Platform	Location	Water Depth (ft)	Year of Installation	Year Designed or API Edition	Physical Characteristics: Number of Legs	Number of Piles	Pile Diameter (inch)	Leg/Pile Annulus	Pile Penetration (ft)	Brace Type in Vertical Frames	Lowest Deck B.O.S. Elev. (ft)	Damages to Jacket Primary Members	Platform Modifications Since Original Design
Bay of Campeche													
U	Nohoch B	125	1979	1977	8	8	48	Ungrouted	700	K & X & Diag	47.5	Minor Damage	Addition of facilities

Table 1-2#: Summary of Basic Platform Information - Operational Characteristics

Platform	Location	Water Depth (ft)	Year of Installation	Operational Characteristics: Type of Facility	Manned / Unmanned Operation & Evacuation During Storm	Number of Wells / Conductors	Future Modifications Planned or Proposed
Bay of Campeche							
U	Nohoch B	125	1979	PDQ	Manned / Evacuated	12	-

Table 3-1#: Summary of Assessment Initiator Triggers

Platform	Addition of Personnel	Addition of Facilities	Increased Loading on Structures > 10%	Inadequate Deck Height	Damage Found During Inspections	Assessment Intiator Triggers
Bay of Campeche						
U	No	Yes	No	No	Yes	Damage/Addition of facilities

Table 3-2#: Summary of Platform Categorization

Platform	Type of Facility	Manning Evaluation: Number of Men	Manned / Unmanned Operation & Evacuation During Storm	Environmental Impact Evaluation: Number of Wells/ Conductors	Oil Storage on Deck (bbl)	Proximity to Shore (miles)	Environmental Impact Category	Metocean Criteria Category
Bay of Campeche								
U	PDQ	15+	Manned / Evacuated	12	Not Significant	50	Significant	Full Population Hurricanes + Winter Storm

Table 3-3#: Summary of Condition Assessment

Platform	Survey Level	Last Survey (year)	Is Platform Damaged ?	Is Deck Height Inadequate ?	Has the Loading Inceased ?	Is Platform Unmanned ?	Does It Have Insignificant Environmental Impact ?	Does Platform Passes Assessment at This Stage ?
Bay of Campeche								
U	III	1993	Yes	No	Yes	No	No	No

Table 3-4#: Summary of Design Basis Checks - Gulf of Mexico Platforms

Platform	Water Depth (ft)	Year Designed (Original)	API RP 2A Edition Used	Is Platform Designed to 9th Edition or Later ?	Original Design Details Available	Does It Pass At This Stage
Bay of Campeche						
U	125	1977	9th	Yes	No	No

Table 3-5#: Summary of Metocean Criteria

Metocean Criteria Category	Platform	Water Depth (ft)	Platform Orientation w.r.t. True North	Section 17 Design Level: Wave Height (ft)	Section 17 Design Level: Current Speed (knots)	Section 17 - Ultimate Strength: Wave Height (ft)	Section 17 - Ultimate Strength: Current Speed (knots)	Section 17 - Ultimate Strength: Required Deck Height (ft)	API RP 2A, 20th Edition *: Wave Height (ft)	API RP 2A, 20th Edition *: Current Speed (knots)
Bay of Campeche										
Combined Hurricane and Winter Storm	U	125	N45W	35.6	0.62	46.6	0.86	29.9	43.3	0.79

* API RP 2A (20th Edition) applied to Bay of Campeche

Table 3-6#: Summary of Design Level Analysis Results

Number of Legs	Platform	Water Depth (ft)	Number of Conductors / J-Tubes	Wave Height (ft)	Maximum Base Shear (kips)	Maximum I.R. (Primary Members)	Member Types with I. R. > 1.0	Pile Axial Capacity (F.O.S.)	Assessment Pass / Fail at Design Level Analysis
Bay of Campeche									
8	U	125	12	35.6	1198	< 1 *	None *	2.6	Pass

* Using Oceanweather data

Table 3-7b#: Summary of Ultimate Strength Analysis Results

Platform	Water Depth (ft)	Base Shear: Section 17 Ult. Load S-17 (kips)	Base Shear: 20th Ed. * Ref. Level S-20 (kips)	Analysis Results: Load at 1st Member with Lin. IR.=1.0 S-1 (kips)	Analysis Results: Load at 1st Member with NLin Event R-1 (kips)	Analysis Results: Ultimate Capacity Ru (kips)	Analysis Results: Platform Failure Mode	Ultimate Capacity / Ultimate Load = Ru/S-17	Assessment Pass / Fail at Ultimate Strength Analsysis	Information to the API TG: Reserve Strength Ratio RSR = Ru/S-20	Information to the API TG: Ultimate Linear Ratio ULR = Ru/S-1	Information to the API TG: Load Reduction Factor LRF = S-1/S-20 #	Information to the API TG: Redundancy Factor RF = Ru/R-1
Bay of Campeche													
U*1	125	2212	1843	4366	8490	9237	Piles/Bracing	4.18	Pass	5.01	2.12	2.37	1.09
U*2	125	2169	1785	4157	8058	9225	Piles/Bracing	4.25	Pass	5.17	2.22	2.33	1.14
U*3	125	2037	1669	3985	6911	7007	Bracing	3.44	Pass	4.20	1.76	2.39	1.01

*1: Boadside wave attack direction *2: Diagonal Wave attack direction *3: Longitudinal wave attack direction # NB New (reduced) metocean data since design

Table 3-8#: Summary of Trial Assessments

Platform	Assessment Intiator Triggers	Metocean Criteria Category	Does Platform Pass at Condition Assessment Stage ?	Does Platform Pass at Design Basis Check Stage ?	Does It Pass at Design Level Analysis Stage ?	Does It Pass at Ultimate Strength Analysis Stage ?
Bay of Campeche						
U	Damage/ Addition of facilities	Full Population Hurricane + Winter Storm	No	No	Yes	Yes

Table 3.5.1a

Metocean Criteria
NOHOCH B Platform

	Reported Data		Assessment criteria US Gulf of Mexico Approach		Assessment criteria Bay of Campeche Approach		Design criteria US GoM
	100 Year Return	100 Year Return	Assessment Design Level	Assessment Ultimate Analysis	Assessment Design Level	Assessment Ultimate Analysis	100 Year Return
		BSC100	0.65*BSC100	1.8*0.65*BSC100	1.0*BSC100	1.8*BSC100	
Metocean Reference	A H Glenn Ref. 7	Oceanweather Ref. 5	Oceanweather Ref. 5	Oceanweather Ref. 5	Oceanweather Ref. 5	Oceanweather Ref. 5	API RP2A,20thEd Ref. 8
Location	Nohoch B	Nohoch B	Nohoch B	Nohoch B	Nohoch B	Nohoch B	U.S. Gulf of Mex.
Item							
Height of Max. Wave, ft	54.80	43.30	35.60	46.60	43.30	54.20	60.00
Water Depth, ft	125.00	125.00	125.00	125.00	125.00	125.00	125.00
Tide, ft	5.90	1.15	0.92	1.26	1.15	1.51	4.00
Period of Max. Wave, sec	16.00	10.30	9.34	10.88	10.30	12.00	12.40
Crest Elevation , ft. (above still water)	35.00	26.10	20.80	28.50	26.10	34.64	39.69
1 Hour Wind, mph	118.00	84.90	69.80	91.20	84.90	107.00	98.40
Current at Surface, knots	1.83	0.79	0.62	0.86	0.79	1.06	1.71
Current at Mudline, knots	0.24	0.79	0.62	0.86	0.79	1.06	1.71
Scale Factor on 100 Year Return Base Shear, Oceanweather, 5	na	1.00	0.65	1.20	1.00	1.80	na

Table 3.5.1b
Metocean Conditions and Base Shear for Various Criteria
for Various Methodology and Metocean Conditions
Nohoch B Platform

Water Depth = 125 ft
Omnidirectional Wave
Wave Direction 1

	Original Design							
Methodology Reference	API RP2A 9th Ed.		API RP2A 20th Ed.		API RP2A 20th Ed.		API RP2A 20th Ed.	
Metocean Reference	A.H.Glenn, Ref 7		A.H.Glenn, Refs 4&7		Oceanweather, Ref 5		API, Ref. 1 & 8	
Location	Nohoch B		Nohoch B		Nohoch B		US Gulf of Mexico	
CRITERIA	Base Shear kip	Wave Height ft	Base Shear kips	Wave Height ft	Base Shear kip	Wave Height ft	Base Shear kips	Wave Height ft
100 Year Return								
Specified Metocean	4562	54.8	na	na	na	na	4436	60
100 Yr Hurricane	na	na	4447	54.8 *	1729	42.1	4436	60
100 Yr Winter Storm	na	na	3515	48.2 *	1208	31.8	du	du
100 Yr Base Shear (Equivalent Hurricane Wave Height)	na	na	4630	55.8 *	1843	43.3	4436	60
Assessment, Gulf of Mexico Approach								
Design Level Analysis								
Specified Metocean	na	na	na	na	na	na	3303	52.5
0.65*100 Yr Base Shear (Equivalent Hurricane Wave Height)	na	na	3125	46.9	1198	35.6	2883	du
Assessment, Gulf of Mexico Approach								
Ultimate Strength Analysis								
Specified Metocean	na	na	na	na	na	na	5868	64.5
1.20*100 Yr Base Shear (Equivalent Hurricane Wave Height)	na	na	5768	61	2212	46.6	5323	du
Assessment, Bay of Campeche Approach								
Design Level Analysis								
Specified Metocean	na	na	na	na	na	na	na	na
1.0*100 Yr Base Shear (Equivalent Hurricane Wave Height)	na	na	4807	55.8	1843	43.3	na	na
Assessment, Bay of Campeche Approach								
Ultimate Strength Analysis								
Specified Metocean	na	na	na	na	na	na	na	na
1.80*100 Yr Base Shear (Equivalent Hurricane Wave Height)	na	na	8653	72.5	3317	54.2	na	na

* Factored values from adjacent site to correlate with Specified Metocean conditions at Nohoch B given in Ref. 7 (see Column 1)

na - Not Applicable du - Data Unavailable

Table 3.5.4a

Nohoch B
Base Shear, BS and Pile Axial Load
for
Assessment, Strength Level, US Gulf of Mexico Approach

	Wave Direction 1		Wave Direction 2		Wave Direction 3	
	BS = 1198 kips		BS = 1102 kips		BS= 1001 kips	
Pile	Pile Load kips	Pile FOS*	Pile Load kips	Pile FOS*	Pile Load kips	Pile FOS*
A1	1686	2.85	1849	2.60	1650	2.91
A2	1544	3.11	1424	3.37	1143	4.20
A3	1339	3.58	1162	4.13	881	5.45
A4	1280	3.75	784	6.12	1280	3.75
B1	609	7.88	1139	4.21	1583	3.03
B2	598	8.03	788	6.09	1114	4.31
B3	546	8.79	644	7.45	910	5.27
B4	422	11.37	213	22.54	401	11.97

* FOS - Factor of Safety = Ultimate Capacity/ Pile Load   Ultimate Capacity = 4800 kips

Table 3.5.5a
Ultimate Strength Analysis Results for Direction 1

Load Step	Displacement at Top of Leg A1 (ft.)	Lateral Load (kips)	Plastic Hinge	Location	Component Failure
1	0.303	2264			
2	0.387	2830			
3	0.476	3396			
4	1.049	6633	1	Elev. -125'	
5	1.183	7302	2	Rows A & 1	
6	1.316	7900	3	Elev. -125'	
			4	Row 2	
			5	Rows A & 2	
7	1.360	8083	6	Elev. -125'	
			7	Row 2	
			8	Row 2	
8	1.405	8258	9	Rows A & 3	
9	1.468	8500	10	Row 2	Yield in Tension (Hinges 4 & 10)
10	1.481	8552	11	Rows B & 1	
11	1.490	8583	12	Rows A & 1	
12	1.494	8597	13	Row 2	Buckle (Hinges 7, 8 & 13)
13	1.586	8839	14	Row 1	
14	1.595	8862	15	Rows A & 4	
15	1.632	8956	16	Row 1	
16	1.660	9023	17	Rows A & 2	
17	1.725	9167	18	Row 1	
			19	Rows B & 3	
18	1.752	9227	20	Rows B & 2	
19	1.758	9237	21	Row 1	Buckle (Hinges 16, 18 & 21)

Table 3.5.5b1
Ultimate Strength Analysis Results for Direction 2

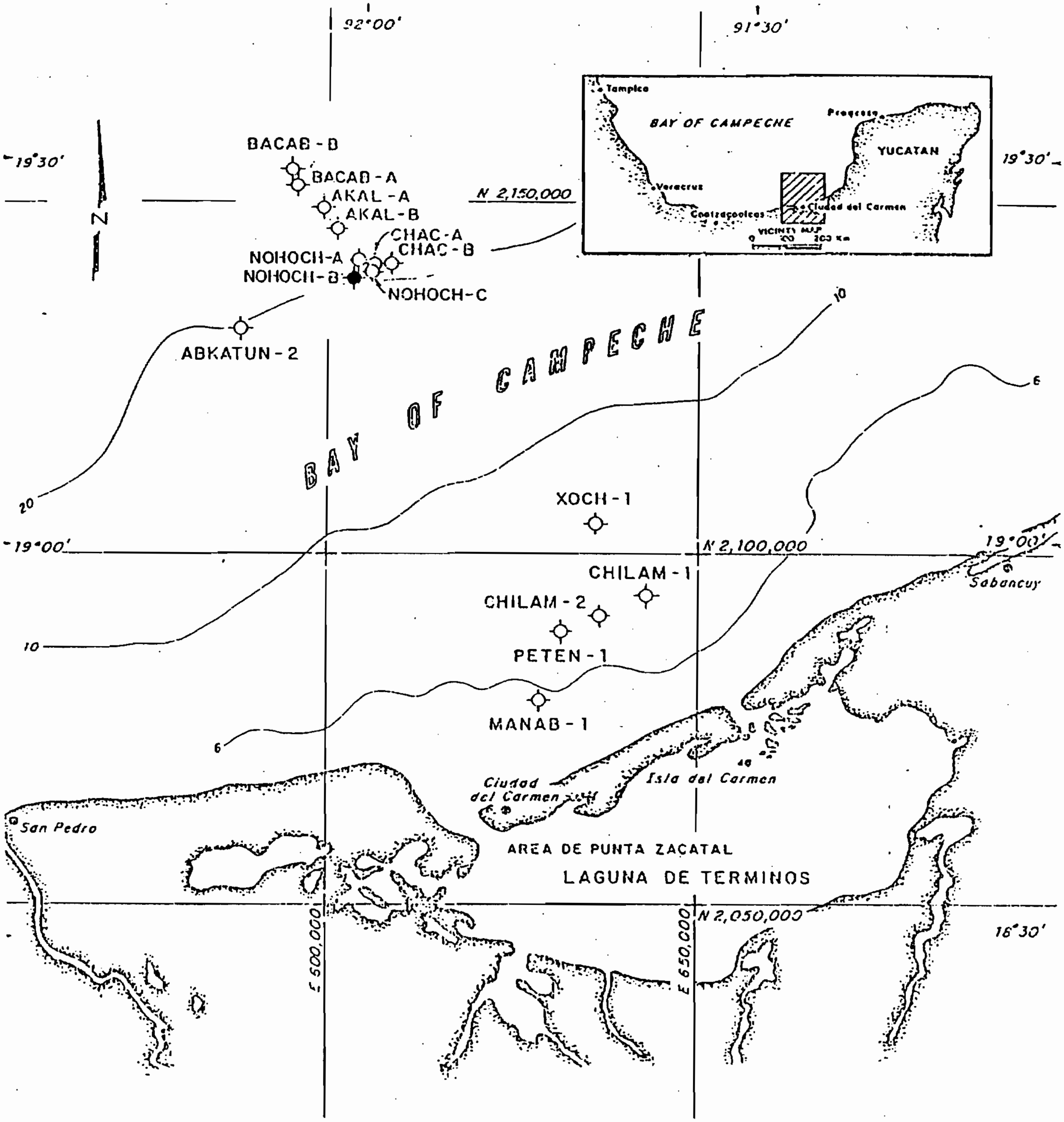
Load Step	Displacement at Top of Leg A1 (ft.)	Lateral Load (kips)	Plastic Hinge	Location	Component Failure
1	0.292	2143			
2	0.465	3215			
3	1.035	6251			
4	1.165	6854	1	Row A	
5	1.298	7406	2	Elev. -125'	
			3	Row B	
6	1.425	7810	4	Row B	
			5	Row A	
			6	Row B	
7	1.502	8032	7	Elev. -125'	
8	1.515	8067	8	Row B	
			9	Row B	Yield in Tension (Hinges 8 & 9)
			10	Row B	
9	1.528	8101	11	Row B	
10	1.553	8168	12	Row B	
11	1.565	8201	13	Row B	
12	1.577	8233	14	Row B	Buckle (Hinges 12, 13 & 14)
			15	Row A	
13	1.579	8236	16	Row B	Yield in Tension (Hinges 3 & 16)
14	1.616	8324	17	Row A	
15	1.655	8398	18	Row B	
16	1.662	8411	19	Row B	
17	1.678	8440	20	Row B	
18	1.795	8637	21	Row A	
19	1.851	8724	22	Row B	
20	1.939	8849	23	(Topside)	
21	1.952	8865	24	Row B	
22	1.996	8908	25	Elev. -73'	
23	2.019	8931	26	Row B	
24	2.057	8965	27	Row B	

Table 3.5.5b2
Ultimate Strength Analysis Results for Direction 2

Load Step	Displacement at Top of Leg A1 (ft.)	Lateral Load (kips)	Plastic Hinge	Location	Component Failure
25	2.090	8991	28	Elev. -125'	Yield in Tension (Hinges 7 & 28)
			29	Row B	
			30	Row A	
26	2.121	9014	31	Row B	
			32	Elev. -125'	
27	2.167	9047	33	Row B	
			34	Row B	
28	2.182	9057	35	Row B	
			36	Elev. -125'	
29	2.244	9099	37	Elev. -73'	
			38	Elev. +12'	
30	2.246	9100	39	Row B	Buckle (Hinges 31, 35 & 39)
31	2.247	9101	40	Elev. -73'	
32	2.252	9104	41	Elev. -125'	
33	2.330	9151	42	(Conductor)	
34	2.340	9157	43	Row B	Yield in Tension (Hinges 18 & 43)
35	2.351	9163	44	Row B	
36	2.362	9169	45	Elev. -73'	
37	2.395	9189	46	Row B	Yield in Tension (Hinges 44 & 46)
			47	(Conductor)	
38	2.419	9203	48	(Conductor)	
39	2.420	9204	49	(Conductor)	
			50	(Conductor)	
40	2.420	9204	51	(Conductor)	
41	2.433	9210	52	(Conductor)	
42	2.451	9219	53	Row B	
43	2.456	9222	54	Row B	
			55	(Conductor)	
44	2.457	9222	56	(Conductor)	
45	2.461	9224	57	Elev. -73'	Buckle (Hinges 37, 45 & 57)
46	2.463	9225			

Table 3.5.5c
Ultimate Strength Analysis Results for Direction 3

Load Step	Displacement at Top of Leg A1 (ft.)	Lateral Load (kips)	Plastic Hinge	Location	Component Failure
1	0.283	2015			
2	0.637	4030			
3	0.957	5642			
4	1.042	6045	1	Row A	
			2	Row B	
5	1.064	6146	3	Row A	
			4	Row B	
6	1.202	6750	5	Row A	
			6	Row B	
7	1.228	6851	7	Row A	
			8	Row B	
			9	Row A	
			10	Row B	
8	1.243	6911	11	Row B	Yield in Tension (Hinges 8 & 11)
9	1.246	6922	12	Row A	Yield in Tension (Hinges 7 & 12)
10	1.248	6932	13	Row B	Yield in Tension (Hinges 2 & 13)
11	1.254	6952	14	Row A	Yield in Tension (Hinges 1 & 14)
12	1.262	6982	15	Row B	
13	1.263	6987	16	Row B	
14	1.264	6990	17	Row B	Yield in Compression (Hinges 4, 16 & 17)
15	1.266	6996	18	Row A	
16	1.266	6996	19	Row A	
17	1.268	7005	20	Row A	Yield in Compression (Hinges 3, 19 & 20)
18	1.269	7007			



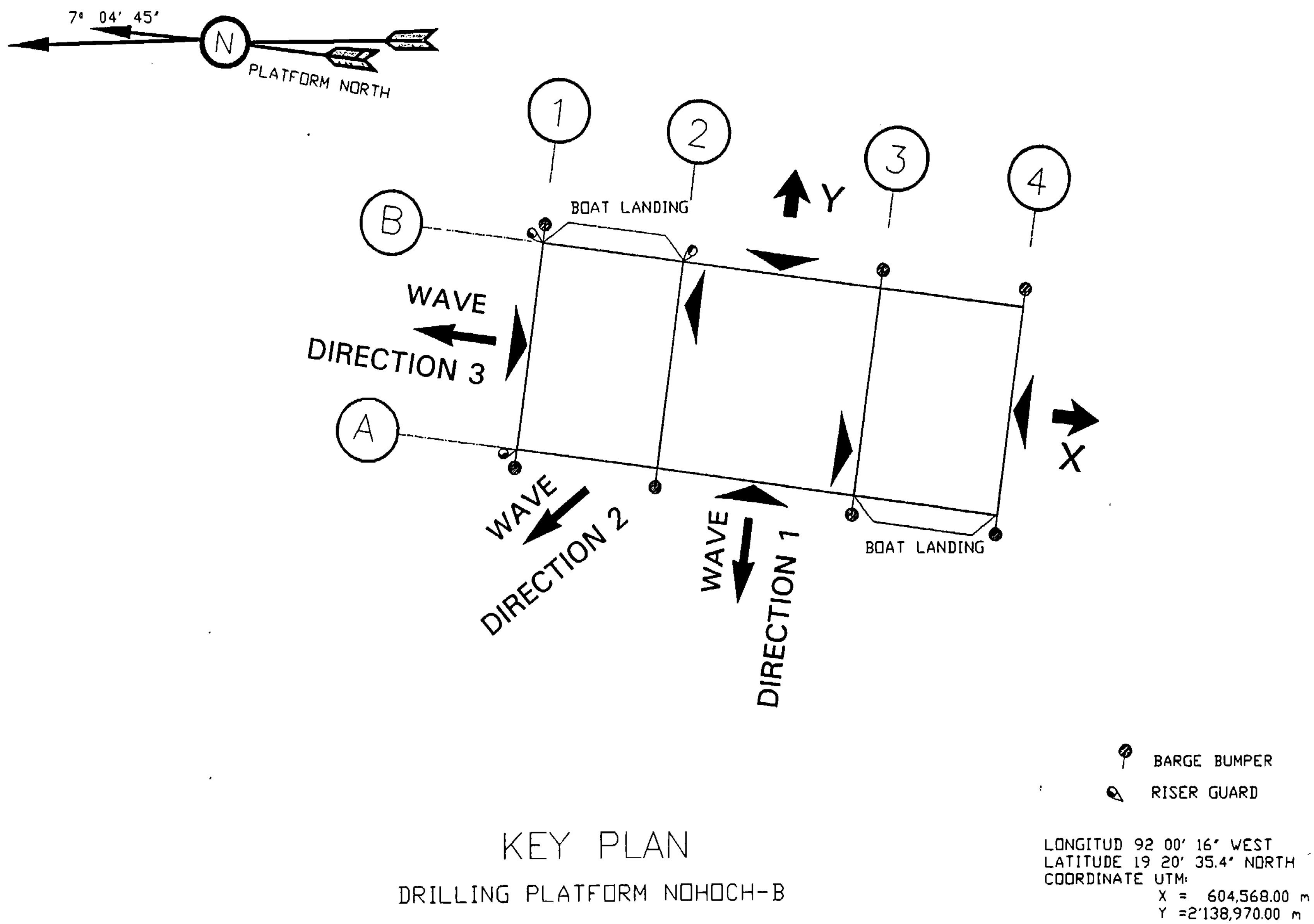
LOCATION MAP OF "NOHOCH B"

PLAN OF BORINGS
Bay of Campeche

0 50 Kilometers

Depth Contours in Fathoms

Figure 2.6a



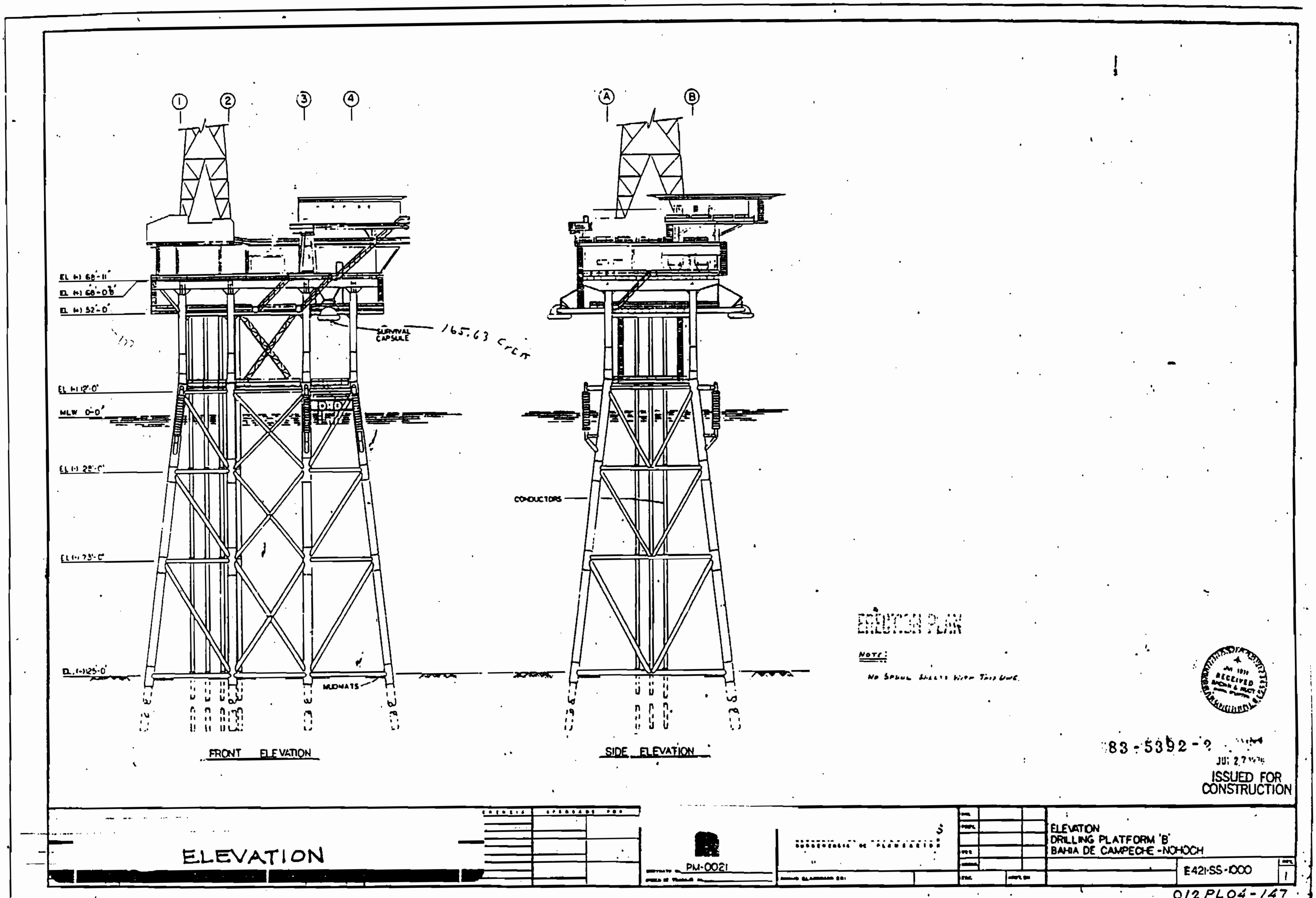
KEY PLAN

DRILLING PLATFORM NOHOCH-B

POINT OF VIEW FROM VERTICAL FRAME

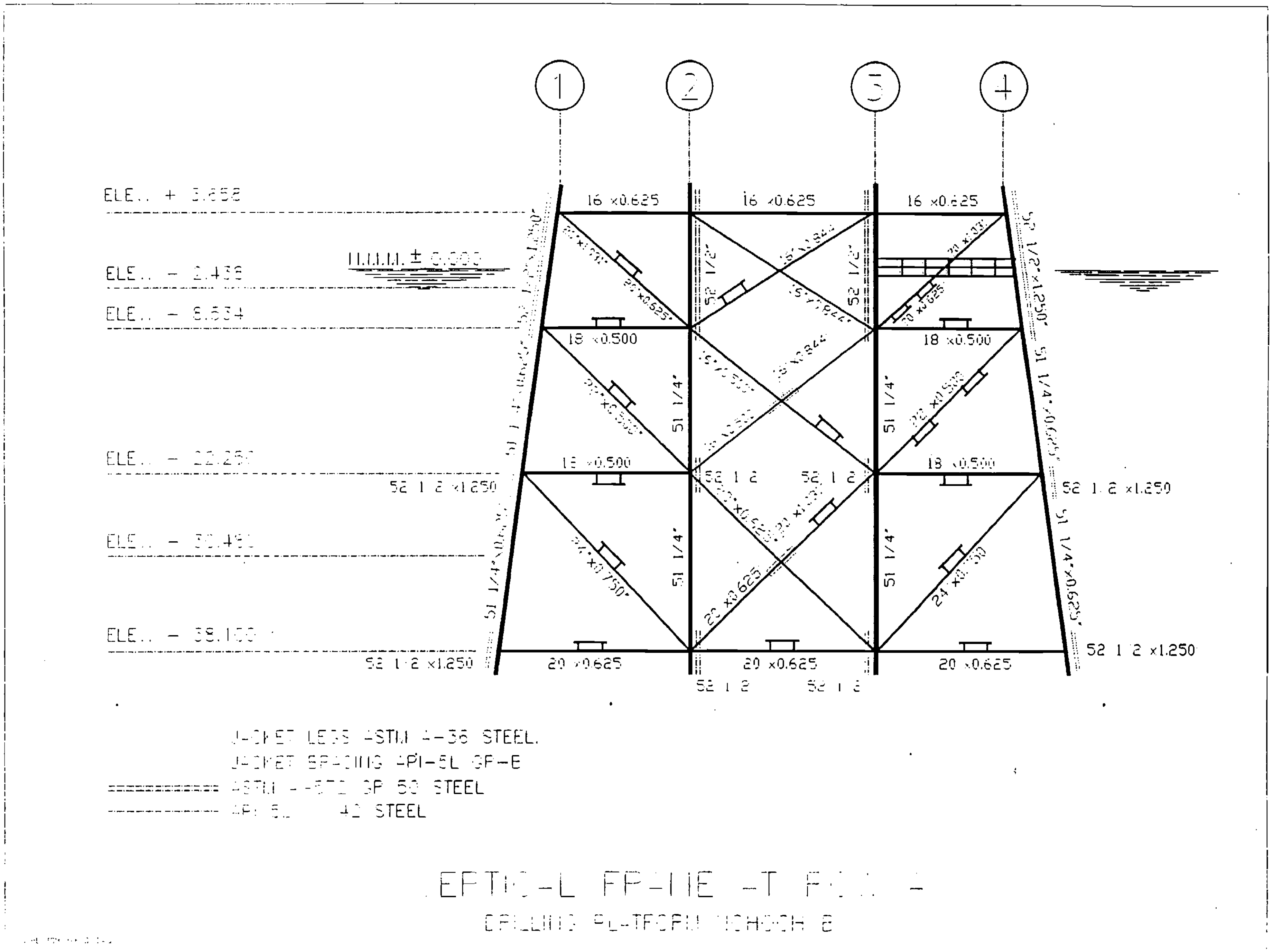
Figu ?.6b

INSPECTOM



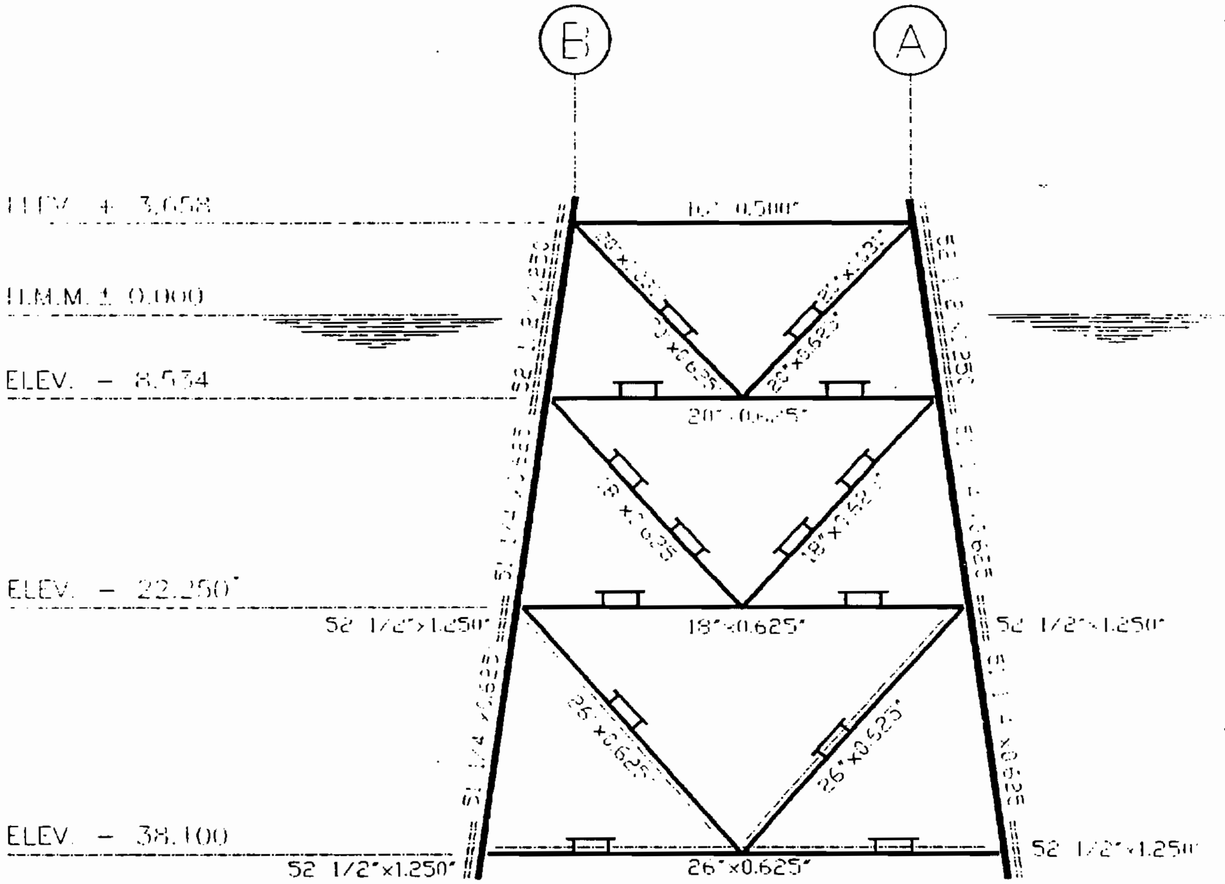
1
2
3
4
A
B
EL (+) 68'-11"
EL (+) 68'-0"
EL (+) 52'-0"
EL (+) 12'-0"
MLW 0'-0"
EL (-) 25'-0"
EL (-) 75'-0"
EL (-) 125'-0"
SURVIVAL CAPSULE
165.63 Crcm
CONDUCTORS
MUDMATS
FRONT ELEVATION
SIDE ELEVATION
NOTE:
83-5392-2
JUL 27
ISSUED FOR CONSTRUCTION
ELEVATION
PM-0021
ELEVATION
DRILLING PLATFORM 'B'
BAHIA DE CAMPECHE -NOHOCH
E421-SS-1000
1
012 PL04-147

Figure 2.6c



1
2
3
4
16 x0.625
18 x0.500
20 x0.625
51 1/4"
52 1/2"
52 1/2"x1.250"
51 1/4"x0.625"
52 1 2 x1.250
ELE.. + 3.658
ELE.. − 2.438
ELE.. − 8.534
ELE.. − 22.250
ELE.. − 38.100
JACKET LEGS ASTM A-36 STEEL.
JACKET BRACING API-5L GR-B
STEEL
42 STEEL

