211A (COPY 2)

STRUCTURAL ASSESSMENT
BOOK 1 OF 4

ASSESSMENT CALCULATIONS

SHELL OIL COMPANY
WEST DELTA 103 "A"
8-PILE, 12-WELL PLATFORM
223' WATER DEPTH

FOR
U.S. MINERALS MANAGEMENT SERVICE

PERFORMED BY:

W. H. LINDER & ASSOCIATES, INC.
3330 WEST ESPLANADE AVENUE
METAIRIE, LOUISIANA 70002
(504) 835-2577

JULY, 1994

Trial Application of the Draft API RP2A-WSD Procedure for Assessment of Shell West Delta 103 "A" Platform

Trial Basis Document

Submitted To:

PMB Engineering, Inc.
San Francisco, California

By:

W. H. Linder & Associates, Inc.
3330 West Esplanade Avenue
Metairie, Louisiana 70002
(504) 835-2577 / (504) 837-5924 (FAX)

Attention: Barry Reed / M. Ekrem Celebi

LINDER — W. H. LINDER & ASSOCIATES, INC. CONSULTING ENGINEERS	SHELL WEST DELTA BLK 103 A / PLATFORM ASSESSMENT	PROJECT NO. 1830 M01
	BY MEC DATE 7/30/94	CHK. ___ DATE ___
		SHEET 1 OF 31

TRIAL DOCUMENT

Table of Contents

1.0 Platform Information (Section 17.7)

As-is condition details related to
- Physical features (water depth, deck elevation, etc.)
- Operational information
- Pertinent inspection information
- Structural assessment data

} REFER TO PLATFORM ASSESSMENT SECTION 6.3.4

Platform sketches (8-1/2" x 11")
- Platform orientation
- Vertical framing elevations (jacket and deck)
- Horizontal framing plans
- Pile makeup and details
- Other information

} REFER TO PART-E OF TRIAL DOCUMENT

PART-A: Trial Application to meet a Regulators Requirements

A.1 Platform Selection (Section 17.3)

A.2 Condition Assessment (Section 17.4)

A.3 Categorization (Section 17.5)

A.4 Design Basis Checks (Section 17.2)

A.5 Analysis Checks (Sections 17.6 and 17.7)
- A.5.1 Metocean, seismic, and ice criteria/ loads
- A.5.2 Screening
- A.5.3 Design level
- A.5.4 Ultimate strength

A.6 Mitigation Alternatives (Section 17.8)

} REFER TO PLATFORM ASSESSMENT SECTIONS 2 & 6

A.7 Summary Note - Part A — SECTION 6.3.4

SECTIONS 6.4 THRU 6.8

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CONSULTING ENGINEERS

SHELL WEST DELTA BLK 103 A
PLATFORM ASSESSMENT

PROJECT NO. 1830M01
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CHK. DATE

TRIAL DOCUMENT

Table of Contents (Contd.)

PART-B: Trial Application to meet the Project Requirements
(In case all or some of this work was already performed in the Part-A, the corresponding sections can be by-passed and reference be made to the appropriate Part-A sections)

B.1 Design Basis Checks (Section 17.2) → REFER TO PLATFORM ASSESSMENT SECTION 3.1.2

B.2 Structural Analysis (Sections 17.6 and 17.7)
- B.2.1 Metocean, seismic, and ice criteria/ loads → SECTION 7.1.3.6
- B.2.2 Screening → SECTIONS 3.1, 3.2
- B.2.3 Design level → SECTION 7.2
- B.2.4 Ultimate strength → SECTION 7.3
- B.2.5 Pushover Analyses I & II → SECTIONS 7.4, 7.5

B.3 Mitigation Alternatives (Section 17.8) → TO BE DECIDED WITH & BY OPERATOR

B.4 Summary Note - Part B → SECTION 3.

PART-C: Review and Feedback to the API TG 92-5 (Voluntary)
(Refer to the relevant Section of Draft Section 17)

PART-D: Miscellaneous Information (Voluntary)

PART-E: PLATFORM SKETCHES

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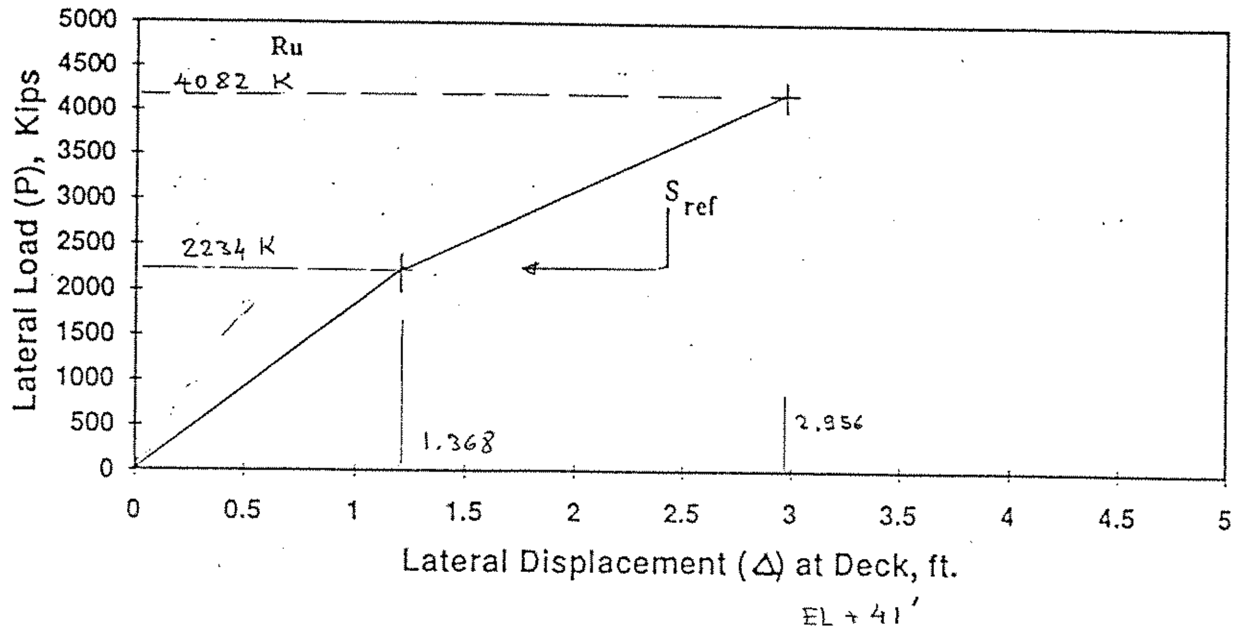
SHELL WEST DELTA BLK 103A
PLATFORM ASSESSMENT

PROJECT NO. 1830M01
BY MEC DATE 7/30/94
CHK. DATE

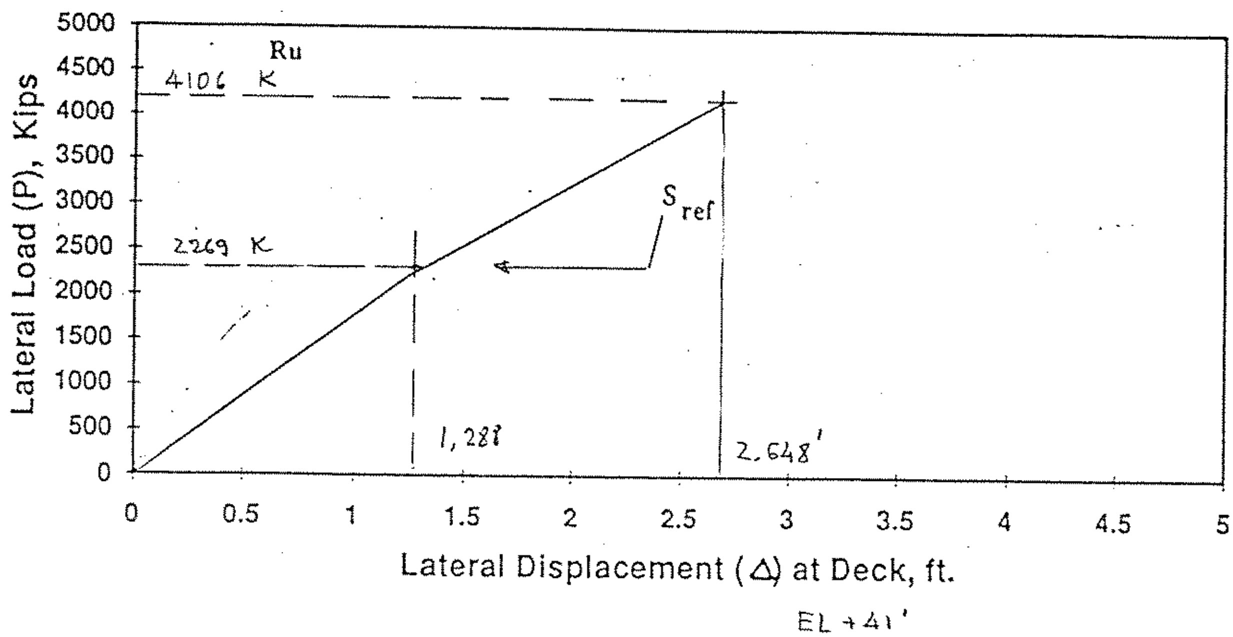
Ultimate Strength Analysis- Minimum Required Results



Reference Level Load (S_{ref})	2234	Kips
Ultimate Capacity (R_u)	4082	Kips
Reserve Strength Ratio (RSR)	1.827	Kips
Platform Failure Mode: Jacket, Pile, Soils, etc.	Pile, Jacket	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 45°

Ultimate Strength Analysis- Minimum Required Results



Reference Level Load (S_{ref})	2269	Kips
Ultimate Capacity (R_u)	4106	Kips
Reserve Strength Ratio (RSR)	1.810	Kips
Platform Failure Mode: Jacket, Pile, Soils, etc.	Jacket	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 67.5°

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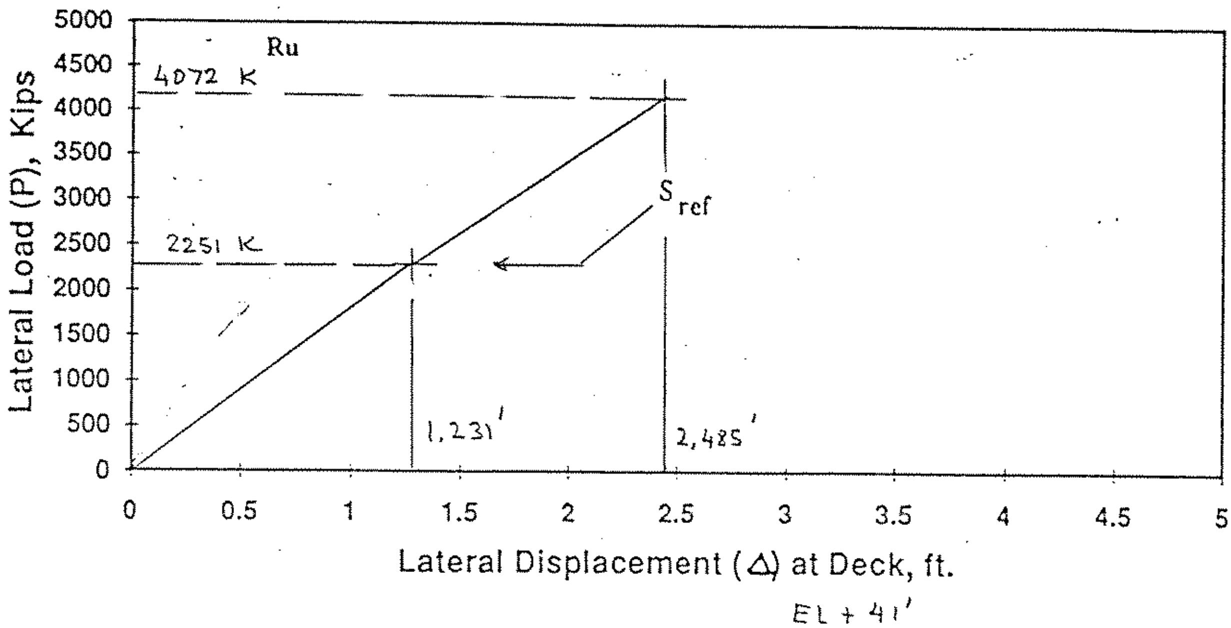
SHELL WEST DELTA BLK 103 A
PLATFORM ASSESSMENT

PROJECT NO. 1836 Mb1
BY MEC DATE 7/30/94
CHK. DATE
SHEET 5 OF 31

Ultimate Strength Analysis- Minimum Required Results



Reference Level Load (S_{ref})	2251	Kips
Ultimate Capacity (R_u)	4071	Kips
Reserve Strength Ratio (RSR)	1.809	Kips
Platform Failure Mode: Jacket, Pile, Soils, etc.	JACKET	

Figure 1: Format for Documenting Load - Displacement Results for Direction- 90°

PROJECT NO. 1836 Md1		
SHELL WEST DELTA BLK 103A	BY MEC	DATE 7/30/94
PLATFORM ASSESSMENT	CHK.	DATE

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'				
1	JNT 154 X = 27,935" = 2,328'	Fx = 2908	FOR JACKET PLANE A & B FAILURES SEE FOLLOWING TWO SHEETS		
2	P. 12 Y = 21.154" = 1.763'	Fy = 2864			
3	LOWER DECK EL +41				
4	JNT 38 X = 28,135" = 2,344'	$(F_x^2+F_y^2)^{1/2}$ = 4082	PILE HEAD FAILURES	SEE SECTION 8.2.2	P. 234
5	P. 3 Y = 21.612" = 1.801'		PILE HEAD		
6	$(X^2+Y^2)^{1/2}$ = 35.478" = 2.95		112	BENDING	U.C = 1.23
7		P. 96 SECTION 8.1.1	122	↓	= 1.09
8	JNT 154 x = 12.504" = 1.042'	Fx = 1580 K	132	↓	= 1.15
9	y = 10.465" = .872'	Fy = 1579 K	142	↓	= 1.26
10	JNT 38 X = 12,525" = 1.044		152	↓	= 1.04
	y = 10,619" = .885	$(F_x^2+F_y^2)^{1/2}$ = 2234 K	162	↓	= 1.31
	$(X^2+Y^2)^{1/2}$ = 16.421" = 1.368'		172	↓	= 1.29
			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.

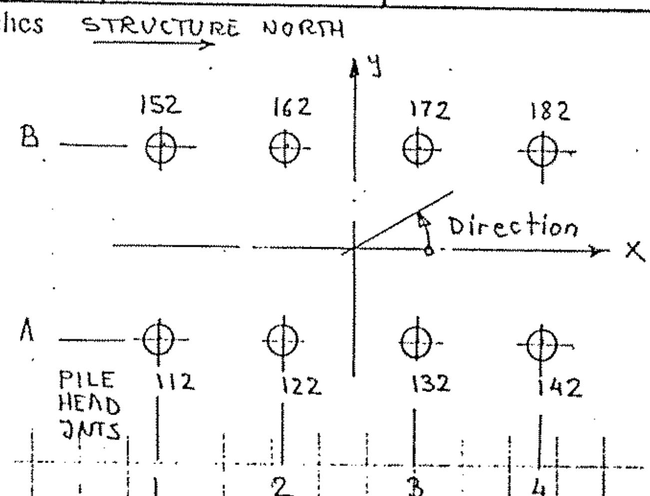


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

(7.3.6 Continued) USED ALSO FOR TRIAL DOCUMENT TABLE 2

SHEET 2 OF 3

SHELL 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

f_{by} = 6.92 — .137

f_{bz} = -1.74 — .035

KL/r = 26.7, U.C. = 1.158

p. 84 STRESS KSI — p. 29 U.C. COMP

f_a = -13.62 — .616

f_{by} = -3.08 — .121

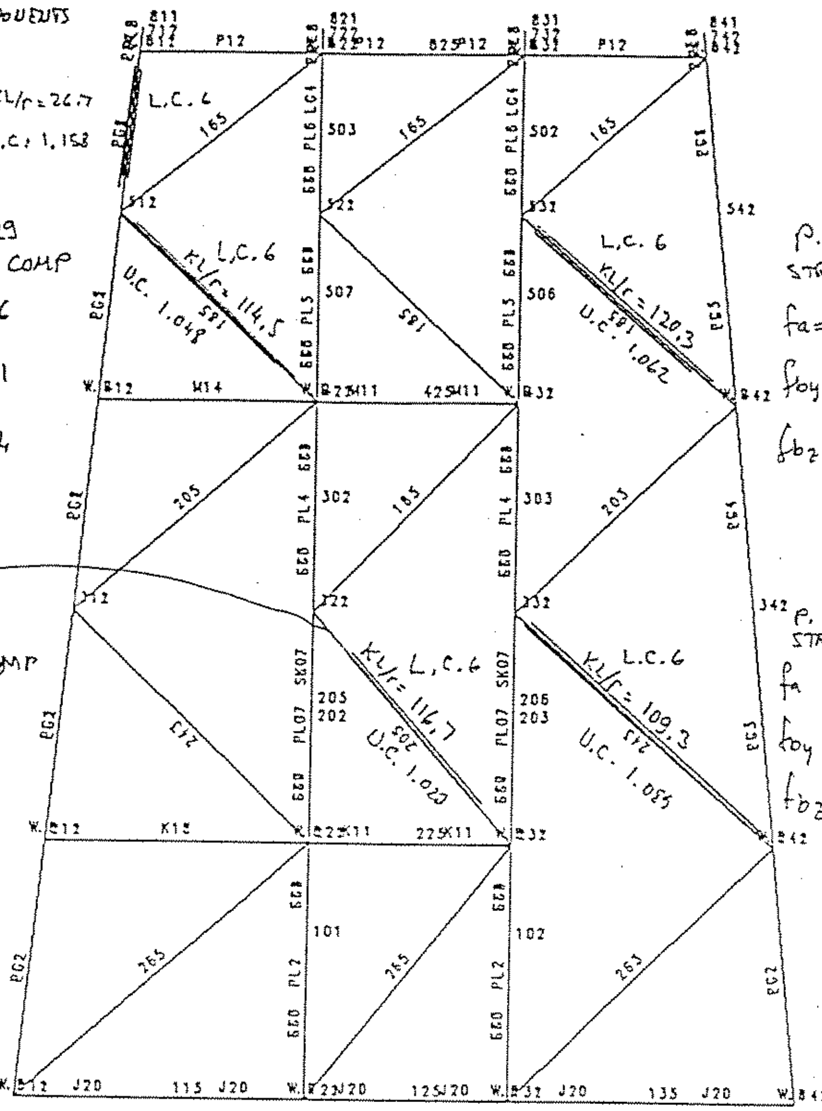
f_{bz} = -10.58 — .414

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

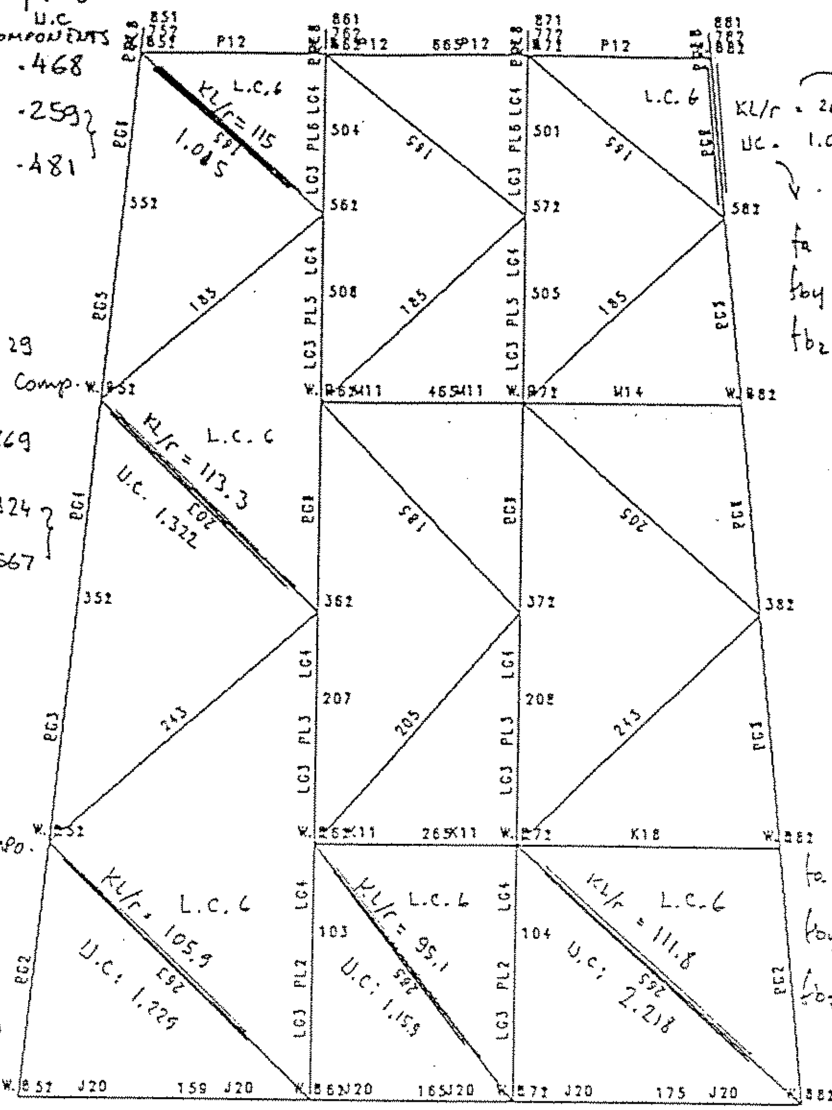
TRUSS DIAGONAL : BUCKLING + BENDING (U.C ARE SLIGHTLY ABOVE 1.0)

(7.3.6 Continued) USED ALSO FOR TRIAL DOCUMENT TABLE 2 SHEET 3 OF 3

SHELL WEST DELTA BLK 103A 223 FT WATERS

BROADSIDE VIEW ROW B

NOTE: STRESS & U.C COMPONENTS FROM WD103US.OT3 OUTPUT output



p. 84 OUTPUT STRESS

f_a = −10.28 KSI
f_{by} = −8.97 KSI
f_{bz} = −16.64 KSI

p. 29 U.C COMPONENTS

.468
.259
.481

p. 36 U.C. Components

Axial .929
Y-Axis .119
Z-Axis .035

KL/r = 26.7
UC = 1.053
L.C. 6

p. 84

f_a = 40.12 KSI
f_{by} = −5.61 "
f_{bz} = 1.64 "

p. 84 STRESS

f_a = −15.00 KSI
f_{by} = −7.06 "
f_{bz} = −12.34 "

p. 29 U.C Comp.

.669
.324
.567

p. 84 STRESS

f_a = −19.30 KSI
f_{by} = −4.64 "
f_{bz} = −5.16 "

p. 31 U.C. COMPO.

.798
.288
.320

p. 83 STRESS

f_a = −21.84
f_{by} = −5.60
f_{bz} = −3.63

p. 31 U.C. COMP.

.956
1.059
.686

p. 84 STRESS

f_a = −22.56 KSI
f_{by} = −6.26 "
f_{bz} = −1.54 "

p. 31 U.C COMPONENTS

0.830 — 0.830
0.320, 0.079 — 0.330
1.160

FAILURES

LEG MEMBER ; INELASTIC BUCKLING

TRUSS DIAGONALS ; BUCKLING + BENDING

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 = 215.781 "
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.

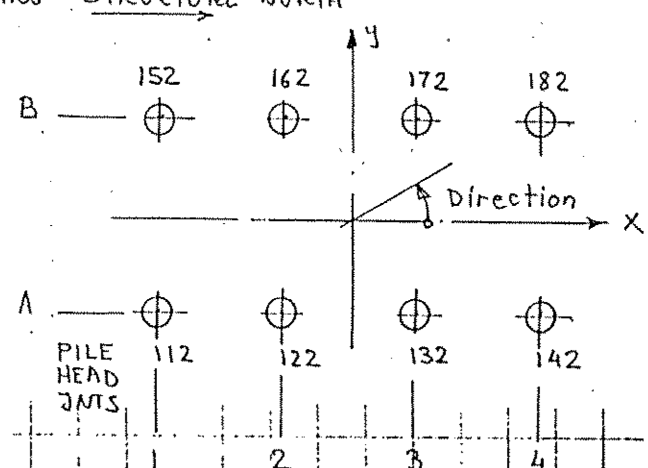


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

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		
1	JNT 154 $x = .532'' = .044'$	$F_x = 57$	JA-JB 145-211	Mainly Buckling + bending	U.C = 1.768
2	p.12 $y = 29.210'' = 2.434'$	$F_y = 4072$			
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 $x = .380'' = .032$	$F_y = 30$ K	JA-JB 381-441	" "	U.C = 2.722
9	$y = 14.567'' = 1.214$	$F_y = 2251$ 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.