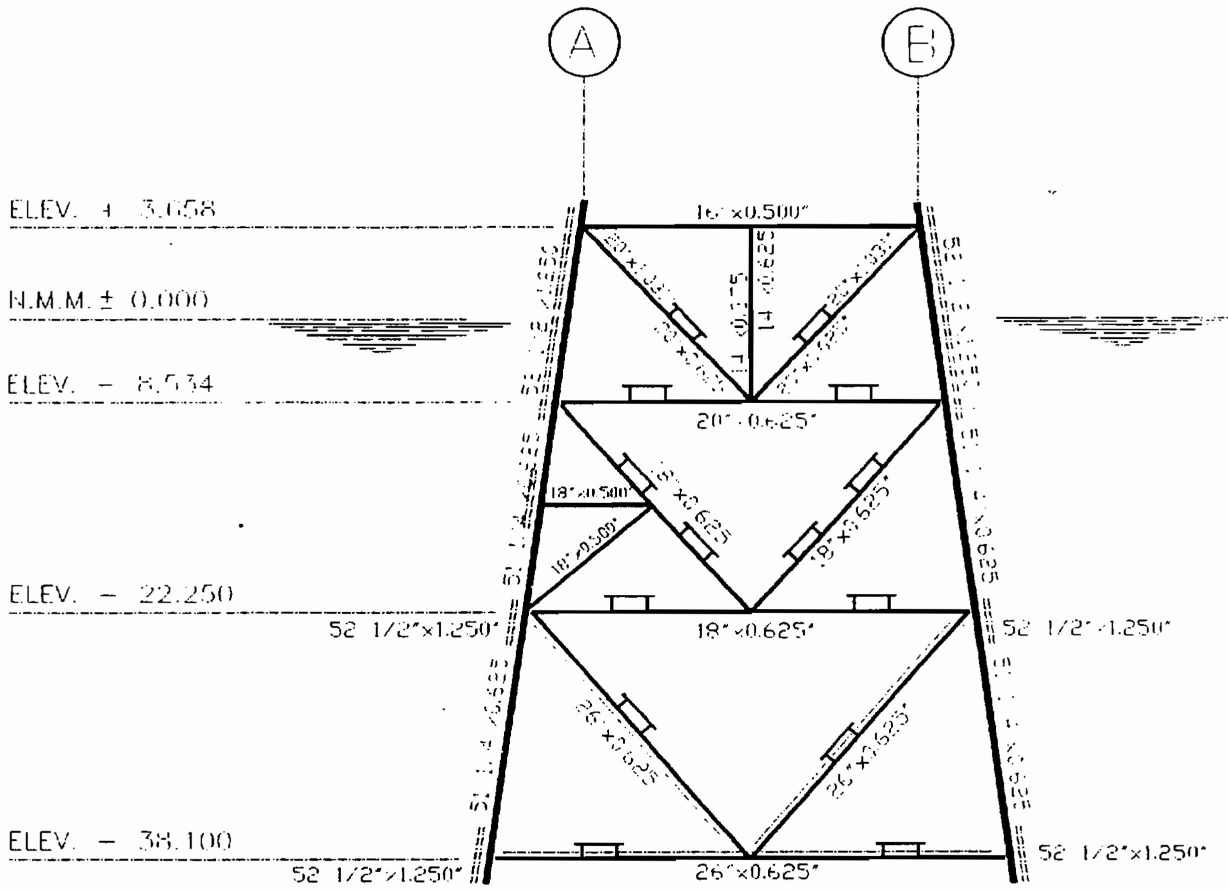
ıre 2.6d



JACKET LEGS ASTM A-36 STEEL
JACKET BRACING API-5L GR-B
ASTM A-572 GR 50 STEEL
API 5LX X42 STEEL

VERTICAL FRAME AT ROW 1
DRILLING PLATFORM NOHOCH-B

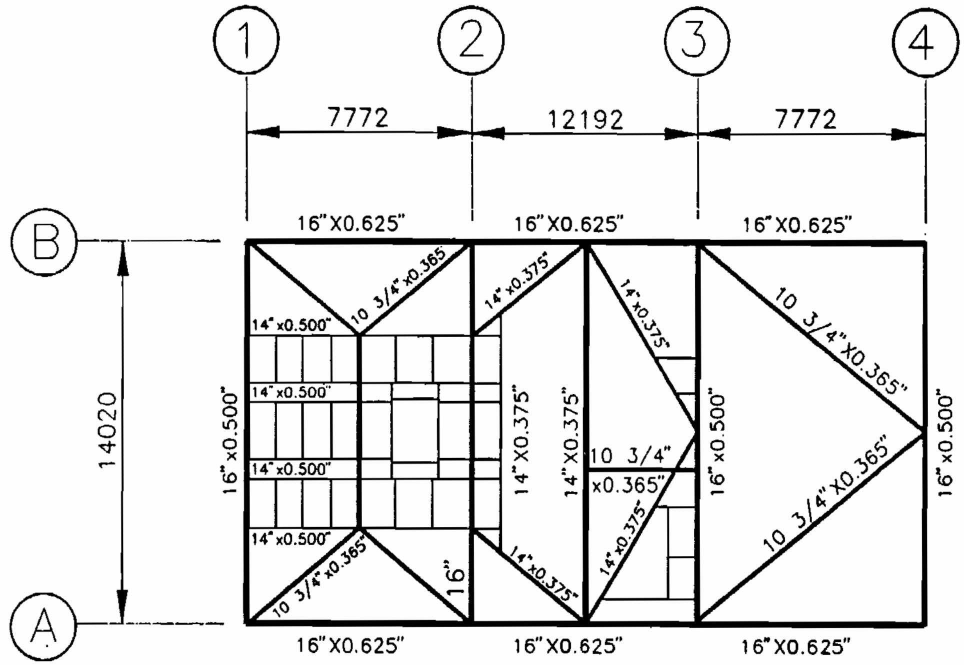
Figure 2.6f



JACKET LEGS ASTM A-36 STEEL
JACKET BRACING API-5L GR-B
========== ASTM A-572 GR 50 STEEL
---------- API 5LX X 42 STEEL

VERTICAL FRAME AT ROW 2
DRILLING PLATFORM NOHOCH-B

Figure 2.6g

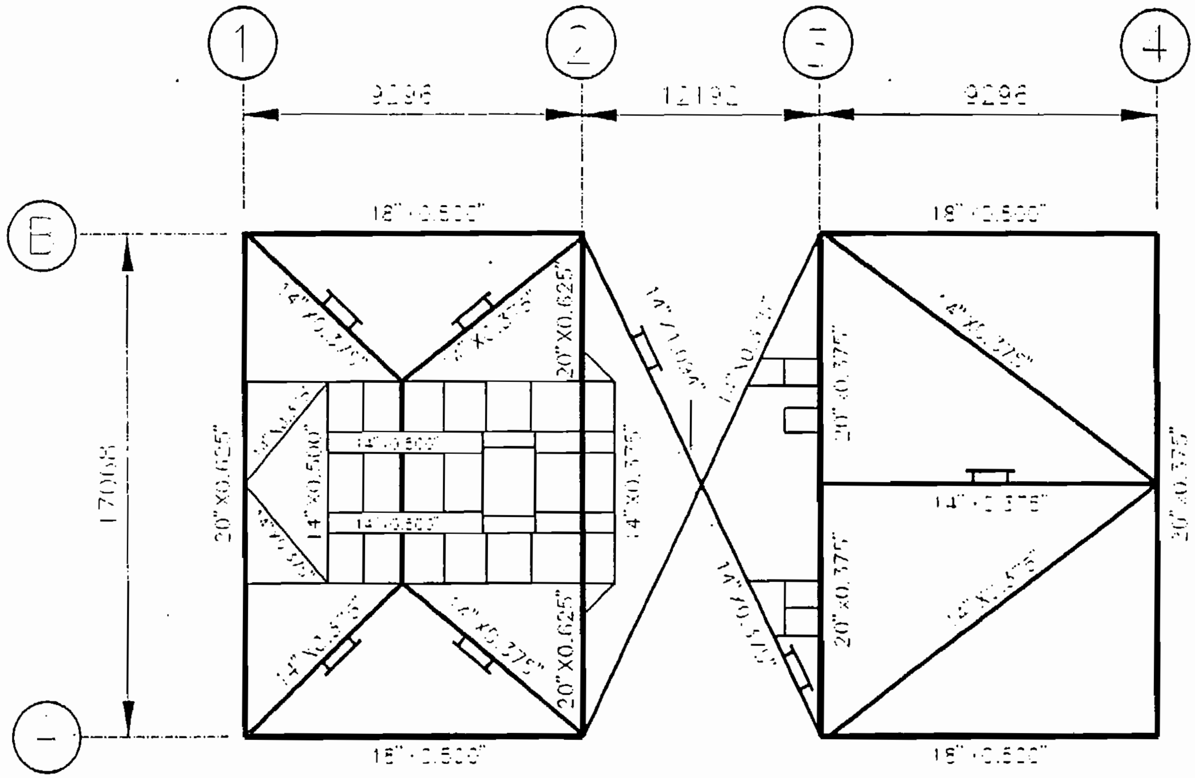


HORIZONTAL FRAME AT EL. + 3.658

DRILLING PLATFORM NOHOCH B

INSPECTON\PL3.DWG

Figure 2.6j



HORIZONTAL FRAME AT EL - 8.534

DRILLING PLATFORM NORTH B

Fig' 2.6k

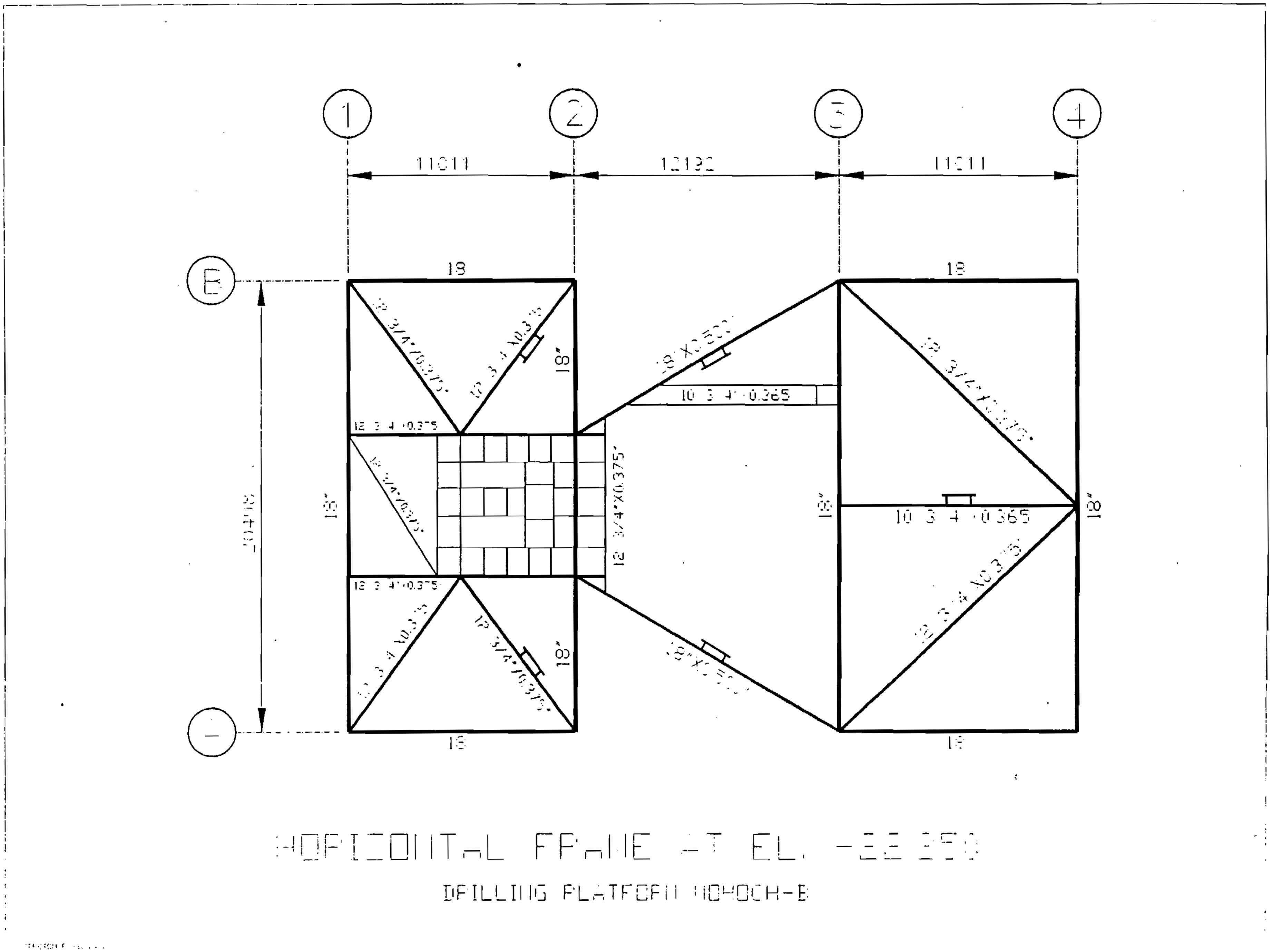
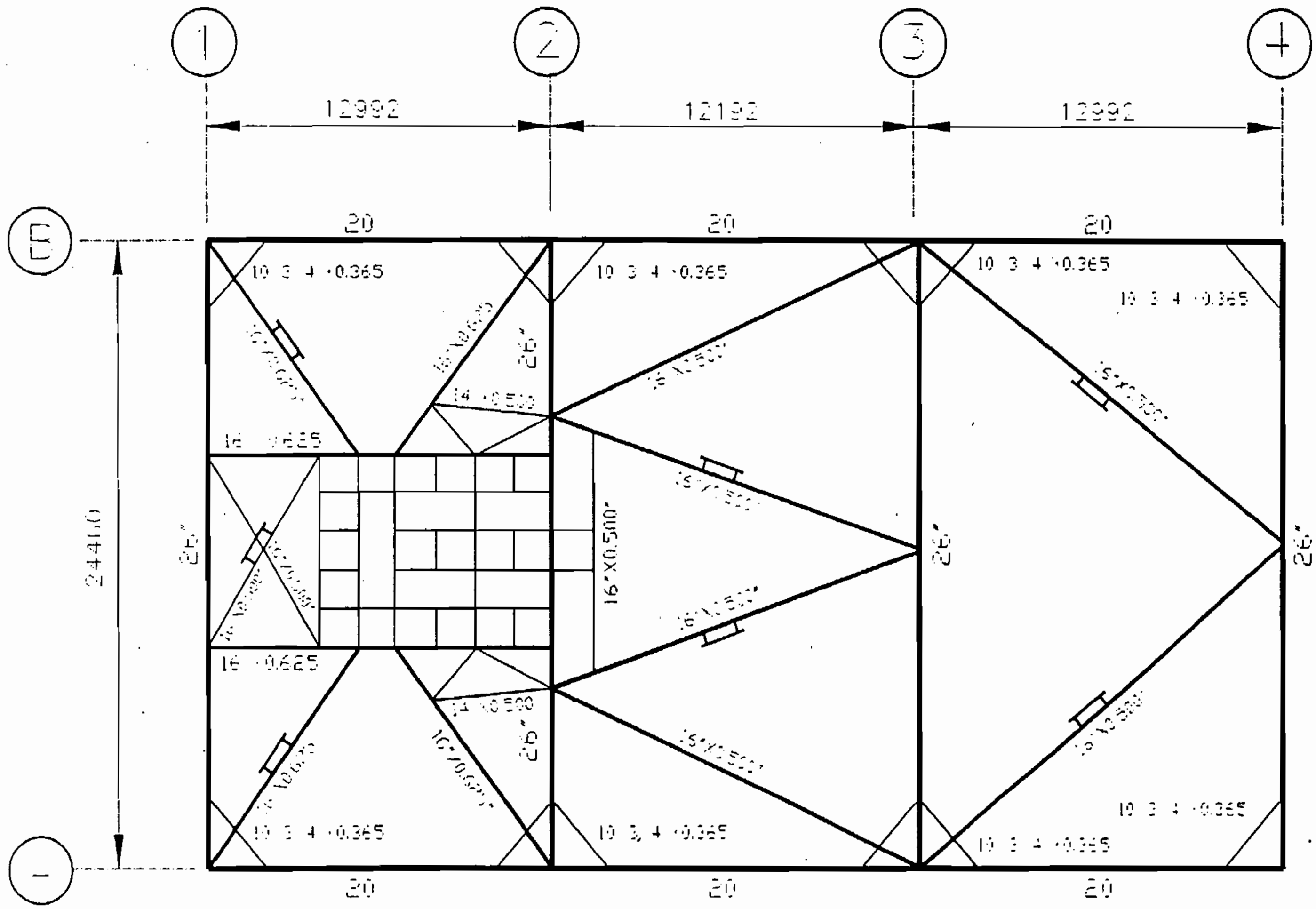

1
2
3
4
E
11011
12192
11011
20498
18"
18" X 0.500"
10 3/4" X 0.365"
12 3/4" X 0.375"
HORIZONTAL FRAME AT EL. -33.350
DRILLING PLATFORM NOHOCH-B

Figure 2.6I



HORIZONTAL FRAME AT EL -38.100

DRILLING PLATFORM NO-DC--3

Figure m

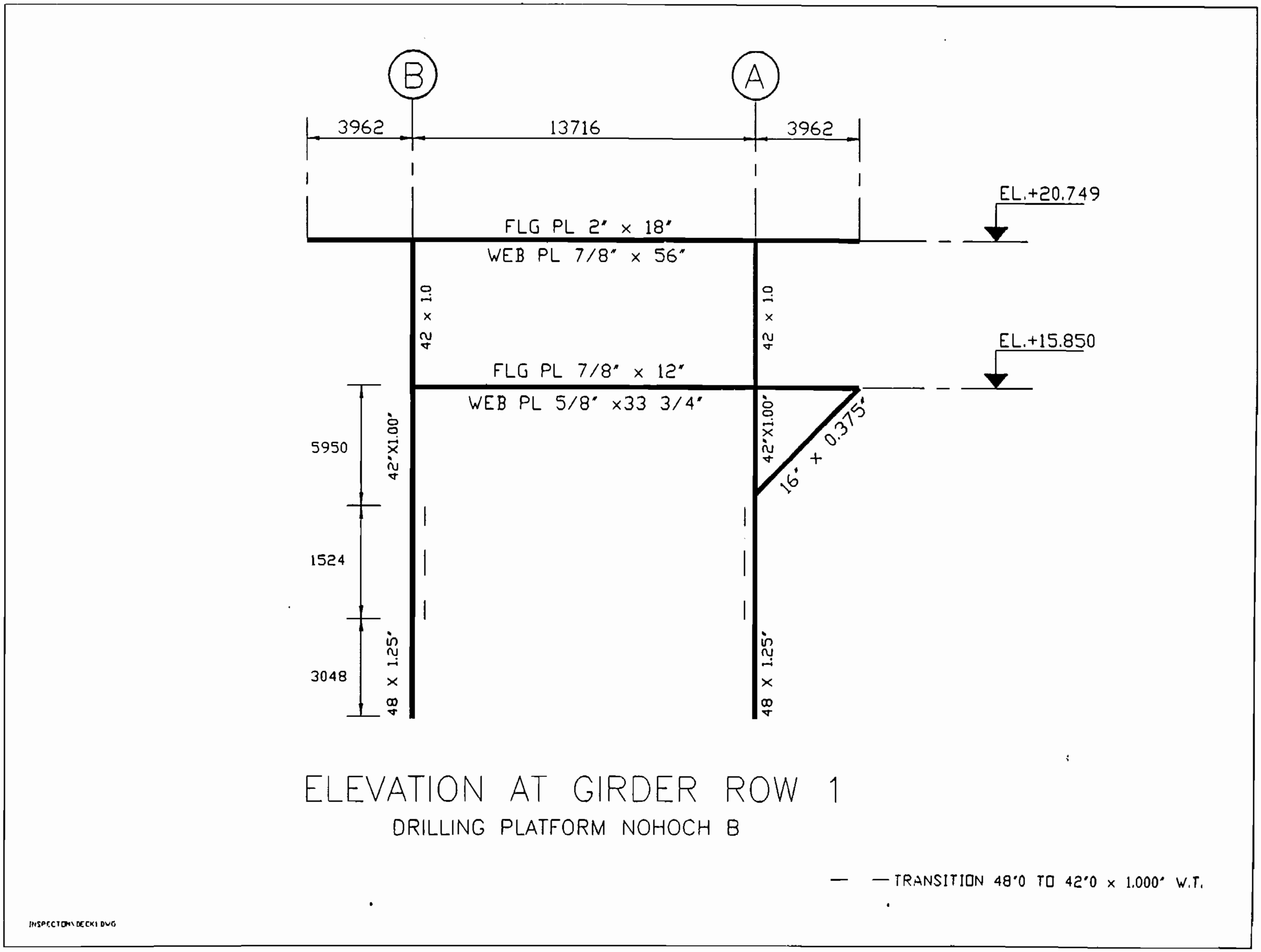
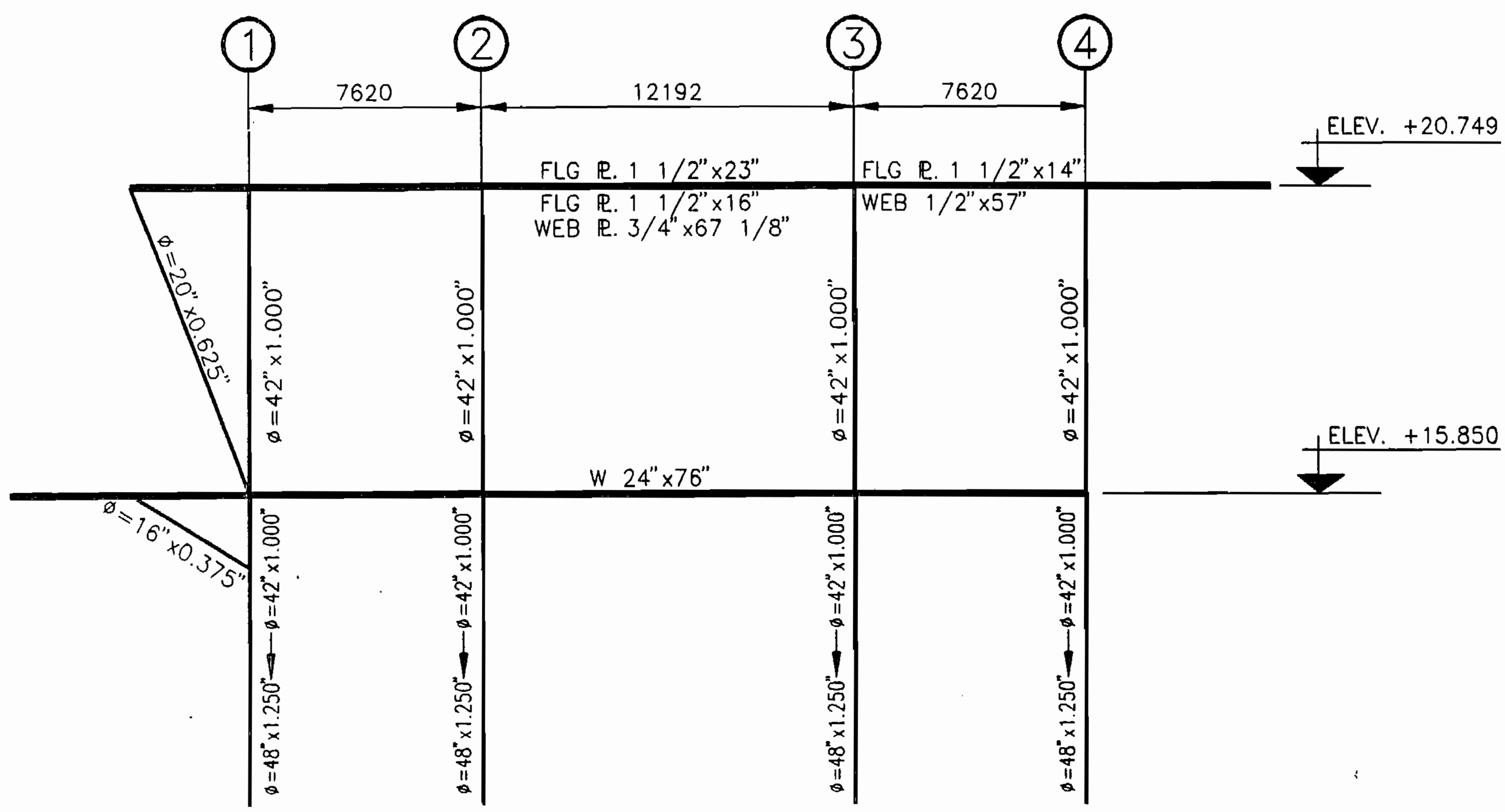
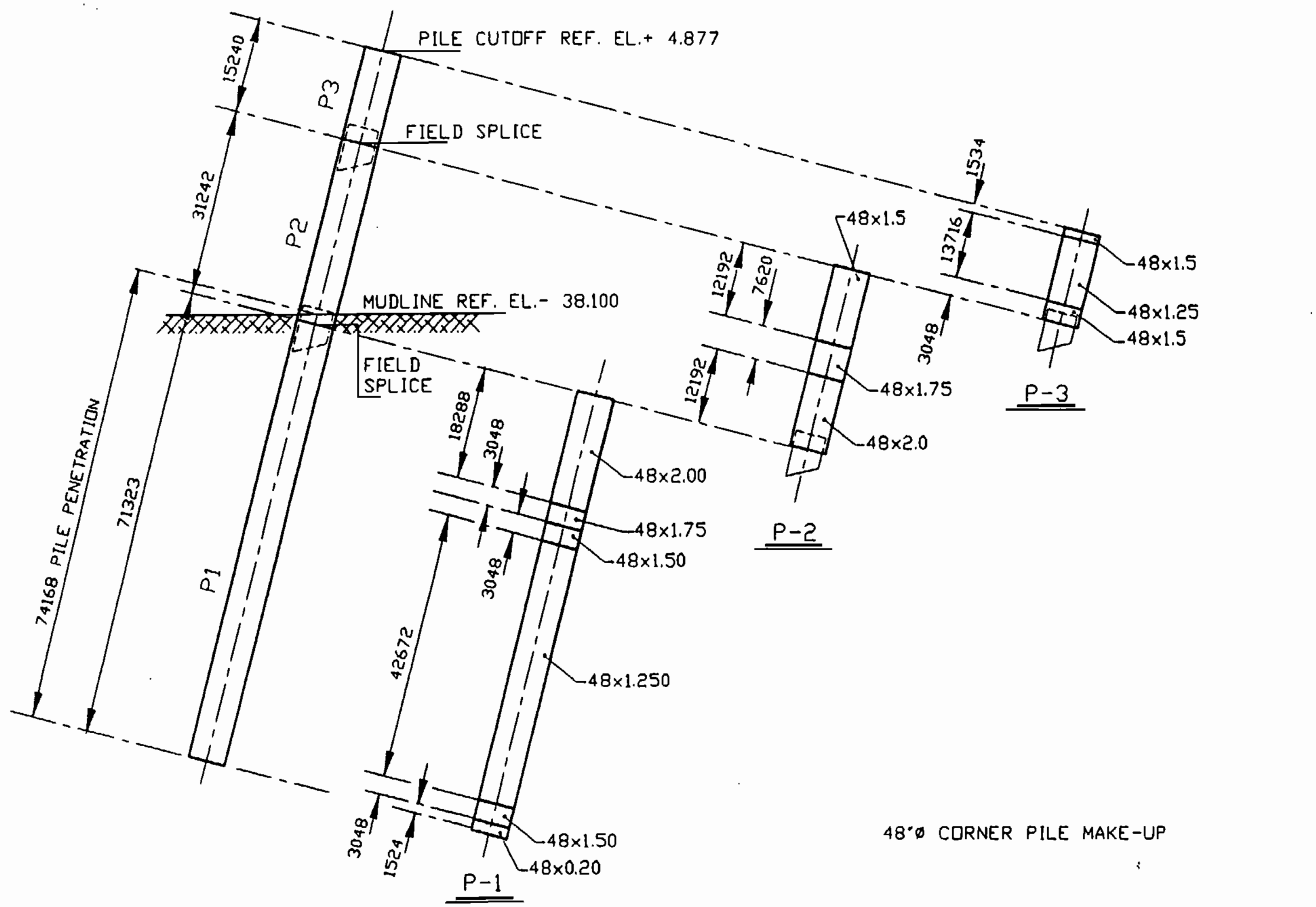

B
A
3962
13716
3962
EL.+20.749
FLG PL 2″ x 18″
WEB PL 7/8″ x 56″
42 x 1.0
42 x 1.0
EL.+15.850
FLG PL 7/8″ x 12″
WEB PL 5/8″ x33 3/4″
5950
42″X1.00″
42″X1.00″
16″ + 0.375″
1524
3048
48 X 1.25″
48 X 1.25″
ELEVATION AT GIRDER ROW 1
DRILLING PLATFORM NOHOCH B
— TRANSITION 48″0 TO 42″0 x 1.000″ W.T.
INSPECTON\DECK1.DWG

Figure 2.6n



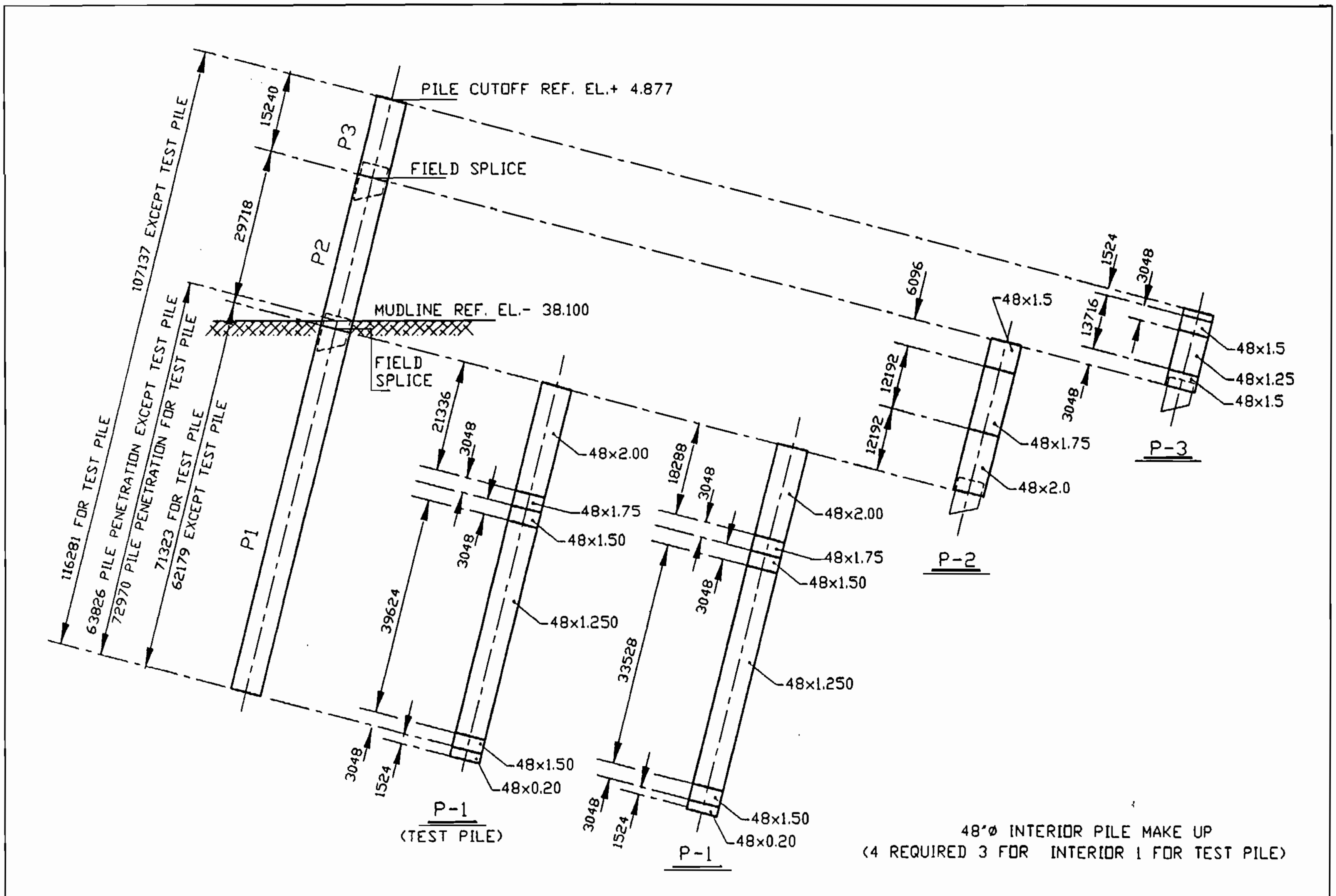
ELEVATION AT GIRDER ROW A
DRILLING PLATFORM NOHOCH-B

Figure 2.6r



CORNER PILE MAKE - UP
DRILLING PLATFORM NOHOCH-B

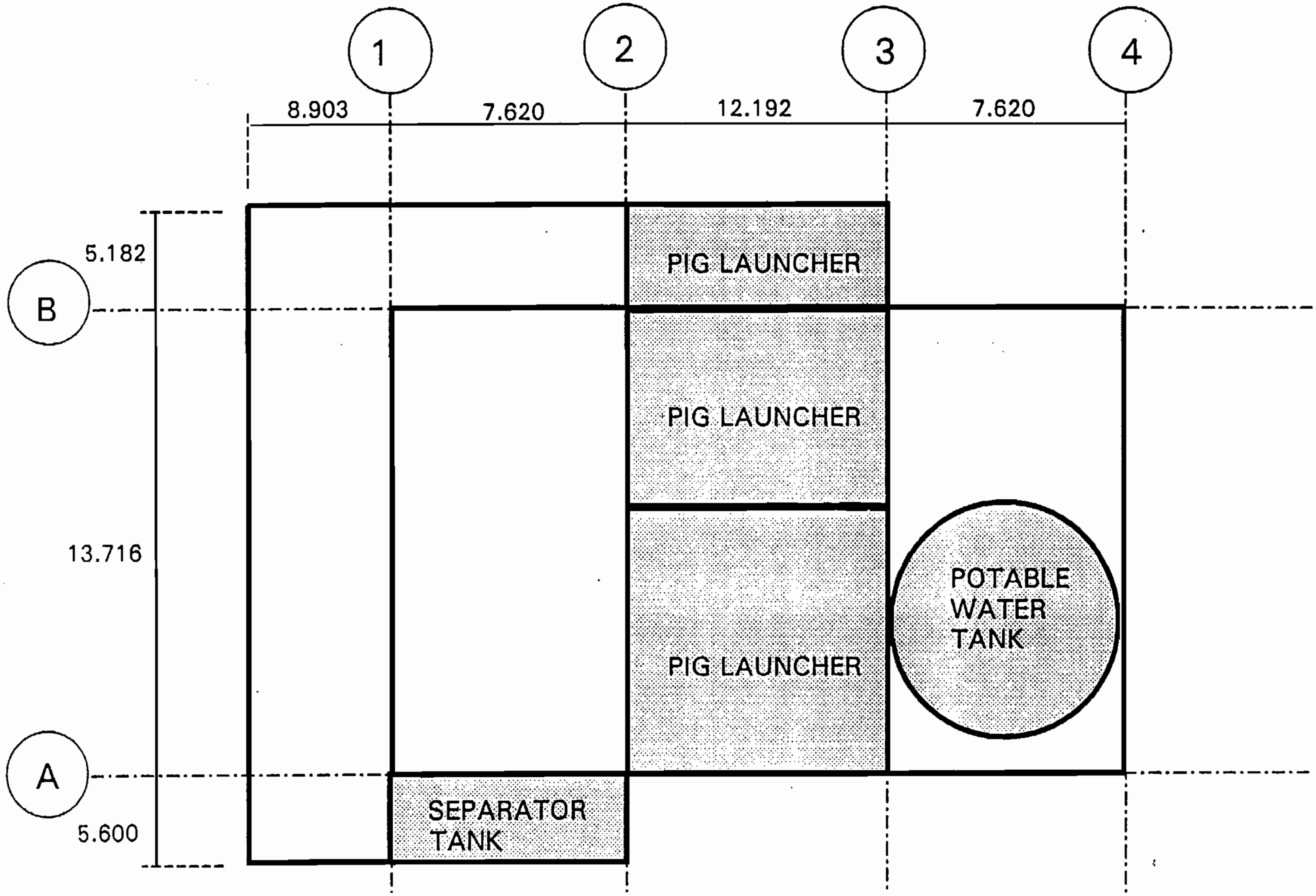
Figure 2.6x



INTERIOR PILE AND TEST PILE MAKE UP

DRILLING PLATFORM 'NOHOCH-B

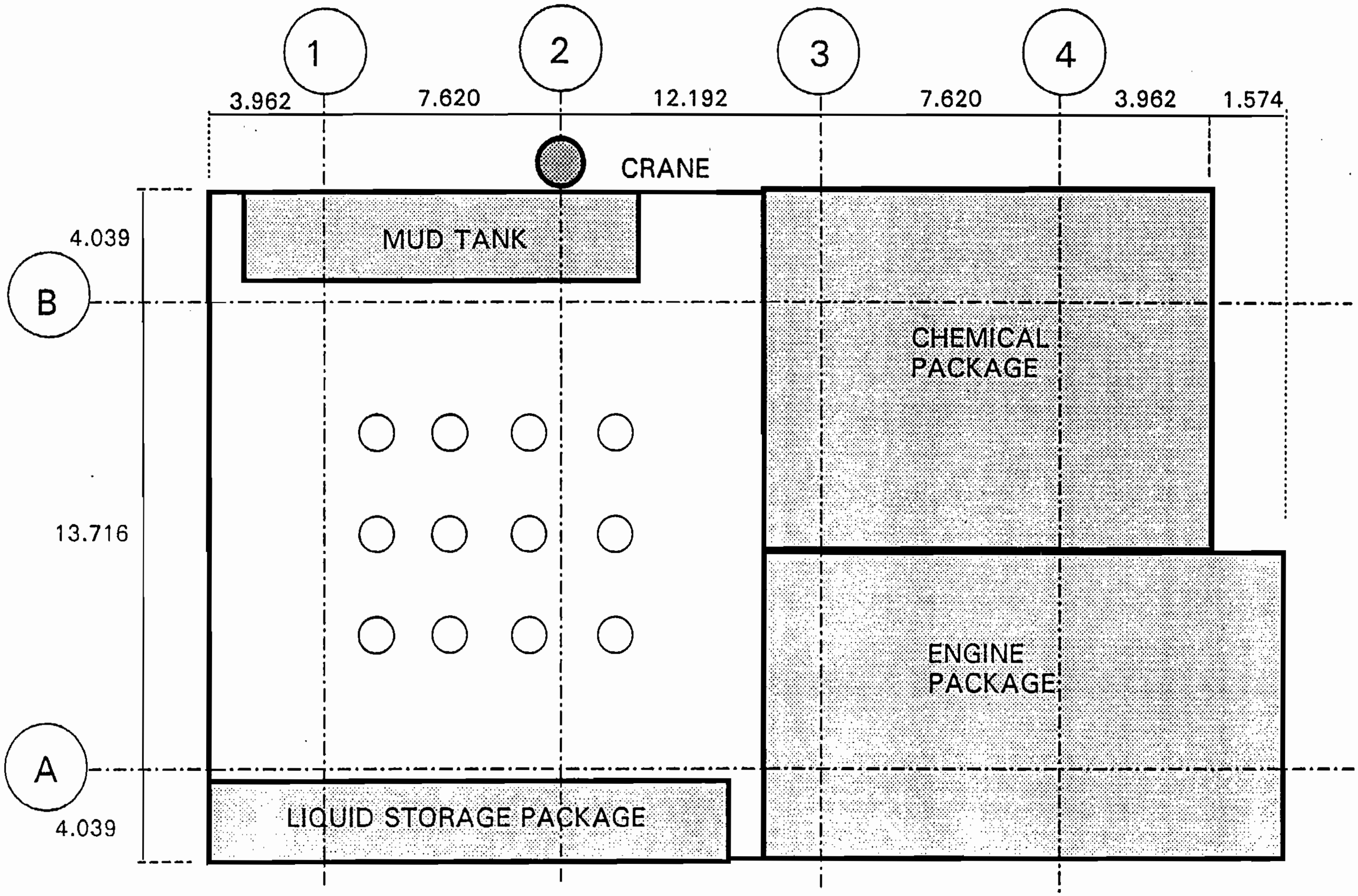
Figure 2



CELLAR DECK LEVEL +15.850 M (+52'-0")
DRILLING PLATFORM NOHOCH-B

JIP4/CUBINF.DRW

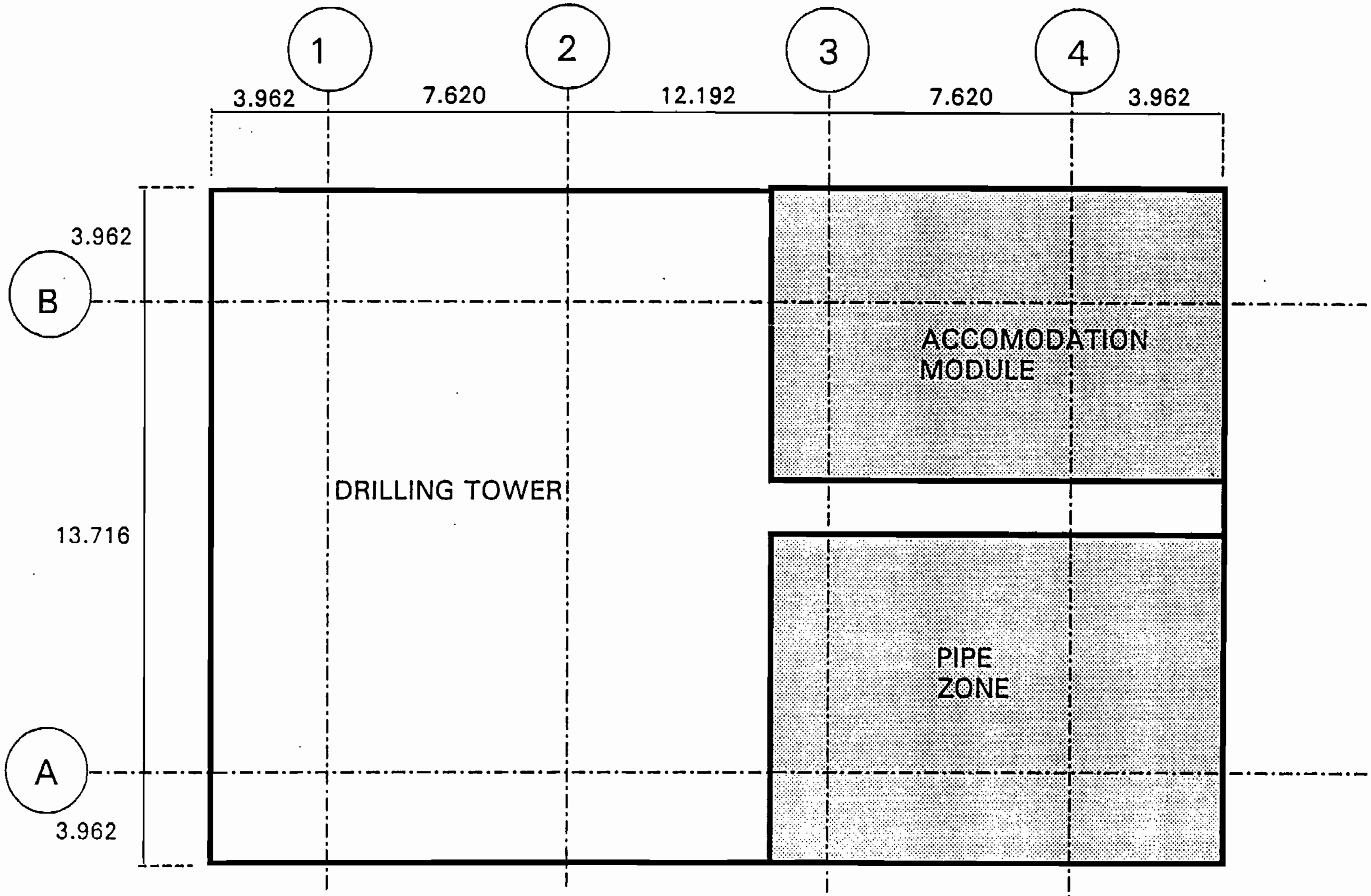
Figure 2.6z



MAIN DECK LEVEL +20.749 M (+68' 7/8")
DRILLING PLATFORM NOHOCH-B

JIP4/CUBPRIN.DRW

Figure ^ 6aa



TOP SIDE DECK
DRILLING PLATFORM NOHOCH-B

JIP4/CUBPERF.DRW

Figure 2.6ab

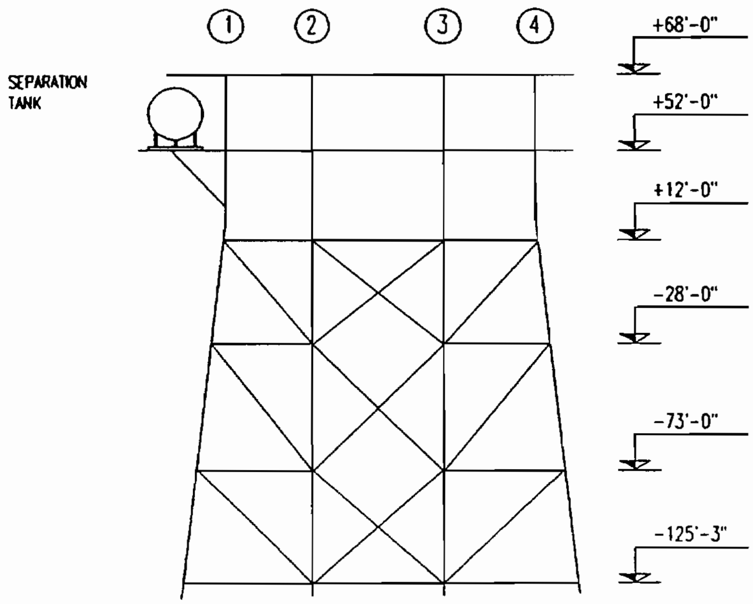


Figure 3.1.3a STRUCTURE ELEVATION

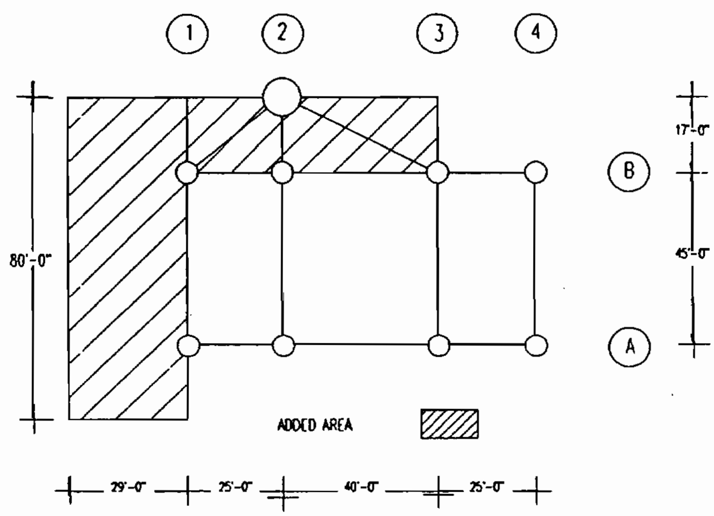
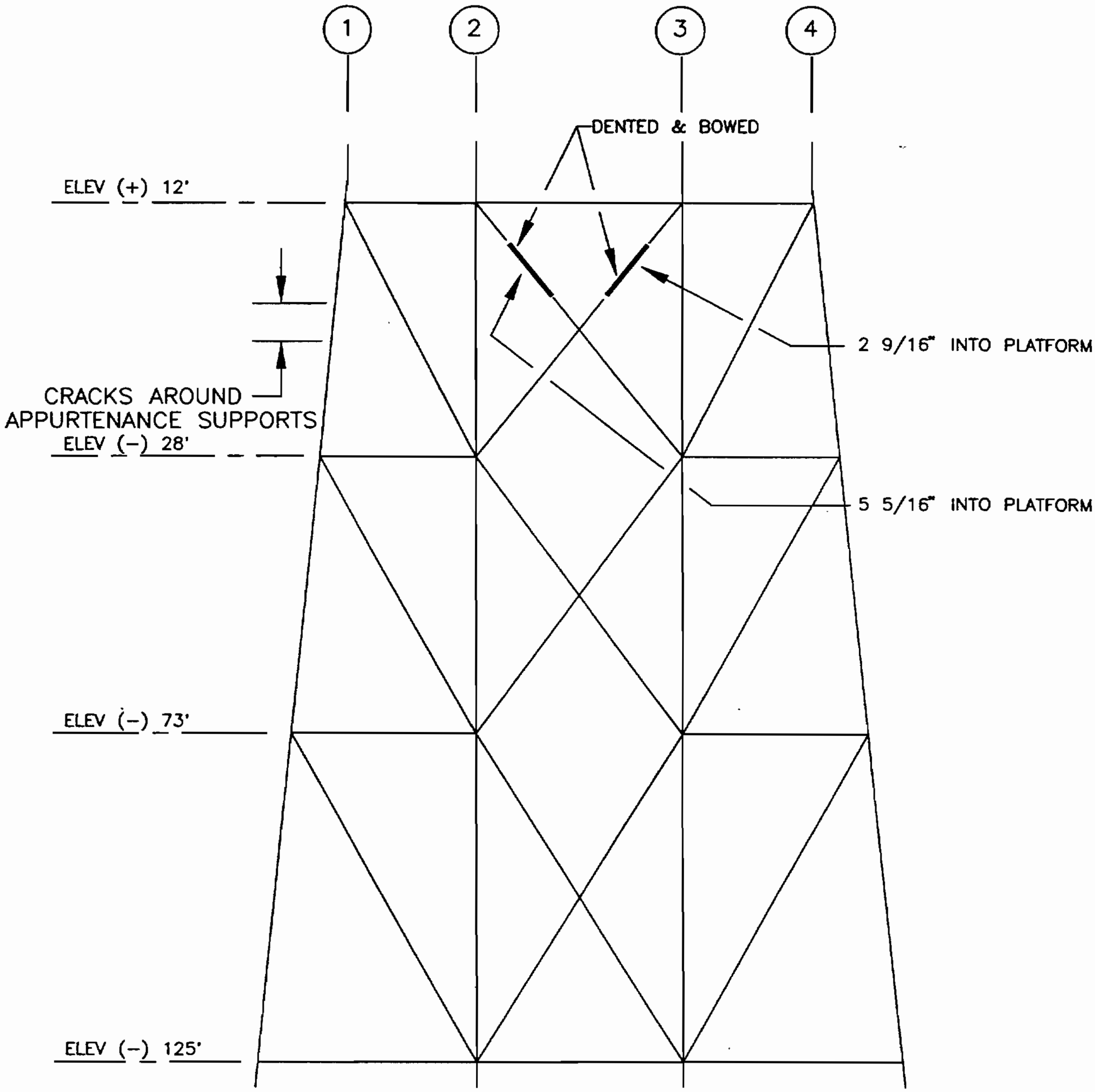