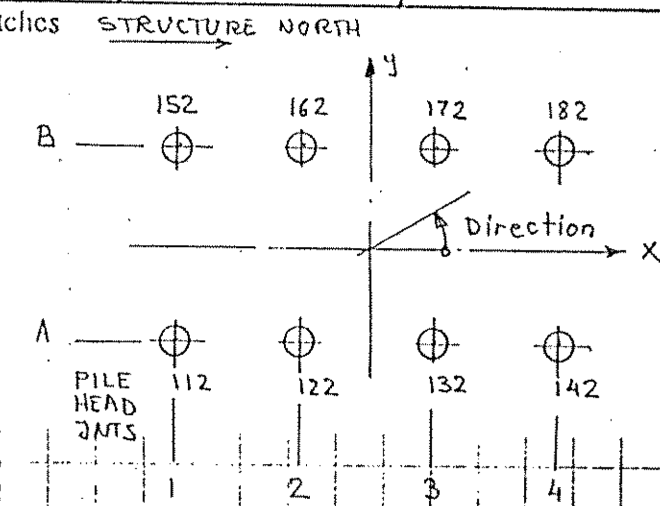


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

PROJECT NO. 1830M01

SHELL WEST DELTA BLK 103A — BY MEC — DATE 7/30/94

PLATFORM ASSESSMENT — CHK. ____ DATE ____

SHEET 11 OF 31

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.

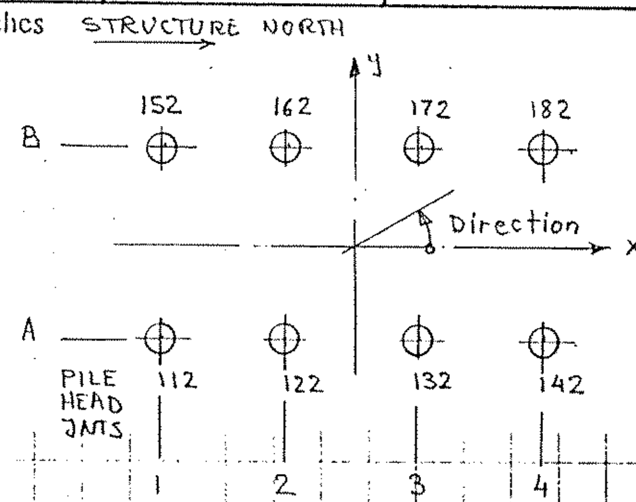
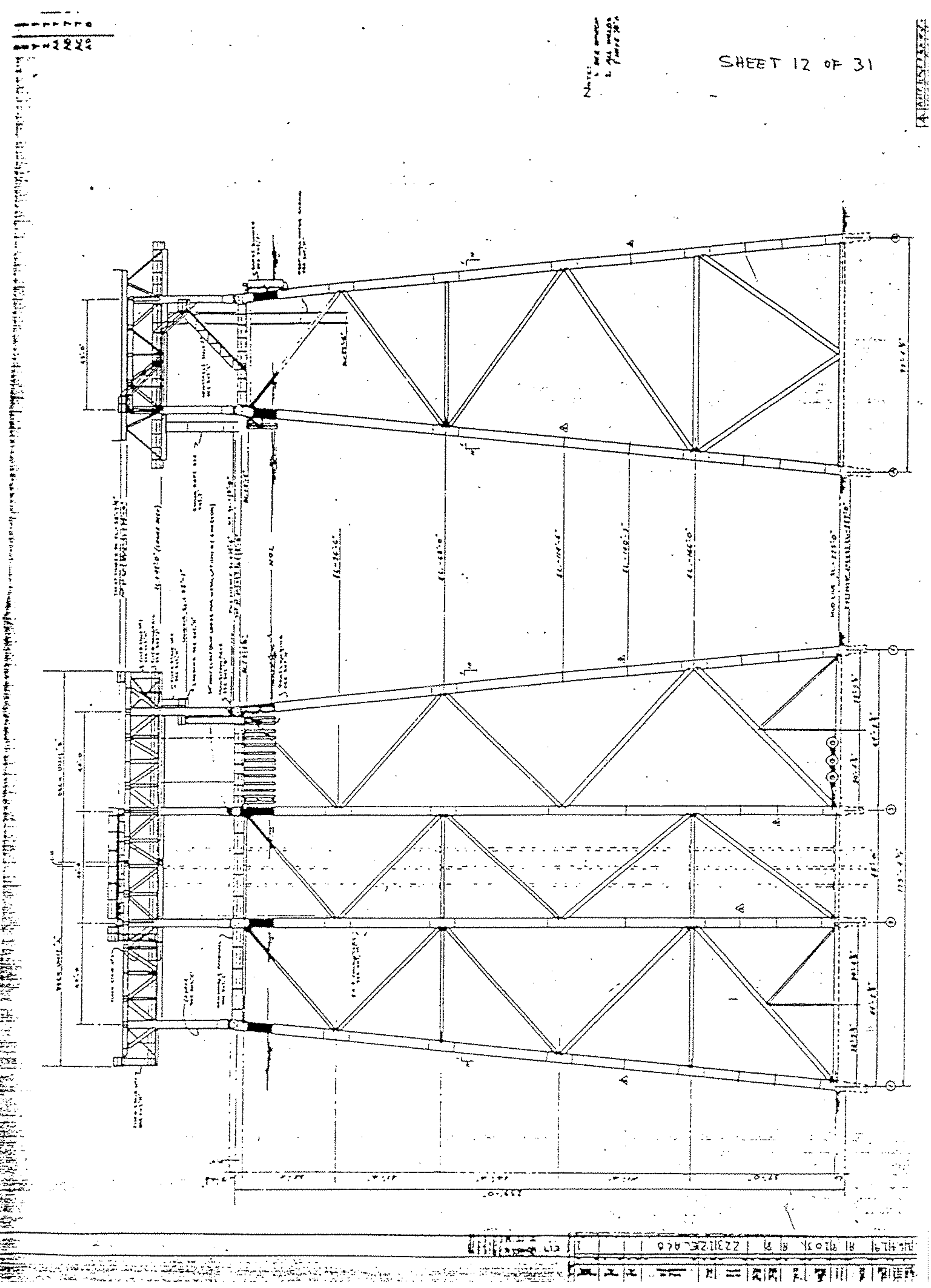
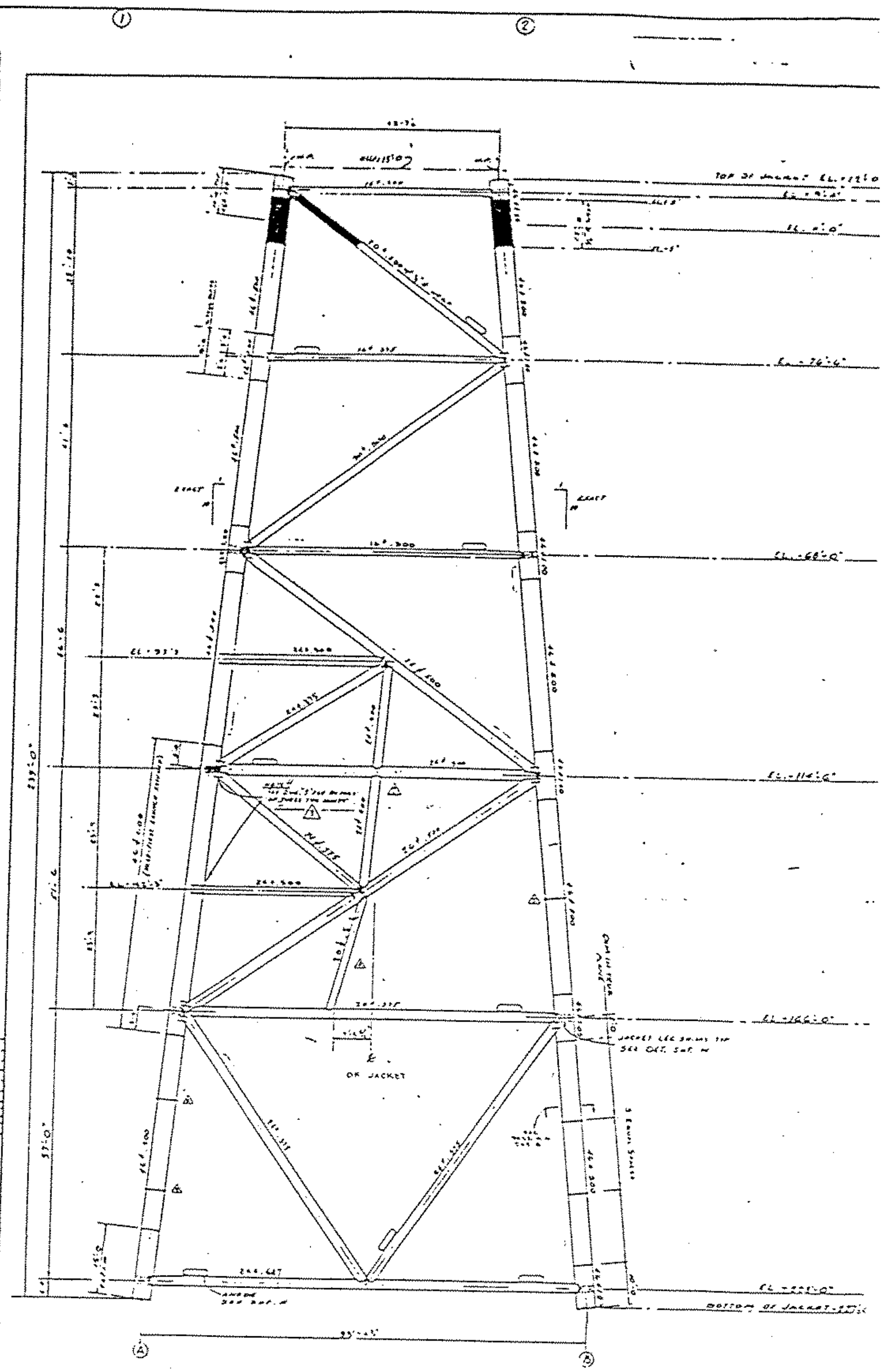
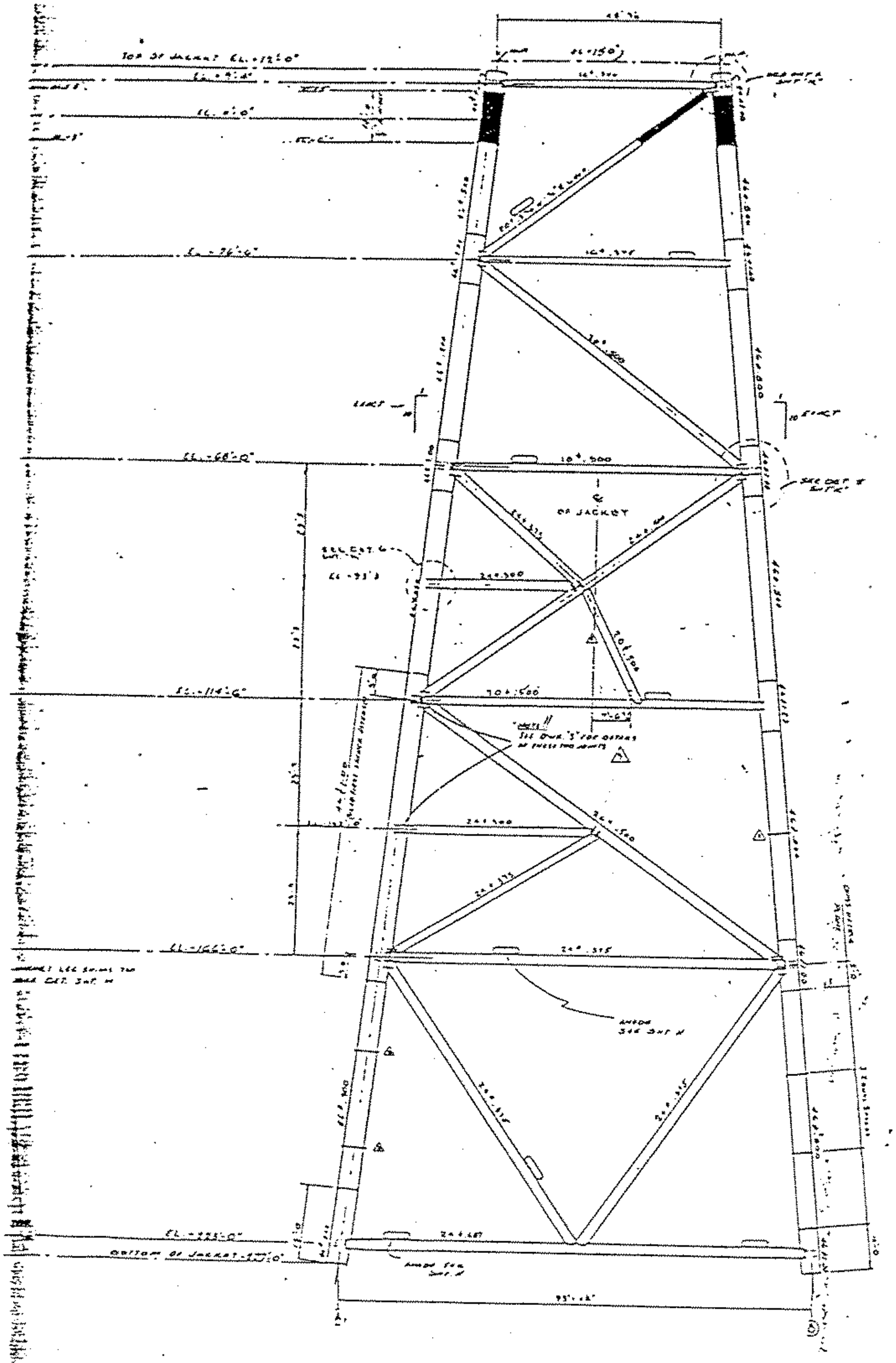


Table 2: Ultimate Strength Analysis - Direction "1" Results
(Provide separate table for each direction analyzed)

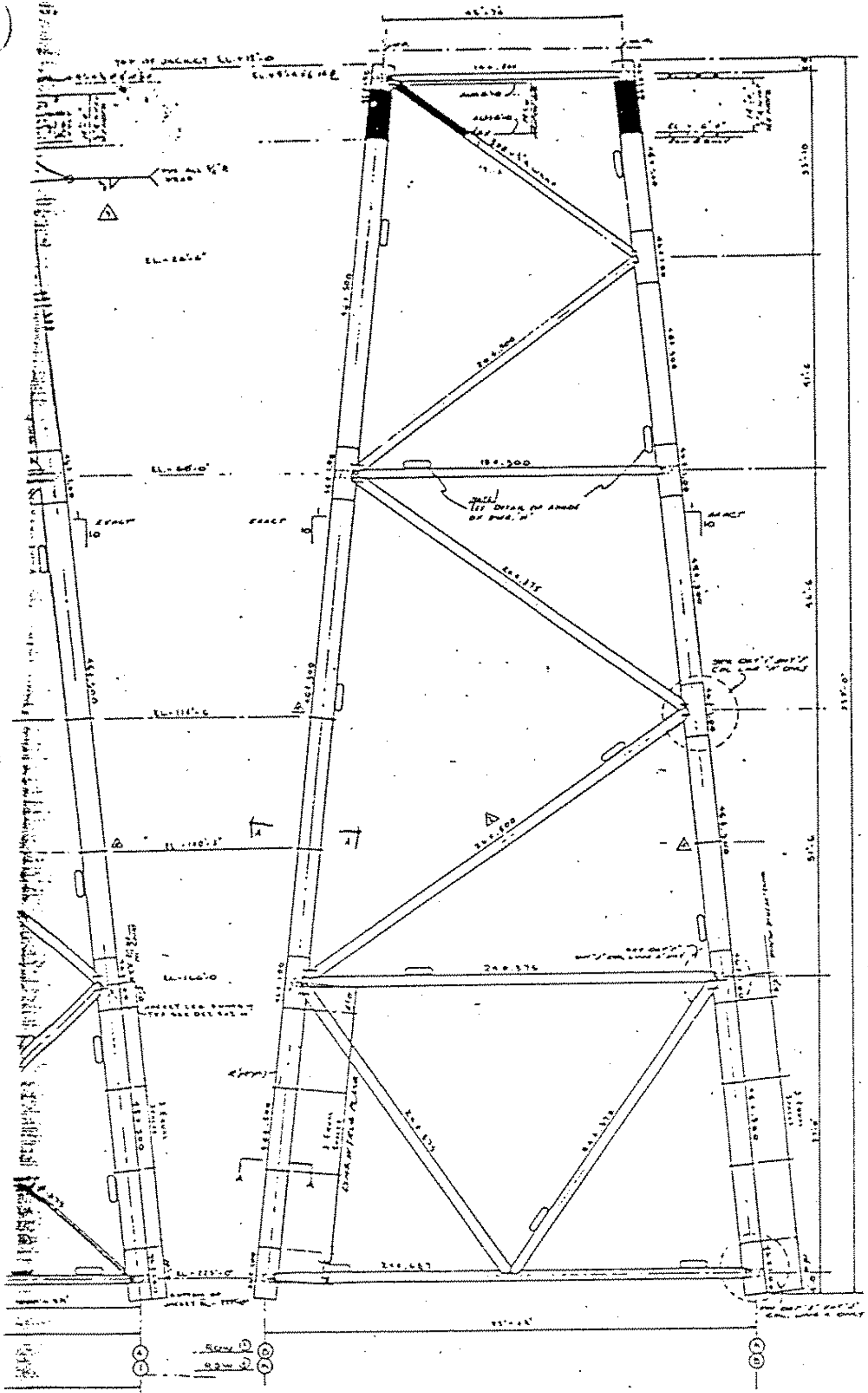




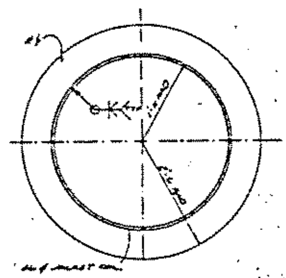
JACKET FRAMING ROW 2



JACKET FRAMING ROW 3



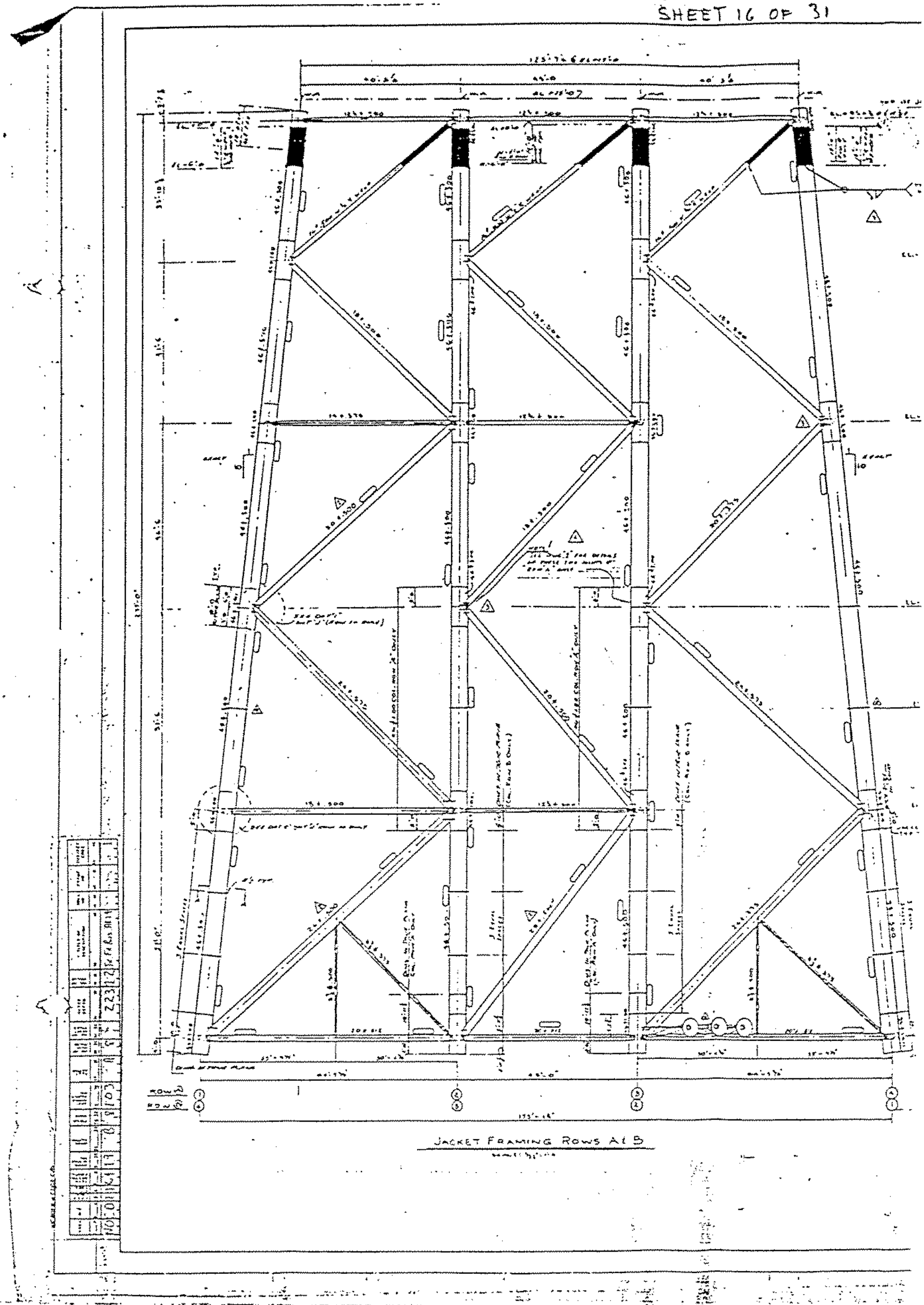
SECTION "A-A" (TYP.)