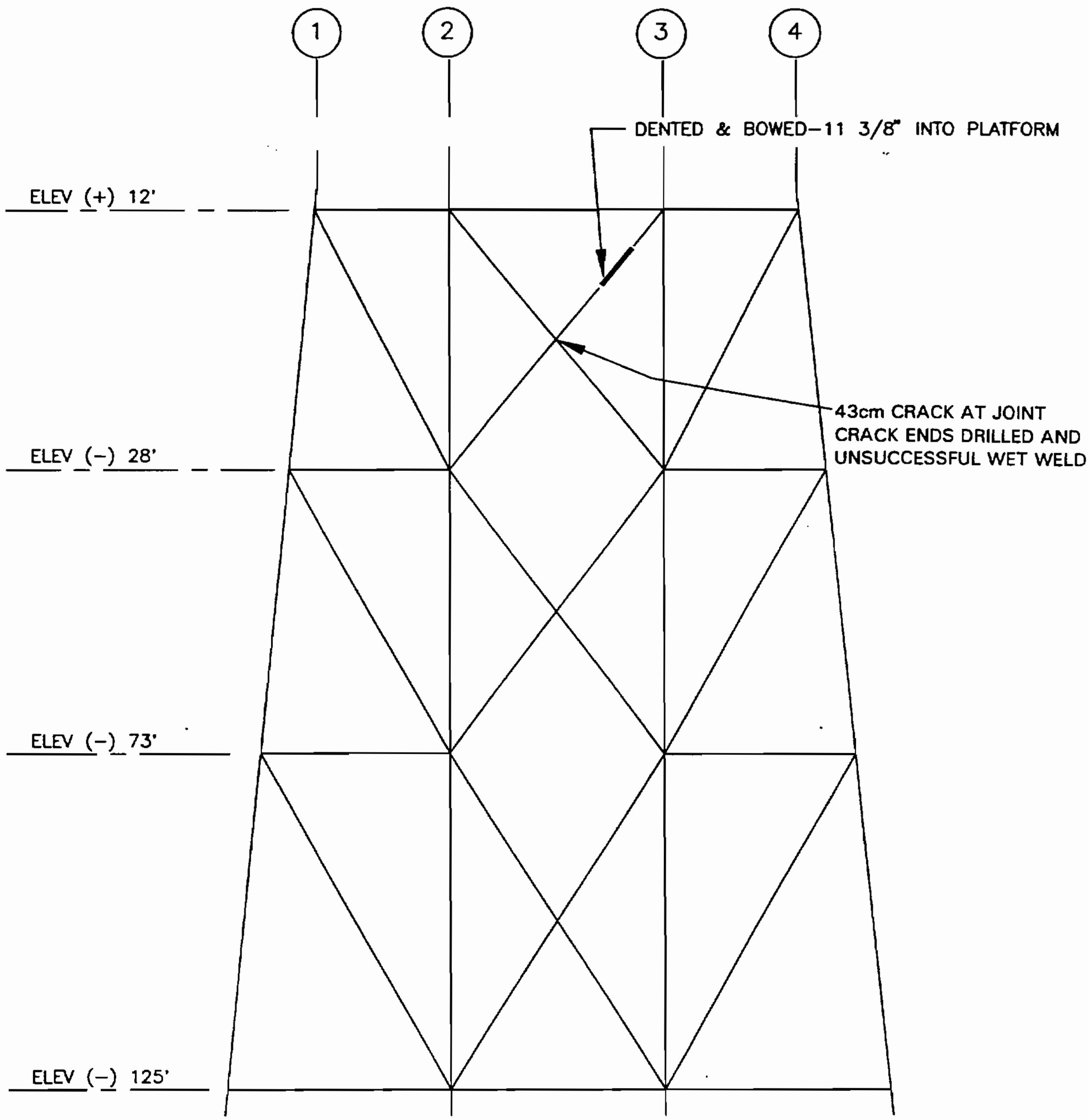
Figure 3.1.3b · "CELLAR DECK" PLAN



NOHOCH B PLATFORM

JACKET DAMAGE ROW-A

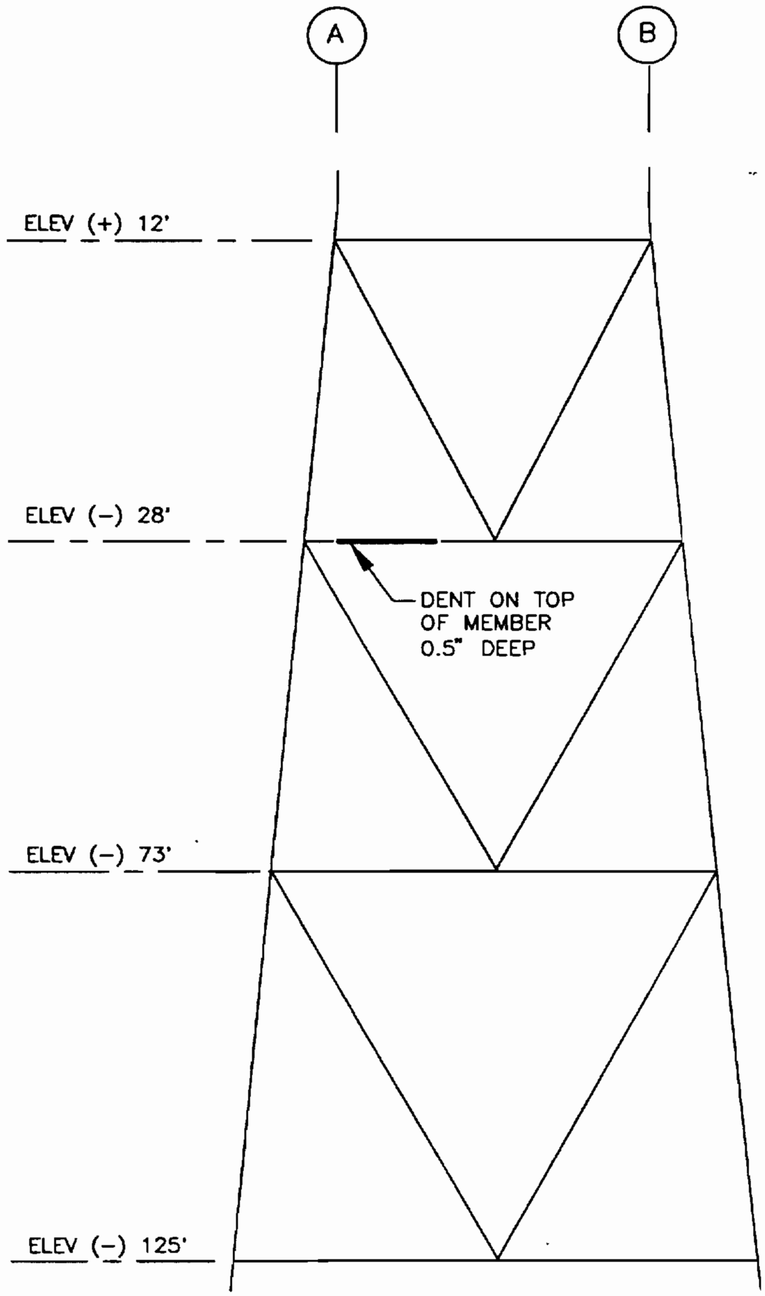
Figure 3.1.5a



NOHOCH B PLATFORM

JACKET DAMAGE ROW-B

Figure 3.1.5b

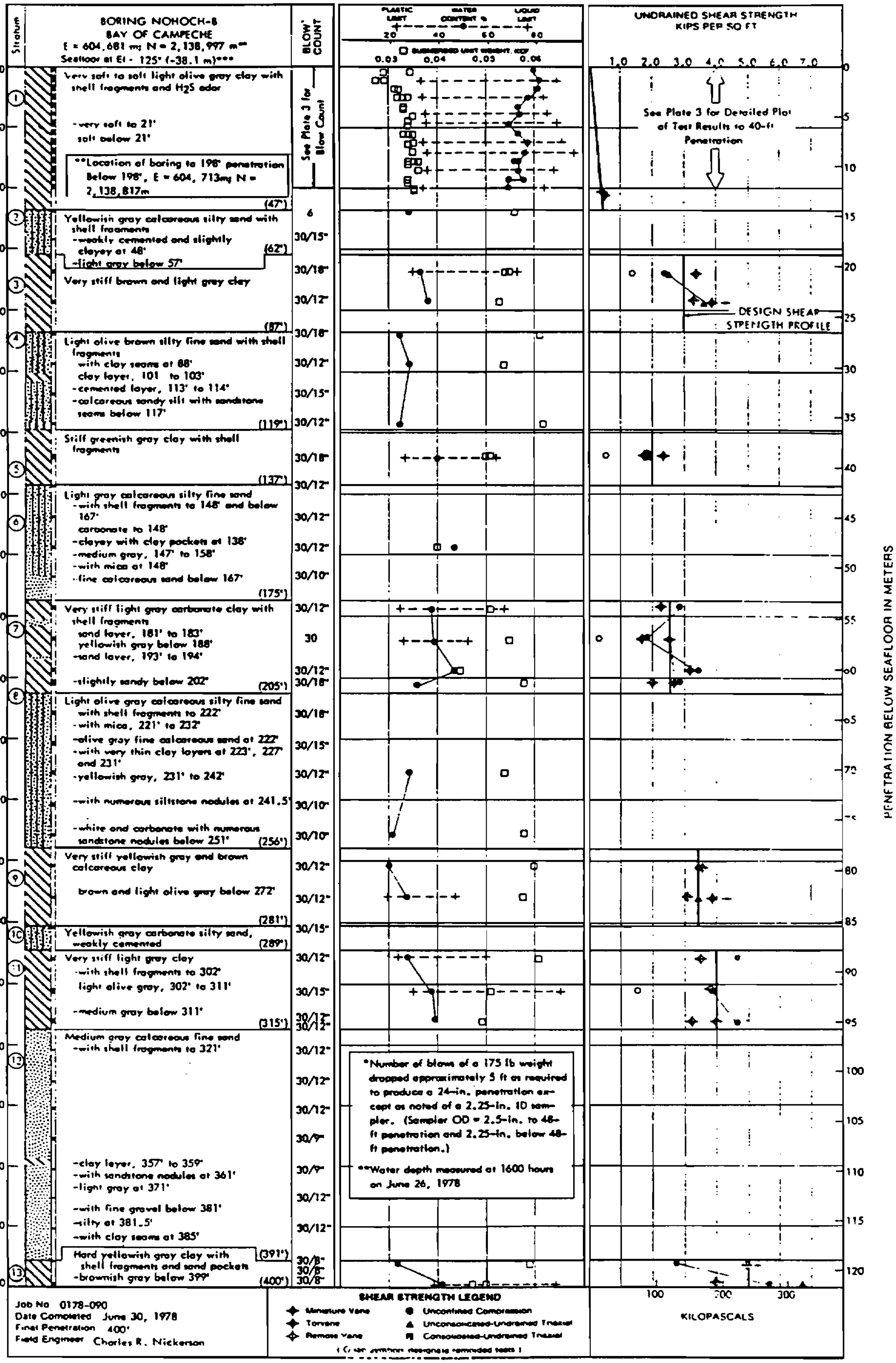


NOTE: DENT SIZE – 29.2 CM LONG
20.3 CM WIDE
1.5 CM DEEP

NOHOCH B PLATFORM

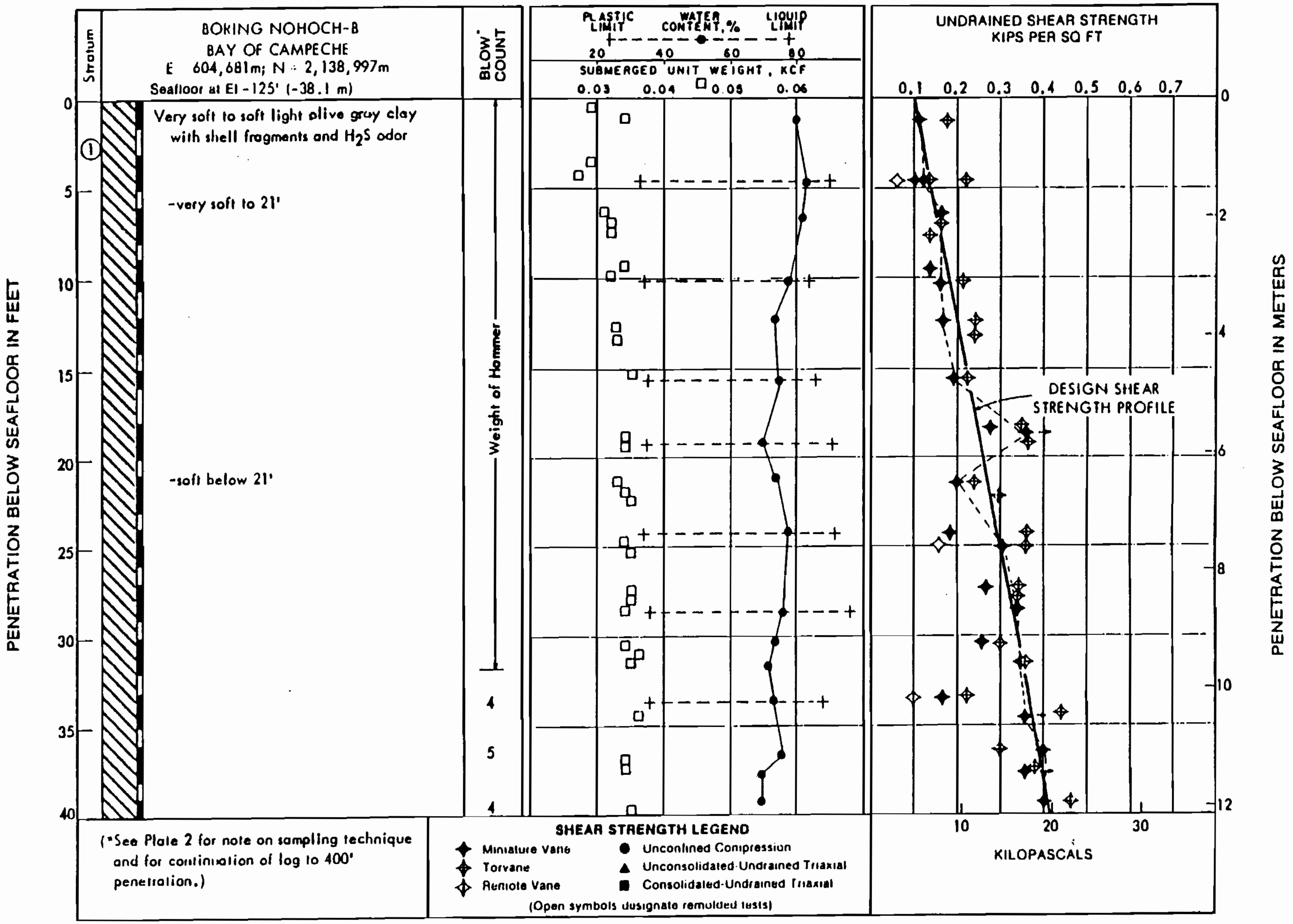
JACKET DAMAGE ROW-4

Figure 3.1.5c



LOG AND TEST RESULTS
BORING NOHOCH-B
BAY OF CAMPECHE

Figure 3.3a

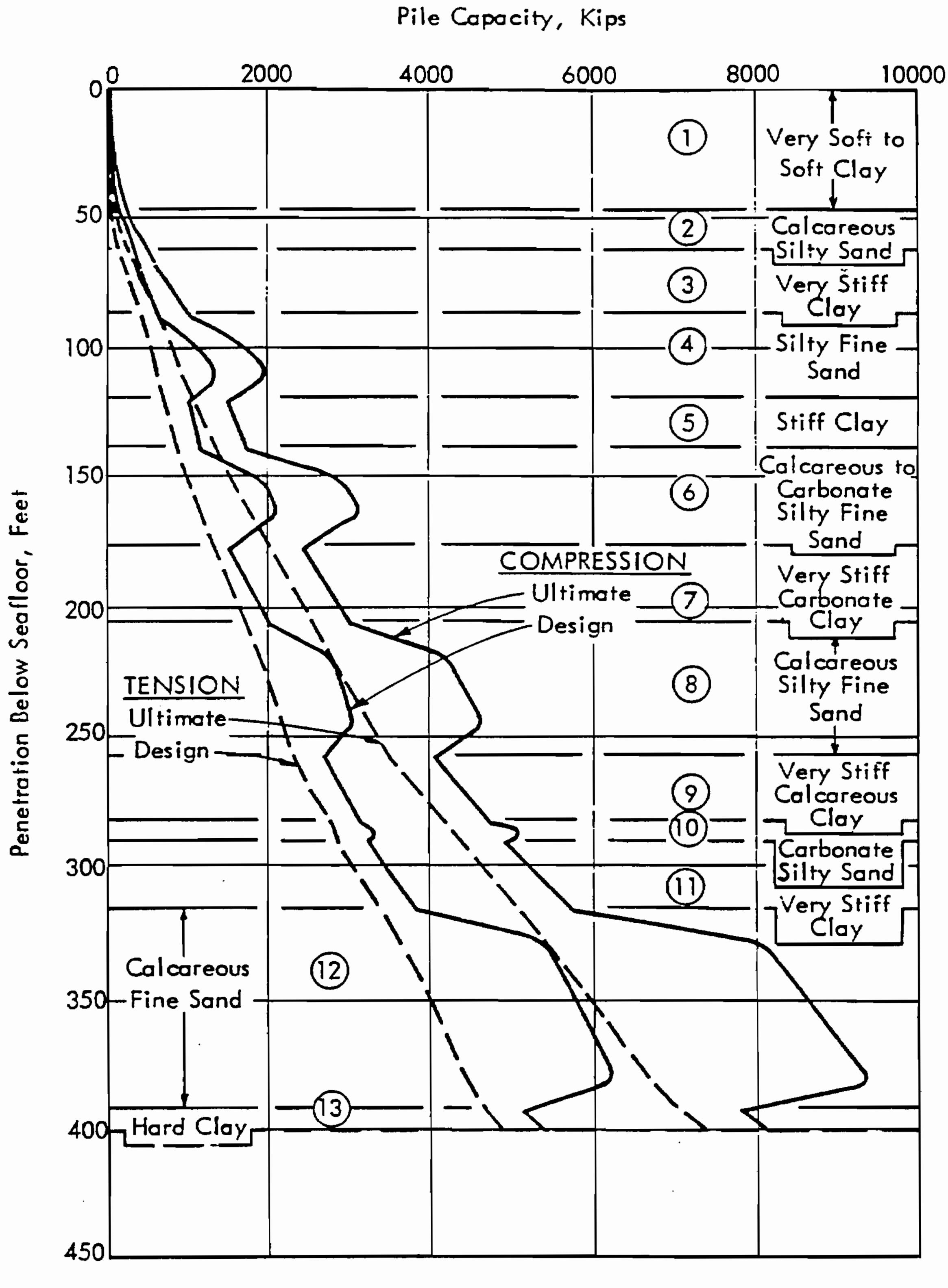


LOG AND TEST RESULTS TO 40-FT PENETRATION

BORING NOHOCH-B

BAY OF CAMPECHE

Figure 3.3b



PILE CAPACITY CURVES

48-in.-Diameter Pipe Pile

λ-Method

Boring NOHOCH-B

Bay of Campeche

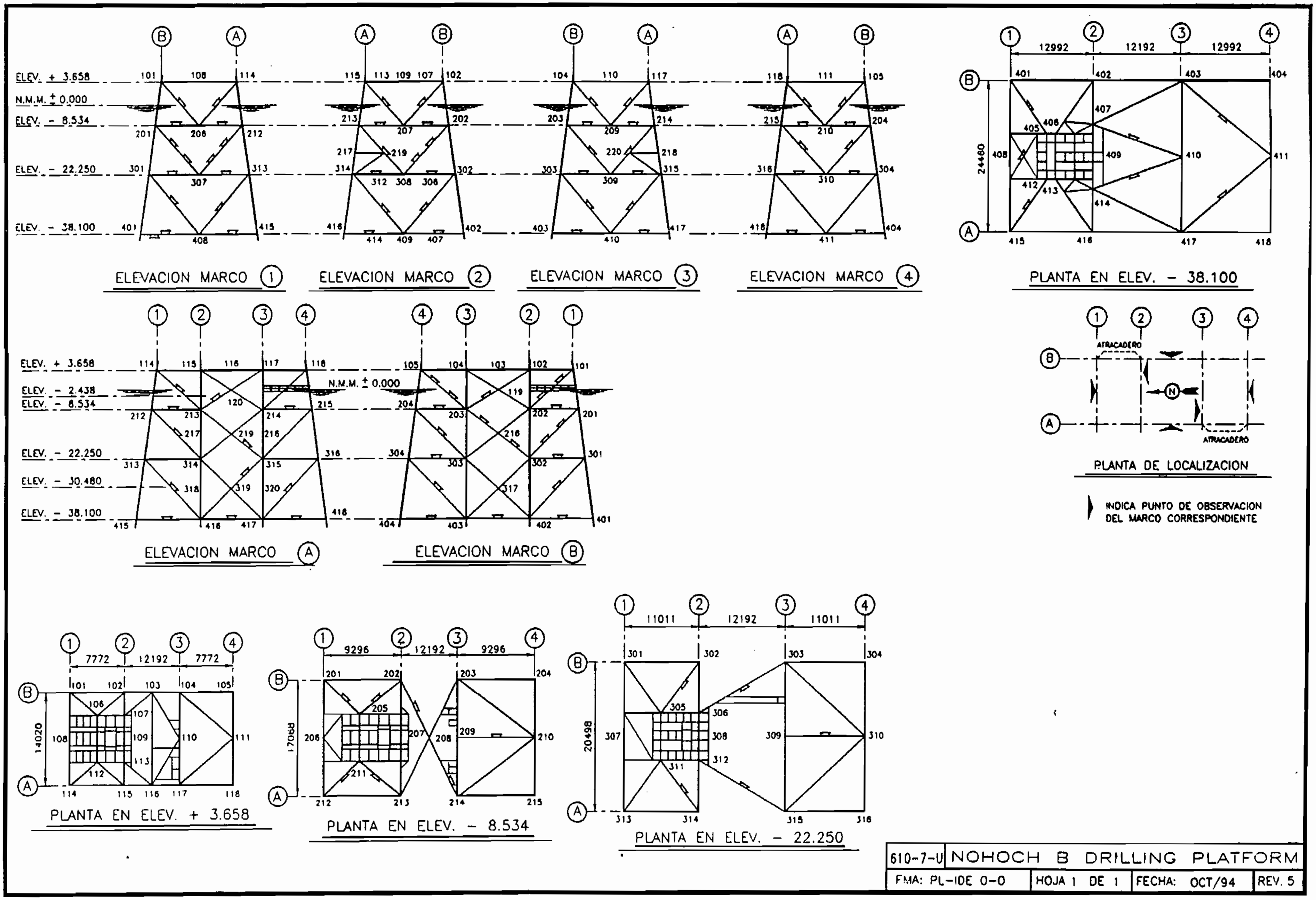
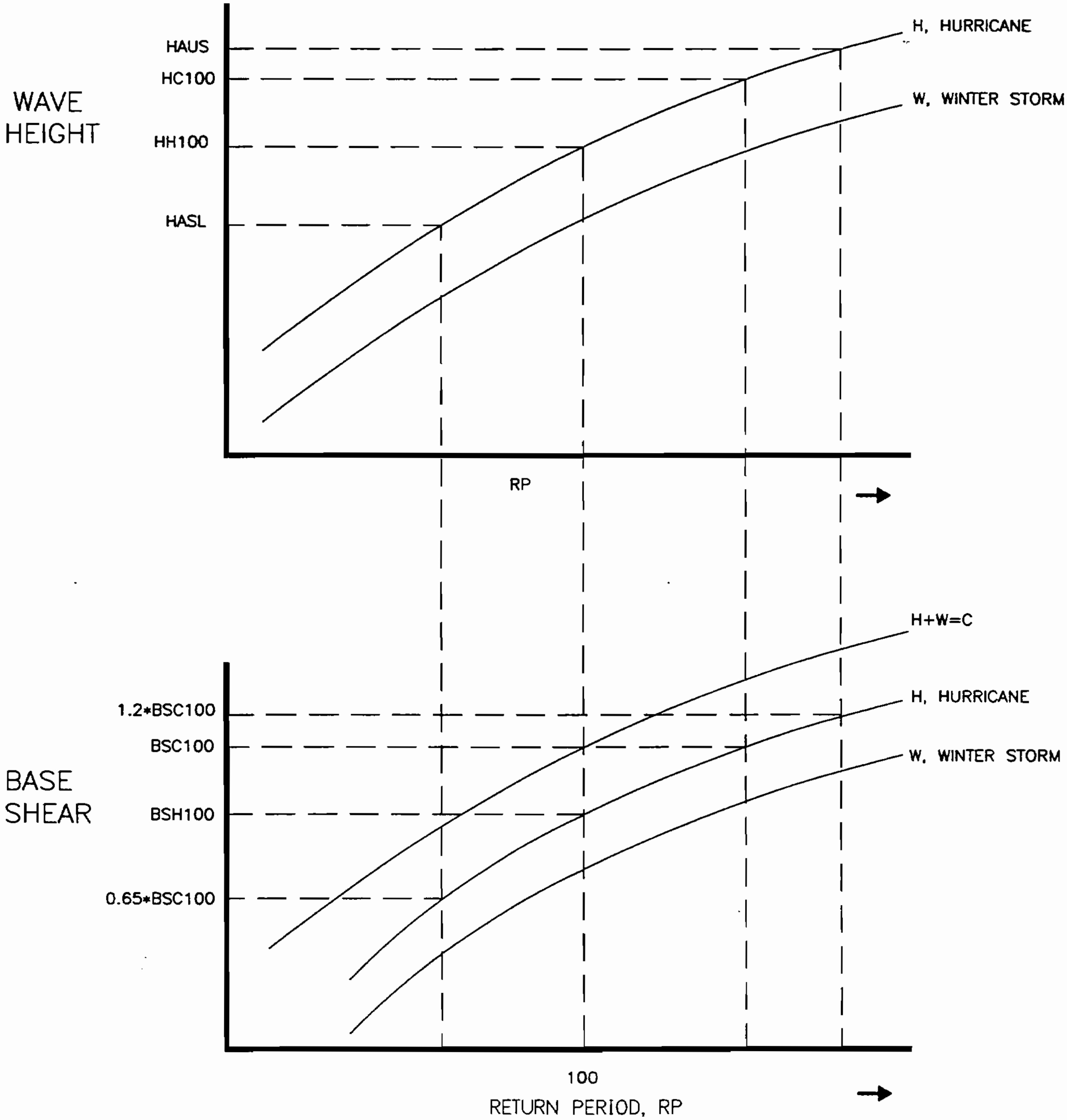


Figure 3.3d Inspection Referencing

HAUS–EQUIVALENT HURICANE WAVE HEIGHT, ASSESSMENT FOR ULTIMATE STRENGTH ANALYSIS, FOR 100 YEAR BASE SHEAR.
HC100–EQUIVALENT HURRICANE WAVE HEIGHT BASED ON 100 YEAR RETURN BASE SHEAR.
HH100–HURRICANE WAVE HEIGHT FOR 100 YEAR RETURN.
HASL–EQUIVALENT HURRICANE WAVE HEIGHT, ASSESSMENT FOR STRENGTH LEVEL ANALYSIS, FOR 100 YEAR BASE SHEAR.
BSC100–BASE SHEAR FOR COMBINED HURRICANE AND WINTER STORM POPULATION, 100 YEAR RETURN.
BSH100–BASE SHEAR FOR HURRICANE POPULATION, 100 YEAR RETURN.



CRITERIA DEVELOPMENT PROCEDCURES
FOR
NOHOCH B SITE

FIGURE 3.5.1a

FIGURE 3.5.1.b
WAVE HEIGHT vs. RETURN PERIOD
(400 Foot Water Depth, Ref.6, Fig. 11)

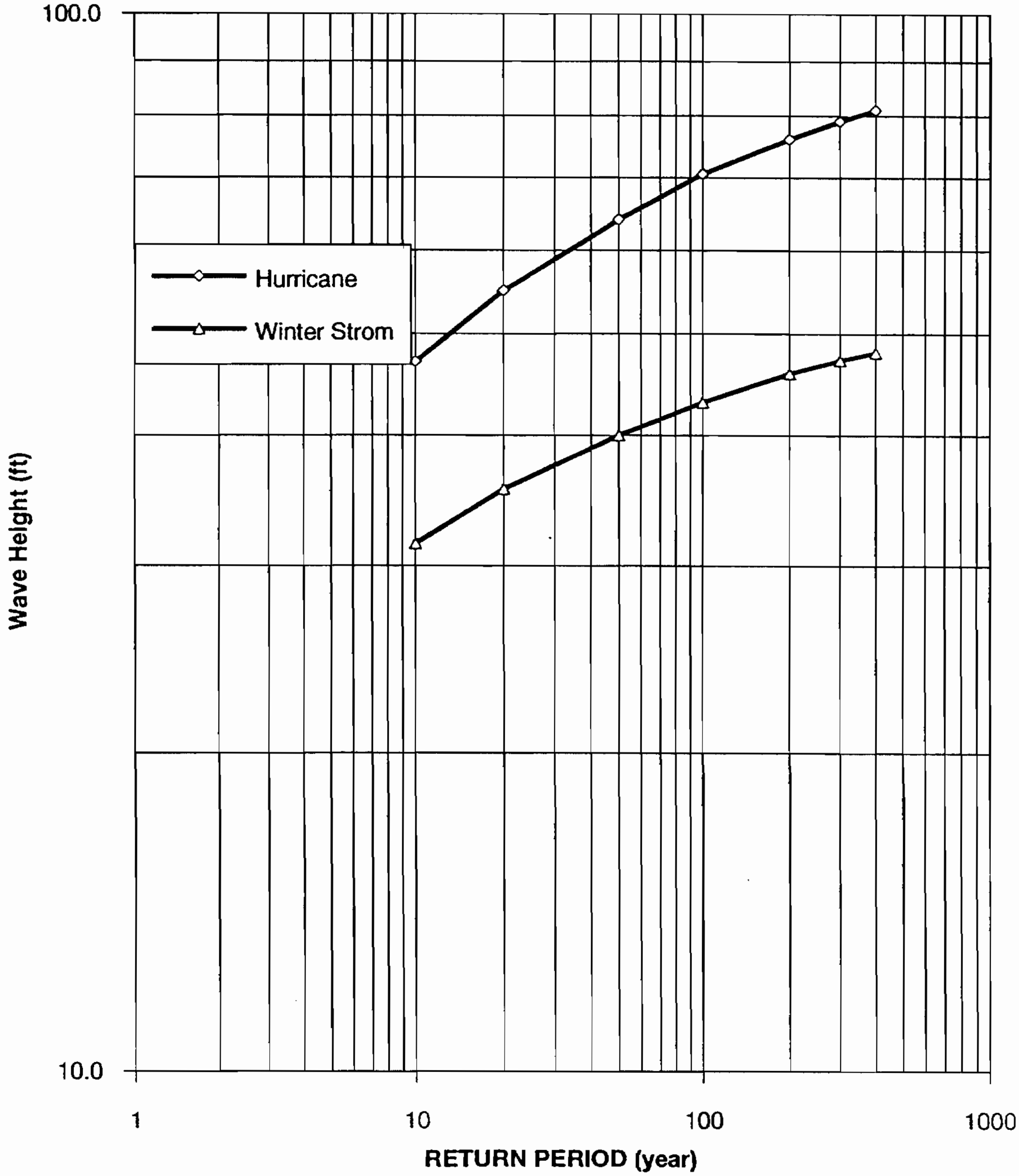


FIGURE 3.5.1.c
BASE SHEAR vs. RETURN PERIOD
(400 Foot Water Depth, Ref.6, Fig.11)

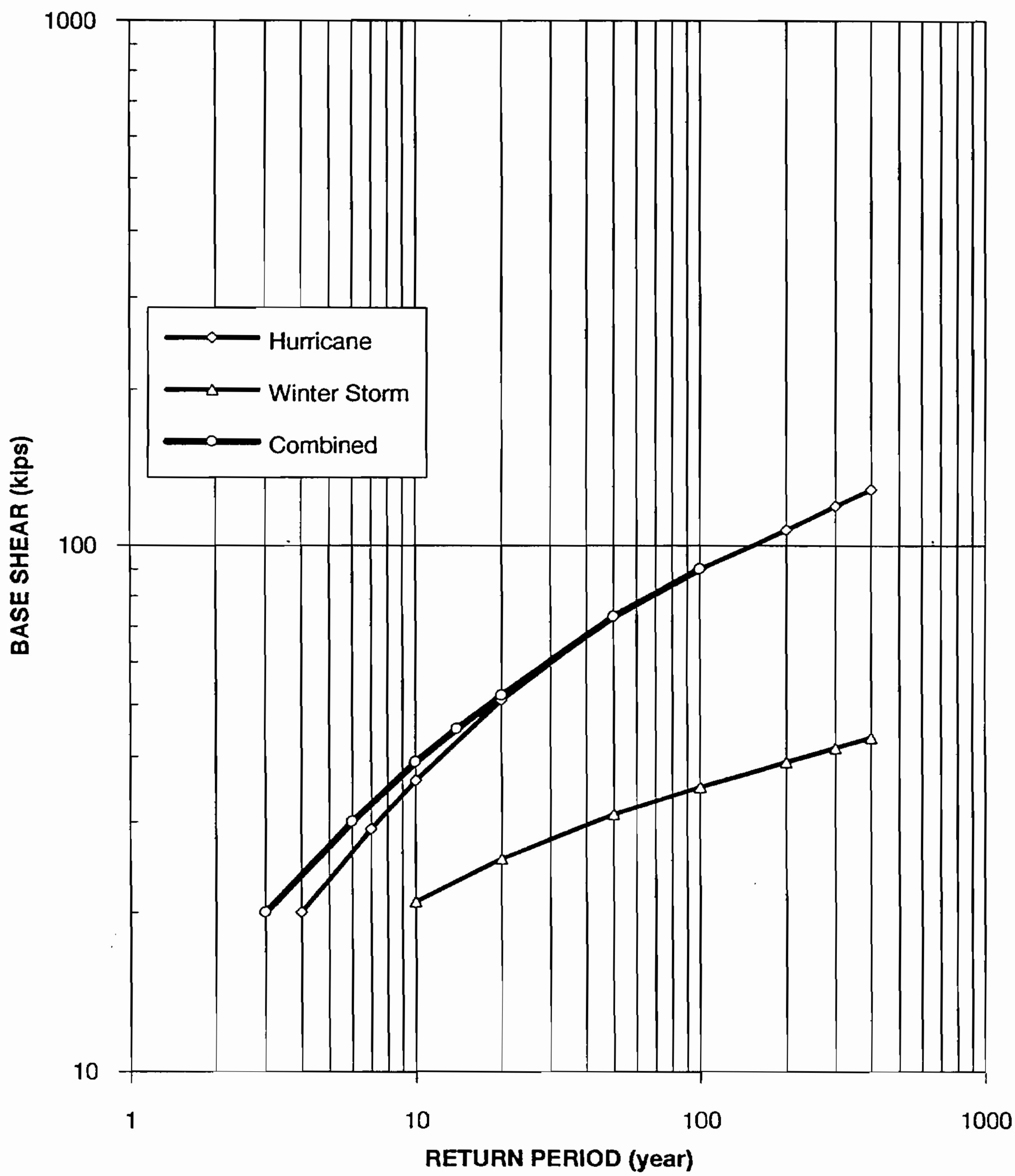


FIGURE 3.5.1.d
WAVE HEIGHT vs. RETURN PERIOD
(A. H. GLEN's Data, 1977&1980 At NOHOCH B Site)

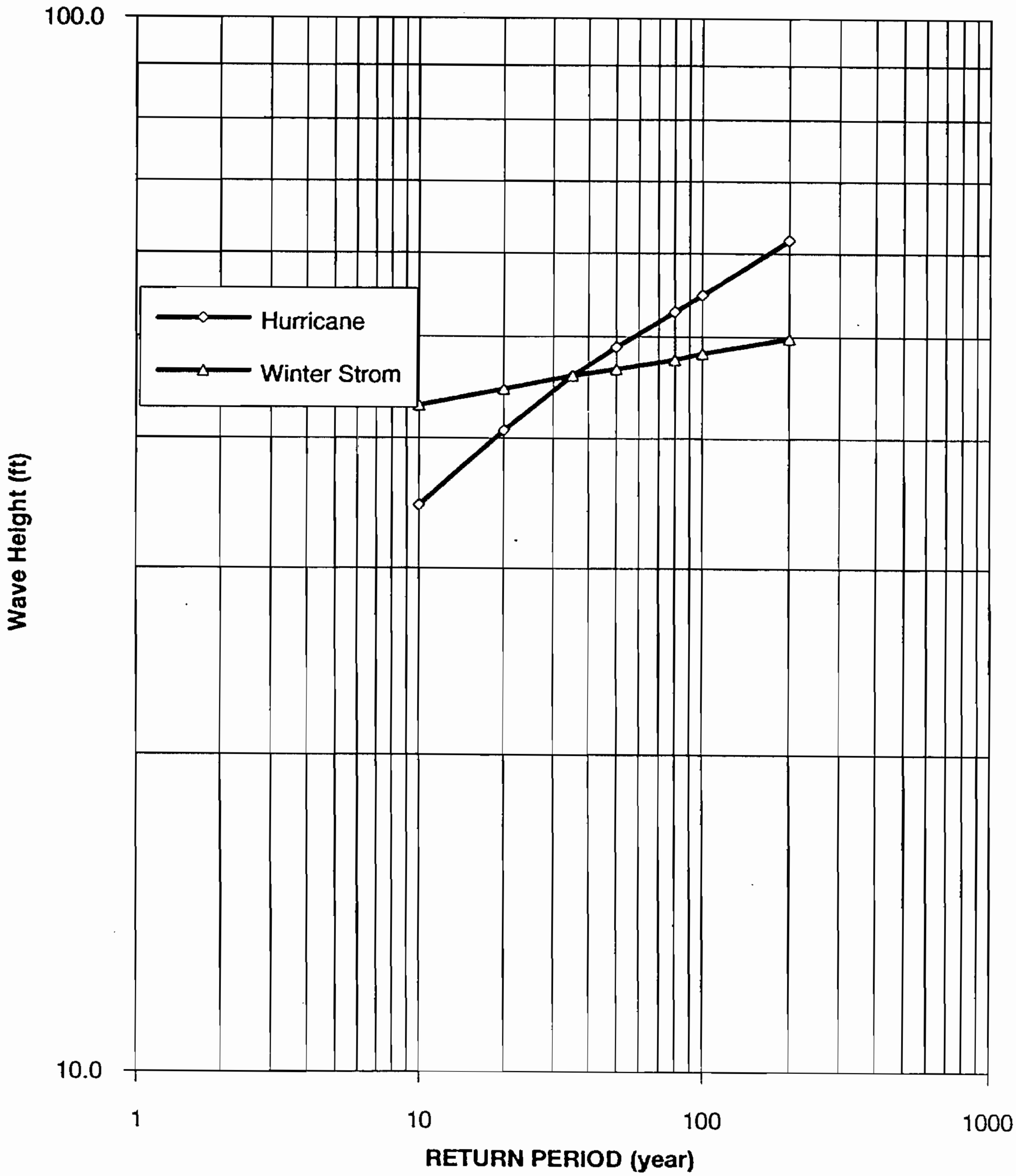


FIGURE 3.5.1.e
BASE SHEAR vs. RETURN PERIOD
(A. H. GLEN's Data, 1977&1980 At NOHOCH B Site)