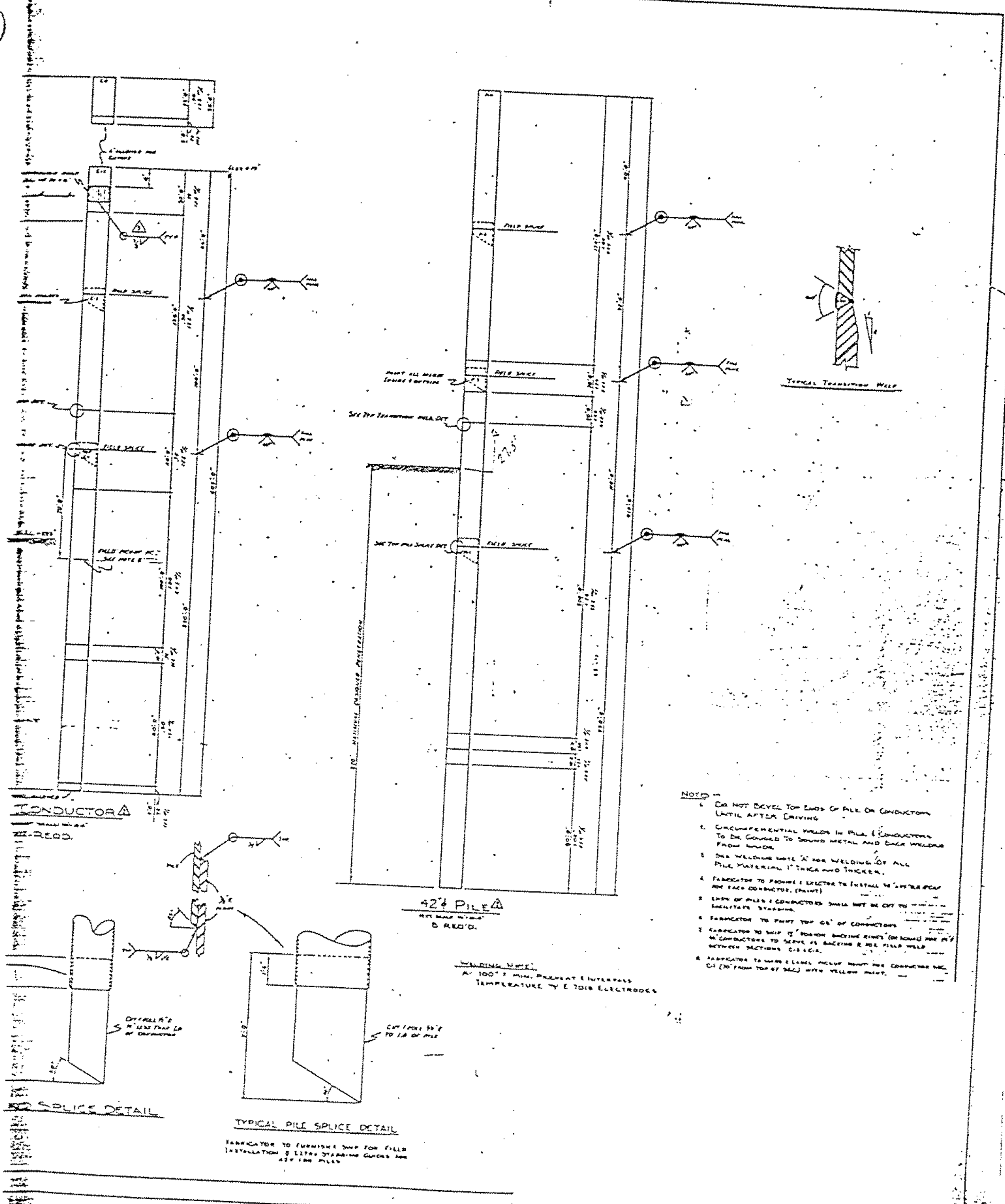
NOTE

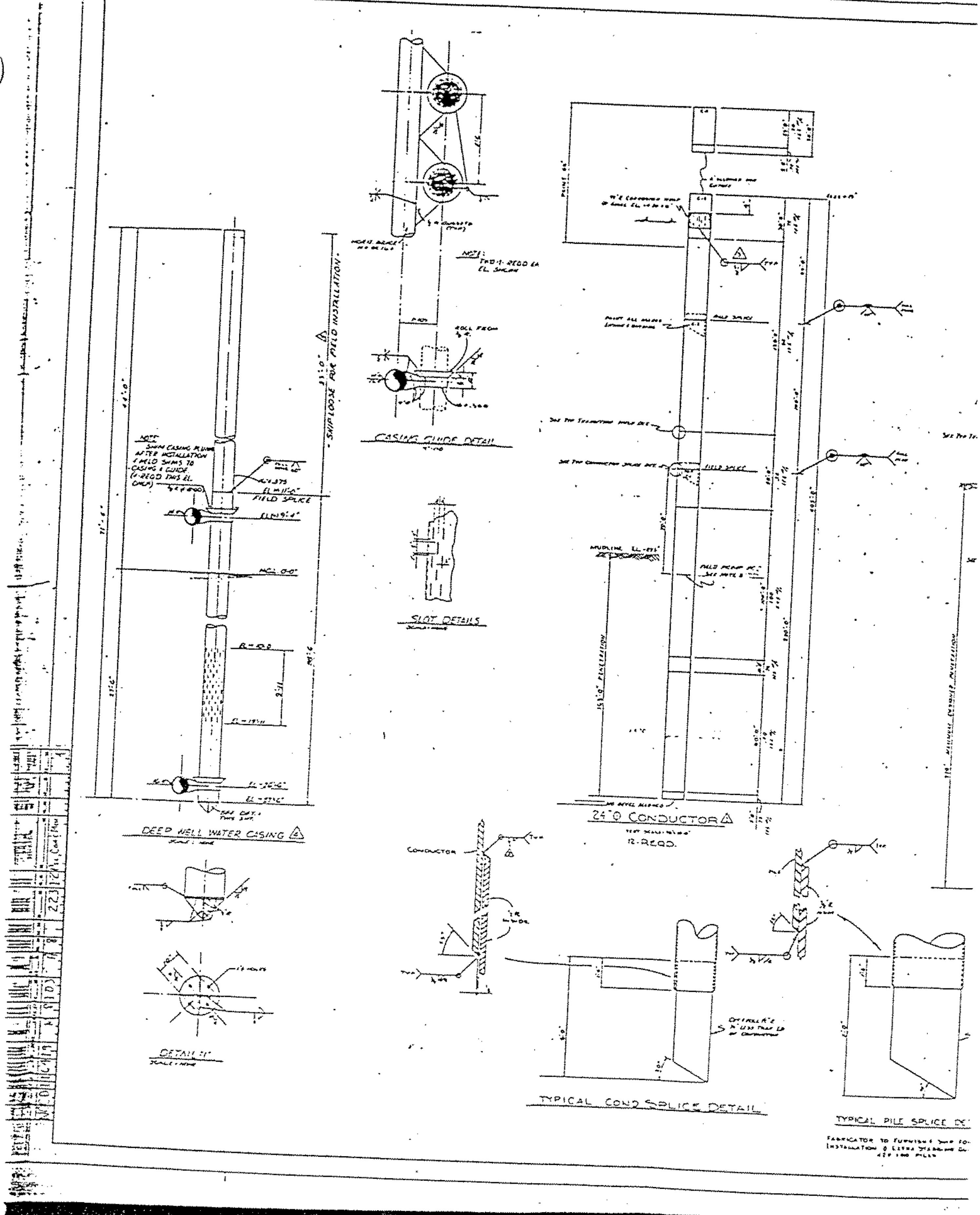
CONNECT WELD TO CLEAR BRACING ON CONNECTING DIAGONALS

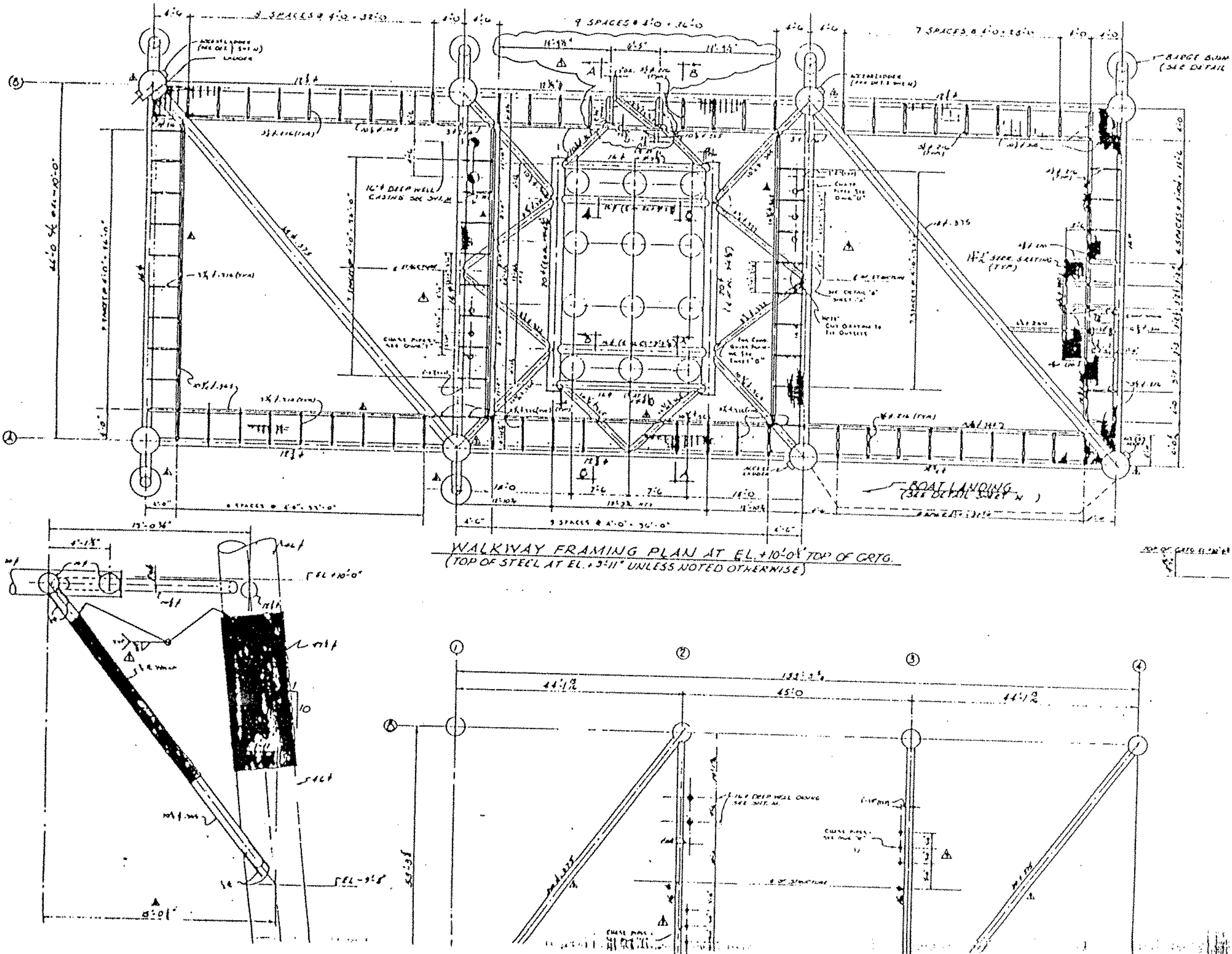
JACKET FRAMING ROWS 1 & 4



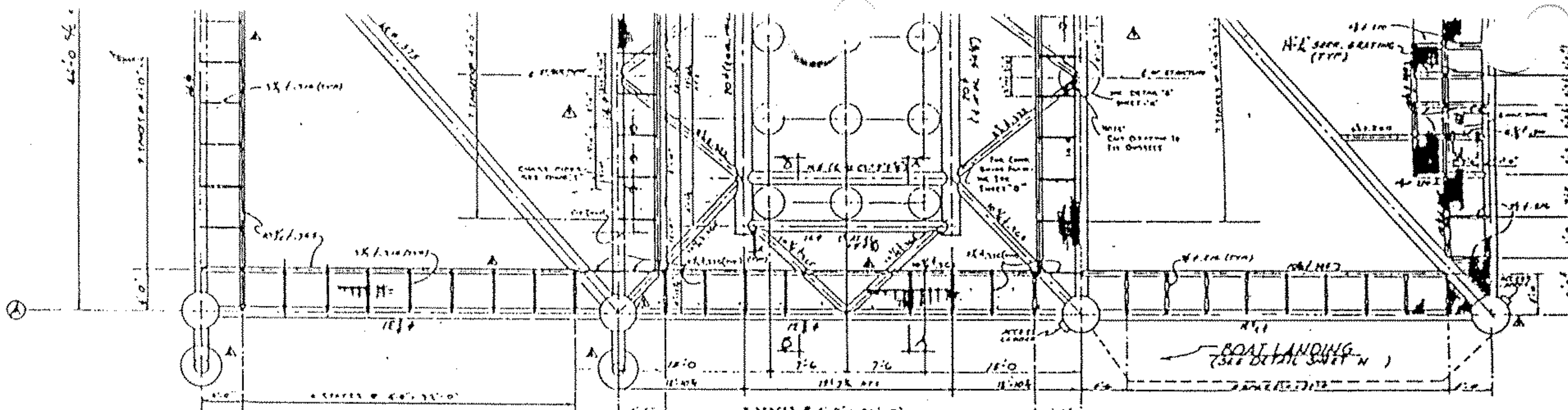
JACKET FRAMING ROWS A & B


PILE SPLICE
FIELD SPLICE
TYPICAL TRANSITION WELD
CONDUCTOR
REQ'D.
42" φ PILE
5 REQ'D.
WELDING NOTE:
A: 100° F MIN. PREHEAT & INTERPASS TEMPERATURE W/ E 7018 ELECTRODES
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.
SPLICE DETAIL
TYPICAL PILE SPLICE DETAIL