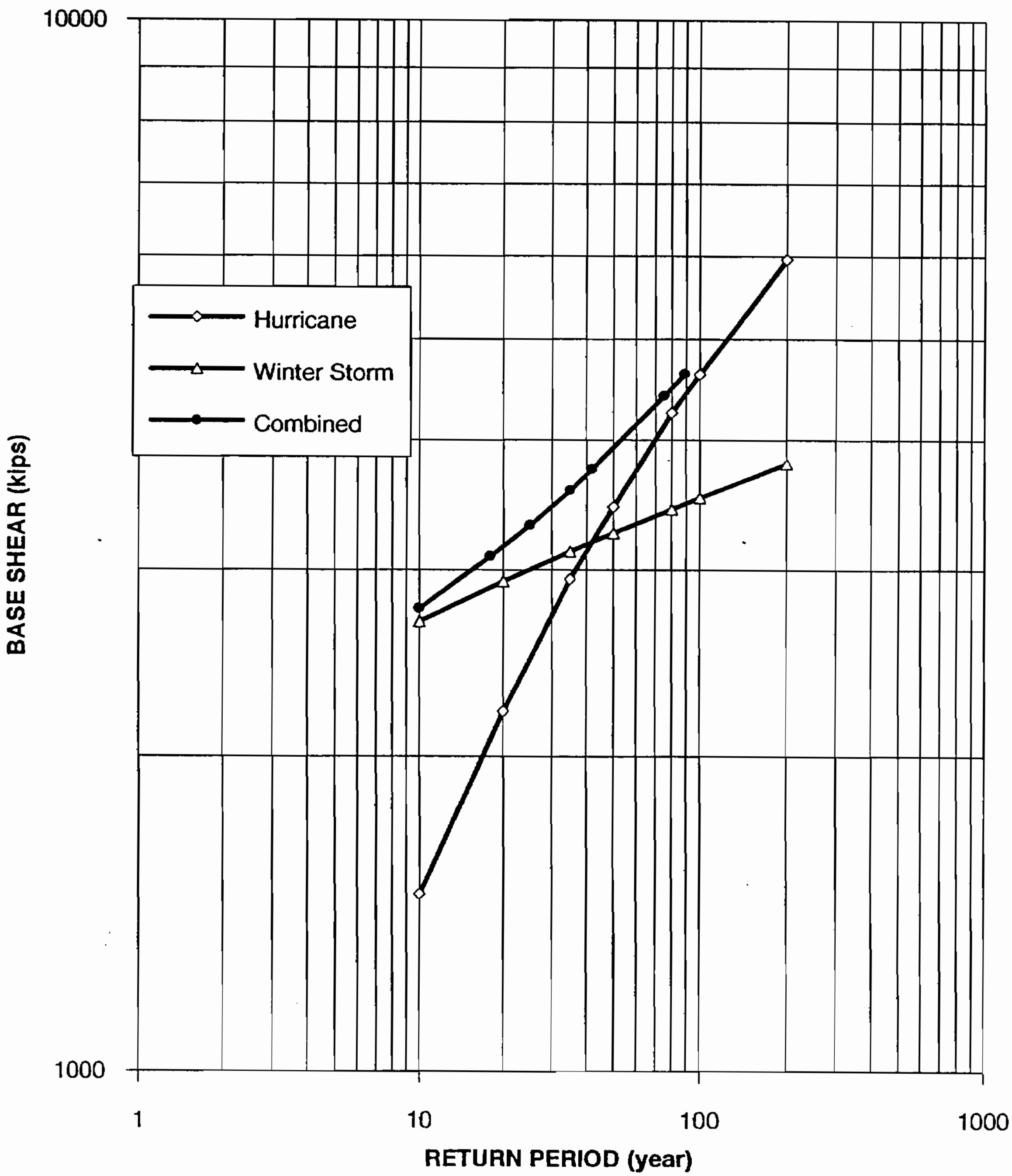


FIGURE 3.5.1.f
WAVE HEIGHT vs. RETURN PERIOD
(Ocean Weather Data, 1994 At NOHOCH B Site)

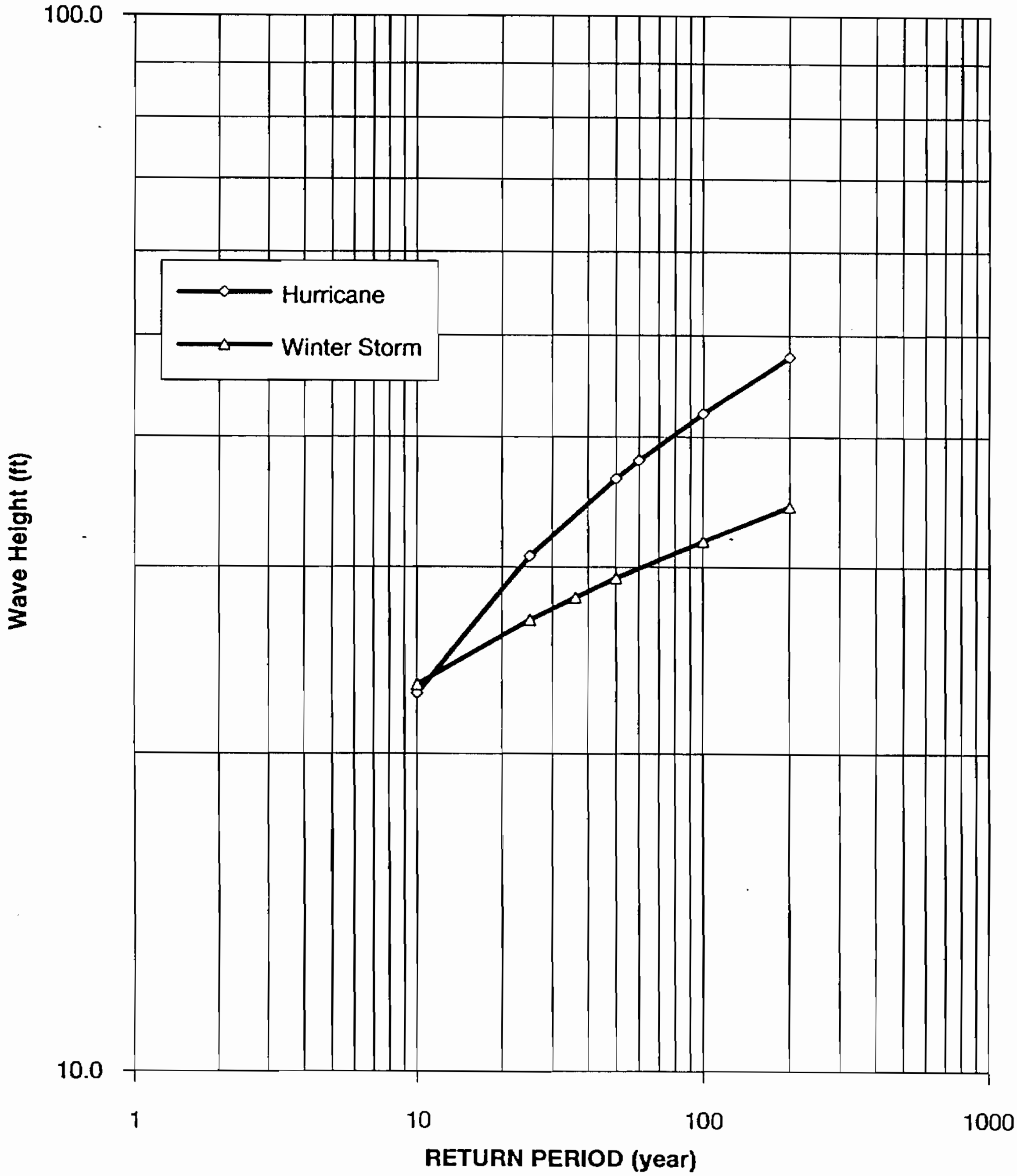
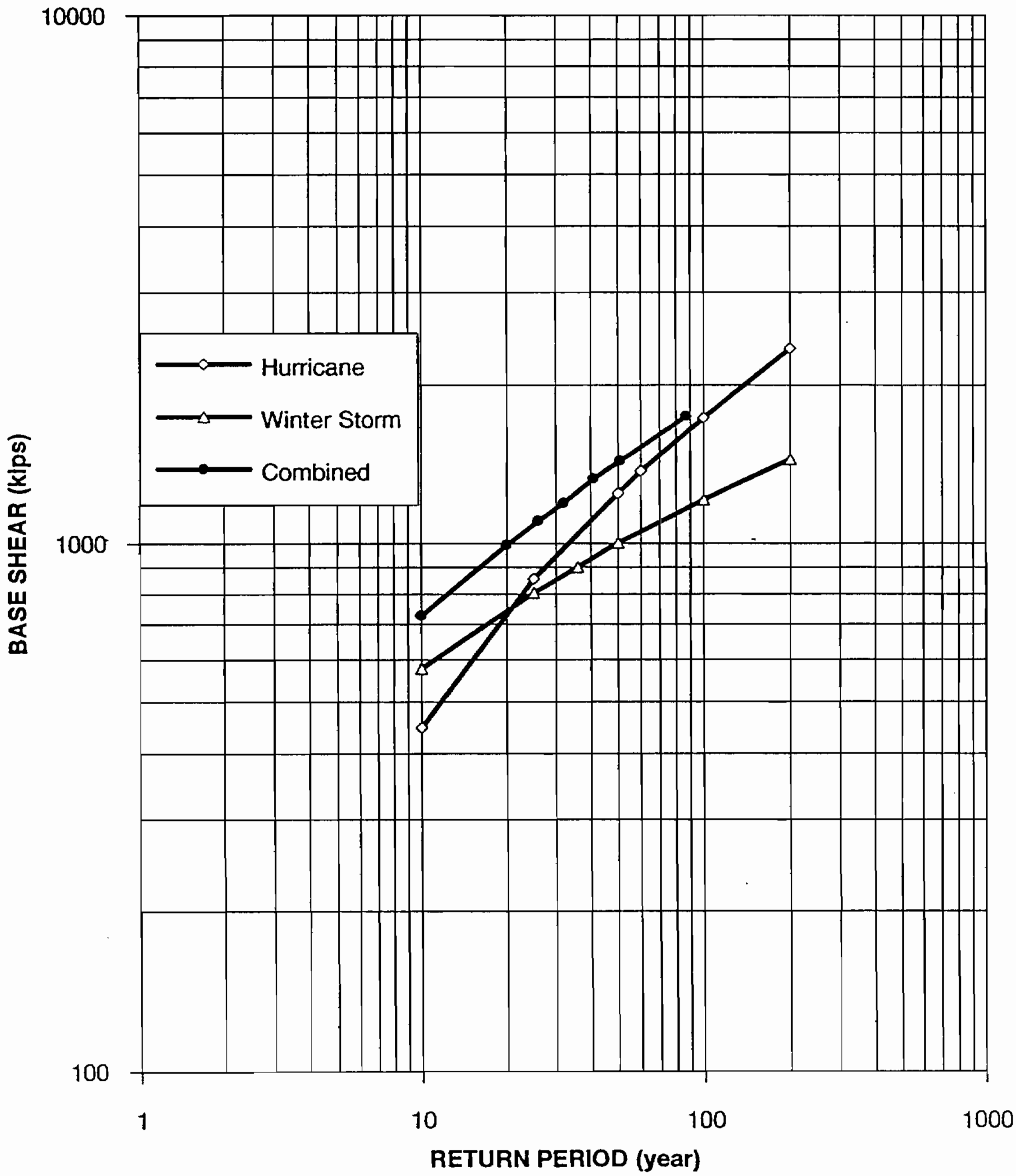
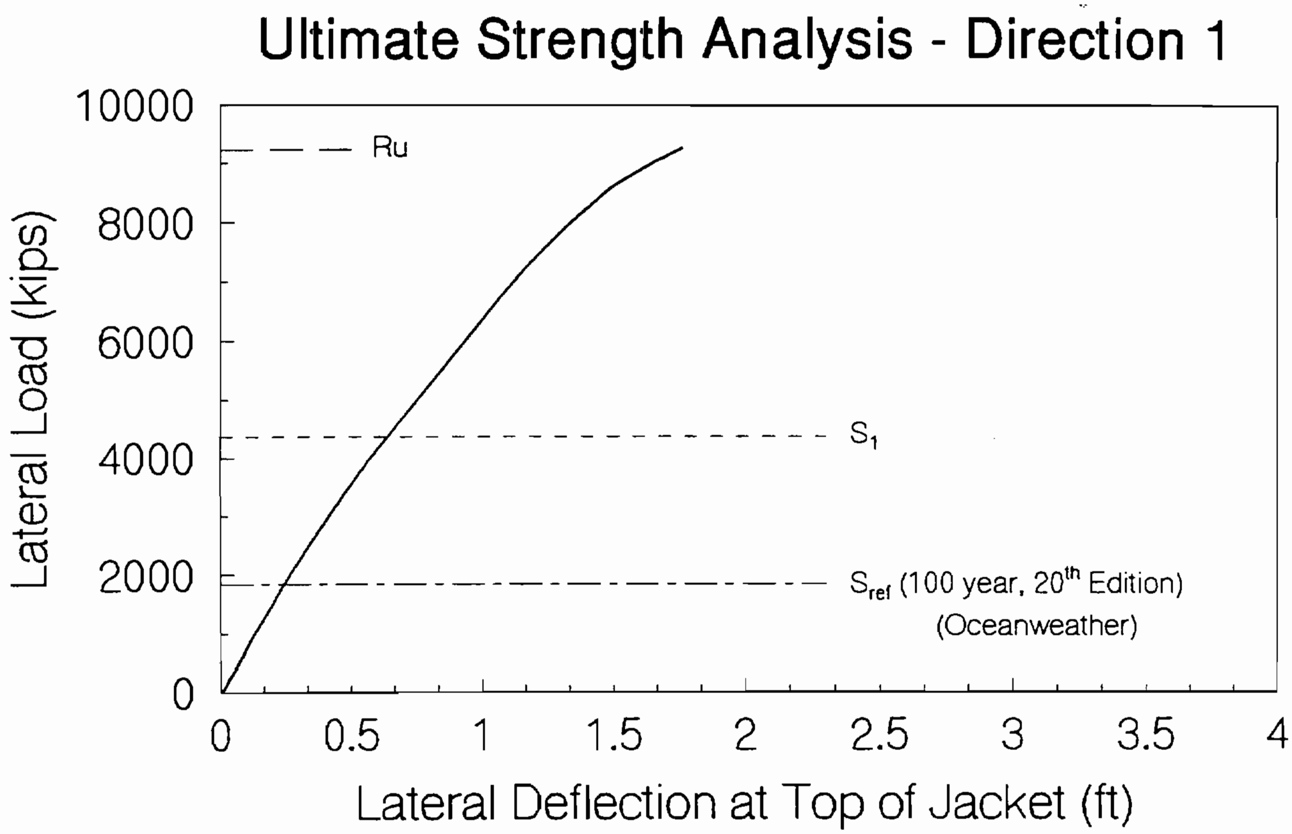


FIGURE 3.5.1.g
BASE SHEAR vs. RETURN PERIOD
(Ocean Weather Data, 1994 At NOHOCH B Site)





Load Level at which First Component Reaches I.R. of 1.0 (S_1)	4366 kips
Reference Level Load (S_{ref})	1843 kips
Ultimate Capacity - Flexible Joints (R_u)	9237 kips
Reserve Strength Ratio - Flexible Joints (RSR)	5.01

Platform Failure Mode: Yielding in Piles and in K Framing at Rows 1 and 2.

Figure 3.5.5a

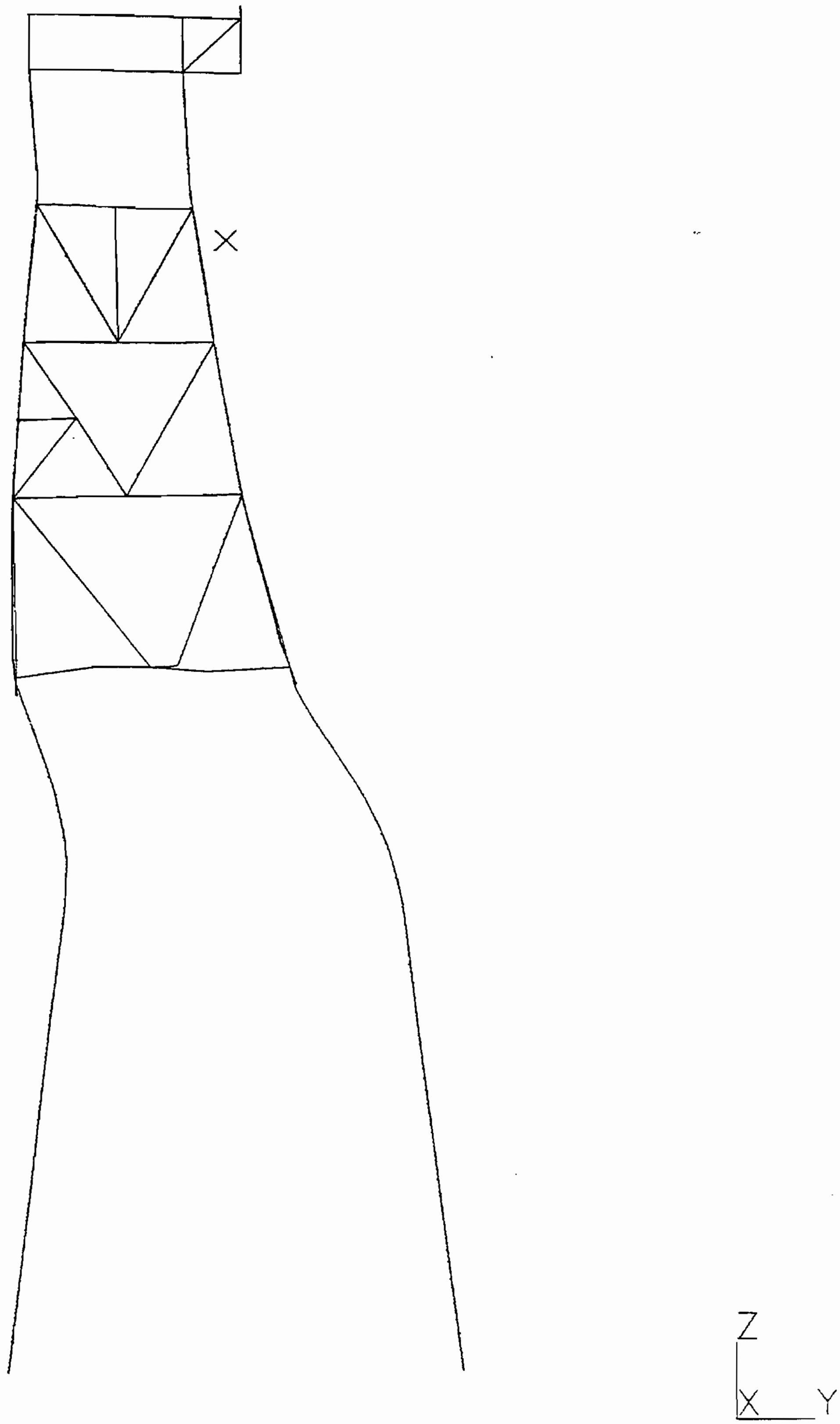
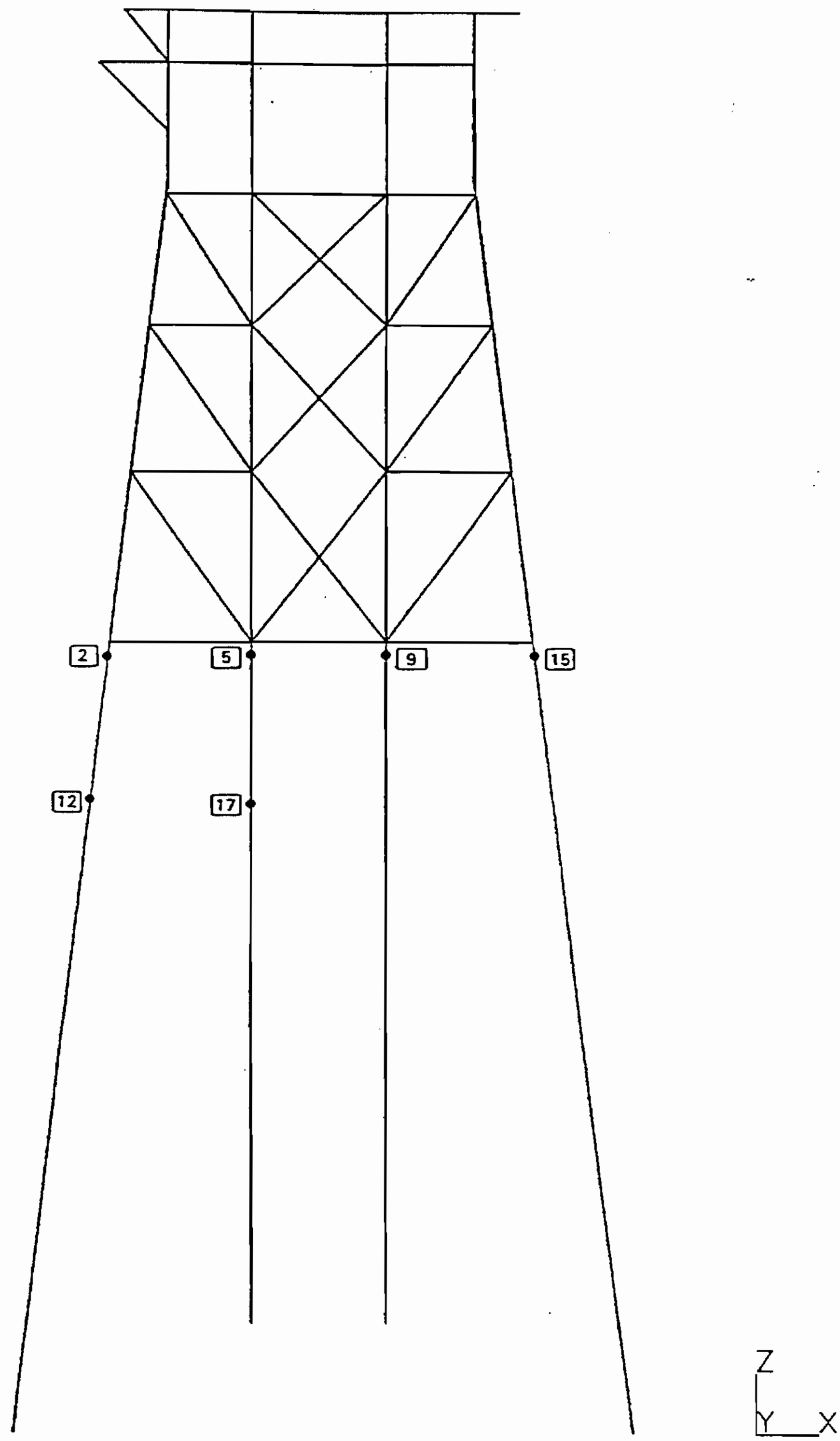


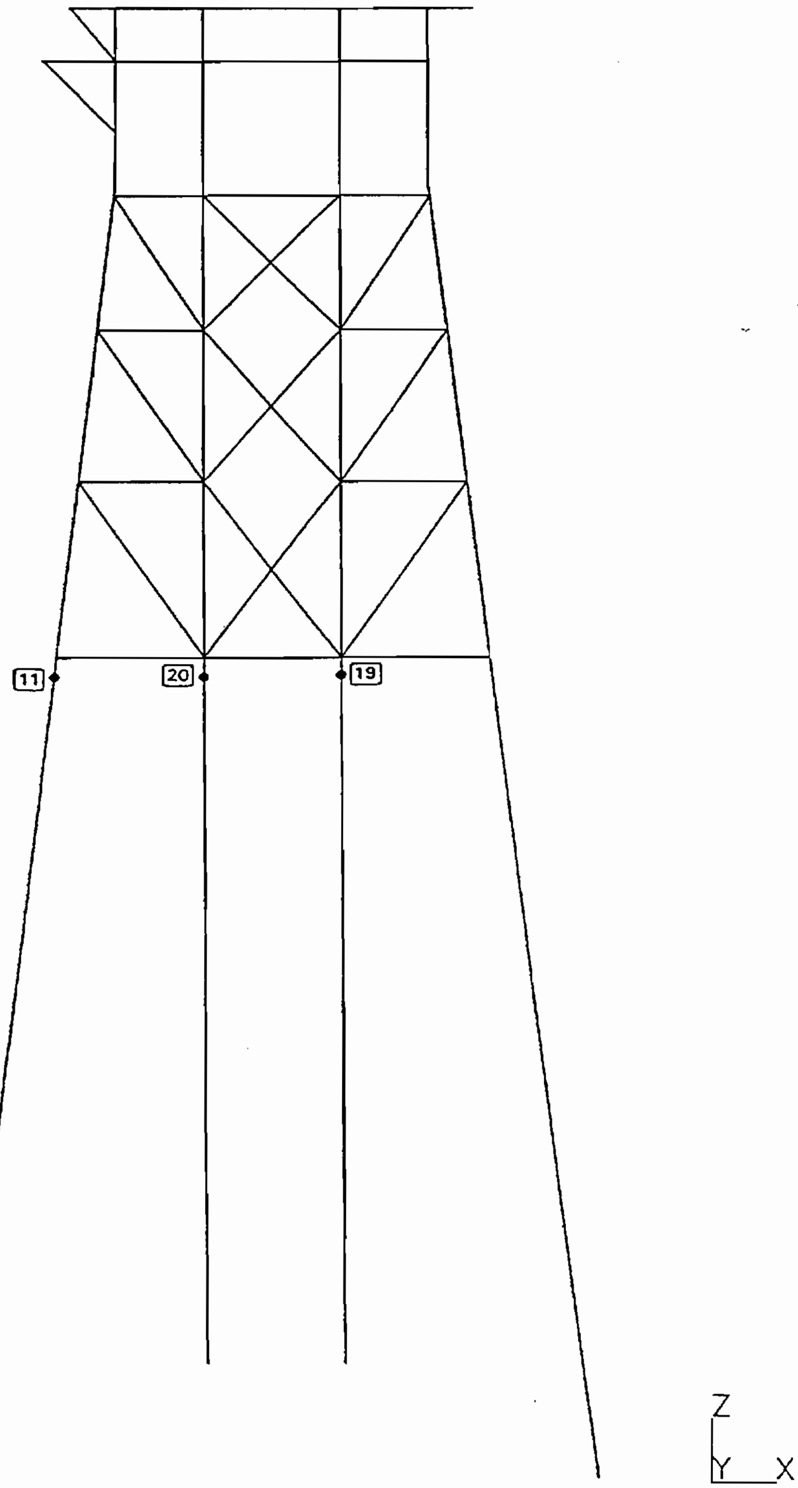
Figure 3.5.5b Deflected Shape for Direction 1 at maximum load



ROW A, DIRECTION 1

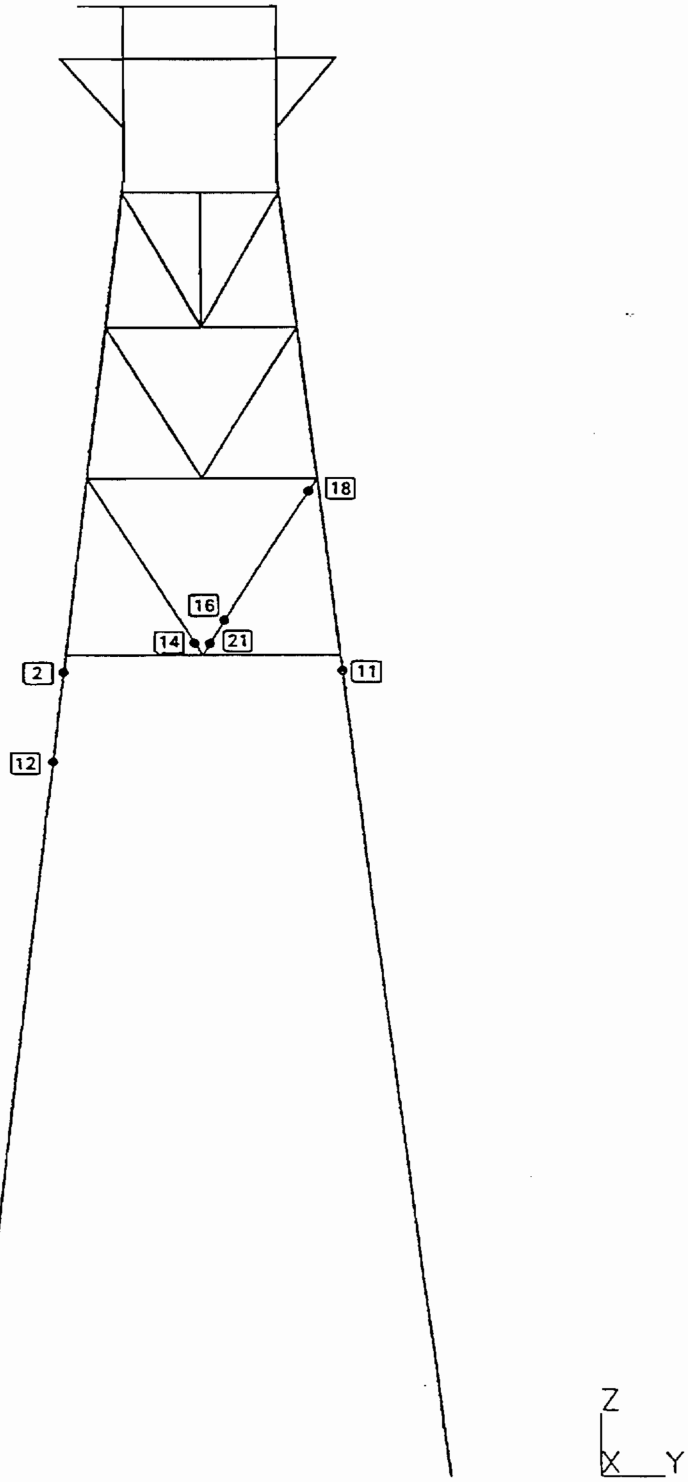
• Location of Plastic Hinge

Figure 3.5.5c1 Plastic hinge locations for Direction 1 at maximum load



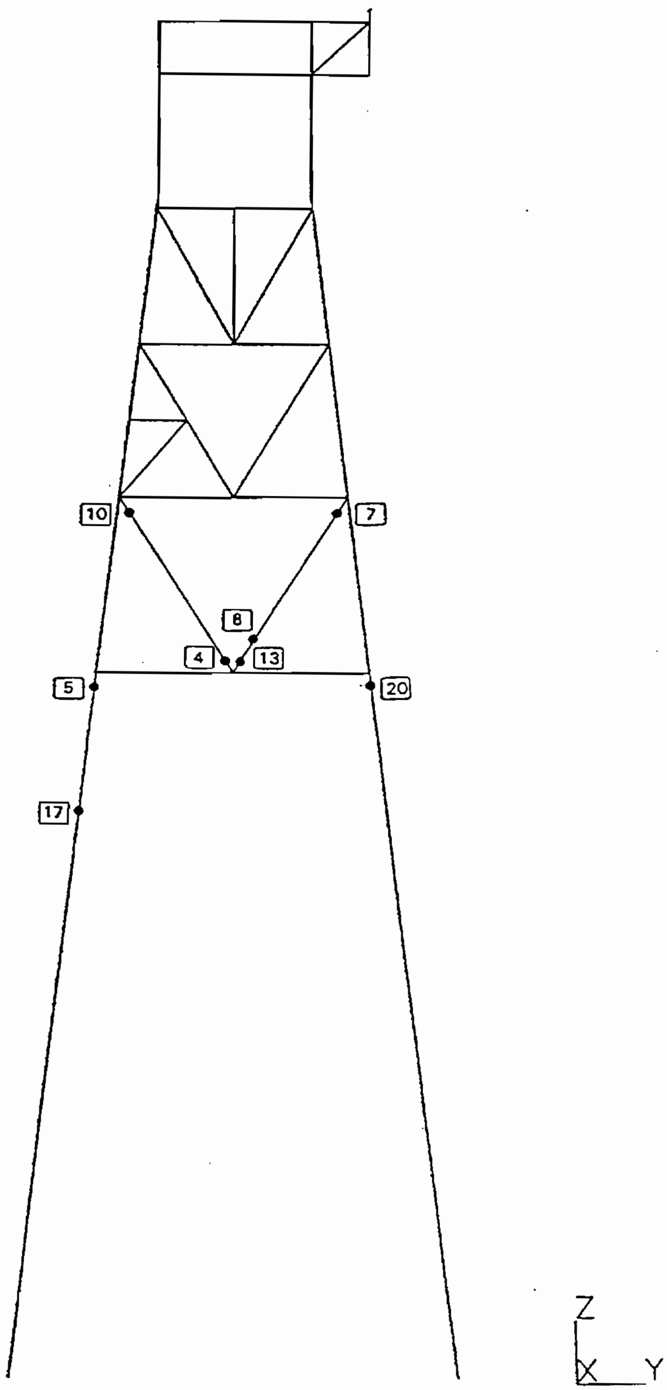
ROW B, DIRECTION 1
• Location of Plastic Hinge

Figure 3.5.5c2 Plastic hinge locations for Direction 1 at maximum load



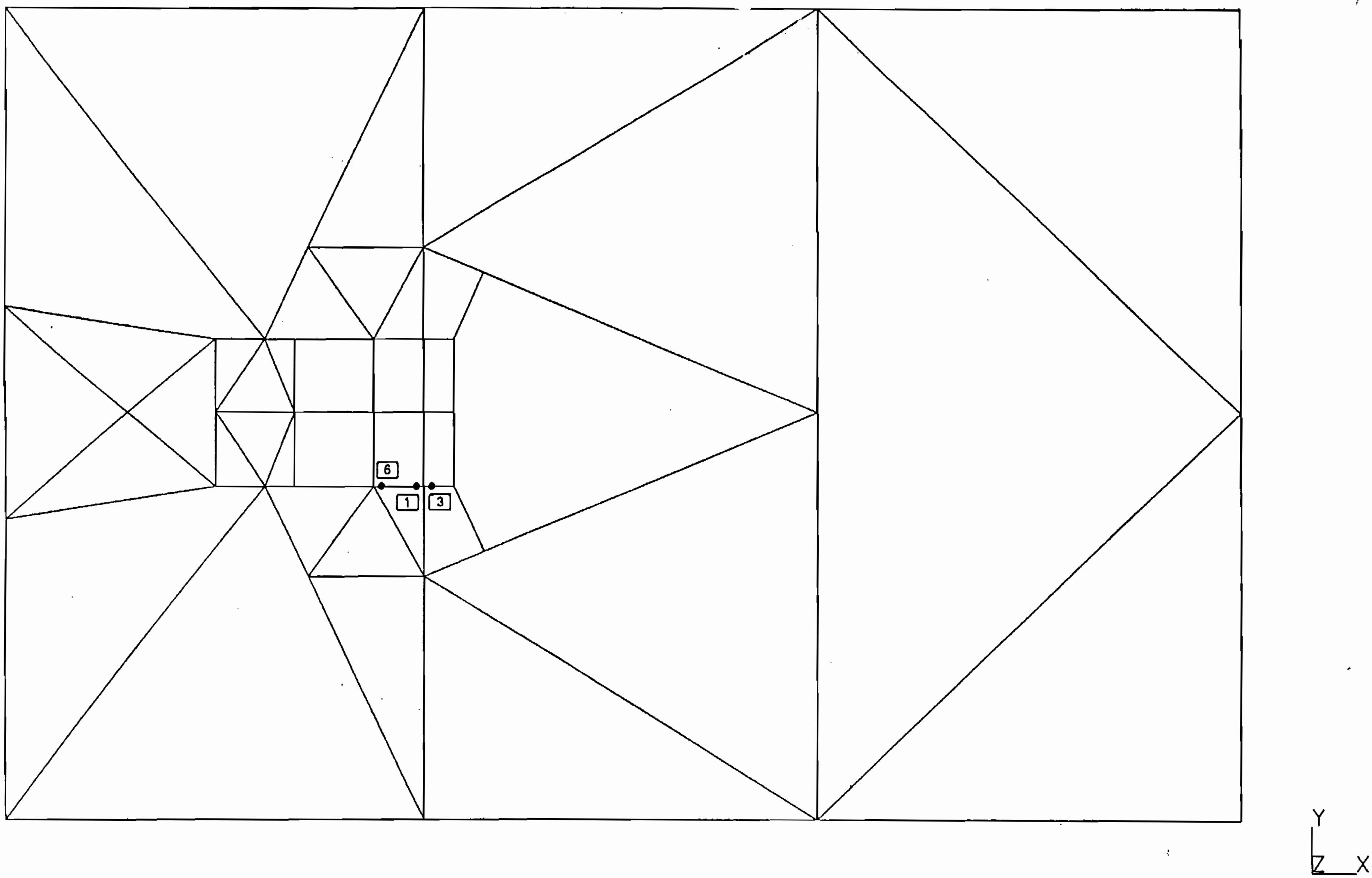
ROW 1, DIRECTION 1
• Location of Plastic Hinge

Figure 3.5.5c3 Plastic hinge locations for Direction 1 at maximum load



ROW 2, DIRECTION 1
• Location of Plastic Hinge

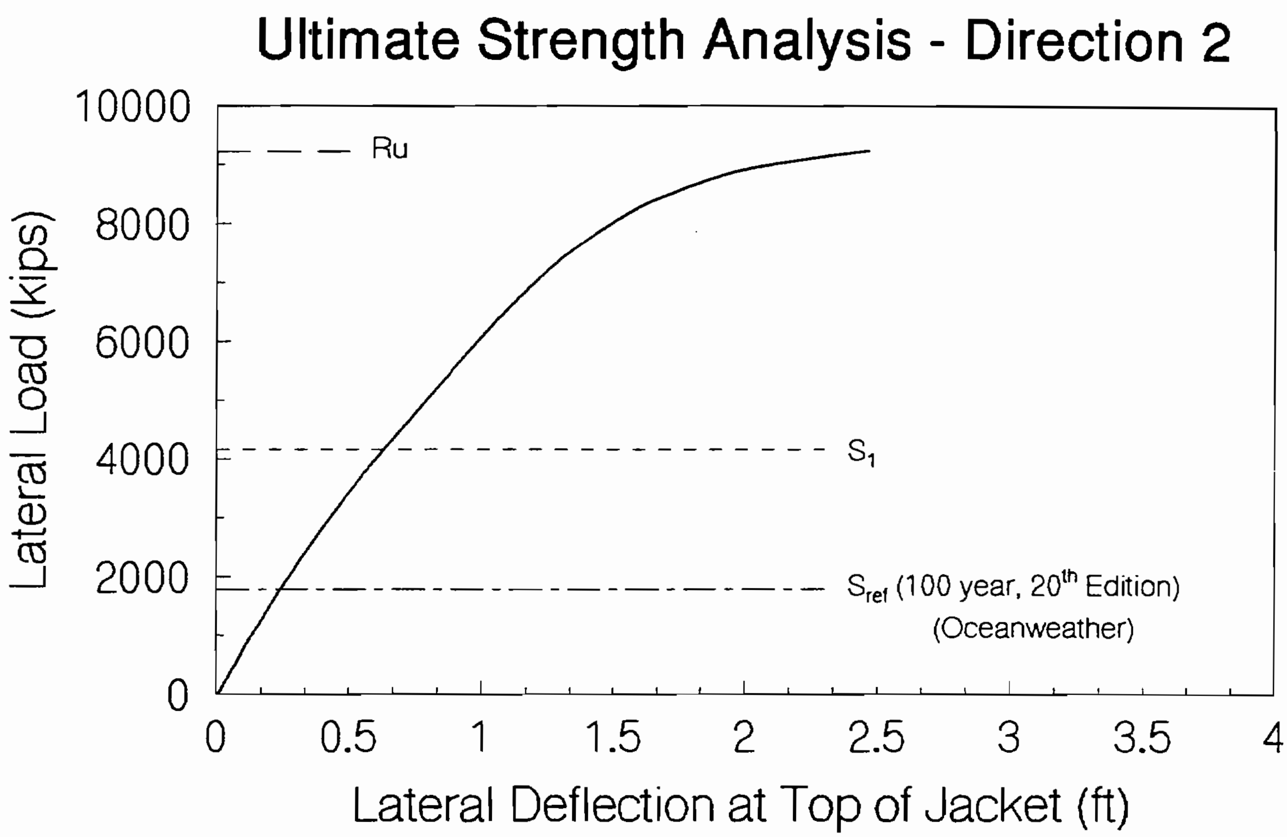
Figure 3.5.5c4 Plastic hinge locations for Direction 1 at maximum load



PLAN AT EL.-125'-0", DIRECTION 1

- Location of Plastic Hinge

Figure 3.5.5c5 Plastic hinge locations for Direction 1 at maximum load



Load Level at which First Component Reaches I.R. of 1.0 (S_1)	4157 kips
Reference Level Load (S_{ref})	1785 kips
Ultimate Capacity (R_u)	9225 kips
Reserve Strength Ratio (RSR)	5.17

Platform Failure Mode: Initial yielding in piles with load redistribution causing yielding and buckling in X framing at Row B.