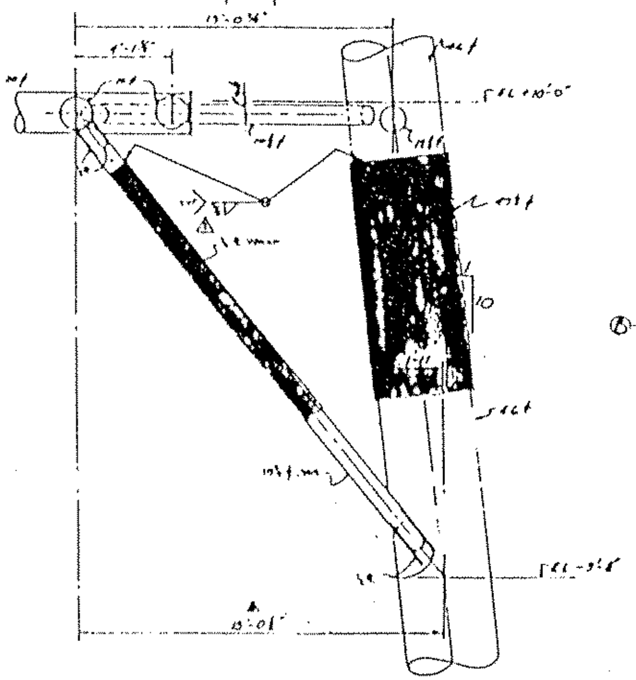

CASING GUIDE DETAIL
SLOT DETAILS
DEEP WELL WATER CASING
SHIP LOOSE FOR FIELD INSTALLATION
FIELD SPLICE
24"Ø CONDUCTOR
12-REQD.
CONDUCTOR
DETAIL "1"
TYPICAL COND SPLICE DETAIL
TYPICAL PILE SPLICE DE



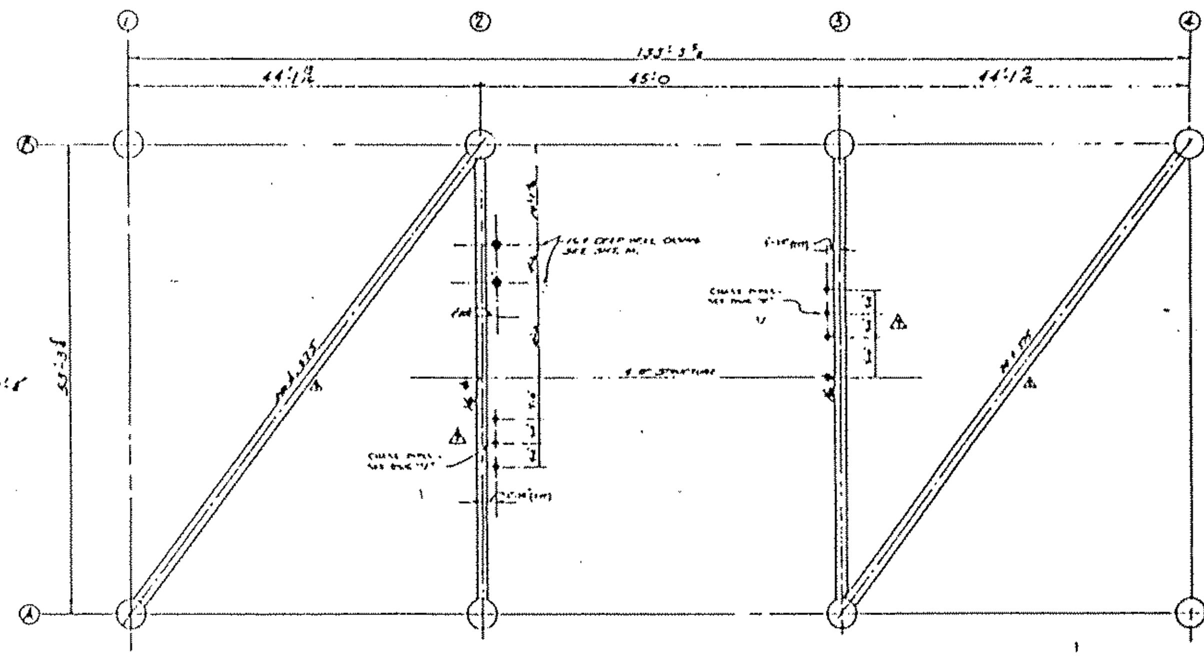
WALKWAY FRAMING PLAN AT EL.+10'-0 1/4" TOP OF GRTG.
(TOP OF STEEL AT EL.+9'-11" UNLESS NOTED OTHERWISE)



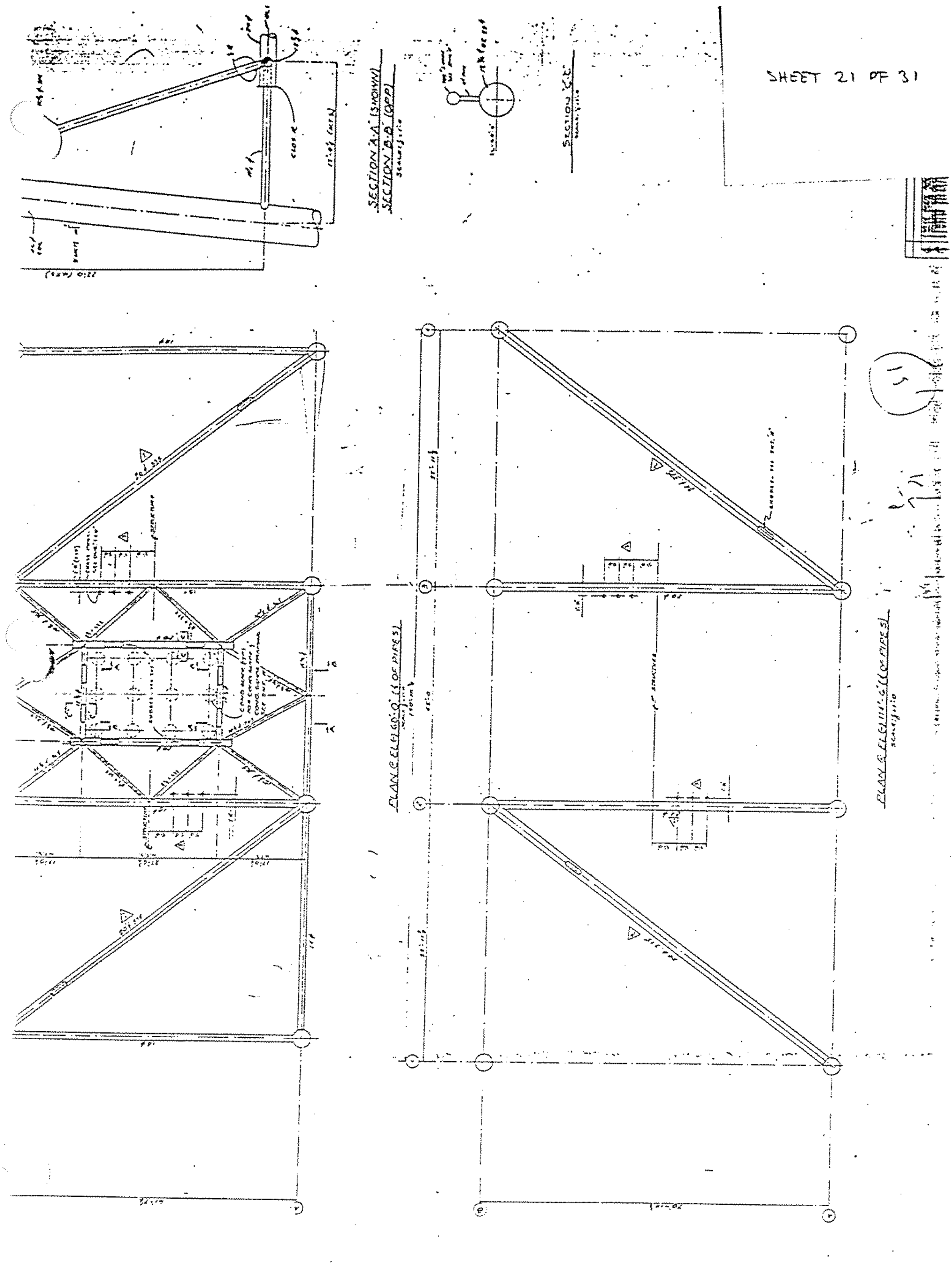
WALKWAY FRAMING PLAN AT EL. +10'-0⅝" TOP OF GRTG.
(TOP OF STEEL AT EL. +9'-11" UNLESS NOTED OTHERWISE)