Figure 3.5.5d

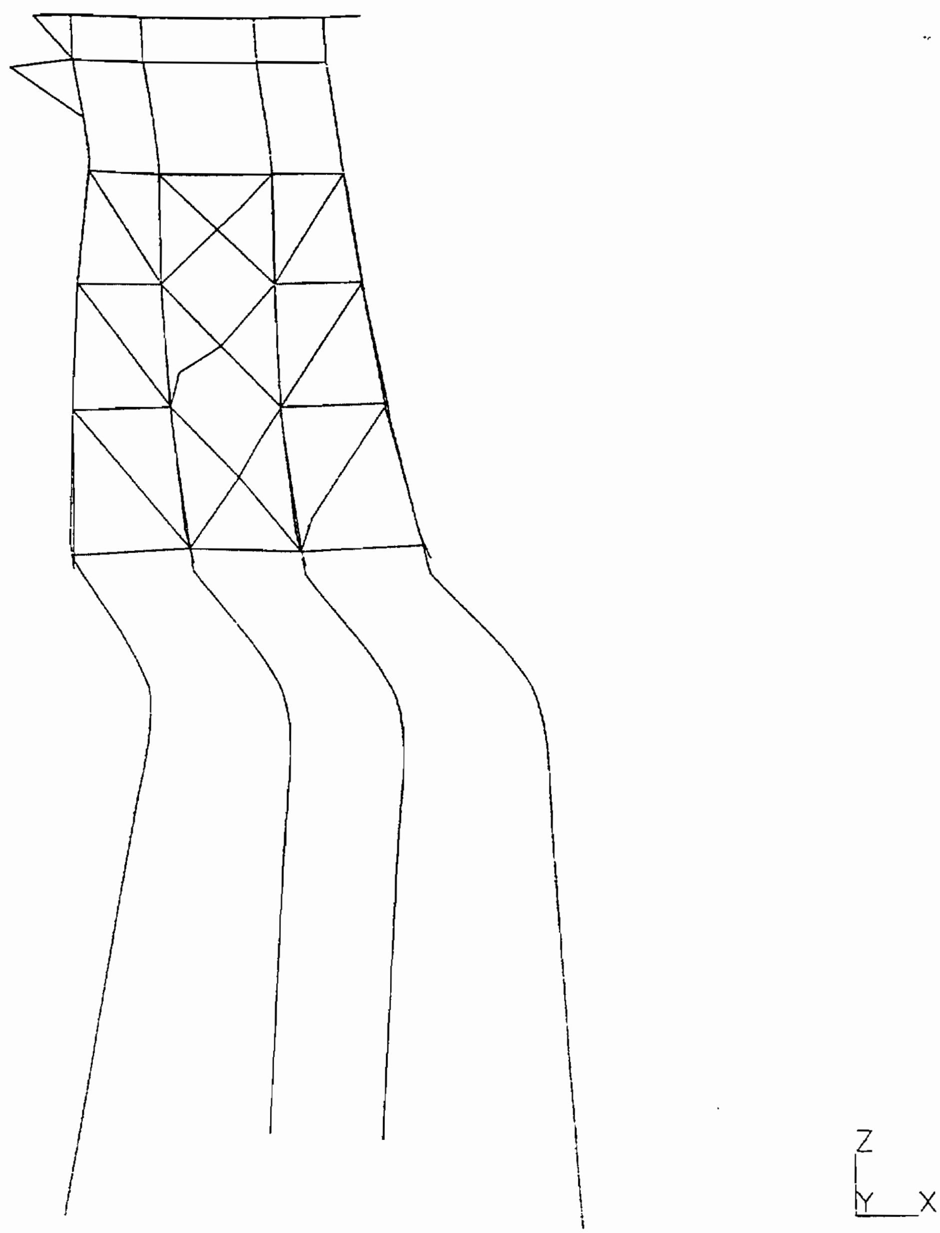
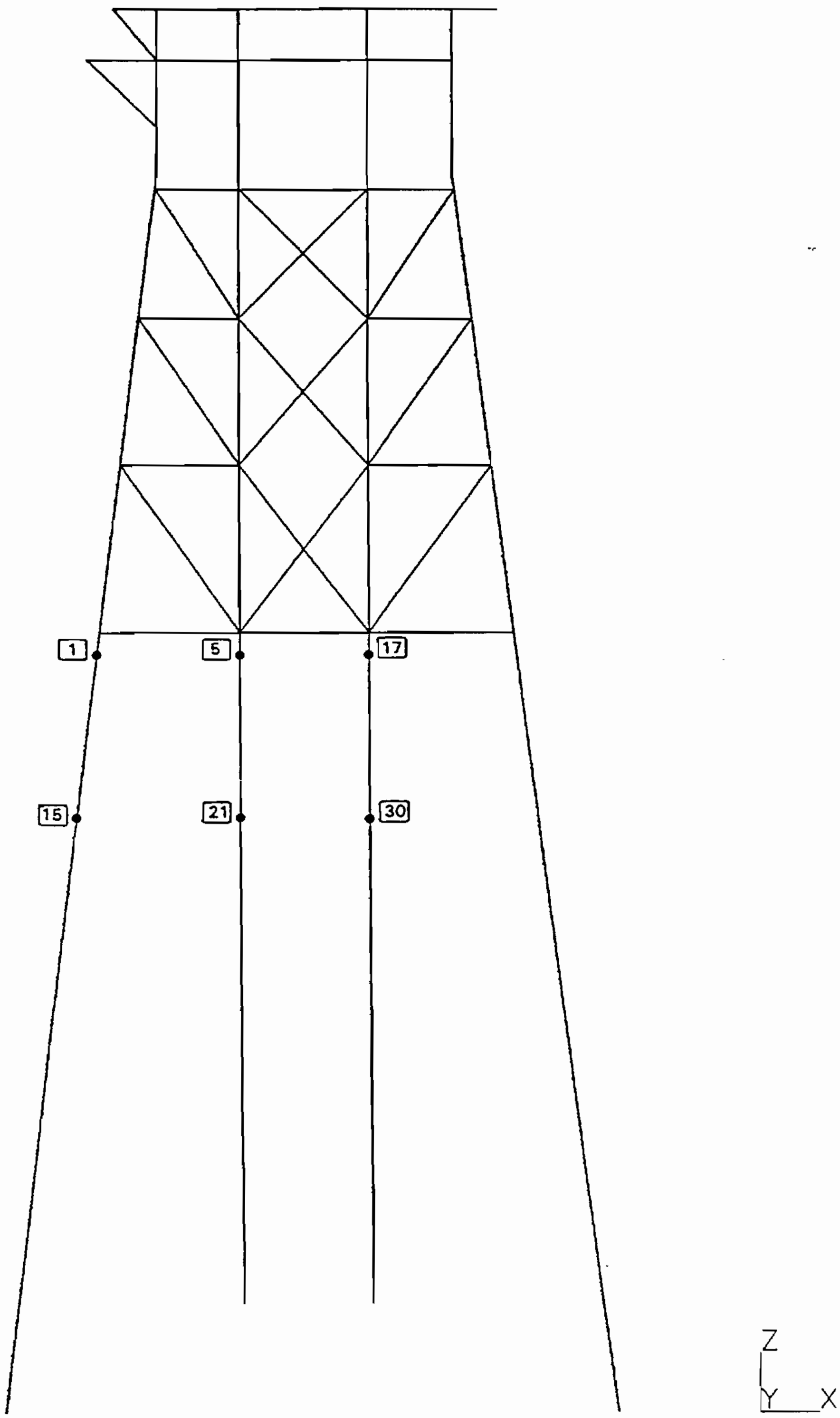
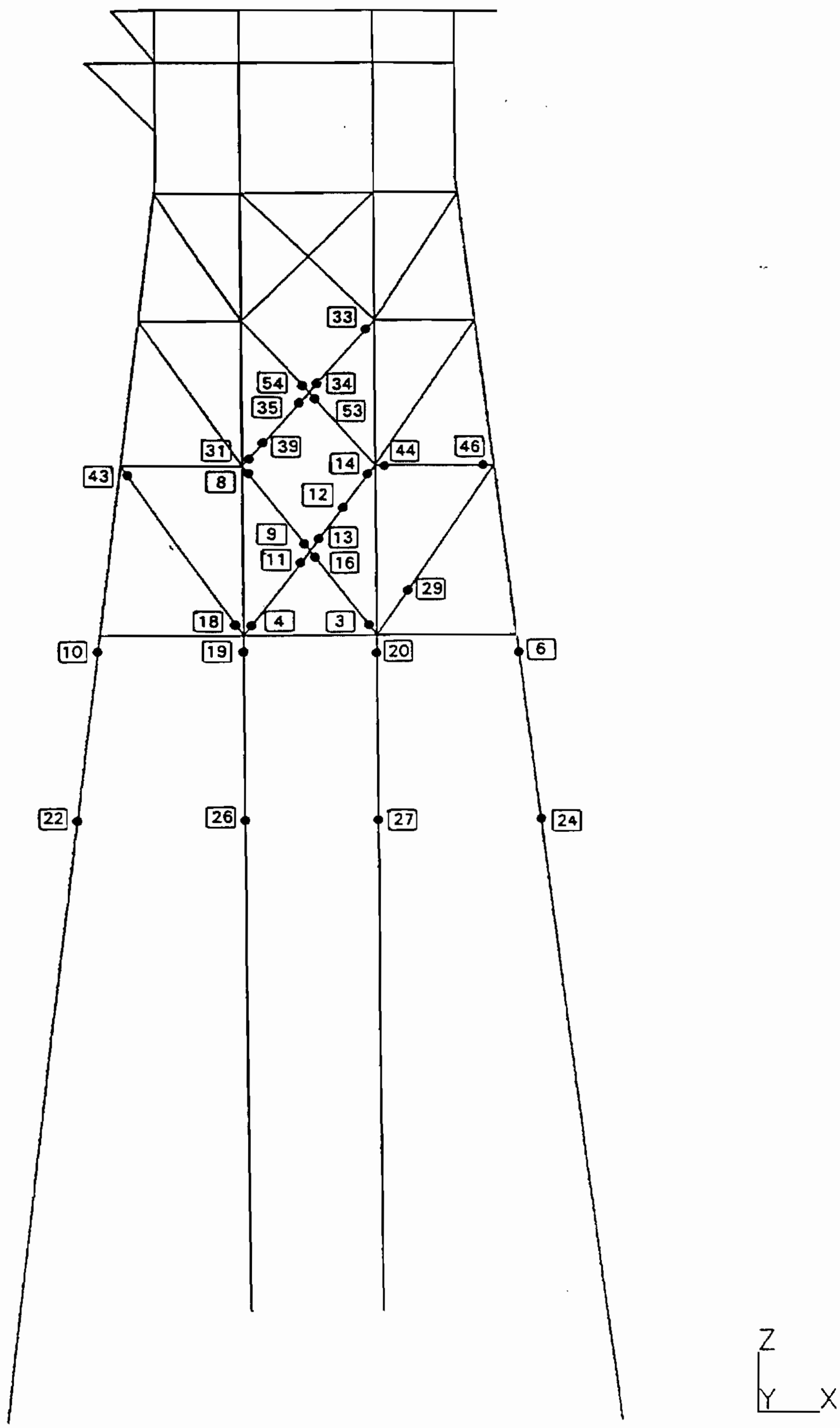


Figure 3.5.5e Deflected Shape for Direction 2 at maximum load



ROW A, DIRECTION 2
• Location of Plastic Hinge

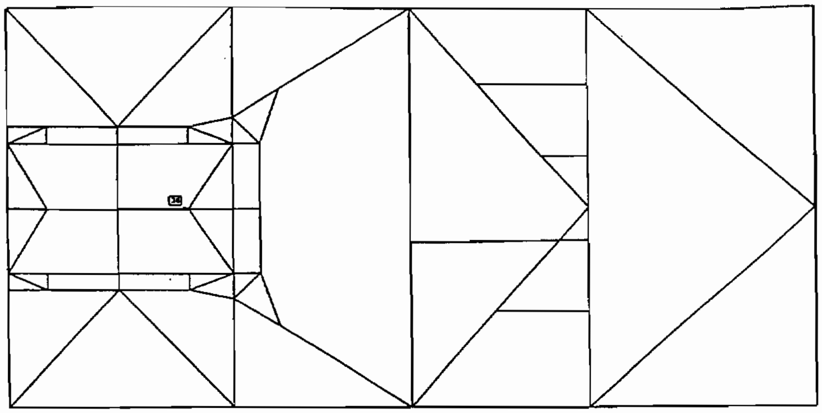
Figure 3.5.5f1 Plastic hinge locations for Direction 2 at maximum load



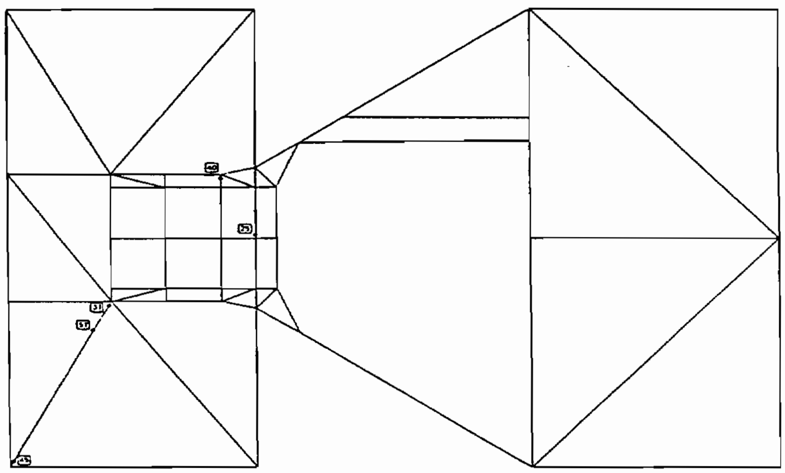
ROW B, DIRECTION 2

• Location of Plastic Hinge

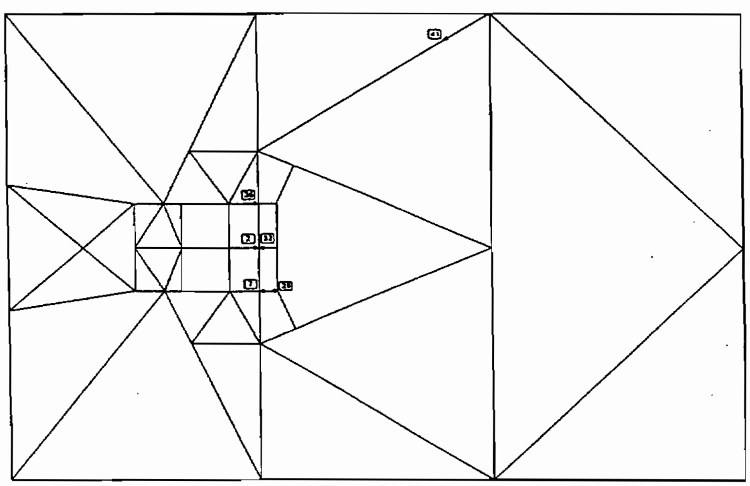
Figure 3.5.5f2 Plastic hinge locations for Direction 2 at maximum load



PLAN AT EL.+12'-0", DIRECTION 2
• Location of Plastic Hinge

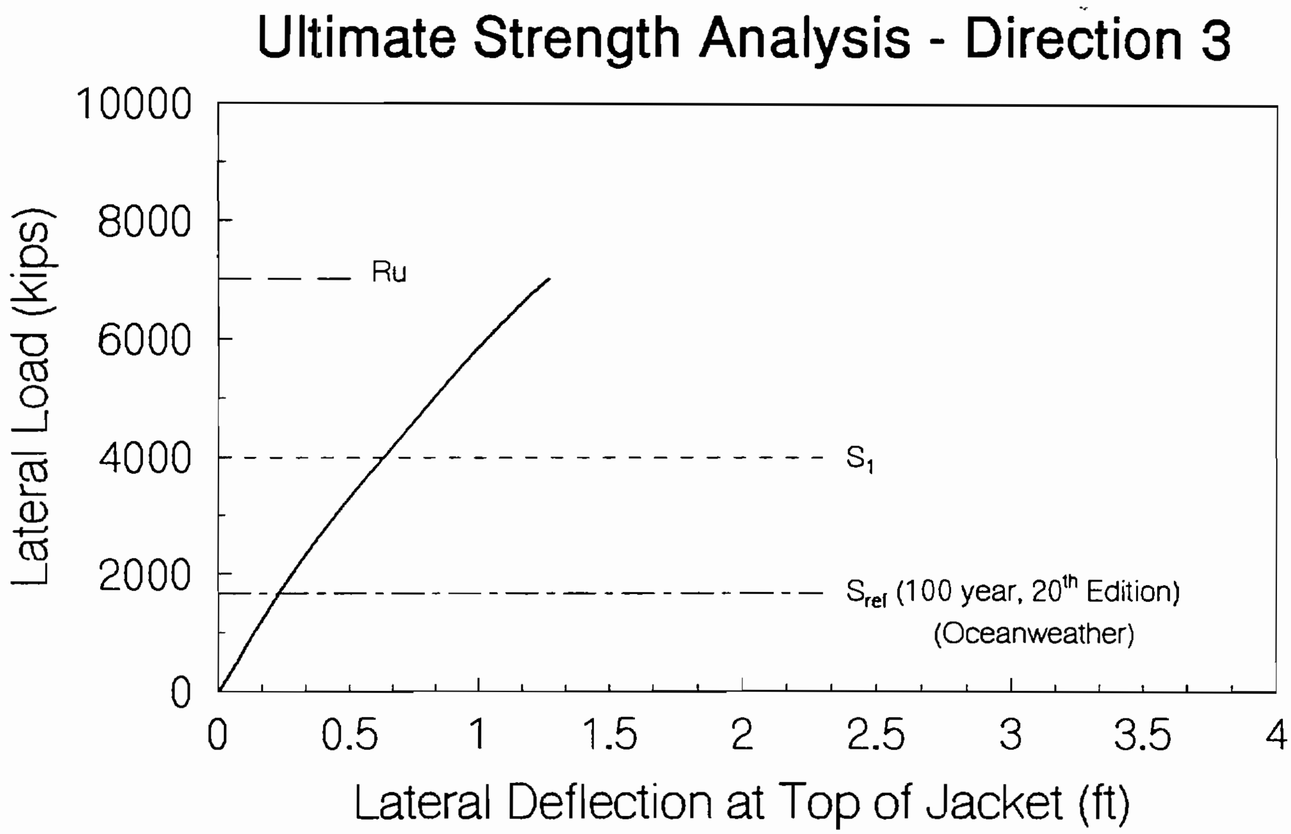


PLAN AT EL.-73'-0", DIRECTION 2
• Location of Plastic Hinge



PLAN AT EL.-125'-0", DIRECTION 2
• Location of Plastic Hinge

Figure 3.5.5f3 Plastic hinge locations for Direction 2 at maximum load



Load Level at which First Component Reaches I.R. of 1.0 (S_1)	3985 kips
Reference Level Load (S_{ref})	1669 kips
Ultimate Capacity (R_u)	7007 kips
Reserve Strength Ratio (RSR)	4.20

Platform Failure Mode: General yielding in Lower Bay X Framing at Rows A and B and Pile/Soil T-Z highly mobilised.

Figure 3.5.5g

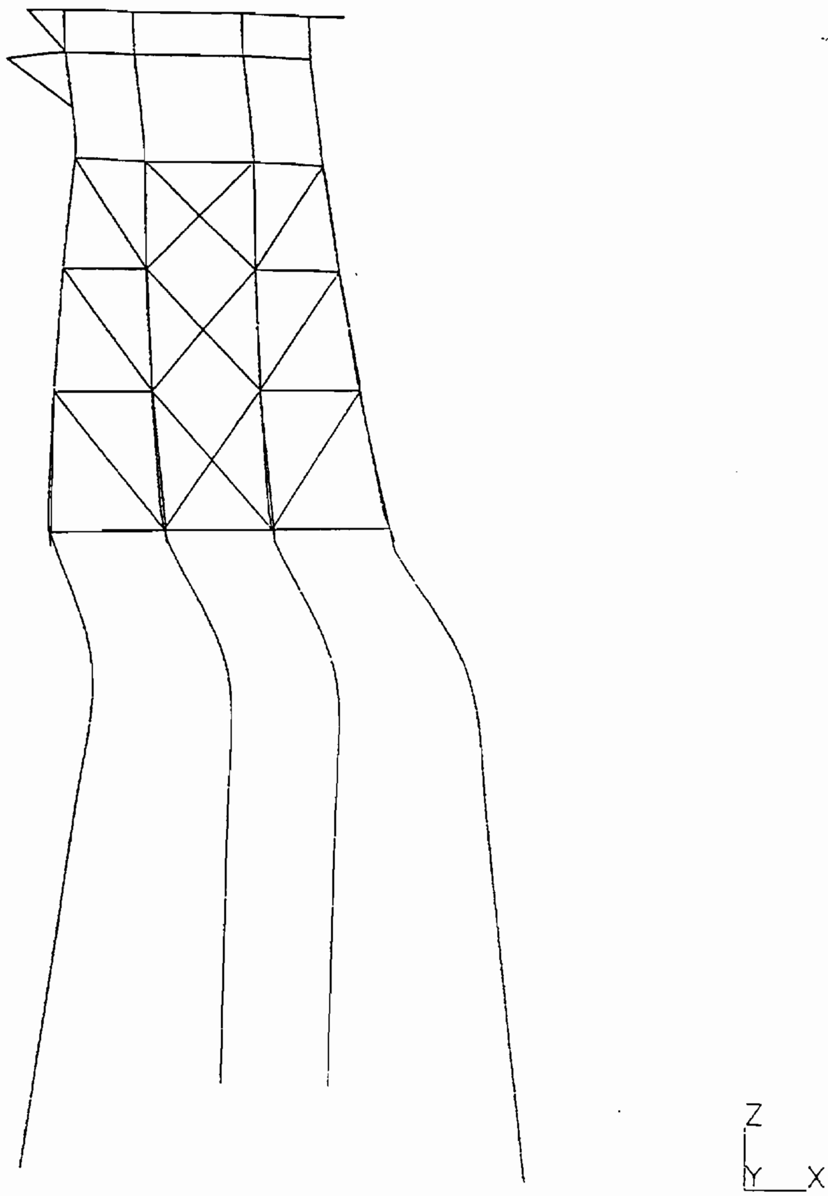
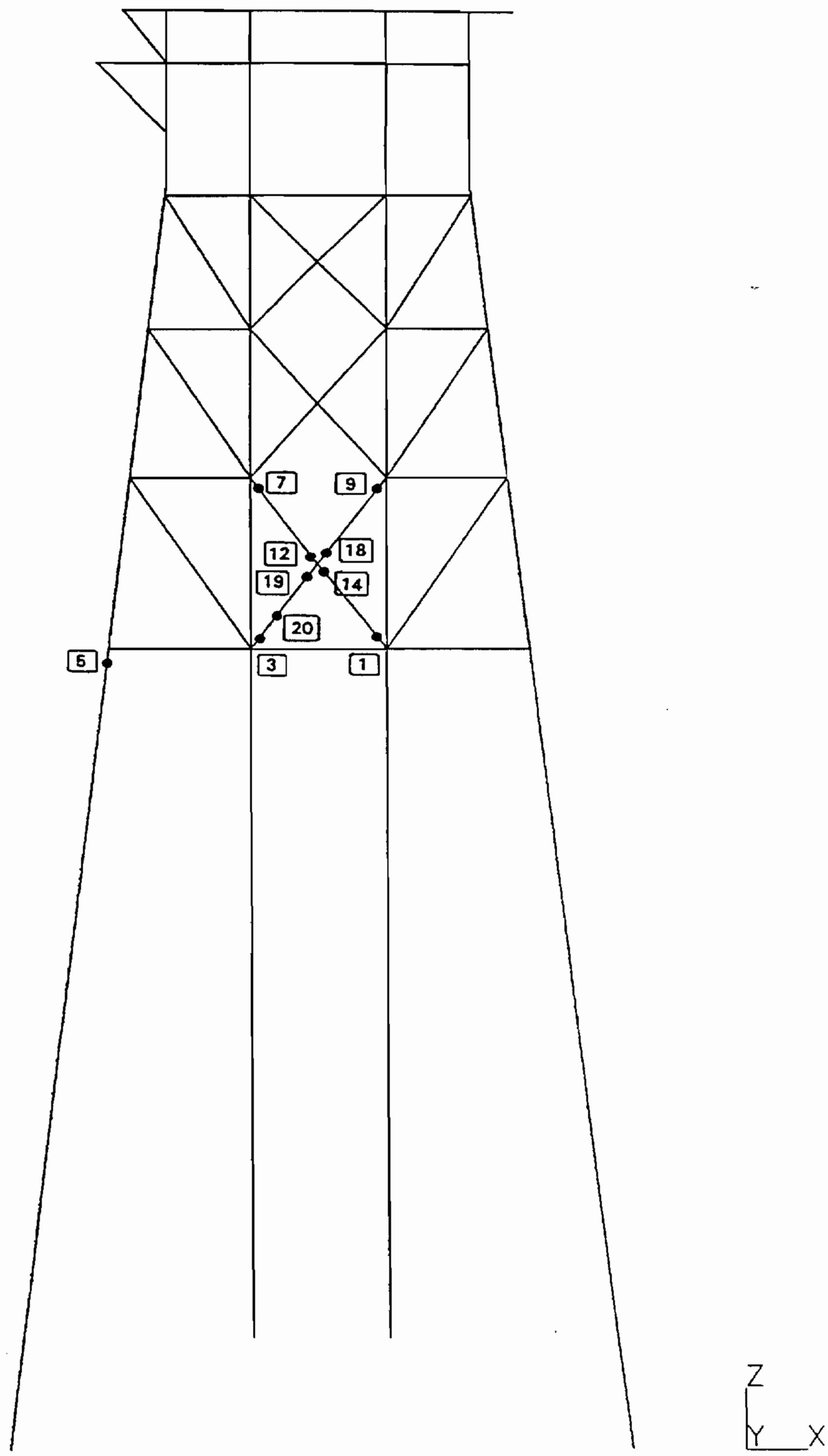
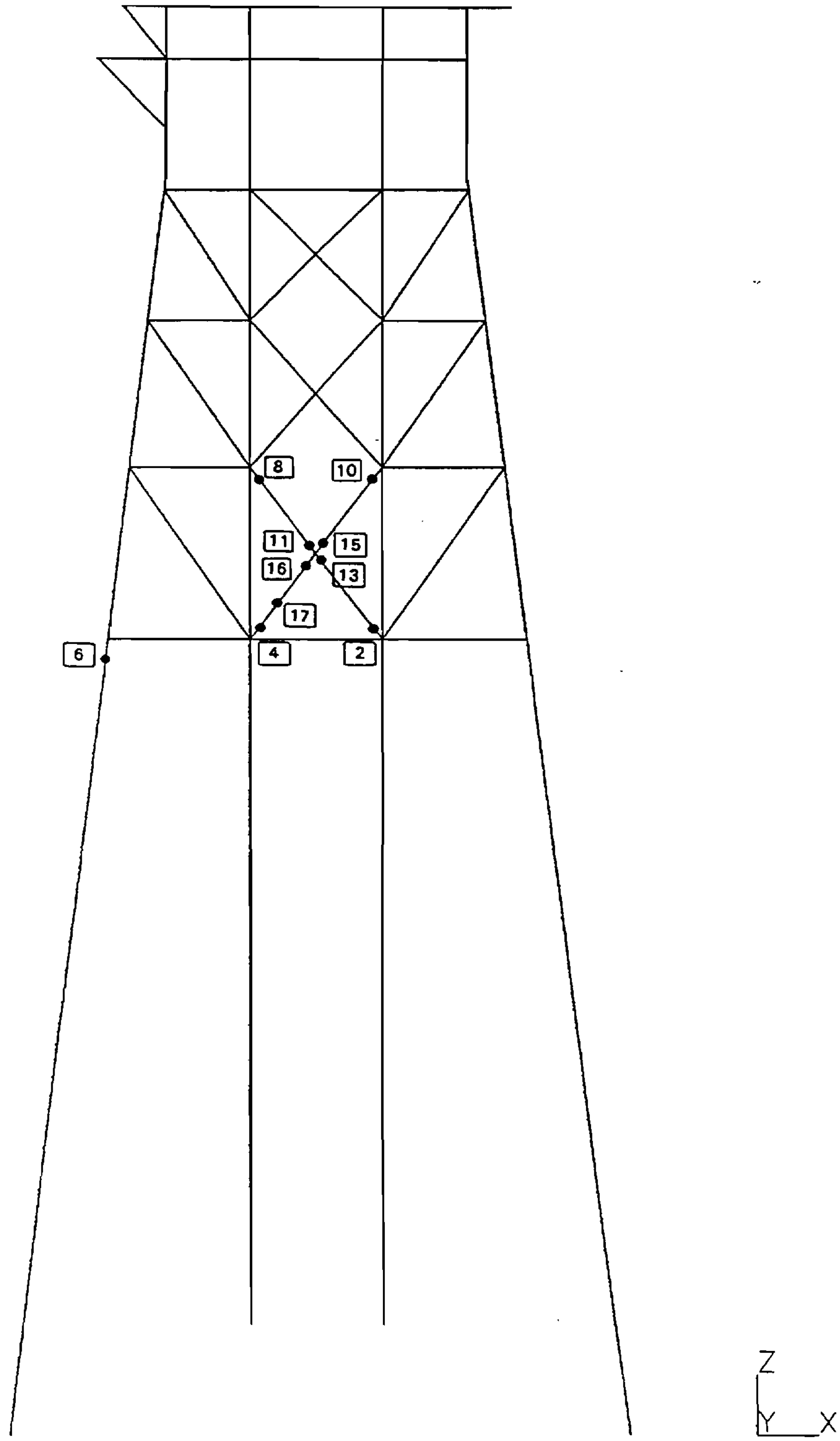


Figure 3.5.5h Deflected Shape for Direction 3 at maximum load



ROW A, DIRECTION 3
• Location of Plastic Hinge

Figure 3.5.5i1 Plastic hinge locations for Direction 3 at maximum load



ROW B, DIRECTION 3
• Location of Plastic Hinge

Figure 3.5.5i2 Plastic hinge locations for Direction 3 at maximum load

Appendix A: Condition assessment - detailed information

A.1 General Information

a. The original and current owner of the platform Nohoch-B is

b. This platform was originally equipped with a drilling module for well development. This equipment is not in use at the moment and the platform mainly serves as a wellhead platform.

c. Nohoch-B is located in a 38.1 m (125') water depth in Campeche Bay in the Gulf of Mexico. Its geographic location is:

Coordinates U.T.M.	X = 604,568.00 m.
	Y = 2'138,970.00 m.
Longitude	W 92° 00' 16"
Latitude	N 19° 20' 35.4"

d. Nohoch-B is a fixed jacket template platform, built with eight steel legs of diameter 52 1/2" and thickness of 0.625" to 1.250". They are braced with four horizontal frames. The piles installed in each leg are 48" diameter and penetrate to 310' below the mudline. The piles are ungrouted.

e. Nohoch-B has 12 wells (8 in service). Also, there are three risers with diameter of 8", 16" and 24", respectively.

f. The platform is manned but evacuated in before extreme environmental events.

g. No adverse performance during past environmental events is known to have been reported.

A.2 Original Design

a. The designer of the jacket was Proyectos Marinos S.C. (PMSC) and the topside structure was designed by Diseno Integral de Ingenieria S.A. DE C.V. (DIISA).

b. The jacket was built with 4 types of steel: ASTM A-36, ASTM A-572 GR-50, API 5L GR B and API 5LX GR X42. The deck was built of steel ASTM A-36 and the piles were built of steel ASTM A-36 and ASTM A-537.

c. The design codes were: API RP 2A 9th Edition, AISC, ANSI and AWS.

d. Material traceability records have not been identified for this study.

e. The piles and conductor driving records were:

Pile / Conductor	1 Section		2 Section		3 Section		4 Section	
Length (feet)	235	205	105	100	60	60		
	215		60		40		30	
Weight (Ton)	95	84	46	43	21	21		
	33		10		6		5	
Thickness (inch)	1.25-2.0	1.25-2.0	2.0-1.50	2.0-1.50	1.5-1.25	1.5-1.25		
	1		1		1		1	
Steel	A-36	A-537	A-537	A-36	A-36	A-36		
	A-36		A-36		A-36		A-36	

Leg	A1	A2	A3	A4	B1	B2	B3	B4
Design penetration	243	209	239	243	243	209	209	243
Penetration	243	209	238	243	243	209	210	243
# of blows	5303	2818	4966	4612	3745	2573	3229	4340

The hammers used for installing piles were M-3000 and M-1800.

Conductor	1	2	3	4	5	6	7	8	9	10	11	12
Design penetration	200	200	200	200	200	200	200		200	200	200	200
Penetration	158	157	158	159	158	158	157		158	158	157	158
# of blows	1369	1408	1460	1332	1366	1338	1350		1395	1486	1421	1483

The hammers used for installing the conductors were M-750 and M-1800.

f. Leg/pile annulus ungrouted.

4. Platform survey history

a. No extreme events are known to have been experienced.

b. Inspection surveys described below indicate damage due to vessel impact.

Coordinates U.T.M.	X =	604,713.00 m.
	Y =	2'138,817.00 m.
Longitude	W	92° 00' 16"
Latitude	N	19° 20' 35.4"

Boring Logs for 450 ft penetration and shear strength profile are shown in Figures 3.3a, b and c.

h. The number, diameter and design penetration of piles and conductors are the following:

PILE	DIAMETER	THICKNESS	PENETRATION
8	48"	1.25" to 2"	70m
CONDUCTOR	DIAMETER	THICKNESS	PENETRATION
11	30"	1"	48m

i. Appurtenances-list and location.

Appurtenances	Location (leg)
Boat Landing	Between legs B1-B2 Between legs A3-A4
Barge Bumpers	A1, A2, A3, A4 B1, B2, B3, B4
Riser Guard	A1, B1, B2

3. Construction

a. The Contractor for the jacket and the topsides fabrication and installation was Brown & Root Inc. and the installation took place in February 1979.

b. Design drawings available are:

Deck:	40 drawings
Jacket:	25 drawings
Pile:	2 drawings
Appurtenances:	10 drawings
Conductor:	3 drawings

c. The specifications were: AWS, API RP2A (9th Edition), AISC and ANSI.

d. Material traceability records have not been identified for this study.

e. The piles and conductor driving records were:

Pile / Conductor	1 Section		2 Section		3 Section		4 Section	
Length	235	205	105	100	60	60		
(feet)	215		60		40		30	
Weight	95	84	46	43	21	21		
(Ton)	33		10		6		5	
Thickness	1.25-2.0	1.25-2.0	2.0-1.50	2.0-1.50	1.5-1.25	1.5-1.25		
(inch)	1		1		1		1	
Steel	A-36	A-537	A-537	A-36	A-36	A-36		
	A-36		A-36		A-36		A-36	

Leg	A1	A2	A3	A4	B1	B2	B3	B4
Design penetration	243	209	239	243	243	209	209	243
Penetration	243	209	238	243	243	209	210	243
# of blows	5303	2818	4966	4612	3745	2573	3229	4340

The hammers used for installing piles were M-3000 and M-1800.

Conductor	1	2	3	4	5	6	7	8	9	10	11	12
Design penetration	200	200	200	200	200	200	200		200	200	200	200
Penetration	158	157	158	159	158	158	157		158	158	157	158
# of blows	1369	1408	1460	1332	1366	1338	1350		1395	1486	1421	1483

The hammers used for installing the conductors were M-750 and M-1800.

f. Leg/pile annulus ungrouted.

4. Platform survey history

a. No extreme events are known to have been experienced.

b. Inspection surveys described below indicate damage due to vessel impact.

c. The following Annual Inspection Reports have been reviewed and evaluated, and the most important findings were:

1983 Program Report No. NHBF2 610-1-U. No information available.

1985 Program Report No. NHBF2 610-2-U. Crack of 92cm length in A-1 leg and a crack of 15cm length in welding of joint 120 were found.

1988 Program Report No. NHBF2 610-3-P/S. Cracks of 33, 34 and 90cm length in A-1 leg were reported, and the crack of 15cm length in joint 120 was confirmed. Also, dented members 105-210, 115-120 and 117-120 were found (see Figure 3.3d).

1990 Program Report No. NHBF2 610-4-U. Cracks of 33, 34 and 90cm length in A-1 leg were confirmed. The crack of 15cm length in joint 120 was repaired by grinding, but a 43cm length was found in joint 119. Dented members reported were 104-119, 115-120, 117-120 and 215-210.

1991 Program Report No. NHBF2 610-5-U. Additional damage than reported in surveying done in 1990 were found. Bent members 104-119, 115-120 and 117-120 with deflections of 14, 7 and 29cm, respectively, were reported. Also, considerable marine growth was reported.

1993 Program Report No. NHBF2 610-6-U. The 43cm length crack located in the joint 119 was repaired using wet welding, but the welding procedure did not satisfy AWS requirements. All damages reported in the previous surveying have no changes, except the marine growth, which was reported with lesser thickness.

d. Due to the damage inspection report (610-3-U) of October 1989, this platform was analyzed and the members 118-214, 105-203, 214-221, 221-314, 203-216, 216-302 were found to have a high utilization according with API RP 2A 17th edition guidelines.

In July 1993 the platform was analyzed again due to the increased damage (report 610-5-U). The element 104-119 and the joints 206, 410, 221, 226 and 208 have a high utilization according to the API RP 2A 19th Edition code of practice.

Appendix B: Mitigation options

Mitigation action are defined as modifications or operational procedures that reduces loads, increase capacities or reduce consequences. These actions can generally be categorized as: 1) Reduction of loading, 2) Increasing of strength and 3) Reduction of consequences.

B.1 Reduction of loading

a. Reduction of vertical gravity loading: A prime example that can be accounted for in the assessment of existing platforms is to accurately determine actual deck weights. Once installed, the actual loading may be substantially less than what was used in the original design. Other related jacket vertical load reductions can come from a partial removal of unneeded deck structures. Conversely, jacket vertical loading can also be reduced by increasing platform buoyancy, and other is the control by evacuation of personnel before extreme pacific storms, or by oil inventory limitations.

b. Wave in deck structures: Lateral loads on platforms increase substantially if the wave crest rise into the deck structure. One mitigation alternative used is to remove most or all of the equipment from the lowest deck elevation. Another alternative is to use deck grating instead of plating which will allow for the encroaching water and trapped air to dissipate through the deck more easily. Unnecessary sub decks and other non-essential structures can be removed.

c. Reduction of hydrodynamic forces on platforms: The simplest method is to reduce the non-essential hydrodynamic loading components as barge bumpers, boat landings, risers, pipelines, drilling caissons, seadeck walkways, stairs, abandoned wells and unused conductors. In some instances structural members can be removed if demonstrated to be overall beneficial. Examples of structural member removal may include launch truss member and redundant elements. Many actions for the reductions of marine growth and its related impact on hydrodynamic forces have been used. In areas of high marine growth copper-nickel cladding and a passive marine growth preventers may be installed to slide along members in wave action scraping any initial growth. Another passive measure is the introduction of marine growth predator colonies, such as starfish communities. Other actions include marine growth removal on a regular basis by means of diver held water jetting equipment.

B.2 Platform strengthening

a. Global platform strengthening: One method is to grout the annulus between the jacket legs and piles. Another method that has been used to increase jacket pile capacity is the addition of soldier piles that are essentially parallel and are in close proximity with existing piles.

b. Local platforms strengthening: A related grouting method for non-leg joints is to fully fill the joint chord member with grout. This procedure is often implemented by fully filling the member that contains the affected joint with grout. As an alternative when other reasons dictate grouting only in the local region near the joint, grout bags (inflatable polypropylene bags placed into a chord through small access holes which are then inflated by injecting grout). Structural members, both intact or damaged also be filled with grout to enhance their bending strength increases near mid span, however, the use of grout can be advantageous. Other grouting options that have received significant application are the use of prestressed grouted split clamps and sleeves. Mechanical clamps have also been used in many platform strengthening projects. These clamps are often used to incorporate new structural members into the platform. The main advantage of using clamps is that they can be installed underwater where underwater welding procedures are deemed unsuitable. The most commonly acceptable underwater wet welding procedure is to restrict the strengthening to filled welded patch plates. The fillet welds can then be oversized such that lower quality strengths are acceptable. Fatigue considerations, however, must be considered.

c. Enhancement for fatigue performance: One procedure is underwater weld toe grinding of fatigue cracks. In many instances, this procedure can be used for cracks up to 60% of the brace wall thickness. Mitigation of fatigue damage can include structural strengthening, preventative grinding, underwater peening and hole drilling.

d. Repair of existing damage: One repair method worthy of mention is removal of the damaged structural member or a portion of a structural component.

B.3 Operations - reduction of consequences

Often one of the best ways to deal with a structural problem is to modify the platform operations so as to either eliminate, or at least minimize, its potential consequences. Where life safety is at issue, this can be accomplished by de-manning the platform, reducing non-essential personnel, or perhaps making the facility a "daylight" operation where no overnight accommodations are used.

Where environmental safety is at issue, platform operations can be modified to minimize potential risks. Some of the significant items that merit special consideration include:

a. production shutdown systems

b. pipeline crossings
c. hazardous materials stored on-board
d. seasonal cantilevered jackup drilling rig capabilities, and
e. knock-on effects from failures of other systems.

Appendix D: Combined Risk of Hurricane and Winter Storms Base Shear (Ref. 14)

The method used to combine the risk of exceeding a given base shear due to both hurricanes and winter storms requires separate specification of wave height versus return period for each type of storm (for example, Figures 3.5.1d and 3.5.1f). Using the appropriate current, wave period, wind speed and storm tide for each return period for each type of storm, the platform base shear versus return period is calculated from the wave force model as shown in Figure 3.5.1a.

The combined probability of not exceeding a given base shear due to both hurricanes and winter storms is:

$$P_c = P_H * P_{WS}$$

In terms of return period for a given base shear:

$$1 - 1/RP_c = (1 - 1/RP_H) * (1 - 1/RP_{WS})$$

For example, in Figure 3.5.1g for BS = 1,000 kips, RP_H = 33 years and RP_{WS} = 50 years. Therefore, RP_C = 20 years.

At short return periods, the "combined" base shear will always be greater than either the hurricane or winter storm base shear of the same return period. At very large return periods (in the range of 1.8*BSC100), the "combined" base shear is essentially the same as the hurricane base shear because the risk of exceeding such a large value in winter storms becomes negligible.

APPENDIX E: ASSESSMENT FOR SEISMIC LOADING.

The seismic activity in Campeche Bay is similar to seismic activity in zone 2 of the U. S. because the acceleration contained in the spectrum of section C2.3.6c of API-RP2A and the acceleration presented in the spectrum developed by Mario Chavez for Campeche Bay are almost the same values.

According to section 17.5.3, the assessment for seismic loading is not applicable for zones 0, 1 and 2 of the U. S., and according to the similarity mentioned above, the seismic assessment would not be applicable for the Campeche Bay. However, in platforms recently designed in 1992 for the Campeche Bay, it was necessary to check and reinforce the deck for seismic loading, however the design was lead by storm conditions.

On the other hand, Nohoch-B platform does not fulfill the Screening for seismic assessment according to section 17.5.3.4 since this structure has several damages located in the jacket, and also there has been an increment in loads on the cellar deck for addition of a separator tank and a cantilever

For this reason we considered necessary to assess the Nohoch-B drilling platform with a design level analysis during a seismic motion of 100 year return period, to know its structural behavior, because for seismic assessment purposes the design level check is felt to be an operator's economic risk decision as stated in 17.6.3.2 of ref 1.

A static seismic analysis was executed for checking Nohoch-B platform.
As mentioned above, Mario Chavez design spectrum for 100-year return period was utilized. The platform was modeled by means of a 7 degree of freedom mass-spring system, in order to obtain the shapes and periods for every vibration mode in three orthogonal directions X, Y and Z (longitudinal, transversal and vertical, respectively). The periods of the first natural vibration mode for each direction are summarized in table 1.

DIRECTION	NATURAL PERIOD (sec)
LONGITUDINAL	2.245
TRANSVERSAL	2.19
VERTICAL	0.503

Table 1. Periods corresponding to natural vibration modes for each main direction of Nohoch-B platform.

Once obtained the seismic forces at each level, these were applied to the structure in order to perform a static analysis. In tables No. 2, 3 and 4 the seismic and shear forces for each level are shown.

LONGITUDINAL DIRECTION		
LEVEL (Elevation)	SEISMIC SHEAR (kips)	SEISMIC FORCE (kips)
1 (-38.100 m)	*2,365.64	114.54
2 (-22.250 m)	2,251.11	174.67
3 (-8.534 m)	2,076.43	112.56
4 (+3.658 m)	1,963.87	50.66
5 (+4.877 m)	1,913.21	0.00
6 (+15.850 m)	1,913.21	393.39
7 (+20.749 m)	1,519.82	1,519.82

Table 2. The maximum overturning moment is 401,926.3 Kip-ft.

* Represents the total base shear.

TRANSVERSAL DIRECTION		
LEVEL (Elevation)	SEISMIC SHEAR (kips)	SEISMIC FORCE (kips)
1 (-38.100 m)	*2,460.35	105.73
2 (-22.250 m)	2,354.62	177.75
3 (-8.534 m)	2,176.87	117.84
4 (+3.658 m)	2,059.03	53.30
5 (+4.877 m)	2,005.73	12.56
6 (+15.850 m)	1,993.17	414.10
7 (+20.749 m)	1,579.07	1,579.07

Table 3. The maximum overturning moment is 420,313.3 Kip-ft.
* Represents the total base shear.

VERTICAL DIRECTION		
LEVEL (Elevation)	SEISMIC SHEAR (kips)	SEISMIC FORCE (kips)
1 (-38.100 m)	3,257.70	187.22
2 (-22.250 m)	3,070.48	273.13
3 (-8.534 m)	2,797.35	240.09
4 (+3.658 m)	2,557.26	151.98
5 (+4.877 m)	2,405.28	35.24
6 (+15.850 m)	2,370.04	550.22
7 (+20.749 m)	1,819.82	1,819.82

Table 4.

Maximum displacements of the structure at each level are shown in tables 5, 6, 7 and 8. The X-axis positive direction is opposite to platform north and goes along longitudinal direction; Y-axis goes along transverse direction, and positive Z-axis goes up in vertical direction.

COMBINED DIRECTION +X+Y-Z(50%)		
LEVEL (Elevation)	X (cm)	Y (cm)
1 (-38.100 m)	30.76800	31.33130
2 (-22.250 m)	27.88593	25.06686
3 (-8.534 m)	15.88052	12.52353
4 (+3.658 m)	15.04720	12.39480
5 (+4.877 m)	12.58253	9.36184
6 (+15.850 m)	11.19209	8.04890
7 (+20.749 m)	9.03110	6.64028

Table 5.