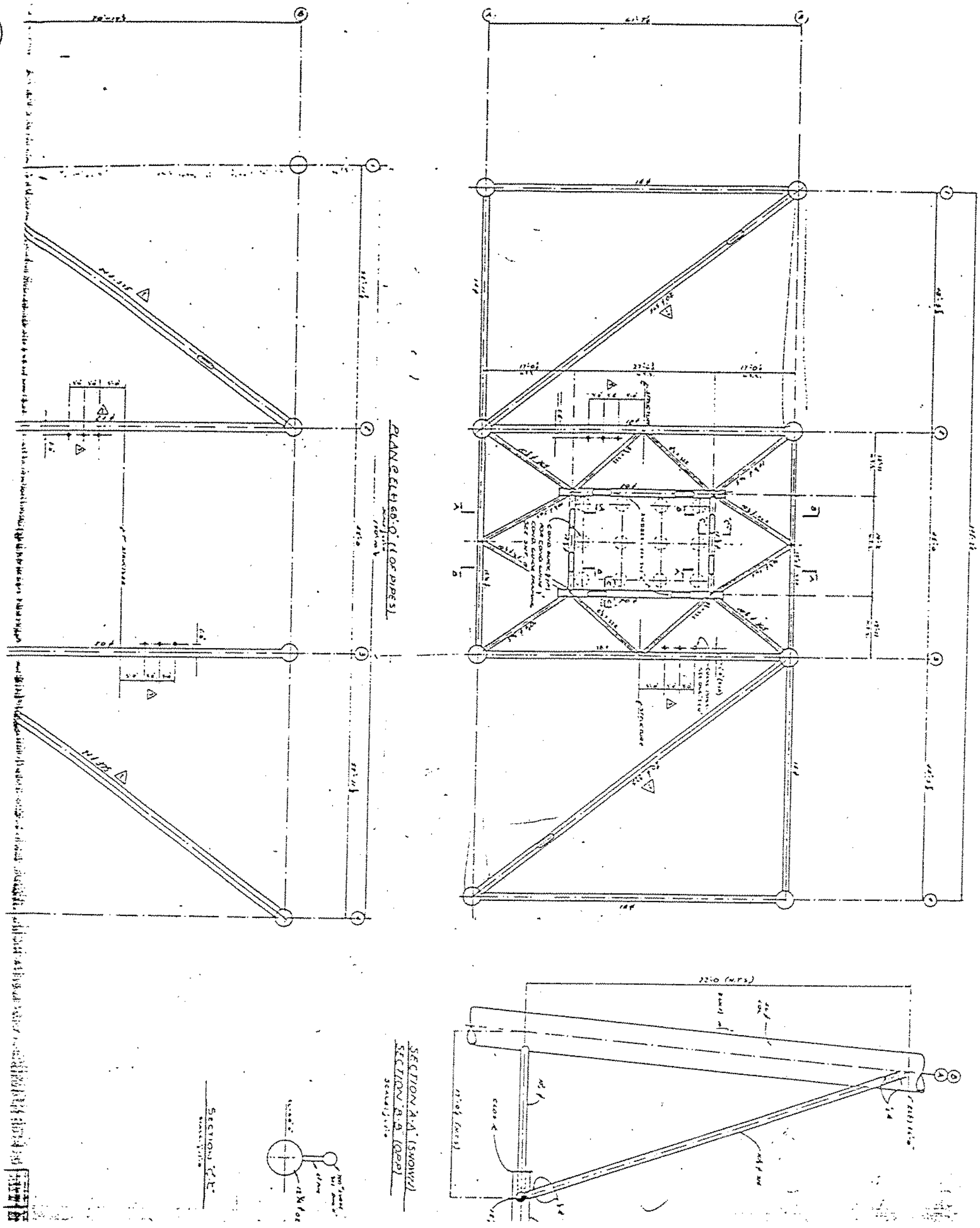


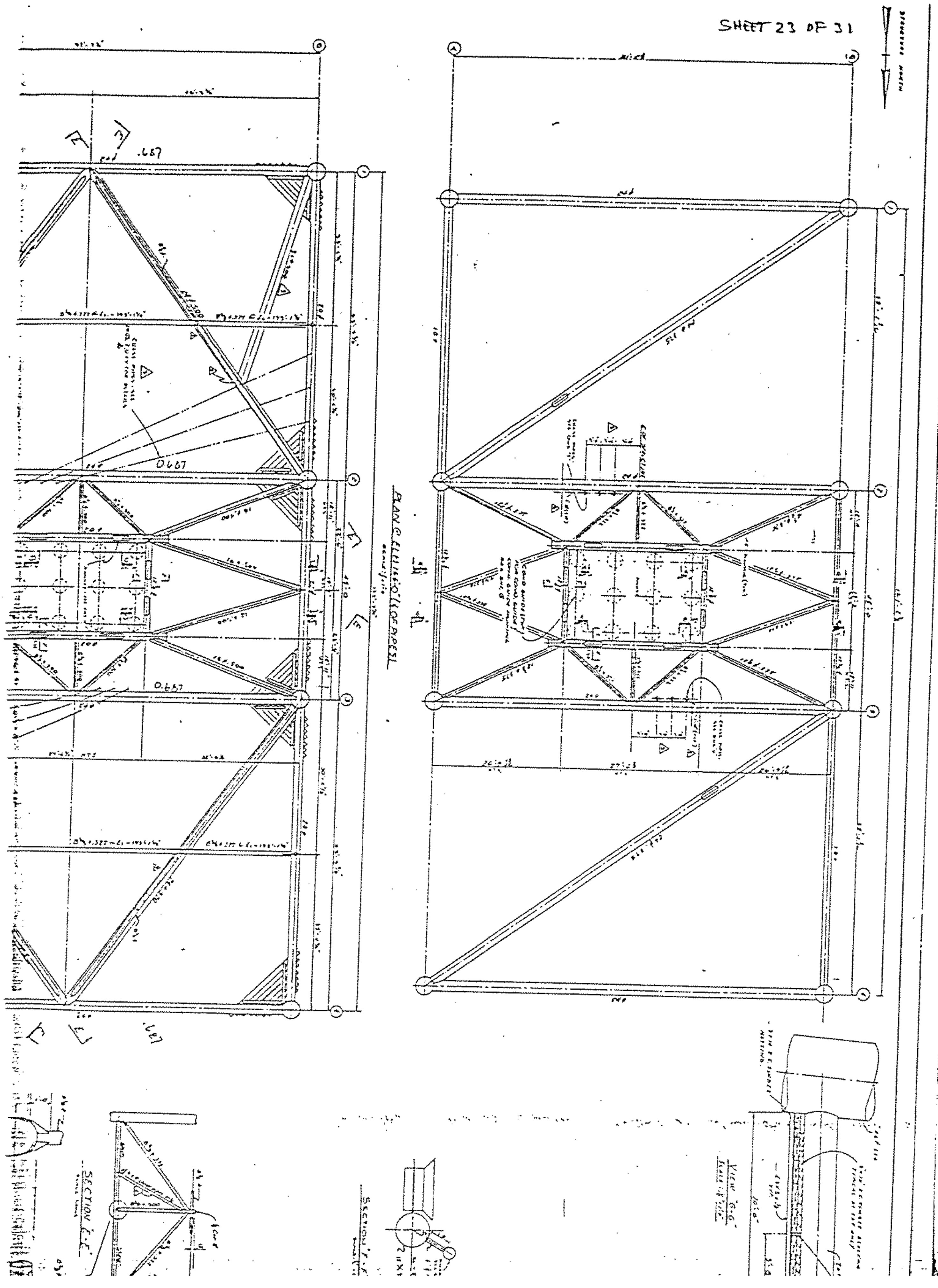
SECTION "A-A" (SHOWN)
SECTION "B-B" (OPPOSITE HAND)
⅜" = 1'-0

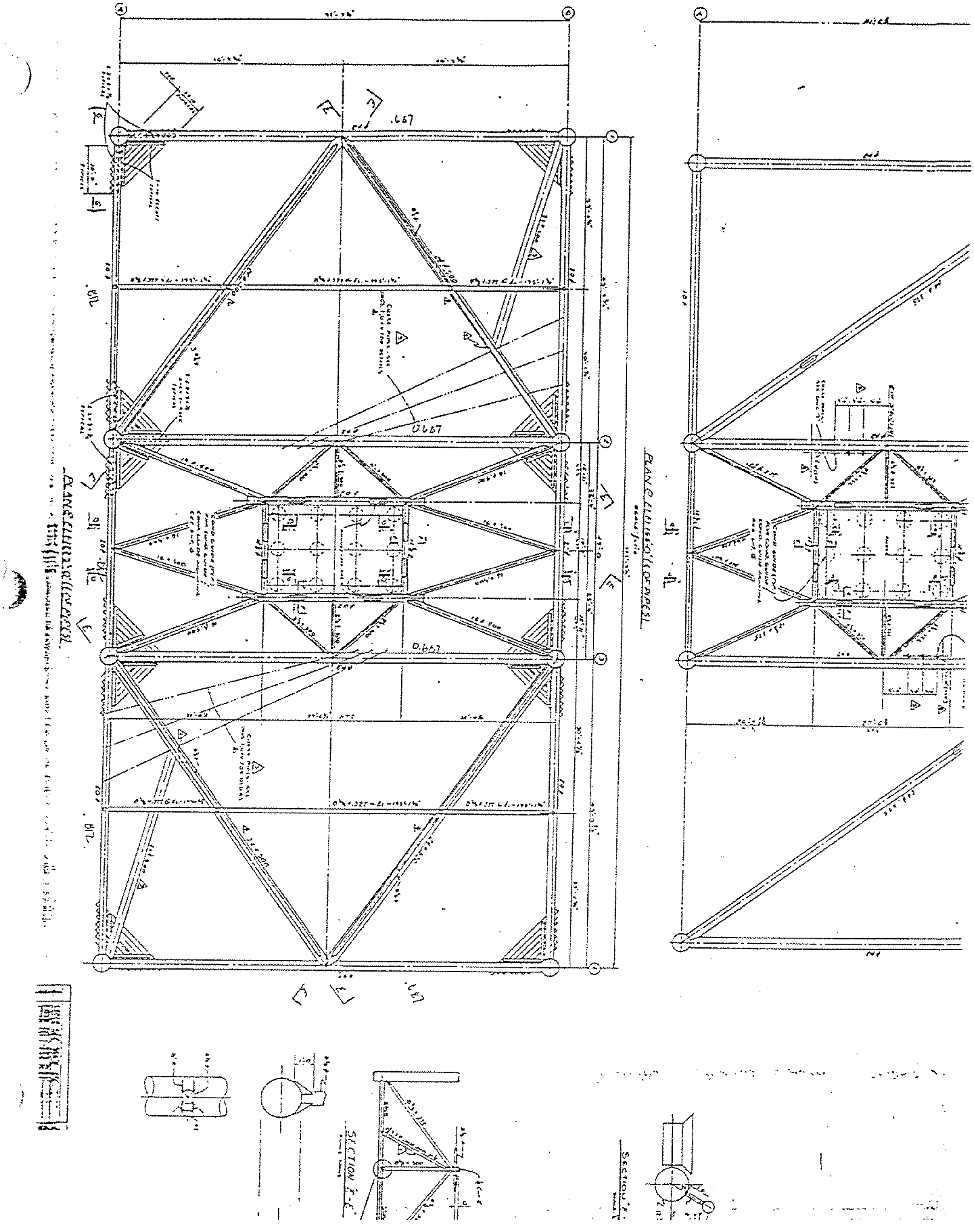


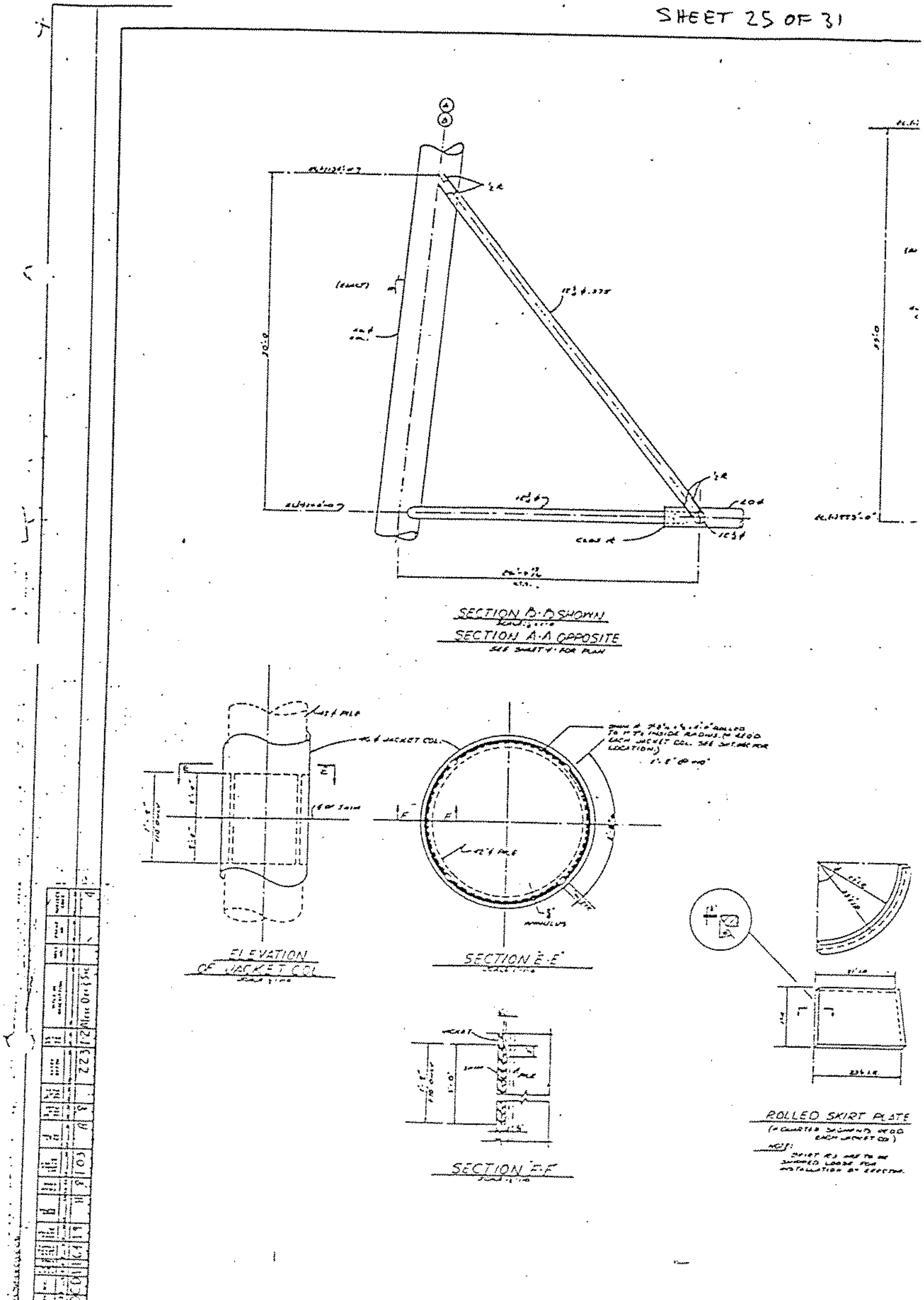
FRAMING PLAN AT EL. -26'-6"
⅜" = 1'-0

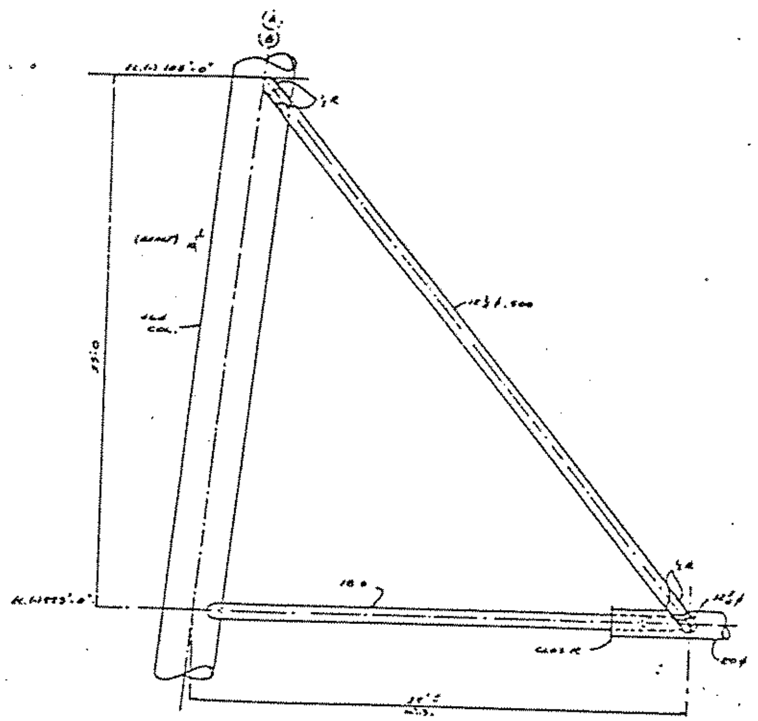

SECTION 'A-A' (SHOWN)
SECTION 'B-B' (OPP)
SECTION 'C-C'