COMBINED DIRECTION -X+Y-Z(50%)		
LEVEL (Elevation)	X (cm)	Y (cm)
1 (-38.100 m)	-34.16438	28.51270
2 (-22.250 m)	-30.54866	25.04600
3 (-8.534 m)	-15.93537	14.15155
4 (+3.658 m)	-15.07772	13.39461
5 (+4.877 m)	-12.84003	10.52288
6 (+15.850 m)	-10.68739	8.78034
7 (+20.749 m)	-8.08546	6.45987

Table 6.

COMBINED DIRECTION +X-Y-Z(50%)		
LEVEL (Elevation)	X (cm)	Y (cm)
1 (-38.100 m)	30.54846	-30.87000
2 (-22.250 m)	27.67681	-24.56074
3 (-8.534 m)	15.58023	-12.56384
4 (+3.658 m)	14.73740	-11.80101
5 (+4.877 m)	13.03406	-10.65814
6 (+15.850 m)	11.31131	-8.96607
7 (+20.749 m)	9.10374	-6.69108

Table 7.

COMBINED DIRECTION -X-Y-Z(50%)		
LEVEL (Elevation)	X (cm)	Y (cm)
1 (-38.100 m)	-34.76797	-30.95707
2 (-22.250 m)	-3107165	-24.31639
3 (-8.534 m)	-15.93465	-14.16320
4 (+3.658 m)	-15.04131	-12.74750
5 (+4.877 m)	-12.35464	-10.65158
6 (+15.850 m)	-10.53756	-7.77993
7 (+20.749 m)	-8.02236	-6.39790

Table 8.

The overall elastic behavior for the seismic load condition shows IR's above the allowable level mainly in members of the deck and in some of the jacket.

According with 17.6.3.2 (ref. 1) an Ultimate Strength Analysis using a 1000-years return period event is required if the platform does not pass the design basis check, that is the case for Nohoch-B, for that reason we are not considering any mitigation alternative to be applied just considering the results of the elastic analysis.

The seismic resultant base shear is shown below and some comparisons have been made in order to obtain some qualitative conclusions.

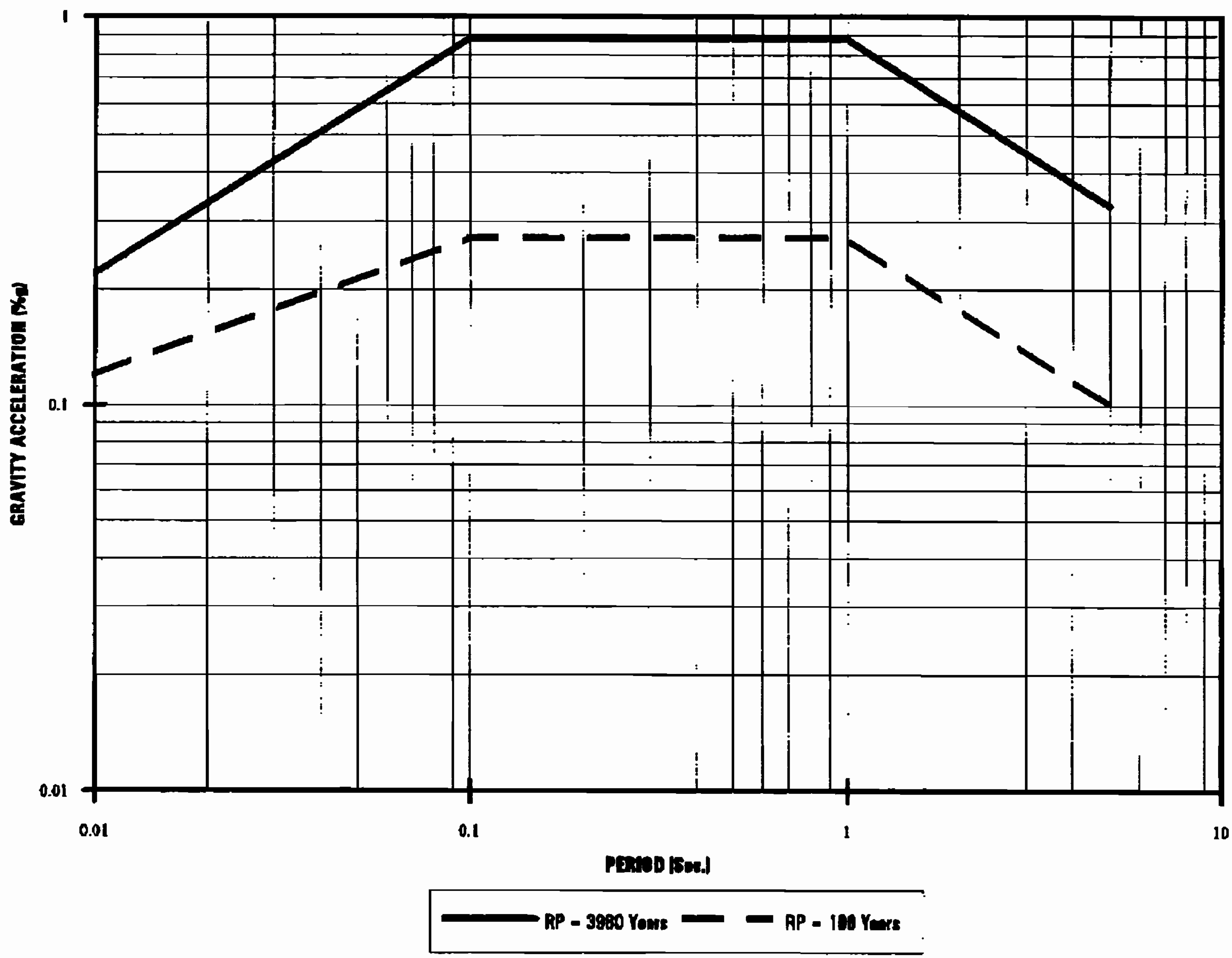
BASE SHEARS COMPARISONS (Kips)				
CONCEPT	SHEAR COMPONENTS		RESULTANT	COMPARISONS
	DIRECTION 1 WIDE	DIRECTION 3 LONG		
EARTHQUAKE MARIO CHAVEZ SPECTRUM RP= 100 YEARS	2,460	2,365	3,412 (1)	(4)/(1) = 2.1
WAVE BASE SHEAR OCEANWEATHER RP= 100 YEARS	1,843 (2)	1,669		(4)/(2) = 3.8
WAVE BASE SHEAR A.H.GLENN RP= 100 YEARS	4,630 (3)	4,193		(4)/(3) = 1.5
ULTIMATE STRENGTH BASE SHEAR OBTAINED USING WAVE FORCES DISTRIBUTION	9,237 (5)	7,007 (4)		(4)/(4) = 1.0
EARTHQUAKE MARIO CHAVEZ SPECTRUM RP= 1000 YEARS (*).	5,486**	5,274**	7,617** (6)	(4)/(6) = 0.92 (5)/(6) = 1.21

(*) Assuming that for Nohoch-B the acceleration for RP=1000 years is 0.413g, this value is preliminar and has to be checked.

** Base shears calculated from a 1000-year earthquake assuming that the acceleration and base shears ratios are the same, eg. (0.413/0.185)3412= 7617 kips

Because the seismic spectra available at IMP is for RP = 100 years and RP = 3980 years, and the only seismic analysis developed was for a 100-year return period event, a new seismic spectra for RP = 1000 years was interpolated considering a log-distribution of the form $a = m \log RP + C$. The values obtained were $m = 0.228$ and $C = -0.271$, the accelerations for Nohoch-B associated with a seismic event of RP = 1000 years using the above formulation is 0.413g. Assuming that the accelerations and base shears ratios are the same, the estimated resultant base shear for RP = 1000 years is 7617 Kips and comparing with the ultimate strength base shear obtained using wave forces distribution, the ratios are 0.92 and 1.21 for the longitudinal and wide directions, respectively. That means in a first estimation that Nohoch-B could probably withstand a seismic event of RP = 1000 years without collapsing. For a definite conclusion a validated seismic spectra for RP = 1000 years, and an ultimate seismic analyses have to be performed.

DESIGN SPECTRUM FOR SITES LOCATED IN CAMPECHE BAY



APPENDIX F: FATIGUE ASSESSMENT

The present analysis was done following recomendations contained in the DRAFT Section 17.7.2d, and Section 5 of the API RP-2A 20th Edition.

According to Sect. 5, a detailed FATIGUE analysis should be performed for template type structures, like the fixed platform under assessment. However, in lieu of detailed fatigue analysis, simplified fatigue analyses, may be applied for tubular joints in template type platforms if the following cosiderations are satisfied:

1.- Are in less than 400 feet (122m) of water.

2.- Are constructed of ductile steels.

3.- Have redundant structural framing.

4.- Have natural periods less than 3 seconds.

In order to check if NOHOCH-B platform fulfill previous requirements, following information was obtained:

Platform's water depth	125'-3" (38.176 m)
Platform's steel	ASTM A572 Gr. 50 ASTM A-36 API-5LX Gr.X-42 API-5L Gr. B
Have redundant structuration	Yes
Natural Period	2.2 secs.

A simplified FATIGUE analysis was performed to the platform under assessment, in order to check fatigue life of all tubular joints according to sect. C5 (Commentary on Fatigue of API RP-2A).

Analysis was performed for the Campeche Bay. The simplified Fatigue assessment involved the checking of all tubular joints in the structure such that the "Peak Hot Spot Stresses" for the fatigue design wave do not exceed the "Allowable Peak Hot Spot Stress".

FATIGUE DESIGN WAVE

Fatigue design wave characteristics were taken from the Oceanweather and Meteorological Report. Following table shows fatigue design wave characteristics used for generating forces and stresses applied to the structure:

	APPLIED WAVE DIRECTIONS	WAVE HEIGHT (meters)	WAVE PERIOD "T" (secs.)	TIDE HEIGHT (m)
1.-	WAVE +X 167°	12.91	11.51	0.65
2.-	WAVE -X-Y 222°	12.91	11.51	0.65
3.-	WAVE -Y 267°	12.91	11.51	0.65
4.-	WAVE +X-Y 312°	12.91	11.51	0.65

This wave was applied to the structure without wind, current and no gravity load effects.

Due that the employed software for computing wave forces does not accept wave kinematics factor modification, a value of 1.00 was used instead of 0.88 as the sect. 5 specifies.

ALLOWABLE PEAK HOT SPOT STRESSES

Allowable Peak Hot Spot Stresses, Sp, were determined from Fig. C.5.1-2 page 159 of API RP-2A, as a function of water depth, member location, S-N curve, and design fatigue life. S-N curve was taken from API Recomendations, Fig. 5.4.1. The X' curve for welds without profile control but conforming to a basic standard flat profile (ANSI/AWS D1.1-92) was applied. The used design fatigue life was taken as 50 years.

PEAK HOT SPOT STRESSES FOR THE FATIGUE DESIGN WAVE.

The Peak Hot Spot Stresses at a joint were taken as the maximum value of the following expressions calculated at both the chord and the brace sides of the tubular joint.

$$\Gamma_H(\Phi) = F_a\ SCF_{ac}\ (1 - R) - F_{ib}\ SCF_{ib} \cos \Phi$$

$$\Gamma_H(\Phi) = F_a\ SCF_{as}\ (R) - F_{ob}\ SCF_{ob} \operatorname{sen} \Phi$$

Where Fa, Fib and Fob are the nominal member in axial, in plane bending and out of plane bending stresses, and SCFac, SCFas, SCFib, and SCFob are the stress concentration factors for axial crown, axial saddle, and in plane and out of plane bending respectively for the chord or the brace side. ϕ is the position of the Peak hot spot around the weld and R is an interpolation factor (see page 346 Fatigue Handbook-Almar-Naes). Stress Concentration Factors were calculated according to **Wordsworth-Smedly** formulas.

Four wave approach directions (end-on, broadside, and two diagonal), with one wave position relative to the platform were considered to identify the "Peak Hot Spot stresses" at each member end for the fatigue design wave. "Peak Hot Spot stresses" were computed for four points around weld toe on each joint at each member end for the fatigue design wave.

RESULTS OF FATIGUE ASSESSMENT.

Results of fatigue analysis showed that "Peak Hot Spot stresses" of tubular joints are lower than the allowable "Peak Hot Spot stresses", which were obtained from API-RP2A Ed. 20th, those values are of 2953 kg/cm2 and 3516 kg/cm2 for waterline and other members respectively. Therefore, it is considered that the platform under assessment does not have any fatigue problem.

PLATFORM "V"

ULTIMATE STRENGTH ANALYSIS OF CAMEROON "E" PLATFORM

INTRODUCTION

The Cameroon Mokoko Abana field consists of 11 platforms off the coast of Cameroon, West Africa in water depths from 150 feet to 165 feet. Most of the platforms are very similar in design and were installed in the early 1980's. The wave climate is mild with the 100 year wave between 18 and 21 feet (Hmax). Due to the mild climate, the platforms were designed with no batter on the platform faces. The majority of the platforms are well jackets with only test facilities. The "E" platform supports some oil production facilities and has in the past supported a small 6 to 10 man living quarters.

An earlier assessment (reference 1) of the Cameroon "E", "G" and "K" platforms was performed to evaluate the marine growth and scour conditions identified in the 1992 underwater inspection program. This assessment utilized the design level procedure described in the Draft Section 17.0 of API RP2A-WSD and concluded that the 3 platforms passed the assessment and no remedial measures were recommended. The assessment procedure in Draft Section 17.0 does not require a more detailed ultimate strength level analysis if a platform passes the less detailed design level analysis. However, to provide data to verify the assessment procedure and to confirm the design level analysis procedure, an ultimate strength level analysis was also performed on the "E" platform. The "E" platform was selected due to current plans to add additional facilities (compressor) on the platform.

SCOPE

The "E" platform was modeled and analyzed with the PMB Capacity Analysis Program (CAP). Three wave approach directions were considered, including endon, broadside and diagonal directions. The applied environmental loads and the identified failure mechanisms were compared to the previous design level analyses to ensure consistency of the models. This organization of this report is similar to the Trial Basis Document.

Platform Information

a.	Name:	Cameroon Mokoko-Abana "E" platform
b.	Function:	Manned Production (currently only production)
c.	Year Designed:	1981
d.	Year Installed	1981
e.	Water Depth:	161 feet
f.	No. of legs:	4
g.	Foundation:	4 ungrouted main piles
h.	Conductors:	9 - 24 inch
i.	Deck Clearance:	+32 feet above mean water level
j.	Deck Payload:	1500 kips
k.	Last inspection:	1992
l.	Structural damage:	None
m.	Soil properties:	From boring on site and nearby

Figures 1.1 through 1.9 show the structural framing of the jacket and the pile makeup.

Part A "E" Platform Assessment

A.1 Platform Selection

The "E" platform was previously selected for a design level analysis based on high marine growth and scour identified in the 1992 underwater inspection. It was selected for an ultimate strength analysis because of plans to add additional topside facilities.

A.2 Condition Assessment

The 1992 underwater inspection revealed no significant structural damage and adequate cathodic protection. Marine growth was reported and was modeled as 2 inch radial growth on members at the surface to 80 feet below water level and .2" growth to the mudline.

A.3 Exposure Category

The "E" platform was previously categorized as manned, non-evacuated, insignificant environmental impact for the design level analysis. This categorization was also used in this ultimate strength analysis, although it is understood that the quarters on "E" platform have been removed. This categorization is conservative and results in the application of a higher environmental loading.

A.4 Design Basis Check

The platform is not in the Gulf of Mexico so the design basis check does not apply.

A.5 Analysis Checks

The structure passed a previous design level analysis and it passes the ultimate strength analysis. The design level analysis was performed using SESAS, an in-house linear structural analysis program and results of this study are shown in the attached report. The ultimate strength analysis was performed using CAP, a nonlinear structural analysis program by PMB Engineering.

A.5.1 Criteria

Reference 2 describes the environmental loading for the ultimate strength analysis and Table 1 list the environmental criteria. The criteria are based on the West Africa Extreme Wave Hindcast Study (WAX) by Oceanweather Inc., 1992. The results of the WAX study showed the 100 year wave height as 18.4 feet (Hmax) which is less than the original design 100 year wave height of 21 feet.

A.5.2 Model Development

The jacket was modeled based on Revision 6 of the "E" platform drawings as shown in Figures 1.1 through 1.9 and included the following appurtenances:

1. 9 drive pipes (24" diameter).
2. 40 foot wide boat landing on the B face.
3. 13 risers as described in the 1992 underwater inspection report.
4. Riser Protector on the A face.
5. Emergency Sump
6. Stairs and walkways

The deck is well above the wave crest, the deck members were not evaluated in this assessment. The deck was modeled as a simple stiff space frame with the actual deck loads applied at the leg joints. The deck loading included the following estimate of the existing equipment loads plus the additional weight of a new compressor:

Deck Dead Load	600 kips
Deck Existing Equipment Load	1520 kips
Workover Rig Load	840 kips
New Upper Deck Compressor	400 kips
Total Topside Load	3360 kips

The jacket was modeled with the following member elements:

Jacket legs and piles	Beam columns
Vertical Diagonals	Marshal Strut
Horizontal bracing	Linear beams
Deck elements	Linear beams

The effective length factors (K factors) were set equal to the recommendations of API RP2A 20th edition. This is conservative and lower K factors may be used to obtain higher capacities. Since the platform capacity exceeded the minimum ultimate strength requirement, further analyses with lower K factors were not considered.

Soil Resistance was modeled with soil elements derived from the soil boring MAE-E through 109 foot penetration and soil boring MAA-F below 109 foot penetration (MAE-E boring stopped at 109 feet).

The wave force area of the model was compared with the model used in the previous design level analysis with the following results for the base shear associated with 21 foot wave combined with wind and current:

Approach Direction	CAPS Model Base Shear (kips)	Previous Model Base Shear (kips)	Difference
End-on	590	616	- 4%
Broad side	705	628	+12%

A.5.3 Ultimate Strength Analysis Results

To pass the assessment process, the ultimate strength of the platform, expressed in terms of base shear, must exceed the base shear from a 24 foot wave combined with wind and current. The following results were obtained for the platform ultimate strength:

Approach Direction	Ultimate Capacity Base Shear (kips)	24' Wave Combined Loading Base Shear (kips)	Component Failure
End-on	1107	766	Vertical Diagonal
Diagonal	1116	789	Vertical Diagonal
Broad side	1120	907	Vertical Diagonal

Expressed in terms of the reserve strength ratio (Ultimate capacity/100 year design base shear), the analysis results are:

Approach Direction	Ultimate Capacity Base Shear (kips)	100 year Loading Base Shear (kips)	Reserve Strength Ratio
End-on	1107	493	2.2
Diagonal	1116	515	2.1
Broad side	1120	592	1.9

Based on these results, the "E" platform passes the ultimate strength assessment. Plots of the ultimate strength versus deck deflection are shown in Figures A.5.3.1 through A.5.3.3 and the tabular results are shown in Tables A.5.3.1 through A.5.3.3.

The 1st component failures for the 3 directions were in the diagonals although earlier initial yielding occurred in the legs and piles. This is consistent with the results of the design level analysis which showed the diagonals as the highest stressed members. Tables A.5.3.4 through A.5.3.6 show the component failures or inelastic events for each load step and deck deflection. The deflected shape and failure mode of the platform at 90 degrees is shown in Figure A.5.3.4. The pile loads and capacities for the highest loaded piles are shown in Figure A.5.3.5 and A.5.3.6. The foundation did not govern the ultimate capacity and based on these results higher deck loads can be supported without reducing the ultimate capacity.

References

1. ., "Assessment of Pecten Cameroon Platforms "E", "G" & "K" , report attached to , March 24, 1994.

2. , April 6, 1994 on "Criteria for Reanalysis of Cameroon Structures" (criteria developed by .

Table 1

Environmental Criteria - Cameroon Platform Ultimate Strength Analysis

Wave: Hmax = 24 feet T= 16 seconds

Current: 5.1 feet/sec. at surface decreasing to zero at mudline

85% current blockage

applied inline with wave

Wind: 81 mph (gust), 63 mph (sustained)

applied inline with wave

Wave Coefficients:

Cd (smooth) = 0.65
Cm (smooth) = 1.60

Cd (rough) = 1.05
Cm (rough) = 1.20

Kinimatics Factor = 1.0

No conductor shielding

Marine Growth: 2" radial to -80 ft
.2" radial from -80 ft to mudline

*** TABLE A.5.3.1 "E" PLATFORM END ON DIRECTION ***

CAP Results Table   Tue Sep 6 21:08:31 1994

Project: mae Model: modell Version: 1

Cut Plane Force Fx - Kip

X DIRECTION APPLIED WAVE

Load Step	X Deck Disp Ft	Force Kips
0	000.000e-3	000.000e-3
1	0.226	44.093
2	0.757	202.485
3	1.287	353.238
4	1.814	491.511
5	2.337	625.280
6	2.468	658.333
7	2.524	672.459
8	2.594	689.731
9	2.601	691.600
10	2.605	692.410
11	2.610	693.813
12	2.613	694.399
13	2.470	650.354
14	2.396	627.342
15	2.267	587.466
16	2.044	518.398
17	1.947	488.494
18	1.905	475.545
19	1.887	469.938
20	1.880	467.511
21	1.866	463.305
22	1.860	461.484
23	1.858	460.695
24	1.853	459.069
25	2.309	585.090
26	2.758	700.595
27	2.870	726.253
28	2.984	751.654
29	3.018	759.431
30	3.078	772.778
31	3.127	779.951
32	3.210	792.310
33	3.354	813.485
34	3.603	849.244
35	3.616	851.117
36	3.639	854.377
37	3.649	855.641
38	3.654	856.225
39	3.662	857.282
40	3.677	859.112
41	3.704	862.279
42	3.749	867.758
43	3.754	868.348
44	3.762	869.375
45	3.766	869.819
46	3.772	870.542
47	3.772	870.043
48	3.777	868.678
49	3.769	869.400
50	3.777	867.894
51	3.778	868.809
52	3.784	868.016
53	3.780	866.386
54	3.262	705.528
55	3.171	678.229
56	3.013	630.999
57	2.996	625.889
58	2.966	617.033
59	2.963	616.075
60	2.957	614.312
61	3.445	750.924
62	3.475	759.443
63	3.488	763.134
64	3.511	769.373
65	3.523	772.344
66	3.543	777.546
67	3.579	786.558
68	3.640	802.166
69	3.733	822.627
70	3.783	832.560
71	3.804	836.794
72	3.814	838.607
73	3.818	839.377
74	3.826	840.691
75	3.829	841.239
76	3.823	840.223
77	3.337	689.992
78	3.216	652.717
79	3.006	588.157
80	2.984	581.171
81	2.944	569.066
82	2.940	567.756
83	2.933	565.408
84	3.455	701.220
85	3.585	735.122
86	3.599	738.791
87	3.605	740.262
88	3.609	741.012
89	3.615	742.370
90	3.626	744.722
91	3.644	748.796
92	3.676	755.852
93	3.731	768.074
94	3.755	773.365
95	3.765	775.657
96	3.783	779.626
97	3.791	781.344
98	3.795	781.937
99	3.797	782.188
100	3.801	782.736
101	3.808	783.686
102	3.819	785.331
103	3.839	788.180
104	3.874	793.116
105	3.935	801.664
106	3.961	805.366
107	4.002	809.792
108	4.021	811.196
109	4.054	813.679
110	4.112	817.980
111	4.212	825.232
112	4.220	825.693
113	4.229	826.272
114	4.246	827.261
115	4.274	828.840
116	4.286	829.515
117	4.307	830.697
118	4.333	832.132
119	4.378	834.592
120	4.456	838.846
121	4.464	839.304
122	4.479	840.081
123	4.491	840.729
124	4.513	841.863
125	4.551	843.826
126	4.596	846.031
127	4.631	847.708
128	4.691	850.612
129	4.796	855.636
130	4.977	864.229
131	5.291	878.983
132	5.397	883.907
133	5.443	885.954
134	5.474	887.189
135	5.527	889.345
136	5.619	893.073
137	5.779	899.869
138	5.831	902.830
139	5.853	904.089
140	5.891	906.272
141	5.908	907.217
142	5.915	907.624
143	5.929	908.091
144	5.952	908.932
145	5.992	910.389
146	6.062	912.911
147	6.183	917.280
148	6.392	924.840
149	6.748	940.119
150	6.815	943.534
151	6.932	949.506
152	6.983	952.088
153	6.988	952.367
154	6.997	952.845
155	7.014	953.675
156	7.042	955.113
157	7.092	957.604
158	7.177	961.916
159	7.325	969.339
160	7.506	978.406
161	7.728	989.456
162	7.950	1000.481
163	8.222	1013.942
164	8.558	1031.541
165	8.817	1044.473
166	8.897	1048.431
167	9.034	1055.249
168	9.076	1057.317
169	9.095	1058.209
170	9.126	1059.734
171	9.181	1062.360
172	9.204	1063.487
173	9.214	1063.977
174	9.232	1064.826
175	9.240	1065.192
176	9.253	1065.829
177	9.259	1066.104
178	9.269	1066.581
179	9.273	1066.788
180	9.280	1067.145
181	9.293	1067.765
182	9.316	1068.836
183	9.322	1069.166
184	9.325	1069.308
185	9.331	1069.555
186	9.339	1069.981
187	9.355	1070.719
188	9.361	1071.039
189	9.373	1071.593
190	9.378	1071.832
191	9.387	1072.248
192	9.390	1072.427
193	9.397	1072.739
194	9.408	1073.278
195	9.413	1073.511
196	9.422	1073.915
197	9.425	1074.091
198	9.432	1074.394
199	9.442	1074.920
200	9.461	1075.829
201	9.470	1076.222
202	9.473	1076.393
203	9.479	1076.683
204	9.487	1077.019
205	9.494	1077.370
206	9.507	1077.978
207	9.512	1078.242
208	9.515	1078.356
209	9.519	1078.553
210	9.526	1078.896
211	9.529	1079.043
212	9.534	1079.298
213	9.542	1079.646
214	9.549	1079.995
215	9.556	1080.342
216	9.564	1080.690
217	9.571	1081.036
218	9.578	1081.380
219	9.586	1081.724
220	9.593	1082.068
221	9.600	1082.411
222	9.608	1082.753
223	9.615	1083.094
224	9.622	1083.435
225	9.629	1083.776
226	9.637	1084.115
227	9.644	1084.454
228	9.651	1084.793
229	9.659	1085.133
230	9.666	1085.475
231	9.673	1085.820
232	9.681	1086.165
233	9.688	1086.509
234	9.695	1086.852
235	9.702	1087.195
236	9.710	1087.536
237	9.717	1087.876
238	9.724	1088.215
239	9.732	1088.551
240	9.739	1088.885
241	9.746	1089.214
242	9.753	1089.543
243	9.761	1089.878
244	9.768	1090.224
245	9.771	1090.375
246	9.777	1090.635
247	9.779	1090.748
248	9.783	1090.943
249	9.790	1091.282
250	9.793	1091.428

251	9.799	1091.679
252	9.806	1092.024
253	9.819	1092.616
254	9.824	1092.874
255	9.831	1093.214
256	9.839	1093.551
257	9.846	1093.889
258	9.853	1094.225
259	9.861	1094.560
260	9.868	1094.889
261	9.875	1095.222
262	9.882	1095.558
263	9.890	1095.894
264	9.897	1096.225
265	9.904	1096.558
266	9.911	1096.889
267	9.919	1097.218
268	9.926	1097.503
269	9.933	1097.759
270	9.940	1098.083
271	9.947	1098.425
272	9.950	1098.576
273	9.956	1098.835
274	9.958	1098.946
275	9.962	1099.134
276	9.969	1099.474
277	9.977	1099.805
278	9.984	1100.140
279	9.991	1100.473
280	9.998	1100.807
281	10.006	1101.139
282	10.013	1101.450
283	10.020	1101.763
284	10.027	1102.090
285	10.035	1102.425
286	10.042	1102.745
287	10.049	1103.054
288	10.056	1103.363
289	10.063	1103.696
290	10.071	1104.038
291	10.074	1104.188
292	10.079	1104.448
293	10.082	1104.552
294	10.089	1104.887
295	10.096	1105.220
296	10.103	1105.549
297	10.111	1105.875
298	10.118	1106.170
299	10.125	1106.432
300	10.132	1106.763
301	10.139	1107.106

*** Table A.5.3.2 Diagonal direction ***

CAP Results Table   Tue Sep 27 18:10:04 1994

Project: mae Model: model1 Version: 1

Cut Plane Force Fx - Kip

Diagonal app. heavy deck

Load Step	X Deck Disp Ft	Force Kips
0	000.000e-3	000.000e-3
1	0.220	45.076
2	1.041	279.455
3	1.860	499.420
4	2.063	550.712
5	2.239	593.996
6	2.543	668.844
7	3.071	793.591
8	3.173	817.109
9	3.204	824.313
10	3.258	836.450
11	3.282	842.135
12	3.292	844.453
13	3.296	845.453
14	3.304	847.154
15	3.317	849.792
16	3.330	852.224
17	3.342	854.792
18	3.355	857.303
19	3.368	859.657
20	3.381	862.108
21	3.394	864.417
22	3.407	866.643
23	3.419	868.949
24	3.432	871.215
25	3.445	873.433
26	3.458	875.608
27	3.471	877.763
28	3.484	879.906
29	3.497	882.039
30	3.509	884.162
31	3.522	886.288
32	3.535	888.413
33	3.548	890.537
34	3.561	892.660
35	3.574	894.782
36	3.587	896.905
37	3.600	899.027
38	3.613	901.150
39	3.625	903.271
40	3.638	905.394
41	3.651	907.513
42	3.664	909.623
43	3.677	911.732
44	3.690	913.841
45	3.703	915.953
46	3.716	918.035
47	3.728	920.131
48	3.741	922.227
49	3.754	924.322
50	3.767	926.417
51	3.780	928.512
52	3.793	930.605
53	3.806	932.699
54	3.819	934.791
55	3.831	936.882
56	3.844	938.974
57	3.857	941.065
58	3.870	943.156
59	3.883	945.246
60	3.896	947.337
61	3.909	949.427
62	3.922	951.517
63	3.934	953.607
64	3.947	955.697
65	3.960	957.786
66	3.973	959.876
67	3.986	961.964
68	3.999	964.054
69	4.012	966.129
70	4.025	968.206
71	4.037	970.282
72	4.050	972.358
73	4.063	974.434
74	4.076	976.509
75	4.089	978.584
76	4.102	980.659
77	4.115	982.733
78	4.128	984.807
79	4.140	986.878
80	4.153	988.536
81	4.166	989.825
82	4.179	991.450
83	4.191	992.930
84	4.204	994.139
85	4.216	995.257
86	4.229	996.438
87	4.242	997.764
88	4.229	992.329
89	4.216	989.846
90	3.393	738.291
91	3.215	684.086
92	3.196	678.221
93	3.162	668.059
94	3.148	663.658
95	3.142	661.753
96	3.131	658.221
97	3.928	871.279
98	4.127	924.330
99	4.212	945.648
100	4.238	951.852
101	4.249	954.435
102	4.261	956.624
103	4.273	958.637
104	4.285	960.396
105	4.297	961.852
106	4.309	964.208
107	4.319	966.604
108	4.324	967.593
109	4.336	969.800
110	4.348	971.725
111	4.360	973.750
112	4.372	975.737
113	4.384	977.619
114	4.396	979.410
115	4.408	981.236
116	4.419	983.001
117	4.431	984.807
118	4.443	986.564
119	4.455	988.269
120	4.467	989.956
121	4.478	991.625
122	4.490	993.304
123	4.502	994.995
124	4.514	996.671
125	4.525	998.353
126	4.537	1000.022
127	4.549	1001.648
128	4.561	1003.220
129	4.573	1004.720
130	4.585	1006.163
131	4.597	1007.557
132	4.609	1008.929
133	4.621	1010.243
134	4.633	1011.545
135	4.645	1012.789
136	4.658	1014.070
137	4.670	1015.348
138	4.682	1016.503
139	4.694	1017.373
140	4.707	1018.265
141	4.719	1019.250
142	4.731	1019.876
143	4.743	1020.643
144	4.755	1021.601
145	4.768	1022.291
146	4.780	1022.955
147	4.792	1023.872
148	4.804	1024.703
149	4.816	1025.214
150	4.829	1025.703
151	4.841	1026.325
152	4.853	1027.271
153	4.865	1028.464
154	4.877	1029.415
155	4.890	1030.156
156	4.902	1030.884
157	4.914	1031.594
158	4.926	1032.348
159	4.938	1033.049
160	4.951	1033.799
161	4.963	1034.433
162	4.975	1035.162
163	4.987	1035.702
164	4.999	1036.221
165	5.011	1036.678
166	5.023	1037.300
167	5.036	1038.143
168	5.048	1039.028
169	5.060	1040.005
170	5.072	1040.603
171	5.084	1041.211
172	5.097	1041.604
173	5.109	1041.909
174	5.121	1042.908
175	5.133	1044.188
176	5.145	1044.939
177	5.157	1045.720
178	5.170	1046.385
179	5.182	1047.078
180	5.194	1047.589
181	5.206	1047.973
182	5.218	1048.619
183	5.230	1049.389
184	5.242	1050.286
185	5.254	1051.160
186	5.266	1051.764
187	5.278	1052.443
188	5.290	1053.110
189	5.302	1053.720
190	5.314	1054.320
191	5.326	1054.931
192	5.338	1055.460
193	5.350	1055.957
194	5.362	1056.334
195	5.374	1056.674
196	5.386	1057.349
197	5.398	1058.186
198	5.410	1059.071
199	5.422	1059.810
200	5.434	1060.394
201	5.446	1060.929
202	5.458	1061.363
203	5.470	1061.436
204	5.482	1060.696
205	5.494	1060.814
206	5.506	1062.124
207	5.518	1063.259
208	5.530	1064.177
209	5.542	1064.993
210	5.554	1065.670
211	5.566	1066.352
212	5.578	1066.962
213	5.590	1067.644
214	5.602	1068.264
215	5.614	1068.818
216	5.626	1069.247
217	5.638	1069.911
218	5.650	1070.487
219	5.662	1070.711
220	5.674	1071.338
221	5.686	1072.105
222	5.698	1072.907
223	5.709	1073.789
224	5.721	1074.544
225	5.733	1075.246
226	5.745	1075.886
227	5.757	1076.481
228	5.769	1077.150
229	5.781	1077.835
230	5.793	1078.279
231	5.805	1078.840
232	5.817	1079.486
233	5.829	1080.106
234	5.841	1080.898
235	5.853	1081.578
236	5.865	1082.252
237	5.877	1082.856
238	5.889	1083.425
239	5.901	1083.907
240	5.913	1084.419
241	5.925	1084.976
242	5.937	1085.469
243	5.949	1086.175
244	5.961	1087.147
245	5.973	1087.899
246	5.985	1088.394
247	5.997	1089.050
248	6.009	1089.616
249	6.021	1090.219
250	6.033	1090.779

251	6.045	1091.309
252	6.057	1091.778
253	6.069	1092.331
254	6.081	1093.183
255	6.093	1093.997
256	6.105	1094.687
257	6.117	1095.356
258	6.129	1095.866
259	6.141	1096.387
260	6.153	1096.972
261	6.165	1097.548
262	6.177	1097.453
263	6.189	1096.928
264	6.201	1097.264
265	6.213	1098.190
266	6.226	1098.807
267	6.238	1099.708
268	6.250	1100.487
269	6.262	1101.114
270	6.274	1101.889
271	6.286	1102.707
272	6.298	1103.350
273	6.311	1103.952
274	6.323	1104.574
275	6.335	1105.205
276	6.347	1105.793
277	6.359	1106.464
278	6.371	1107.087
279	6.384	1107.635
280	6.396	1108.212
281	6.408	1108.816
282	6.420	1109.408
283	6.432	1110.074
284	6.444	1110.668
285	6.456	1111.259
286	6.469	1111.750
287	6.481	1112.222
288	6.493	1112.466
289	6.505	1112.446
290	6.517	1113.022
291	6.529	1114.031
292	6.541	1114.924
293	6.553	1115.665
294	6.565	1116.308
295	6.569	1103.210
296	6.566	1115.906
297	6.565	1100.818
298	6.552	1097.167
299	5.735	857.516
300	5.684	842.554
301	5.596	816.618

*** TABLE A.5.3.3 "E" PLATFORM BROAD SIDE DIRECTION ***

CAP Results Table Mon Sep 26 21:53:04 1994

Project: mae Model: modell Version: 1

Cut Plane Force Fy - Kip

CUTPLANE 90 WITH HEAVY DECK

Load Step	Y Deck Disp Ft	Force Kips
0	000.000e-3	000.000e-3
1	0.216	45.439
2	1.192	312.301
3	2.158	551.199
4	2.398	608.540
5	2.814	706.786
6	2.840	712.743
7	2.851	715.326
8	2.856	716.445
9	2.864	718.382
10	2.849	713.481
11	2.832	709.635
12	2.574	633.431
13	2.126	502.579
14	2.078	488.412
15	2.057	482.279
16	2.048	479.623
17	2.032	475.022
18	2.025	472.821
19	2.876	694.209
20	3.086	740.522
21	3.132	750.191
22	3.188	762.075
23	3.194	763.361
24	3.205	765.520
25	3.214	766.800
26	3.231	769.144
27	3.258	773.208
28	3.307	780.213
29	3.391	792.181
30	3.427	797.335
31	3.442	799.568
32	3.436	797.719
33	3.420	793.547
34	3.179	724.048
35	2.761	604.915
36	2.716	592.023
37	2.638	569.685
38	2.629	567.268
39	2.643	571.043
40	2.667	577.476
41	2.709	588.619
42	2.781	607.918
43	2.906	641.345
44	3.123	699.241
45	3.486	793.836
46	3.525	800.342
47	3.574	808.264
48	3.595	811.701
49	3.605	812.928
50	3.610	813.442
51	3.619	814.451
52	3.634	816.196
53	3.661	819.221
54	3.708	824.447
55	3.788	833.344
56	3.813	836.016
57	3.824	837.049
58	3.813	835.831
59	3.796	829.105
60	3.802	830.886
61	3.791	827.780
62	3.773	822.397
63	3.741	813.073
64	3.686	796.924
65	3.591	768.953
66	3.426	720.505
67	3.140	636.591
68	3.017	600.258
69	3.003	596.326
70	2.980	589.513
71	2.971	586.328
72	3.211	646.408
73	3.316	672.191
74	3.497	716.852
75	3.575	736.190
76	3.584	738.100
77	3.589	739.222
78	3.599	741.319
79	3.616	744.953
80	3.624	746.526
81	3.637	749.008
82	3.645	750.021
83	3.659	751.920
84	3.682	755.211
85	3.724	760.910
86	3.795	770.781
87	3.918	787.877
88	4.108	805.635
89	4.199	812.397
90	4.239	814.998
91	4.307	819.049
92	4.425	825.990
93	4.630	837.940
94	4.682	840.724
95	4.692	841.211
96	4.702	841.689
97	4.720	842.536
98	4.750	843.996
99	4.763	844.626
100	4.785	845.702
101	4.824	847.423
102	4.862	849.050
103	4.929	851.891
104	5.044	856.809
105	5.094	858.920
106	5.180	862.569
107	5.218	864.149
108	5.222	864.317
109	5.229	864.505
110	5.242	864.837
111	5.265	865.411
112	5.304	866.406
113	5.371	868.129
114	5.489	871.107
115	5.692	876.229
116	6.043	884.977
117	6.665	908.583
118	7.142	932.285
119	7.619	955.833
120	8.452	999.707
121	8.614	1008.245
122	8.684	1011.955
123	8.714	1013.557
124	8.766	1016.332
125	8.789	1017.523
126	8.828	1019.566
127	8.840	1020.186
128	8.861	1021.265
129	8.870	1021.706
130	8.885	1022.477
131	8.912	1023.826
132	8.924	1024.408
133	8.929	1024.660
134	8.937	1025.098
135	8.953	1025.805
136	8.979	1027.152
137	8.990	1027.708
138	9.010	1028.672
139	9.018	1029.089
140	9.033	1029.812
141	9.059	1031.065
142	9.070	1031.604
143	9.074	1031.838
144	9.083	1032.244
145	9.097	1032.948
146	9.122	1034.168
147	9.133	1034.667
148	9.146	1035.314
149	9.160	1035.982
150	9.183	1037.126
151	9.190	1037.474
152	9.202	1038.074
153	9.216	1038.709
154	9.229	1039.357
155	9.243	1039.998
156	9.256	1040.634
157	9.270	1041.275
158	9.283	1041.925
159	9.297	1042.581
160	9.310	1043.230
161	9.323	1043.871
162	9.337	1044.516
163	9.350	1045.158
164	9.364	1045.795
165	9.377	1046.404
166	9.390	1047.004
167	9.404	1047.640
168	9.417	1048.276
169	9.431	1048.910
170	9.444	1049.535
171	9.457	1050.160
172	9.471	1050.784
173	9.484	1051.410
174	9.498	1052.039
175	9.511	1052.667
176	9.524	1053.294
177	9.538	1053.917
178	9.551	1054.535
179	9.564	1055.150
180	9.578	1055.758
181	9.591	1056.370
182	9.605	1056.987
183	9.618	1057.594
184	9.631	1058.201
185	9.645	1058.804
186	9.658	1059.410
187	9.671	1060.016
188	9.685	1060.619
189	9.698	1061.219
190	9.711	1061.819
191	9.725	1062.424
192	9.738	1063.031
193	9.752	1063.638
194	9.765	1064.244
195	9.778	1064.848
196	9.791	1065.385
197	9.804	1065.921
198	9.817	1066.449
199	9.830	1066.976
200	9.843	1067.502
201	9.855	1068.028
202	9.868	1068.549
203	9.881	1069.072
204	9.894	1069.590
205	9.907	1070.109
206	9.919	1070.621
207	9.932	1071.131
208	9.945	1071.634
209	9.958	1072.131
210	9.970	1072.615
211	9.983	1073.095
212	9.995	1073.582
213	10.008	1074.109
214	10.021	1074.674
215	10.034	1075.246
216	10.047	1075.806
217	10.060	1076.362
218	10.073	1076.903
219	10.086	1077.453
220	10.099	1077.971
221	10.112	1078.508
222	10.125	1079.051
223	10.138	1079.594
224	10.151	1080.138
225	10.164	1080.663
226	10.176	1081.155
227	10.189	1081.691
228	10.202	1082.227
229	10.215	1082.748
230	10.228	1083.269
231	10.241	1083.788
232	10.253	1084.298
233	10.266	1084.811
234	10.279	1085.345
235	10.292	1085.886
236	10.305	1086.418
237	10.318	1086.940
238	10.330	1087.421
239	10.343	1087.849
240	10.356	1088.365
241	10.369	1088.929
242	10.382	1089.523
243	10.395	1090.112
244	10.408	1090.666
245	10.421	1091.213
246	10.434	1091.767
247	10.447	1092.303
248	10.460	1092.831
249	10.473	1093.352
250	10.485	1093.870

251	10.498	1094.387
252	10.511	1094.900
253	10.524	1095.414
254	10.537	1095.924
255	10.549	1096.435
256	10.562	1096.940
257	10.575	1097.444
258	10.588	1097.941
259	10.600	1098.435
260	10.613	1098.922
261	10.626	1099.411
262	10.638	1099.916
263	10.651	1100.449
264	10.664	1100.984
265	10.677	1101.545
266	10.690	1102.083
267	10.703	1102.633
268	10.716	1103.157
269	10.729	1103.693
270	10.742	1104.220
271	10.755	1104.751
272	10.768	1105.280
273	10.780	1105.805
274	10.793	1106.328
275	10.806	1106.835
276	10.819	1107.338
277	10.832	1107.853
278	10.845	1108.371
279	10.857	1108.882
280	10.870	1109.392
281	10.883	1109.900
282	10.896	1110.414
283	10.909	1110.915
284	10.921	1111.417
285	10.934	1111.925
286	10.947	1112.424
287	10.960	1112.913
288	10.973	1113.406
289	10.985	1113.892
290	10.998	1114.390
291	11.011	1114.899
292	11.024	1115.417
293	11.037	1115.946
294	11.049	1116.480
295	11.062	1116.997
296	11.075	1117.510
297	11.088	1118.026
298	11.101	1118.538
299	11.113	1118.999
300	11.126	1119.474
301	11.139	1120.007

Cameron 'E' 0° Hm = 21'

** TABLE A.5.3.4 -"E" END-ON ***

CAP Inelastic Event Detailed Report
21:08:31 1994 Tue Sep 6

Project: mae Model: model1 Version: 1

mber Name	Load Step	Time	Element Type	Event Description
diag1-2400	12	000.000e-3	Strut	Strut Buckling
PileA2-1816	27	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1817	27	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2217	27	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2216	28	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA1-2711	29	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-2710	30	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
diag1-2404	30	000.000e-3	Strut	Plastic Strut/NLTruss
PileB1-2016	32	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB1-2017	32	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag2-2405	37	000.000e-3	Strut	Plastic Strut/NLTruss
diag3-2409	46	000.000e-3	Strut	Strut Buckling
PileA1-2710	47	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA1-2711	47	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
diag1-2400	47	000.000e-3	Strut	Strut Reloading
diag2-2405	47	000.000e-3	Strut	Elastic
diag3-2409	47	000.000e-3	Strut	Strut Reloading
PileA1-2710	48	000.000e-3	Beam Column	Elastic (0,0)
PileB1-2016	48	000.000e-3	Beam Column	Elastic (0,0)
PileB1-2017	48	000.000e-3	Beam Column	Elastic (0,0)
PileB2-2216	48	000.000e-3	Beam Column	Elastic (0,0)
PileB2-2217	48	000.000e-3	Beam Column	Elastic (0,0)
diag1-2404	48	000.000e-3	Strut	Elastic
diag3-2409	48	000.000e-3	Strut	Strut Buckling
PileA1-2711	49	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1816	49	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	49	000.000e-3	Beam Column	Elastic (0,0)
diag3-2409	49	000.000e-3	Strut	Strut Reloading
PileA1-2710	50	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA1-2710	51	000.000e-3	Beam Column	Elastic (0,0)
PileA1-2711	51	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag1-2404	51	000.000e-3	Strut	Plastic Strut/NLTruss
diag3-2409	51	000.000e-3	Strut	Strut Buckling
PileA2-1816	52	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1817	52	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag1-2404	53	000.000e-3	Strut	Elastic
PileA1-2711	54	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1816	54	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	54	000.000e-3	Beam Column	Elastic (0,0)
diag2-2405	64	000.000e-3	Strut	Plastic Strut/NLTruss
diag1-2400	69	000.000e-3	Strut	Strut Buckling
PileA2-1816	71	000,000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1817	71	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1817	74	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
diag2-2407	75	000.000e-3	Strut	Strut Buckling
PileA2-1816	76	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	76	000.000e-3	Beam Column	Elastic (0,0)
diag1-2400	76	000.000e-3	Strut	Strut Reloading
diag2-2405	76	000.000e-3	Strut	Elastic
diag3-2409	76	000.000e-3	Strut	Strut Reloading
diag2-2405	87	000.000e-3	Strut	Plastic Strut/NLTruss
diag1-2404	98	000.000e-3	Strut	Plastic Strut/NLTruss
diag1-2400	107	000.000e-3	Strut	Strut Buckling
diag3-2409	107	000.000e-3	Strut	Strut Buckling
PileB2-2217	111	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1817	112	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1817	113	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
PileA2-1816	117	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2016	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB1-2017	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2217	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileB2-2216	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2217	122	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileB2-2216	125	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2016	126	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2017	126	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
LegA1-1508	129	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-2711	132	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-2710	133	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2017	135	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileA2-1817	136	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
diag1-2400	137	000.000e-3	Strut	Strut Residual
LegA1-1509	138	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1709	141	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2218	141	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag3-2412	142	000.000e-3	Strut	Plastic Strut/NLTruss
LegA1-1508	148	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1910	149	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
diag2-2407	149	000.000e-3	Strut	Strut Residual
LegA2-1708	152	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegB2-2110	152	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA1-2711	153	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
LegA2-1706	158	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1707	159	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegB1-1907	159	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1806	159	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1807	159	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1705	160	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1706	160	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1707	160	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1507	161	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1506	162	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegB2-2108	162	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1703	163	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	163	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1705	163	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1908	163	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA1-1506	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2107	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2205	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2206	164	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
diag3-2409	164	000.000e-3	Strut	Strut Residual
LegA2-1703	165	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1505	167	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-1606	171	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-1607	172	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1703	175	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	175	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1704	176	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1703	182	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileA2-1809	182	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1808	183	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	199	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1907	201	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1705	203	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	204	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1705	204	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1703	206	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1705	217	000.000e-3	Beam Column	Beam Clmn Initial Tield (1,1)
LegB1-1910	217	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
.ileB1-2005	229	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA1-1504	231	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB1-2006	231	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1705	249	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	250	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1705	250	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	252	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	253	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	254	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2110	264	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
LegA1-1504	268	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	268	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1707	269	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	269	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
LegA2-1706	270	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	270	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
LegA2-1706	271	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1707	272	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1504	276	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	281	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA1-1504	282	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	282	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	282	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1706	282	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	283	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1704	283	000.000e-3	Beam Column	Elastic (0,0)
LegA1-1504	284	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1703	284	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	284	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1706	284	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1703	286	000.000e-3	Beam Column	Elastic (0,0)
LegA1-1504	287	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	287	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1707	287	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	288	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1704	288	000.000e-3	Beam Column	Elastic (0,0)
PileB2-2208	288	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1703	289	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1707	289	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1504	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	290	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	293	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1706	293	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1704	294	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1706	294	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1703	295	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1703	296	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1704	296	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2207	296	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1704	297	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1504	298	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1704	298	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1706	298	000.000e-3	Beam Column	Elastic (0,0)
LegA2-1703	299	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1817	299	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
LegA2-1703	300	000.000e-3	Beam Column	Elastic (0,0)
PileA2-1817	300	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
LegA1-1504	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
egA2-1703	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
egA2-1704	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
egA2-1706	301	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)

*** Table A.5.3.5 "E" Platform Diagonal Approach **

CAP Inelastic Event Short Report Tue Sep 27 18:
10:04 1994

Project: mae Model: model1 Version: 1

Member Name	First Load Step	Yield Time	Element Type	Worst Event Description
LegA1-1503	12	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1504	8	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1505	43	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1506	32	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1507	29	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1508	41	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2103	11	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2104	9	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2105	53	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2106	44	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2107	43	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB2-2108	46	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileA1-2710	16	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA1-2711	11	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
PileA2-1816	24	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA2-1817	21	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
PileB1-2016	20	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2017	16	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
PileB2-2205	292	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2206	289	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2208	10	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileB2-2209	10	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileB2-2216	7	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB2-2217	7	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileB2-2218	100	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag1-2401	87	000.000e-3	Strut	Strut Buckling
diag1-2402	183	000.000e-3	Strut	Plastic Strut/NLTruss
diag2-2408	294	000.000e-3	Strut	Strut Buckling

*** TABLE A.5.3.6 "E" PLATFORM BROADSIDE APPROACH

Heavy deck analysis 90 deg. approach

CAP Inelastic Event Short Report   Mon Sep 26 21:
53:04 1994

Project: mae Model: modell Version: 1

Member Name	First Load Step	Yield Time	Element Type	Worst Event Description
LegA1-1503	169	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1504	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1505	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA1-1506	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1507	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1508	98	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA1-1509	116	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA1-1510	275	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1705	270	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegA2-1706	121	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1707	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegA2-1708	119	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegA2-1710	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
LegB1-1903	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1904	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1905	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1906	117	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1907	117	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
LegB1-1908	122	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegB1-1909	117	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
LegB2-2107	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
LegB2-2108	119	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
LegB2-2110	120	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA1-1606	276	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA1-1607	278	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA1-2710	25	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA1-2711	24	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,2)
PileA2-1805	171	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileA2-1806	171	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileA2-1816	29	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileA2-1817	29	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileB1-2006	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,1)
PileB1-2007	118	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB1-2008	137	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB1-2009	135	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB1-2016	5	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB1-2017	5	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileB1-2018	124	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2205	119	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2206	119	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2207	212	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
PileB2-2208	206	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,0)
PileB2-2216	5	000.000e-3	Beam Column	Beam Clmn Initial Yield (2,0)
PileB2-2217	5	000.000e-3	Beam Column	Beam Clmn Initial Yield (1,2)
PileB2-2218	103	000.000e-3	Beam Column	Beam Clmn Initial Yield (0,1)
diag1-2401	9	000.000e-3	Strut	Strut Residual
diag1-2402	24	000.000e-3	Strut	Plastic Strut/NLTruss
diag2-2406	45	000.000e-3	Strut	Plastic Strut/NLTruss
diag2-2408	57	000.000e-3	Strut	Strut Residual
diag3-2410	31	000.000e-3	Strut	Strut Residual
diag3-2411	108	000.000e-3	Strut	Plastic Strut/NLTruss
diag4-2413	195	000.000e-3	Strut	Plastic Strut/NLTruss

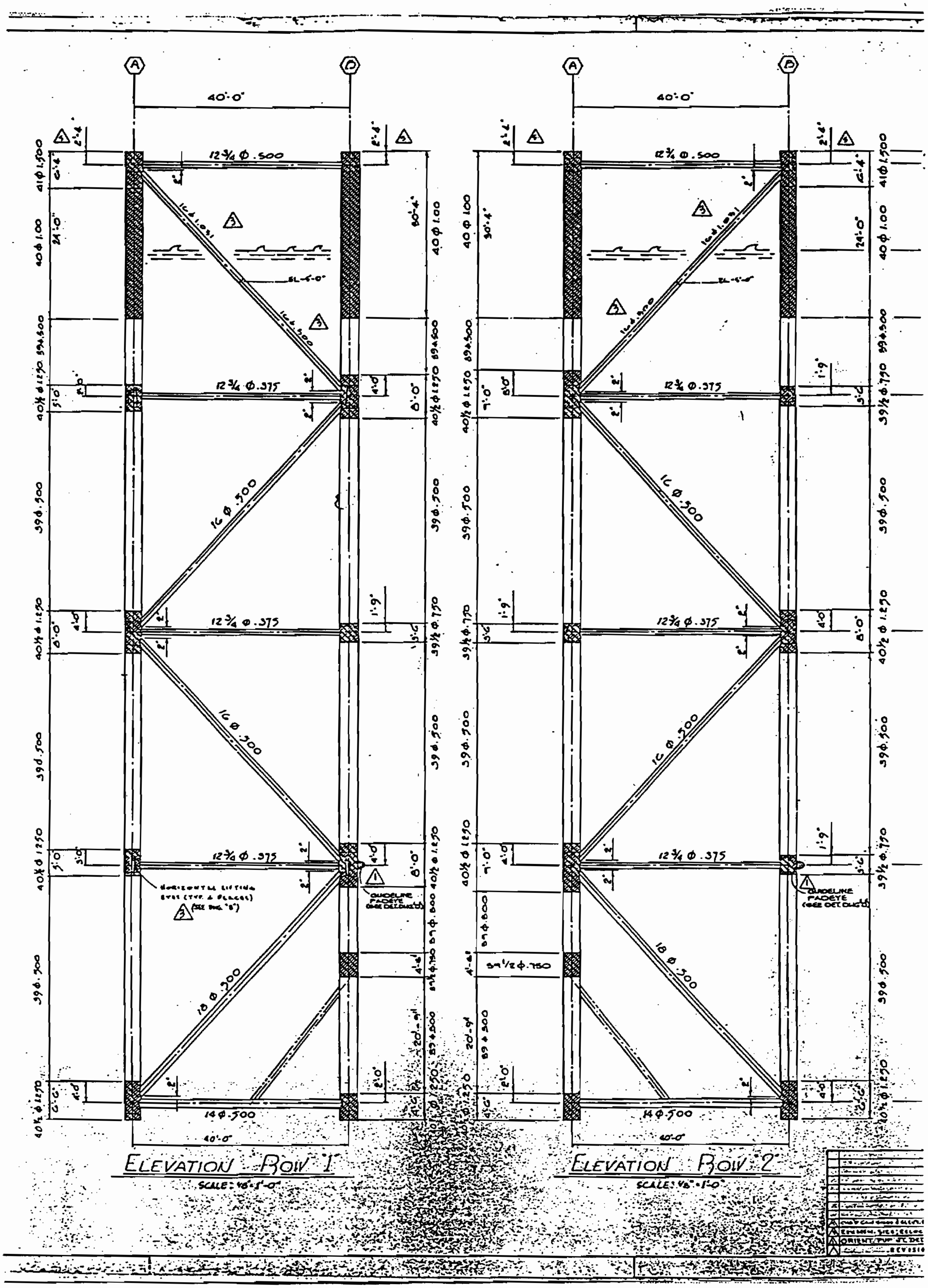


FIGURE 1.1 "E" PLATFORM

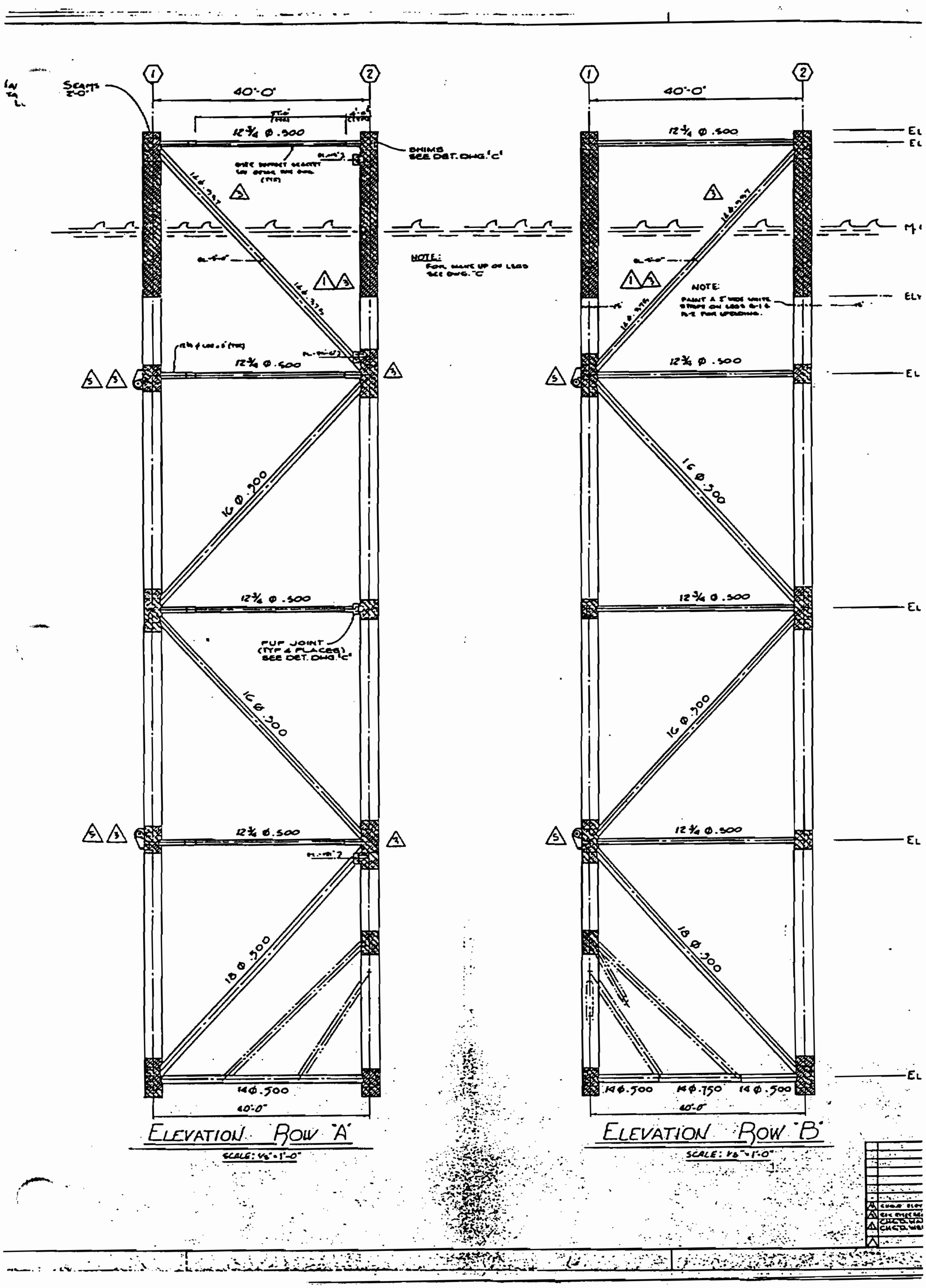


FIGURE 1.2 "E" PLATFORM

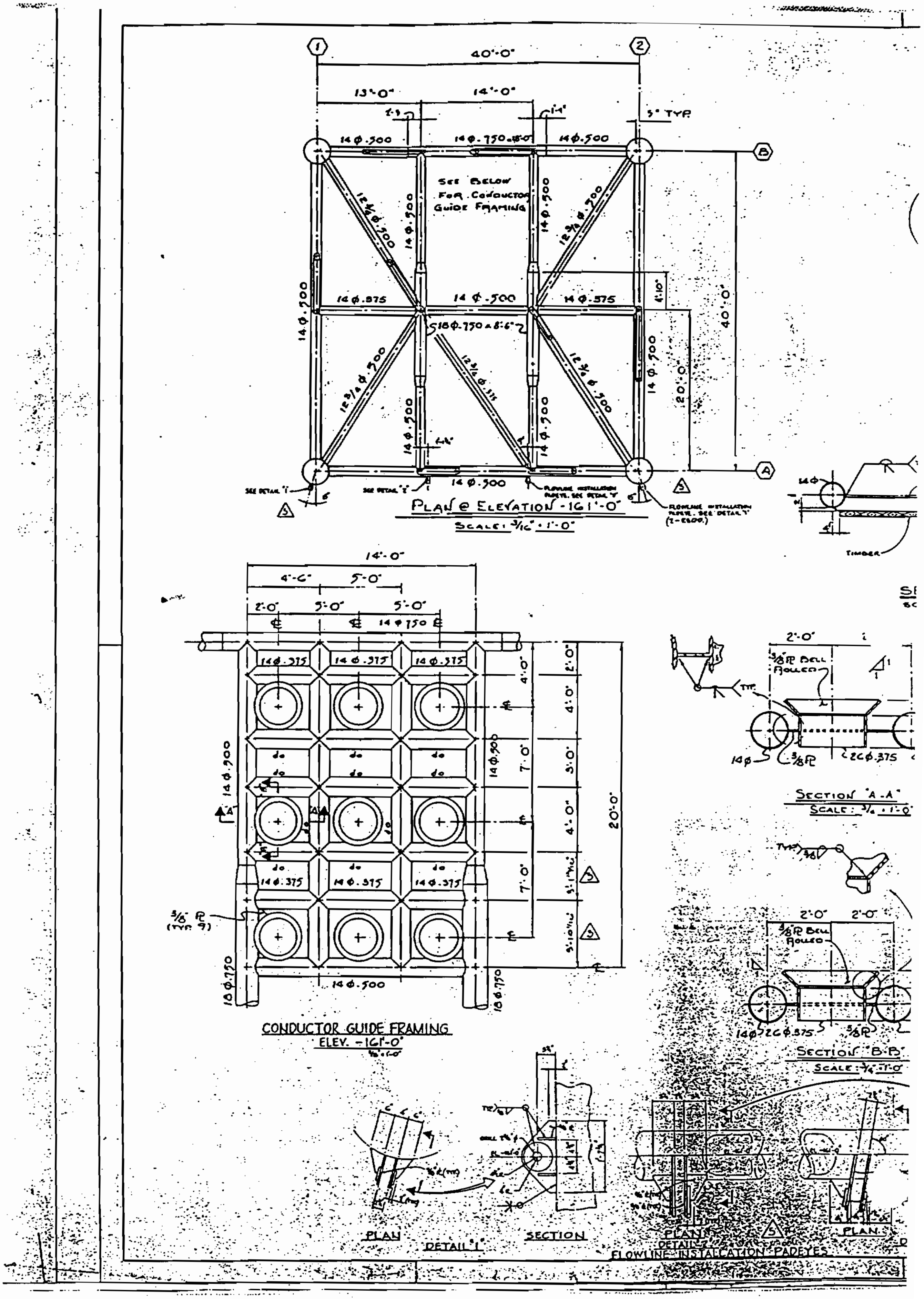
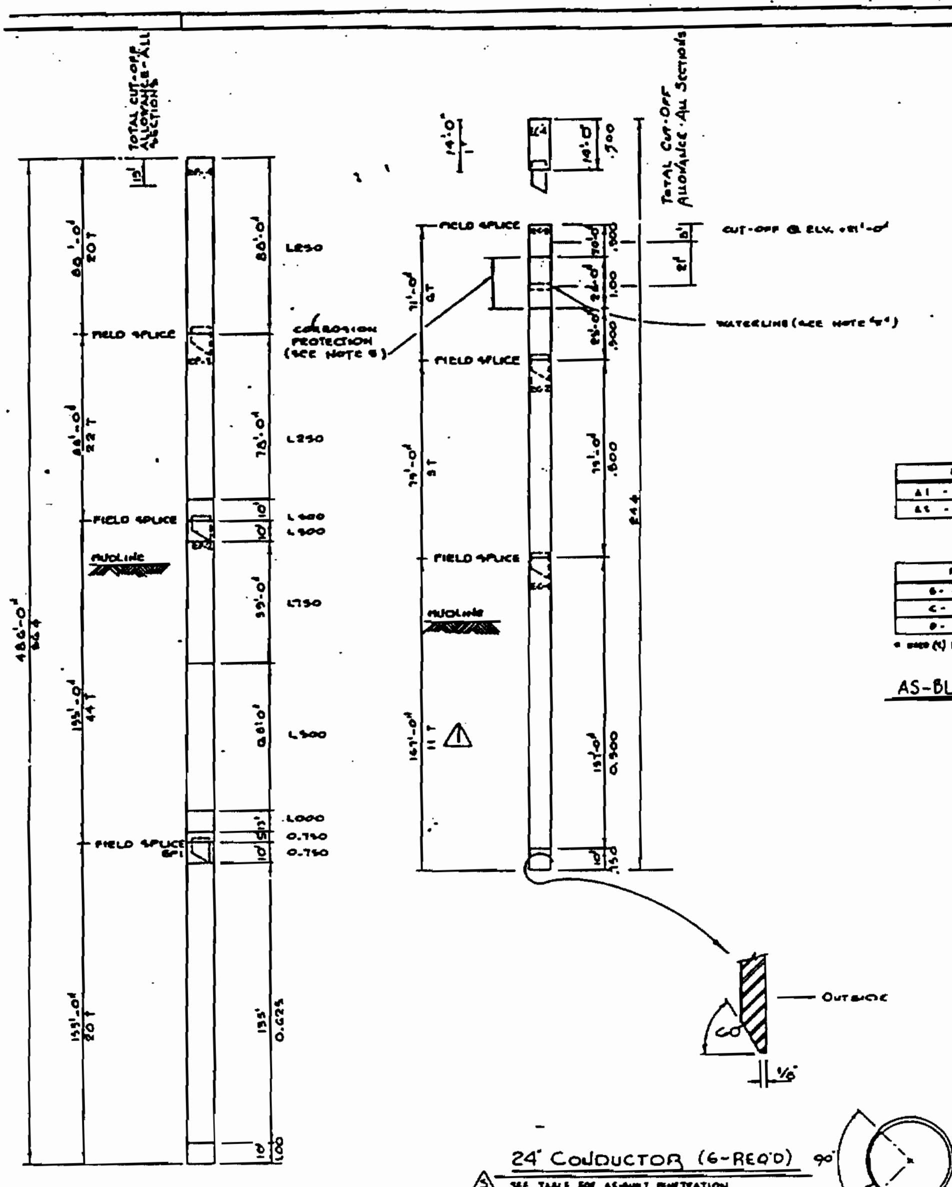


FIGURE 1.4 "E" PLATFORM



FINAL PILE PENETRATIONS	
A1 - 292'	B1 - 292'
A2 - 291'	B2 - 290'

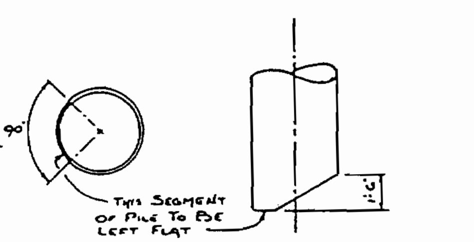
FINAL CONDUCTOR PENETRATIONS	
B - 130'	E - 124'
C - 130'	H - 129'
D - 205' *	I - 129'

* USED (X) EC-1 SECTIONS IN MAKE-UP.

AS-BUILT PENETRATIONS (5)

24" CONDUCTOR (6-REQ'D)

(5) SEE TABLE FOR AS-BUILT PENETRATION



TYPICAL PILE BOTTOM DETAIL

36" PILE (4-REQ'D)

(5) SEE TABLE FOR AS-BUILT PENETRATION

GENERAL NOTES

1. ALL WELDS SHALL BE FULL PENETRATION UNLESS OTHERWISE NOTED. SEE TYPICAL JOINT DETAILS, STEEL & WELDING SPEC. CE-00-EX FOR PIPE MILL AND FABRICATION YARD BUTT WELD DETAILS & STEEL REQUIREMENTS.
2. UNLESS OTHERWISE QUALIFIED & APPROVED BY PECTEN ALL OFFSHORE STRUCTURAL WELDS SHALL BE WITH E7018 LOW HYDROGEN ELECTRODES, DRIED 2 HRS. AT 450° F., STORED IN OVENS AT 250° MIN. & LIMITED TO 8 HRS EXPOSURE. MINIMUM PRE-HEAT SHALL BE 200° F.
3. ALL CIRCUMFERENTIAL WELDS IN TOP 30 FEET OF PILE SHALL BE GROUND FLUSH BY FABRICATOR
4. PILE AND CONDUCTORS SHALL BE FABRICATED TO SPECIFIED LENGTH ± 3 IN. FOR EACH SECTION, OR AS REQUIRED TO MAINTAIN A TOLERANCE OF ± 1 FOOT ON TOTAL LENGTH.
5. FABRICATOR SHALL PROVIDE THE FOLLOWING 2" WIDE PAINT STRIPES AROUND EACH CONDUCTOR (OR PILE) AS INDICATED ON THE DRAWING:

   WHITE STRIPE AT WATERLINE

   RED STRIPE TO BE USED FOR LOCATING CORROSION PROTECTION DURING CONDUCTOR INSTALLATION

   YELLOW TO INDICATE PICK-UP POINT (TO BE ACCOMPANIED BY LEGEND "PICK-UP-POINT" MAKE 6" HIGH LETTERS)
6. TOP OF EACH PILE & CONDUCTOR SHALL BE MARKED WITH 6" HIGH PAINTED IDENTIFICATION AS SHOWN ON DRAWING.
7. FABRICATOR TO PAINT TOP OF ALL CONDUCTORS DOWN TO 25' BELOW INDICATED WATERLINE

FIGURE 1.9 "E" PLATFOR[M]

(5)	REV. PILE & COND. (AS BUILT)			
	REV. COND. WT.			4-13-81
	REVISION	CONSULTANT	PECTEN	DATE

RELEASED FOR

	CONSULTANT ENGR.	CH. ENG.	DATE
BIDDING			
FABRICATION			
INSTALLATION			
DRAWN BY			
CHK.			

DATE 1-30-81

PILES & CONDUCTORS
MOKOKO EAST DRILLING PLATFORM "E"
MOKOKO-ABANA FIELD
4 PILES 9 WELLS 141' WATER
PECTEN APP'D DATE 1-20-81

CE-80-6H (5)

FIGURE A.5.3.1 – ENDON DIRECTION – "E" PLATFORM

Load at 1st component I.R.=1.0 (S1) = 405 kips

100 year, 20th ed. ref. load (Sref) = 493 kips

Ultimate Strength Analysis Load (Susa) = 766 kips

Ultimate Capacity (Ru) = 1107 kips

Reserve Strength Ratio = 2.2

Platform Failure Mode = Diagonal

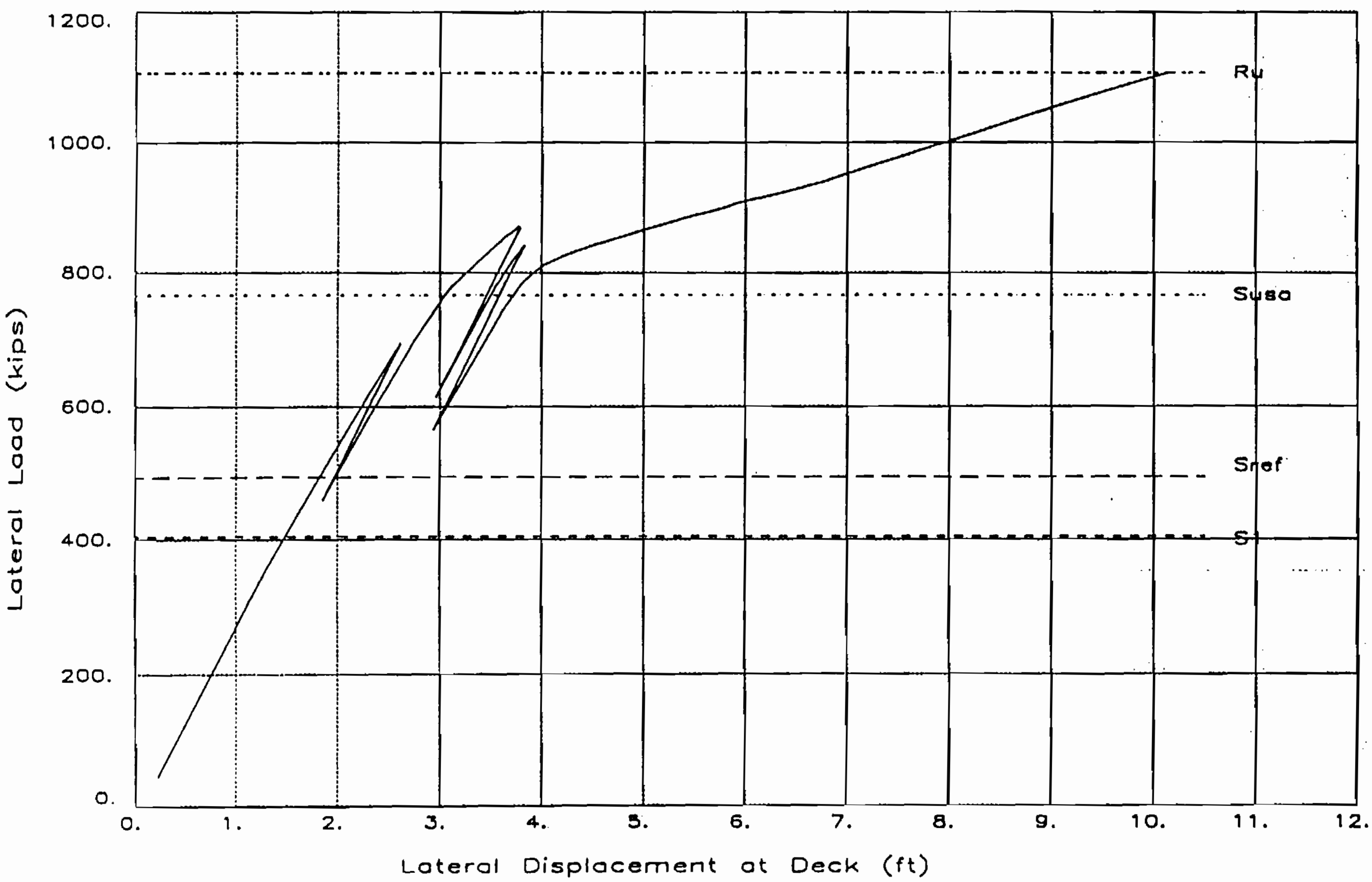


FIGURE A.5.3.2 – DIAGONAL DIRECTION – "E" PLATFORM

Load at 1st component I.R.=1.0 (S1) = 596 kips

100 year, 20th ed. ref. load (Sref) = 515 kips

Ultimate Strength Analysis Load (Susa) = 762 kips

Ultimate Capacity (Ru) = 1116 kips

Reserve Strength Ratio = 2.1

Platform Failure Mode = Diagonal

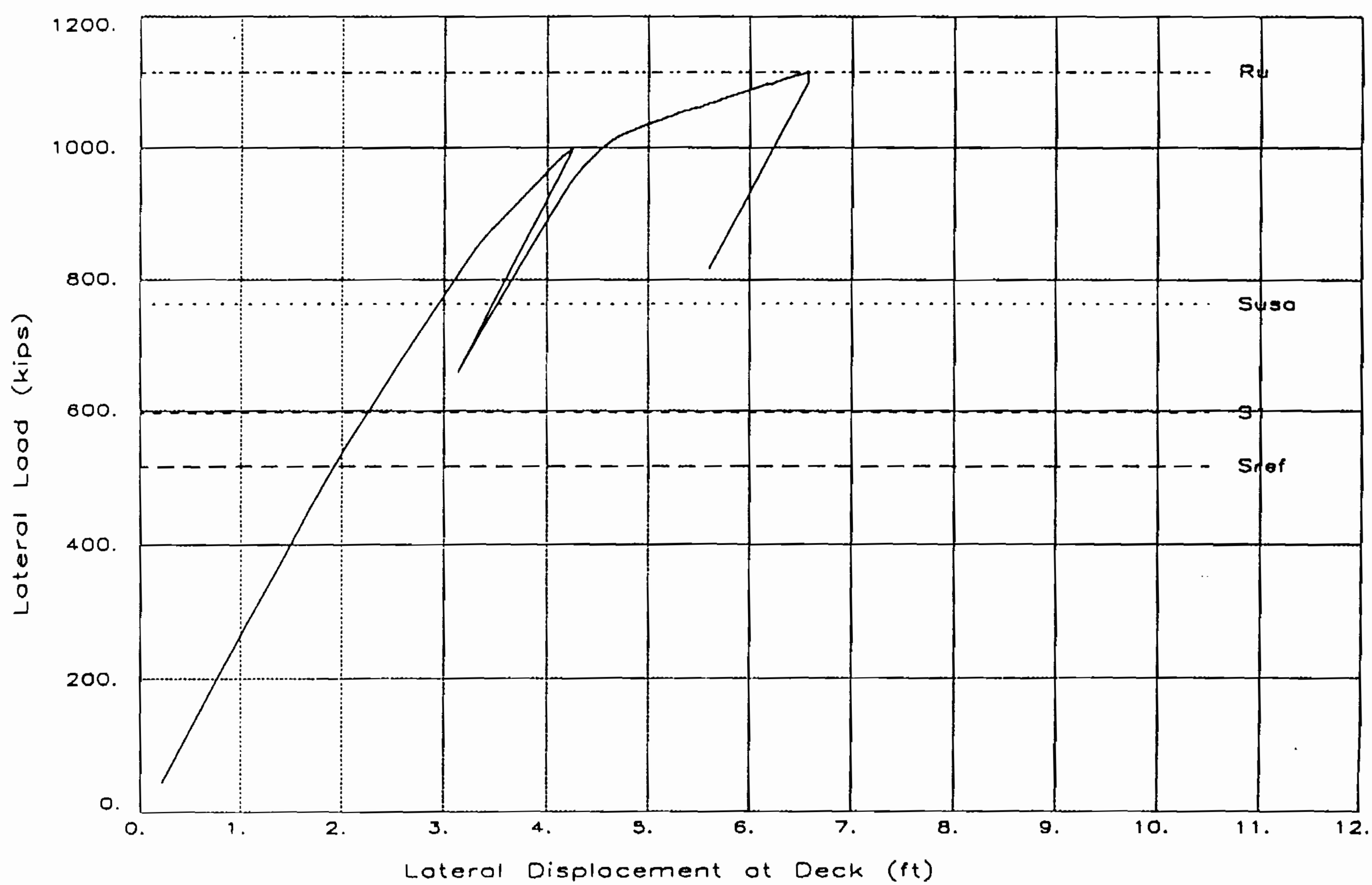


FIGURE A.5.3.3 – BROADSIDE DIRECTION – "E" PLATFORM

Load at 1st component I.R.=1.0 (S1) = 405 kips

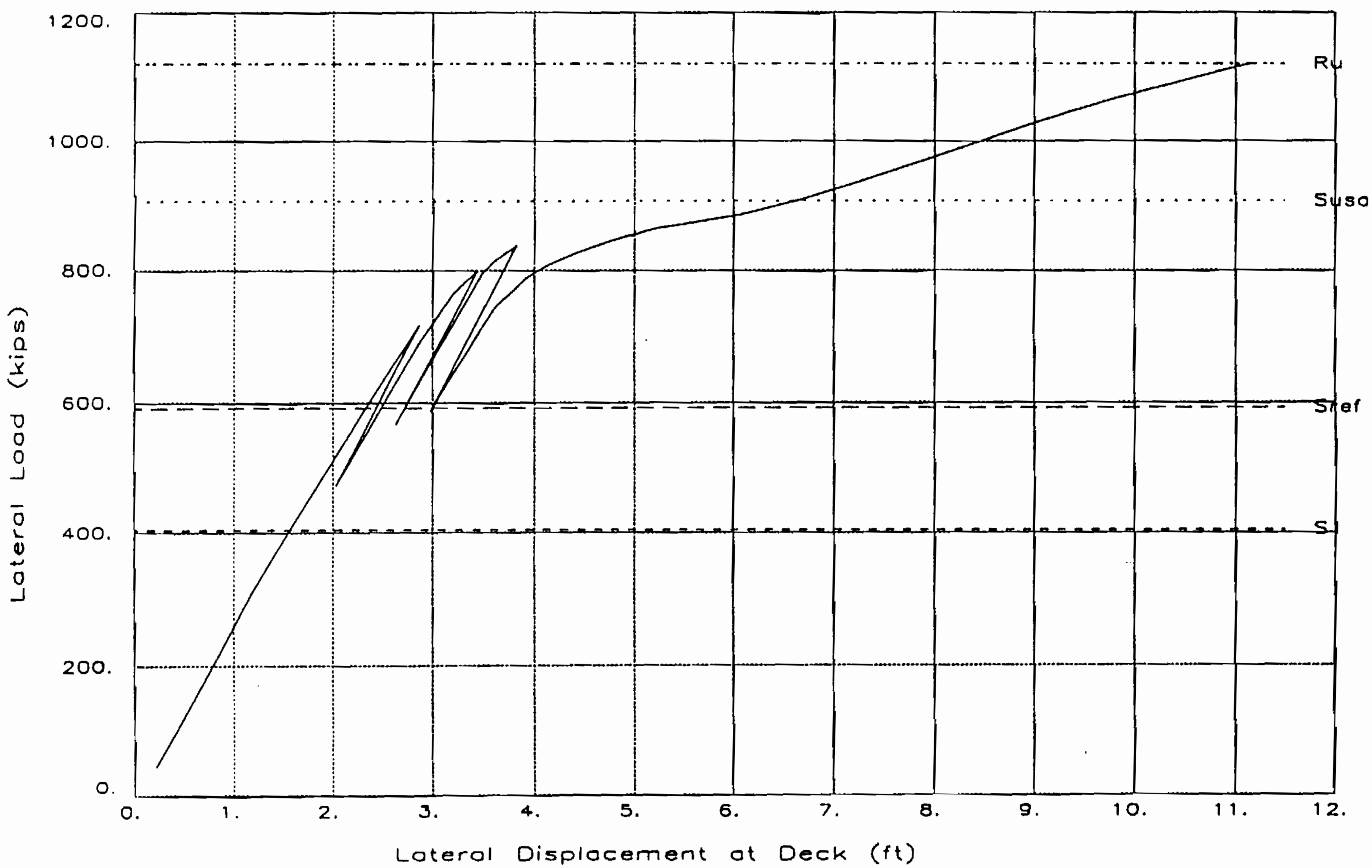
100 year, 20th ed. ref. load (Sref) = 592 kips

Ultimate Strength Analysis Load (Susa) = 907 kips

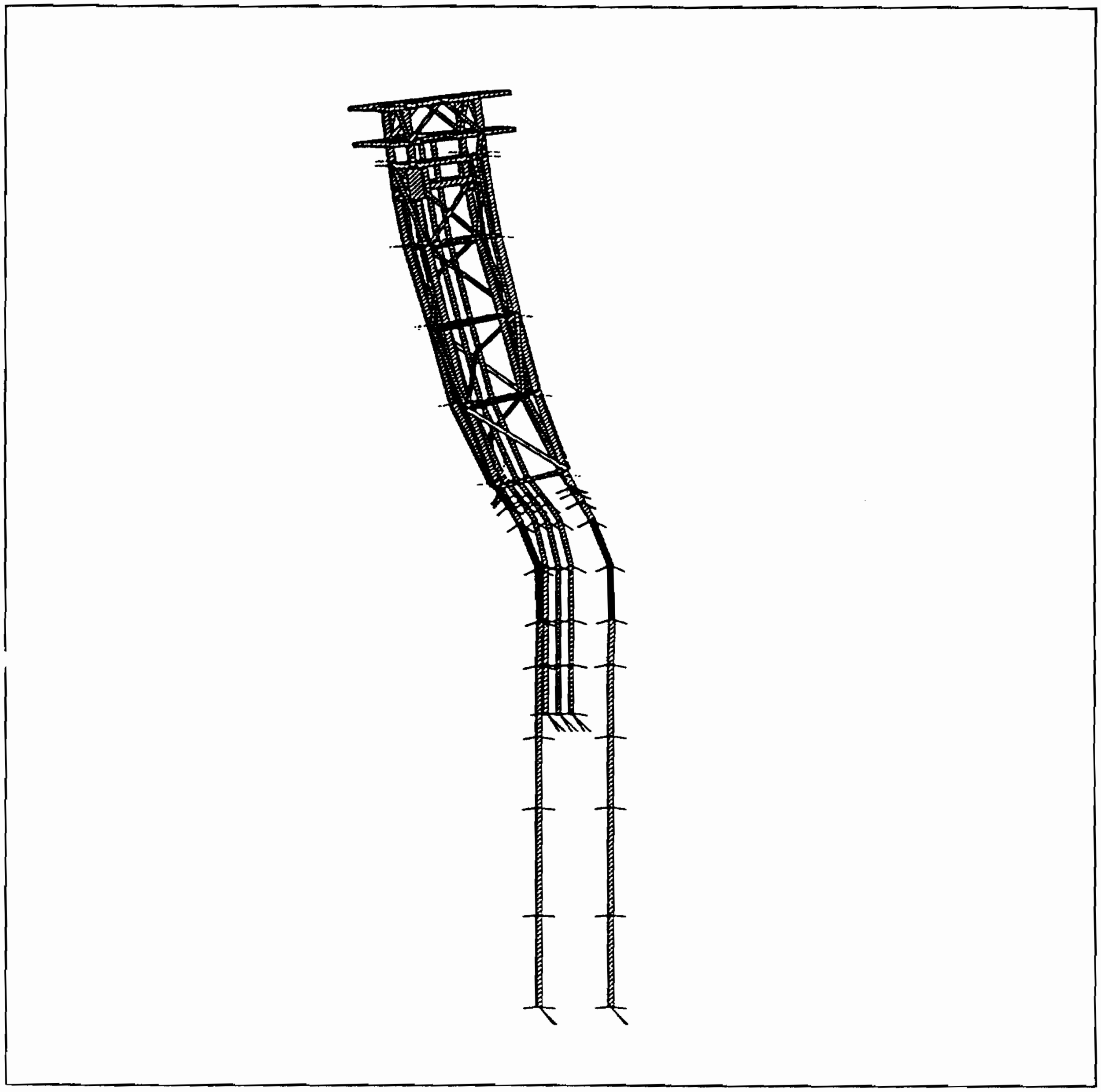
Ultimate Capacity (Ru) = 1120 kips

Reserve Strength Ratio = 1.9

Platform Failure Mode = Diagonal



Project: mae Model: model1 Version: 1 Thu Sep 8 07:36:47 1994 *



CAP --1 CAMEROON E AT 90 DEG FIGURE A.5.3.4

Inelastic Events Legend

	Elastic		Strut Buckling
	Strut Residual		Strut Reloading
	Plastic Strut/NLTruss		Beam Clmn Initial Yield
	Beam Clmn Fully Plastic		Fracture

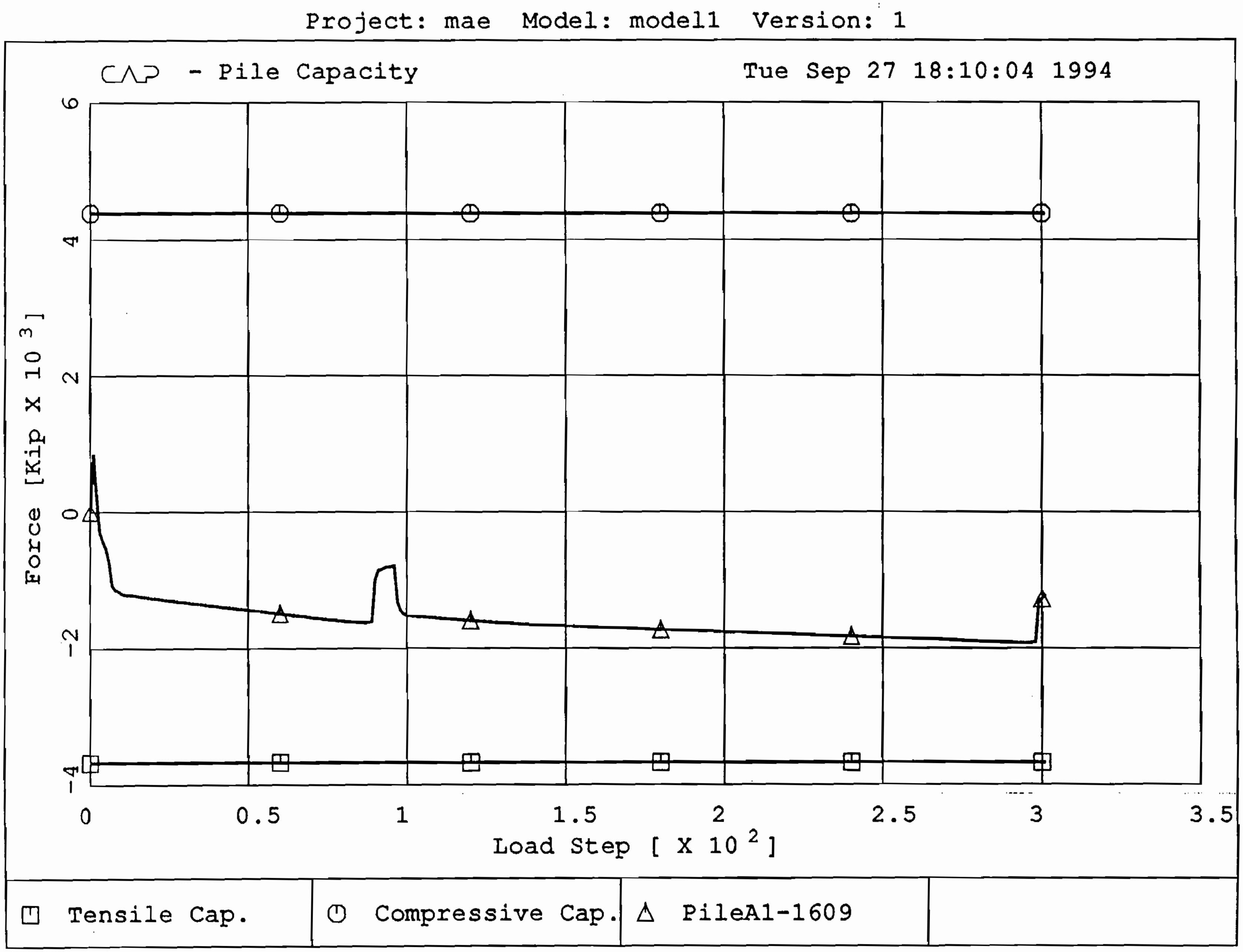


Figure A.5.3.5 "E" PILE CAP. @45

Project: mae Model: model1 Version: 1

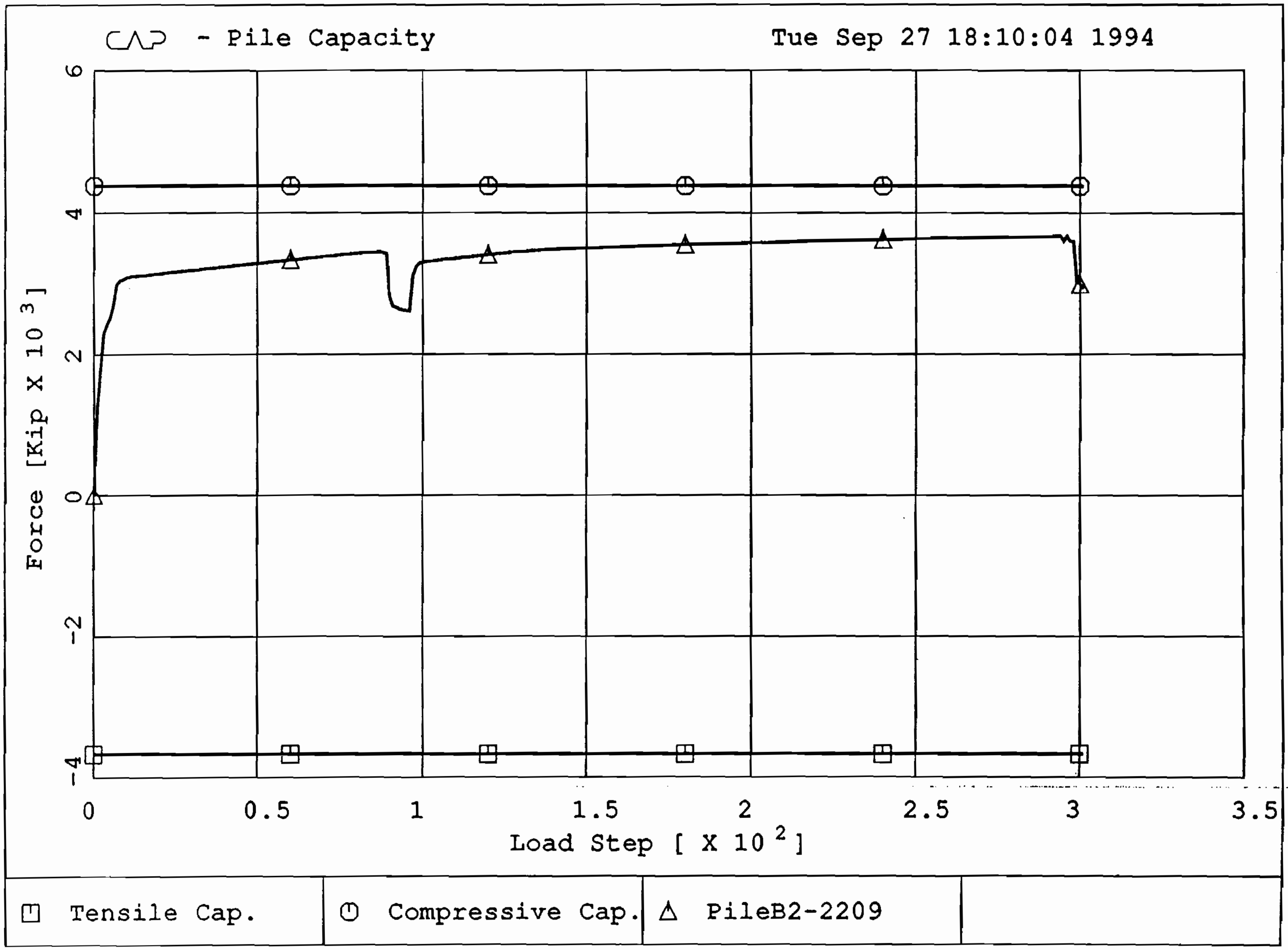


Figure A.5.3.6 "E" PILE CAP. @45

Attachment 1

Design Level Analysis - Cameroon Platforms "E", "G" & "K"

ASSESSMENT OF CAMEROON PLATFORMS "E", "G" & "K"

I. INTRODUCTION

The 1992 underwater inspection of Cameroon platforms identified high marine growth and scour conditions. requested an evaluation of the effects of these conditions on the "E", "G" and "K" platforms. The "E" and "G" platforms were selected for evaluation because they were recently modified to support additional drive pipes and a riser protector. The "K" platform was selected because of high scour conditions.

To assess the existing condition of the platforms, a reanalysis was performed utilizing the revised wave force calculation methodology from RP2A 20th edition. The revised methodology to determine wave force was used because it represents industry's best estimate of the actual force. The new wave force calculation method produces a significant increase in lateral platform loading.

All of the Cameroon platforms were designed prior to the revision in the wave force calculation of RP2A, including the modifications to the "E" and "G" platforms. The approach used for this study was to evaluate the combined effects of the higher forces and the as surveyed conditions with the assessment procedures described in the Draft Section 17.0 of API RP2A-WSD.

II. SCOPE

The scope was to perform the assessment procedure outlined in Draft Section 17.0 of API RP2A. The results of earlier analyses (references 1, 2 & 3) were used along with limited new analyses.

The need for the assessment was established because the increase in marine growth and the effects of the scour were judged to be significant. The assessment procedure is shown in the attached Figure 17.2.2 from RP2A and can be summarized as follows for the Cameroon platforms:

a. Determine the exposure category for the platforms for environment and life safety.

b. Check for adequate deck height.

c. If the deck height is adequate, perform a design level analysis. The design level analysis includes a 100 year return period environmental loading and the wave force methodology from RP2A 20th ed.

d. Evaluate the results of the design level analyses.

If the platforms meet or exceed the minimum assessment requirements, then more detail analyses are not required.

e. If the platforms do not met the minimum requirements, then detailed ultimate strength analyses would be recommended.

III. CRITERIA

Following the methodology used to develop the assessment criteria in RP2A Section 17.0 for Gulf of Mexico and U.S. West Coast,). developed criteria for the assessment of the Cameroon platforms (see attached note dated 18 March 1994). The recommended criteria for design level analyses are as follows:

Unmanned, Insignificant Environmental Impact Platforms:
60% of lateral loading of 100 year environmental conditions.

Manned, Insignificant Environment Impact Platforms:
85% of lateral loading of 100 year environmental conditions.

A 21 foot maximum wave height with a 16 second period was used for the 100 year return period wave. This was the 100 year wave used for the original design. The attached note, from recommends a 18.4 foot/16 second wave for the 100 year wave based on a recent joint industry study for West Africa. However, since the assessment analyses had been already been completed with the higher 21' wave and the platforms passed the assessment, the analyses were not repeated with the lower 18.4' wave.

The criteria used in analyses are shown in Table 1.

IV. ASSESSMENT RESULTS

1.0 Condition Assessment

Platforms "E", "G" and "K" were inspected to API RP2A level III survey standards in 1992. The findings of the inspection were the basis for this assessment.

2.0 Categorization

Following the guidelines in section 17.5 of RP2A, the platforms were categorized as follows:

"E" - Manned, non-evacuated, insignificant environmental impact. The "E" platform supports a small quarters for about 6 men. The platform is normally manned with 1 or 2 lease operators and there has been some discussion of removing the quarters. However, it was conservatively categorized as manned for this study. The 100 year storm can not be adequately forecast to allow for evacuation, so the platform was considered non-evacuated.

"G" - Unmanned, insignificant environmental impact. The "G" platform does support production facilities, but is categorized as insignificant environmental impact considering the safety shutdown measures. There are no quarters.

"K" - Unmanned, insignificant environmental impact. The "K" platform supports only test facilities with no quarters.

The general area of the Cameroon Mokoko Abana field supports a large number of oil platforms from several companies. While Pecten's platforms operate without spillage or pollution, it is not uncommon to see oil spillage in the area from other companies' platforms. With such operations in the area, there is no indication that the Pecten platforms are in an environmentally sensitive area. Therefore, the platforms are categorized as having insignificant environmental impact.

3.0 Deck Height Check

The three platforms were checked for deck height clearance using a 21 foot/16 second wave. The results are as follows:

Platform	Water Depth	Storm Tide	Crest Elev.	Deck Height	Air Gap
"E"	161'	7'	+11.5'	+32'	13.5'
"G"	151'	7'	+11.5'	+20'	1.5'
"K"	157'	7'	+11.5	+28'	9.5'

All platform have a positive air gap which means that the 100 year wave will not hit the deck. The deck height values above are based on the bottom of steel of the lower deck and include as-built information.

4.0 Design Level Analysis Results

In-place analyses were performed on the platforms for the 100 year environmental conditions listed in Table 1 and in accordance the calculation methods of RP2A 20th edition. The governing stress ratios for the platforms are as follows:

"E" - Max. stress ratio = 0.97 (diagonal on Row 1)

"G" - Max. stress ratio = 1.21 (diagonal on Row 1)

"K" - Max. stress ratio = 1.11 (diagonal on Row B)

Assuming that the stress in the diagonal members are linear with lateral loading, the platforms pass the assessment for the following % of lateral loading caused by 100 year environmental conditions:

"E" = 103% of RP2A 20th ed. loading (Required = 85%)

"G" = 83% of RP2A 20th ed. loading (Required = 60%)

"K" = 90% of RP2A 20th ed. loading (Required = 60%)

The platforms were checked for the following:

Member in-place stress and hydrostatic collapse assuming flooded and buoyant conditions.

Joint in-place strength (simplified fatigue was also checked but is not required for the assessment).

Axial pile capacity & Combined lateral and axial pile in-place stress.

One third stress increases were included in all component checks. Details of these checks are discussed below:

Members

Tables 2 - 4 show the governing member stresses for platforms "E", "G" and "K" respectively. An effective length factor (K) of 0.65 was used for the vertical diagonals. K values of 1.0 and 0.8 (0.7 for minor bracing) were used for the legs and horizontals respectively.

The members were checked for the in-place stresses assuming they were buoyant or flooded. Hydrostatic collapse was also checked but did not govern.

Joints

All joints on the "E" and "G" platforms had stress ratios less than 1.0 for the joint strength checks. The maximum joint stress ratio for the "K" platform was 1.09 for the strength checks. Several joints on the platforms had high stress ratios for the joint simplified fatigue checks. Tables 5 - 7 list the governing joint stress ratios for strength and simplified fatigue. Draft section 17.7.4e of RP2A states that no additional analytical demonstration of future fatigue life is required if Level III underwater surveys are made and no fatigue damage is observed. Since these platforms were inspected to Level III in 1992 with no observed fatigue damage, no further analytical studies are required. However, the joint fatigue lives are a concern that needs to be addressed through continual underwater inspections.

Piles

The maximum reactions from the 100 year RP2A 20th ed. loadings are listed below with ultimate pile capacity.

Platform	Reaction	Ultimate Capacity	Factor of Safety
"E"	2397 kips	4400 kips	1.83
"G"	2521 kips	4200 kips	1.66
"K"	2154 kips	2700 kips	1.35

Tension load cases did not govern the minimum factor of safety.

As shown above, the "K" platform does not meet the minimum factor of safety of 1.5 recommended by RP2A for design environmental conditions, but the factor of safety is greater than 1.0. Additionally, the use of a 21 foot maximum wave height appears to be conservative.

The piles were also checked for combined axial and bending stress due to in-place loadings. The resulting governing stress ratios are summarized below:

"E" - S.R. max. = 0.84

"G" - S.R. max. = 1.16

"K" - S.R. max. = 0.98

V. SUMMARY

All three platforms pass the assessment procedure described in the Draft Section 17 of API RP2A. The platforms do not have the same factors of safety that would be included in new designs in accordance with API RP2A 20th edition. However the existing factors of safety are adequate and no remedial measures are recommended.

This assessment analysis did not include some conservative measures that are normal in new designs, such as using a conservative K value of 0.8 for diagonal members. However the use of the K value of 0.65 in this assessment is justified and can be shown to be conservative with more detailed analysis and when compared with model tests. Other conservative measures were included such as using nominal yield stress and using member lengths based on the joint center to center dimensions. An additional conservatism may also exist in the selection of the 100 year wave height.

Extra care should be taken to avoid adding extra wave force to platforms "G" and "K". Adding equipment to the +16' elevation on the platforms should not be allowed. Future underwater inspections should include the critical fatigue joints identified in this study.

REFERENCES

1. s, "Reanalysis of "G", "E" & "K" Platforms" 1993 Design Notebook.

2. , "Mokoko Abana Platform "K" Design", 1989 Design Notebook.

3. , "Cameroon MAA "G" Platform Reanalysis", 1992 Design Notebook.

4. Blodgett, O.W., Design of Welded Structures, James F. Lincoln Arc Welding Foundation, Cleveland Ohio, 1966.

5. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design, 20th ed., American Petroleum Institute, 1993.

Table 1 - Analysis Criteria - 100 year return period

Wave: Hmax = 21 feet T = 16 seconds

Current: 5.1 feet/second at surface decreasing to zero at the mudline.

85% current blockage

Wind: 81 mph (gust), 63 mph (sustained)

Wave Coefficients:

Cd (smooth) = 0.65
Cm (smooth) = 1.6

Cd (rough) = 1.05
Cm (rough) = 1.2

Kinematics Coefficient = 1.0

No conductor shielding

Marine Growth:

"G" = 3.5" radial from -70′ to water surface/1" below -70′ to the mudline.

"E" = 2" radial from -80′ to water surface/.2" below -80′ to the mudline.

"K" = 2.1" radial from -80′ to water surface/3" below -80′ to the mudline.

Scour:

"G" = no scour

"E" = 4 foot

"K" = 8 foot

Effective length (K) factors and yield strength:

A K factor of 0.65 was used for vertical diagonals framing into the legs. Lengths were based on center to center dimensions between joints.

K factors for other members conformed to the table in 3.3.1d of RP2A.

Yield strengths were set at nominal yield with no increase for actual strengths from mill certificates.

Table 2 "E" Platform Member Stress Summary

Number	Location	S.R.(K=.8)	S.R.(K=.65)
2101	Row A diag. +16' to -29'	1.08	0.77
2102	Row B diag. +16' to -29'	1.06	0.76
2111	Row A diag. -73' to -117'	1.24	0.90
2112	Row B diag. -73' to -117'	1.33	0.93
2113	Row 1 diag. -73' to -117'	1.55	0.97
2114	Row 2 diag. -73' to -117'	1.22	0.89

Notes:

1. Center to center lengths used.

2. Nominal yield strengths used (36 ksi).

3. Hydrostatic collapse stress ratios were less than 1.0.

4. S. R. = stress ratio

5. Effective length factor (K) of 0.65 was used in the study for vertical diagonals. A K factor of 0.8 is recommended by RP2A for new designs and is shown for information.

Table 3 "G" Platform Member Stress Summary

Number	Location	S.R.(K=.8)	S.R.(K=.65)
2101	Row A diag. +16′ to -26′	1.67	1.03
2102	Row B diag. +16′ to -26′	9999	1.03
2105	Row A diag. -26′ to -68′	1.19	0.88
2106	Row B diag. -26′ to -68′	1.40	0.96
2107	Row 1 diag. -26′ to -68′	1.47	0.98
2108	Row 2 diag. -26′ to -68′	1.27	0.92
2111	Row A diag. -68′ to -110′	4.75	1.07
2112	Row B diag. -68′ to -110′	9999	1.19
2113	Row 1 diag. -68′ to -110′	9999	1.21
2114	Row 2 diag. -68′ to -110′	9999	1.20
2115	Row A diag. -110′ to -151′	1.25	1.02
2116	Row B diag. -110′ to -151′	1.47	1.12
2117	Row 1 diag. -110′ to -151′	1.47	1.14
2118	Row 2 diag. -110′ to -151′	1.47	0.99
		(K=1.0)	
2013	Leg A1 +16′ to -26′	1.09	
2033	Leg B1 +16′ to -26′	1.13	
2023	Leg A2 +16′ to -26′	1.19	

Notes:

1. Center to center lengths used.
2. Nominal yield strengths used (36 ksi).
3. Hydrostatic collapse stress ratios were less than 1.0.
4. S.R. = stress ratio
5. S.R. = 9999 = buckling load exceeded
6. Effective length factor (K) of 0.65 was used in the study for vertical diagonals. A K factor of 0.8 is recommended by RP2A for new designs and is shown for information.

Table 4 "K" Platform Member Stress Summary

Number	Location	S.R.(K=.8)	S.R.(K=.65)
1203	Row A diag. -98′ to -156′	1.03	0.90
1212	Row B diag. -41′ to -98′	1.16	0.99
1213	Row B diag. -98′ to -156′	1.29	1.11
1222	Row 1 diag. -41′ to -98′	1.04	0.90
1232	Row 2 diag. -41′ to -98′	1.06	0.92

Notes:

1. Center to center lengths used.

2. Nominal yield strengths used (36 ksi).

3. Hydrostatic collapse stress ratios were less than 1.0.

4. S.R. = stress ratio

5. Effective length factor (K) of 0.65 was used in the study for vertical diagonals. A K factor of 0.8 is recommended by RP2A for new designs and is shown for information. The K factor for member 1213 was calculated to be 0.55 based on the method described in "Design of Welded Structures" by Boldgett.

Table 5 "E" Platform Joint Stress Summary

#	Location	Strength S.R.	Fatigue S.R.
102	A2 @ +16′	.18	1.16
103	B1 @ +16′	.18	1.60
163	B1 @ -4′	.24	2.00
164	B2 @ -4′	.23	1.90
202	A2 @ -29′	.40	1.35
203	B1 @ -29′	.42	1.39
204	B2 @ -29′	.27	1.71
301	A1 @ -73′	.52	1.45
302	A2 @ -73′	.18	1.18
303	B1 @ -73′	.21	1.37
304	B2 @ -73′	.50	1.39
402	A2 @ -117′	.48	1.27
403	B1 @ -117′	.52	1.38
501	A1 @ -161′	.46	1.42
502	A2 @ -161′	.34	1.59
503	B1 @ -161′	.32	1.34
504	B2 @ -161′	.41	1.69
141	Hor. jt @ +16′	.15	1.17
207	Hor. jt @ -29′	.47	1.49
209	Hor. jt @ -29′	.42	1.29
309	Hor. jt @ -73′	.40	1.33

Notes:

1. The following allowable hot spot stresses were used for the simplified fatigue checks based on 40 year life and API X S-N curve:

	Waterline Mbrs.	Other Mbrs.
"E"	31.5 ksi	34.5 ksi

2. S. R. = Stress ratio

Table 6 "G" Platform Joint Stress Summary

#	Location	Strength S.R.	Fatigue S.R.
102	A2 @ +16'	.27	1.14
103	B1 @ +16'	.27	1.52
202	A2 @ -26'	.53	1.17
203	B1 @ -26'	.56	1.21
204	B2 @ -26'	.28	1.48
301	A1 @ -68'	.67	1.19
303	B1 @ -68'	.21	1.18
304	B2 @ -68'	.67	1.17
402	A2 @ -110'	.67	1.15
403	B1 @ -110'	.75	1.28
501	A1 @ -151'	.65	1.36
502	A2 @ -151'	.45	1.42
503	B1 @ -151'	.43	1.42
504	B2 @ -151'	.60	1.67
207	Hor. jt @ -26'	.54	1.29
209	Hor. jt @ -26'	.43	1.22
407	Hor. jt @ -110'	.45	1.12
506	Hor. jt @ -151'	.40	1.54
507	Hor. jt @ -151'	.38	1.28
509	Hor. jt @ -151'	.30	1.06
512	Hor. jt @ -151'	.28	1.19

Notes:

1. The following allowable hot spot stresses were used for the simplified fatigue checks based on 40 year life and API X S-N curve:

	Waterline Mbrs.	Other Mbrs.
"G"	32.5 ksi	35.4 ksi

2. S. R. = Stress ratio

Table 7 "K" Platform Joint Stress Summary

#	Location	Strength S.R.	Fatigue S.R.
201	A1 @ +11′	.15	1.21
302	A2 @ +7′	.19	1.28
501	A1 @ -4′	.25	1.58
502	A2 @ -4′	.22	1.52
602	A2 @ -41′	.26	1.38
603	B2 @ -41′	.23	1.41
604	B1 @ -41′	.28	1.61
704	B1 @ -98′	.17	1.11
701	A1 @ -98′	.38	0.85
801	A1 @ -156′	.30	0.86
802	A2 @ -156′	.37	0.92
709	Hor. jt @ -98′	.20	1.04
735	Hor. jt @ -98′	.39	1.18
831	Hor. jt @ -156′	.29	1.12
832	Hor. jt @ -156′	.26	1.26
841	Hor. jt @ -156′	1.09	3.00
842	Hor. jt @ -156′	.73	2.50
843	Hor. jt @ -156′	.70	2.30
844	Hor. jt @ -156′	.56	1.90
854	Hor. jt @ -156′	.41	1.00
861	Hor. jt @ -156′	.46	1.90
865	Hor. jt @ -156′	.28	1.20

Notes:

1. The following allowable hot spot stresses were used for the simplified fatigue checks based on 40 year life and API X S-N curve:

	Waterline Mbrs.	Other Mbrs.
"K"	39.9 ksi	43.6 ksi

2. S. R. = Stress ratio

Attachment 2

Criteria for Reanalysis of CameroonStructures

April 6, 1994

CRITERIA FOR REANALYSIS OF CAMEROON STRUCTURES
REFERENCE:

This letter transmits criteria for assessment of Cameroon structures. The criteria have been derived to match the reliability inherent in the assessment criteria for the non-GoM existing structures as recommended in the draft API guidelines.

<table>
<tr><th colspan="2">Exposure Category</th><th>Design Level Analysis</th><th>Ultimate Strength Analysis</th></tr>
<tr><td rowspan="2">Sig. Env. Impact</td><td>Manned, non-evac.</td><td rowspan="3">85% of lateral loading caused by 100-year environmental conditions (100-year H_{max} = 18.4 ft.)</td><td rowspan="3">Loading from a wave of height 24 ft., associated storm tide and other environmental conditions.</td></tr>
<tr><td>Unmanned</td></tr>
<tr><td rowspan="2">Insig. Env Impact</td><td>Manned, non-evac.</td></tr>
<tr><td>Unmanned</td><td>60% of lateral loading caused by 100-year environmental conditions (100-year H_{max} = 18.4 ft.)</td><td>Loading from a wave of height 18.4 ft., associated storm tide and other environmental conditions.</td></tr>
</table>

The minimum deck height should be adequate to clear the crest of ultimate-strength-analysis wave and associated storm tide with zero airgap. Excerpts from supporting calculations are enclosed.

Note that the wave heights are based on the West Africa Extreme Wave Hindcast Study (WAX).

Very truly yours,

RSD:rml

Attachment

11. Bea, R.G. and des Roches, R., "Development and Verification of a Simplified Procedure to Estimate the Capacity of Template-Type Platforms," Proc. 5th Intl. Symposium, Integrity of Offshore Structures, Glasgow, June 1993.

12. - Puskar, F.J., Aggarwal, R.K., Cornell, C.A., Moses, F., Petrauskas, C., "A Comparison of Analytically Predicted Platform Damage to Actual Platform Damage During Hurricane Andrew," Proc. Offshore Technology Conf., OTC 7473, Houston, Texas, 1994.

13. - "Gulf of Mexico Storm Hindcast of Oceanographic Extremes (GUMSHOE)," Report to the Joint Industry Project, Oceanweather Inc., August 1990.

14. Hellan, O., Tandberg, T. and Hellevig, N. C.: "Nonlinear Assessment of Jacket Structures Under Extreme Storm Cyclic Loading: Part IV - Case Studies on Existing North-Sea Platforms," Proc., OMAE, ASME, Glasgow, 1993.

15. Lloyd, J. R. and Clawson, W. C.: "Reserve and Residual Strength of Pile Founded Offshore Platforms," Proc., Symp. on Role of Design, Inspection, and Redundancy in Marine Structural Reliability, National Academic Press, 1983.

16. PMB Engineering, Inc., "Trial Application of the Draft API RP 2A-WSD Procedure for Assessment of Existing Platform," (joint industry project in progress).

Exposure Category	α	LRF	ULR	RSR	DLRP	ULRP	P_f
GOM Min. Conseq.	1.8	0.3	1.67	0.5	6.5	28	5.e-02
GOM Min. Conseq.	2.7	0.3	1.67	0.5	6.5	16	7.e-02
WC Min. Conseq.	1.8	0.5	1.67	0.84	4.3	34	6.e-02
WC Min. Conseq.	2.7	0.5	1.67	0.84	9.5	46	3.e-02
WC High Conseq.	1.8	0.85	1.9	1.6	36	4600	2.e-03
WC High Conseq.	2.7	0.85	1.9	1.6	50	1100	2.e-03
WC New Design	1.8	1.0	1.9	1.9	100	23500	8.e-04
WC New Design	2.7	1.0	1.9	1.9	100	2900	1.e-03
GOM Manned*	2.2	1.0	1.8	1.8	200	1800	1.e-03
GOM Manned*	2.7	1.0	1.8	1.8	200	1100	1.e-03

* Relative to sudden hurricanes and winter storms

Table 4 - Sample Reliability Results for Gulf of Mexico (GOM) and West Coast (WC)

RELIABILITY BASIS

Selection of Criteria consistent with the
risk level of draft API Ch 17 guidelines, which is:

Pf = 0.06 for alpha = 1.8 , α exponent in $F = cH^{\alpha}$ relationship
Pf = 0.03 for alpha = 2.7
for West Coast Min. consequence Case

Criterion for Cameroon Min. Consequence Case:
Load reduction factor, lrf = 0.6
i.e., use 60% of 100 year load for "design level" analysis
Corresponding return period for" ultimate strength"
analysis is around 105 years and the corresponding hult,
the wave ht. for ult. str. analysis = 18.4 ft

Also note that the 100 year Hmax is 18.4ft and not 21 ft

Min. deck ht.= crest ht. of hult(hmax=18.4 ft with T=16sec)+
storm tide associated with that wave height

The crest ht. should be calculated using the wave theory
recommended in API RP, 20th ed.
no airgap allowance is reqd.

Selection of Criteria consistent with the
risk level of draft API Ch 17 guidelines which is:

Acceptable failure probability for existing structures
= half for that of new structures
= 0.002 /year for the
West Coast High Consequence Case

Criterion for Cameroon High Consequence Case:
Load reduction factor, lrf = 0.85
i.e., use 85% of 100 year load for design level analysis
also hult, the wave ht. for ult. str. analysis = 24 ft
corresponding to the alpha =1.8 case.

Also note that the 100 year Hmax is 18.4ft and not 21 ft

Min. deck ht.= crest ht. ofhult (hmax=24.4 ft. w/ approp. T) +
storm tide associated with that wave height

The crest ht. should be calculated using the wave theory
recommended in API RP, 20th ed.
no airgap allowance is reqd.

CAMEROON MIN. CONSEQUENCE CASE

$\alpha = 1.8$

```
**********************************************************
3/18/94;Case: a CONTINUED;
Cameroon Minimum Consequence Structure
Reliability ~~Implications of~~ selecting LRF=0.45 to 0.75
 for the "Design Level" Analysis
ENVIRON.:
WAX data base;  interploated for water depth = 150 ft.= 45.7 m
5 yr Hs=2.52m  => 5yr H =1.76*2.52/.3048=14.6ft.
50 yr Hs=3.04 ft => 50yr H =1.76*3.04/.3048=17.6ft.
100 yr Hs=3.18 ft => 100yr H =1.76*3.18/.3048=18.4ft.
             alpha=1.8, ulr=1.67
**********************************************************

** hdl=design level wave ht., lrf =load reduction factor
** dlrp= wave ht. ret. period for "design level" analysis;
** ulrp= wave ht. ret. per. for "ult. strength"  analysis;
** hult= wave ht. for  ultimate strength analysis;
** beta= annual reliability index against collapse (ult.);
** pf= corresponding annual failure probability

TableForm[Transpose[{lrf,dlrp,ulrp,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"dlrp(yrs)","ulrp(yrs)","Annual Beta","Annual Pf"}}]
```

lrf	dlrp(yrs)	ulrp(yrs)	Annual Beta	Annual Pf
0.45	1.42089	10.5647	0.910989	$1.81151\ 10^{-1}$
0.5	1.77985	22.0771	1.17448	$1.20101\ 10^{-1}$
0.6	3.24604	106.326	1.63045	$5.15036\ 10^{-2}$
0.65	4.65479	239.608	1.83062	$3.35784\ 10^{-2}$
0.7	6.88236	544.145	2.01596	$2.19022\ 10^{-2}$
0.75	10.4303	1239.14	2.1885	$1.43166\ 10^{-2}$

```
TableForm[Transpose[{lrf,hdl,hult,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"hdl(ft.)","hult(ft.)","Annual Beta","Annual Pf"}}]
```

lrf	hdl(ft.)	hult(ft.)	Annual Beta	Annual Pf
0.45	11.8075	15.6996	0.910989	$1.81151\ 10^{-1}$
0.5	12.5193	16.646	1.17448	$1.20101\ 10^{-1}$
0.6	13.8538	18.4204	1.63045	$5.15036\ 10^{-2}$
0.65	14.4837	19.258	1.83062	$3.35784\ 10^{-2}$
0.7	15.0925	20.0675	2.01596	$2.19022\ 10^{-2}$
0.75	15.6822	20.8516	2.1885	$1.43166\ 10^{-2}$

CAMEROON MIN. CONSEQUENCE CASE

$\alpha = 2.7$ CASE

```
********************************************************
3/18/94;Case: b CONTINUED;Cameroon Minimum Consequence Structure
Reliability Implications of selecting LRF=0.45 to 0.75
 for the "Design Level" Analysis
ENVIRON.: WAX data base;interploated for water depth=150 ft.= 45.7 m
5 yr Hs=2.52m  => 5yr H =1.76*2.52/.3048=14.6ft.
50 yr Hs=3.04 ft => 50yr H =1.76*3.04/.3048=17.6ft.
100 yr Hs=3.18 ft => 100yr H =1.76*3.18/.3048=18.4ft.
              alpha=2.7  ulr=1.67
********************************************************

** hdl=design level wave ht., lrf =load reduction factor
** dlrp= wave ht. ret. period for "design level" analysis;
** ulrp= wave ht. ret. per. for "ult. strength"  analysis;
** hult= wave ht. for  ultimate strength analysis;
** beta= annual reliability index against collapse (ult.);
** pf= corresponding annual failure probability

TableForm[Transpose[{lrf,dlrp,ulrp,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"dlrp(yrs)","ulrp(yrs)","Annual Beta","Annual Pf"}}]
```

lrf	dlrp(yrs)	ulrp(yrs)	Annual Beta	Annual Pf
0.45	2.97752	20.4566	1.36183	$8.66259\ 10^{-2}$
0.5	4.0124	35.5382	1.57	$5.82071\ 10^{-2}$
0.6	7.55039	105.643	1.93024	$2.67885\ 10^{-2}$
0.65	10.4633	179.987	2.08839	$1.83813\ 10^{-2}$
0.7	14.5473	303.772	2.23482	$1.27147\ 10^{-2}$
0.75	20.2544	507.623	2.37113	$8.86679\ 10^{-3}$

```
TableForm[Transpose[{lrf,hdl,hult,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"hdl(ft.)","hult(ft.)","Annual Beta","Annual Pf"}}]
```

lrf	hdl(ft.)	hult(ft.)	Annual Beta	Annual Pf
0.45	13.6892	16.5526	1.36183	$8.66259\ 10^{-2}$
0.5	-14.2339	17.2113	1.57	$5.82071\ 10^{-2}$
0.6	15.2283	18.4136	1.93024	$2.67885\ 10^{-2}$
0.65	15.6865	18.9677	2.08839	$1.83813\ 10^{-2}$
0.7	16.123	19.4955	2.23482	$1.27147\ 10^{-2}$
0.75	16.5403	20.0001	2.37113	$8.86679\ 10^{-3}$

CAMEROON HIGH CONSEQUENCE CASE

$\alpha = 1.8$

```
************************************************************
3/18/94;Case: a CONTINUED;
Cameroon High Consequence Structure
Reliability Implications of selecting LRF=0.75 to 1.0
for the "Design Level" Analysis
ENVIRON.:
WAX data base;  interploated for water depth = 150 ft.= 45.7 m
50 yr Hs=3.04 ft => 50yr H =1.76*3.04/.3048=17.6ft.
100 yr Hs=3.18 ft => 100yr H =1.76*3.18/.3048=18.4ft.
             alpha=1.8, ulr=1.9
************************************************************

** hdl=design level wave ht., lrf =load reduction factor
** dlrp= wave ht. ret. period for "design level" analysis;
** ulrp= wave ht. ret. per. for "ult. strength"  analysis;
** hult= wave ht. for  ultimate strength analysis;
** beta= annual reliability index against collapse (ult.);
** pf= corresponding annual failure probability

TableForm[Transpose[{lrf,dlrp,ulrp,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"dlrp(yrs)","ulrp(yrs)","Annual Beta","Annual Pf"}}]
```

lrf	dlrp(yrs)	ulrp(yrs)	Annual Beta	Annual Pf
0.75	11.2217	4867.42	2.52231	$5.82935\ 10^{-3}$
0.8	17.0471	11439.9	2.67929	$3.68895\ 10^{-3}$
0.85	26.2173	26618.3	2.82675	$2.35118\ 10^{-3}$
0.9	40.7017	61253.6	2.96577	$1.50962\ 10^{-3}$
0.95	63.635	139314.	3.09728	$9.76523\ 10^{-4}$
1.	100.	313048.	3.22204	$6.36402\ 10^{-4}$

```
TableForm[Transpose[{lrf,hdl,hult,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"hdl(ft.)","hult(ft.)","Annual Beta","Annual Pf"}}]
```

lrf	hdl(ft.)	hult(ft.)	Annual Beta	Annual Pf
0.75	15.6822	22.4012	2.52231	$5.82935\ 10^{-3}$
0.8	16.2547	23.2189	2.67929	$3.68895\ 10^{-3}$
0.85	16.8115	24.0143	2.82675	$2.35118\ 10^{-3}$
0.9	17.3539	24.7891	2.96577	$1.50962\ 10^{-3}$
0.95	17.8831	25.545	3.09728	$9.76523\ 10^{-4}$
1.	18.4	26.2834	3.22204	$6.36402\ 10^{-4}$

CAMEROON HIGH CONSEQUENCE CASE

$\alpha = 2.7$

```
****************************************************************
3/18/94;Case: b CONTINUED;Cameroon High Consequence Structure
Reliability Implications of selecting LRF=0.75 to 1.0
for the "Design Level" Analysis
ENVIRON.: WAX data base;interploated for water depth=150 ft.= 45.7 m
50 yr Hs=3.04 ft => 50yr H =1.76*3.04/.3048=17.6ft.
100 yr Hs=3.18 ft => 100yr H =1.76*3.18/.3048=18.4ft.
alpha=2.7, ulr=1.9
****************************************************************

** hdl=design level wave ht., lrf =load reduction factor
** dlrp= wave ht. ret. period for "design level" analysis;
** ulrp= wave ht. ret. per. for "ult. strength"  analysis;
** hult= wave ht. for  ultimate strength analysis;
** beta= annual reliability index against collapse (ult.);
** pf= corresponding annual failure probability

TableForm[Transpose[{lrf,dlrp,ulrp,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"dlrp(yrs)","ulrp(yrs)","Annual Beta","Annual Pf"}}]
```

lrf	dlrp(yrs)	ulrp(yrs)	Annual Beta	Annual Pf
0.75	21.2001	1147.14	2.62035	$4.39202\ 10^{-3}$
0.8	29.0565	1908.7	2.74303	$3.04372\ 10^{-3}$
0.85	39.7445	3136.3	2.85828	$2.12972\ 10^{-3}$
0.9	54.2239	5091.69	2.96694	$1.50391\ 10^{-3}$
0.95	73.7576	8171.19	3.06972	$1.0713\ 10^{-3}$
1.	100.	12968.8	3.16723	$7.69503\ 10^{-4}$

```
TableForm[Transpose[{lrf,hdl,hult,betaT,Map[ScientificForm,
pfT] }],TableSpacing->{1,6},TableHeadings->{{},{"lrf",
"hdl(ft.)","hult(ft.)","Annual Beta","Annual Pf"}}]
```

lrf	hdl(ft.)	hult(ft.)	Annual Beta	Annual Pf
0.75	16.5403	20.9791	2.62035	$4.39202\ 10^{-3}$
0.8	16.9405	21.4866	2.74303	$3.04372\ 10^{-3}$
0.85	17.3251	21.9745	2.85828	$2.12972\ 10^{-3}$
0.9	17.6958	22.4446	2.96694	$1.50391\ 10^{-3}$
0.95	18.0537	22.8986	3.06972	$1.0713\ 10^{-3}$
1.	18.4	23.3378	3.16723	$7.69503\ 10^{-4}$