PLAN @ EL(+)60'-0" (C.L OF PIPES)
SECTION 'A-A' (SHOWN)
SECTION 'B-B' (OPP)
SECTION 'C-C'





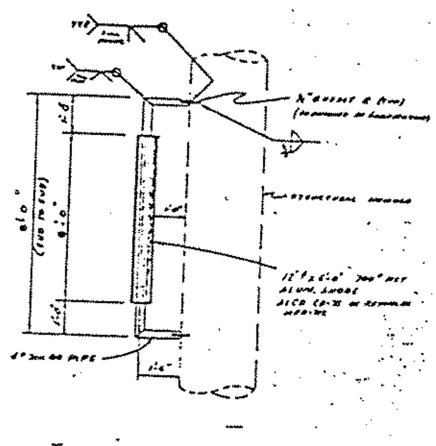

SECTION B-B SHOWN
SECTION A-A OPPOSITE
ELEVATION
OF JACKET COL
SECTION E-E
SECTION F-F
ROLLED SKIRT PLATE



SECTION D-D SHOWN

SECTION C-C OPPOSITE

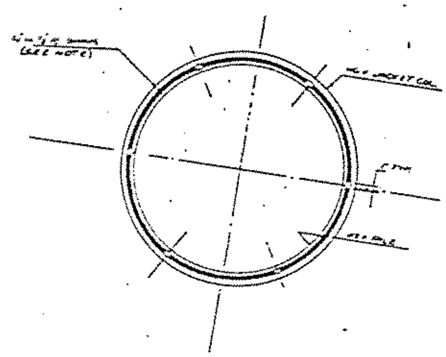
SEE SHEET-5- FOR PLAN



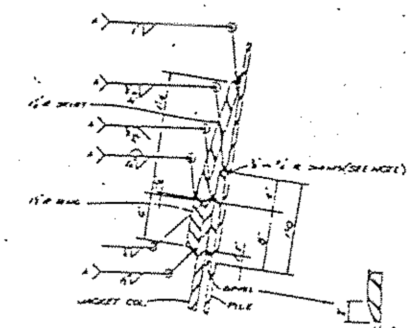
TYPICAL ANODE INSTALLATION

NO SCALE

170-REQD.

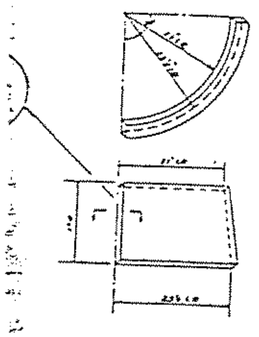


JACKET TO PILE CONNECTION



DETAIL "1"

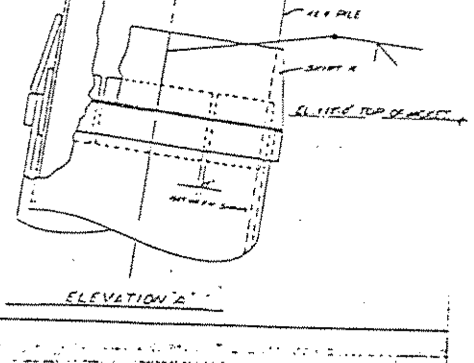
NOTE 1:
100°F MINIMUM PREHEAT & INTERPASS TEMPERATURE W/E 70[illegible] ELECTRODES.



ROLLED SKIRT PLATE

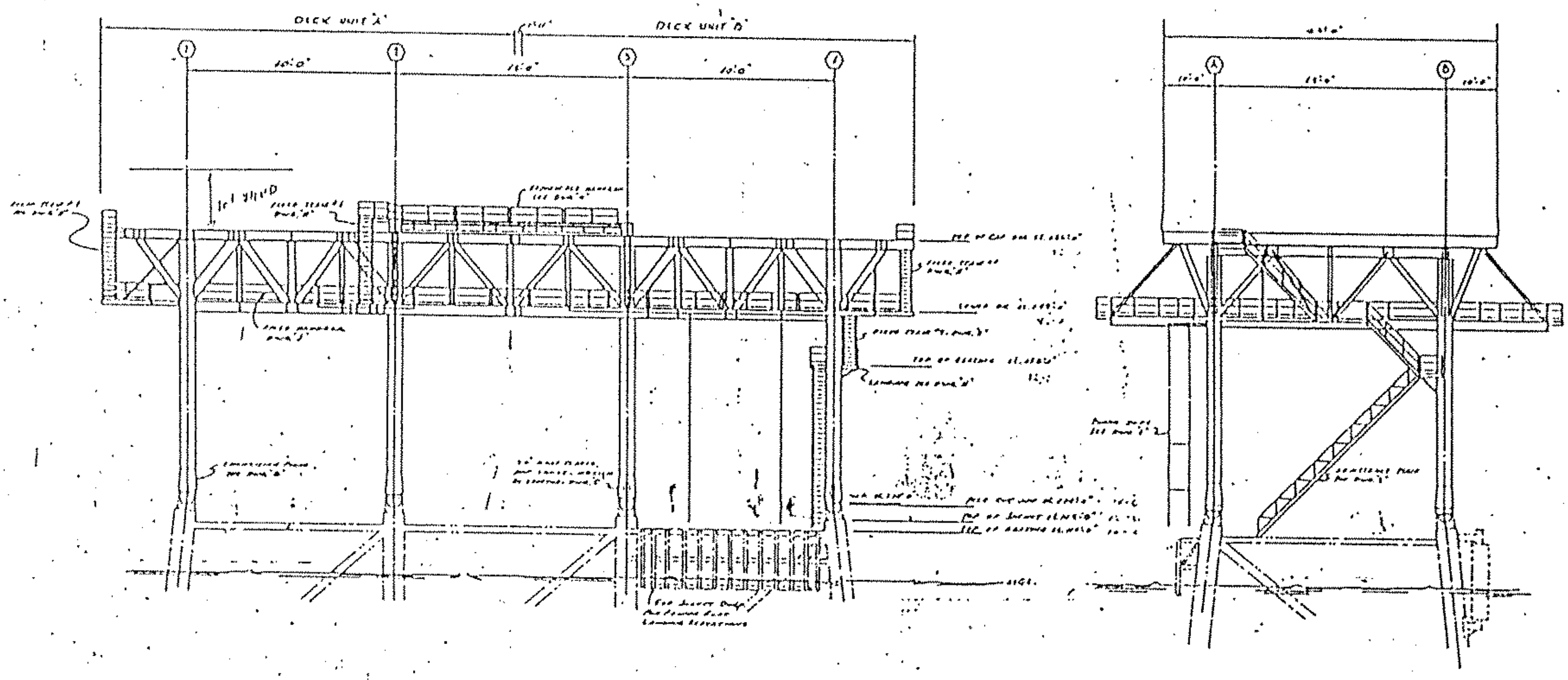
(4 QUARTER SEGMENTS REQD EACH JACKET COL.)

NOTE:
SKIRT PL. ARE TO BE SHIPPED LOOSE FOR INSTALLATION BY ERECTOR.

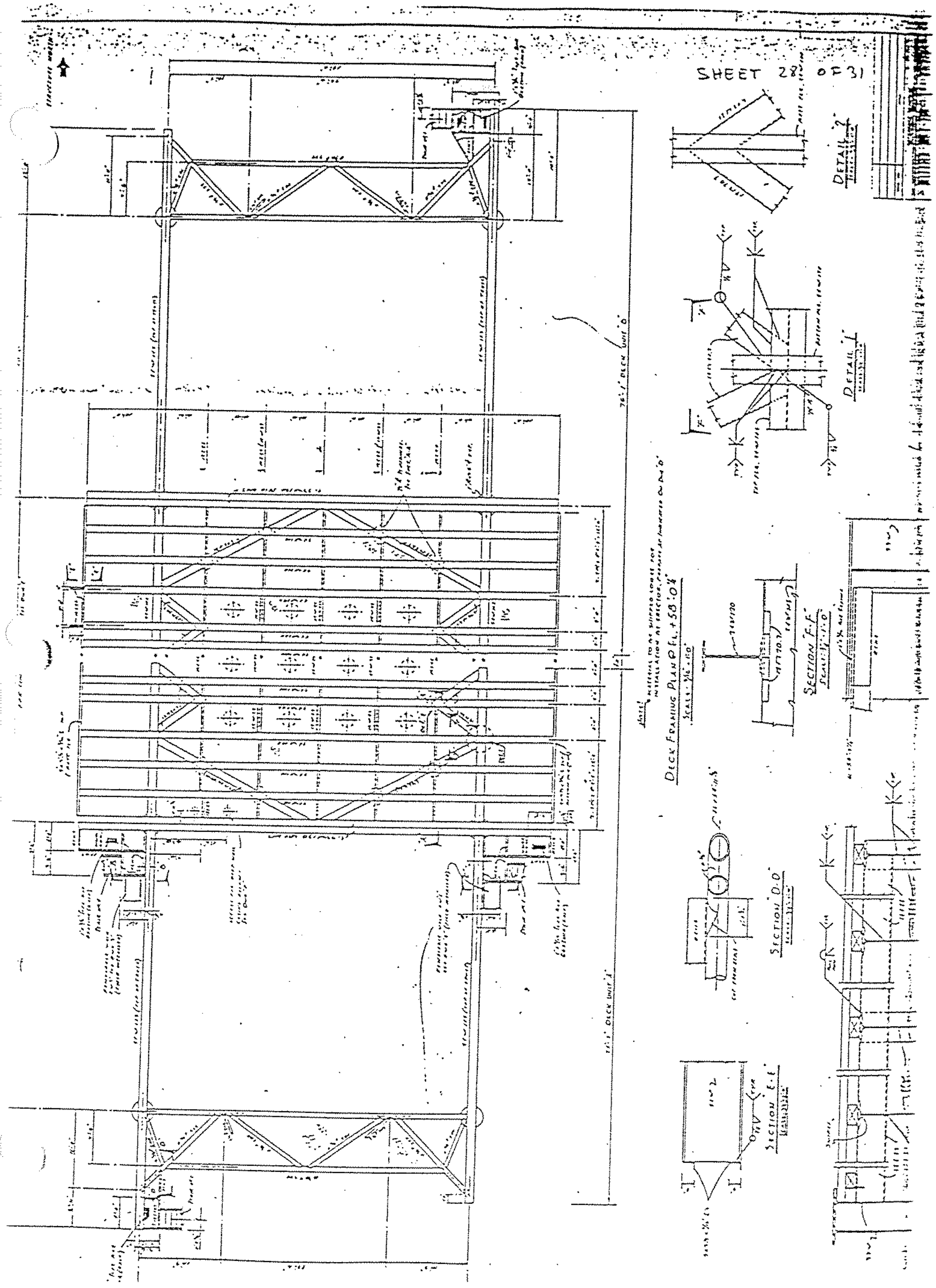


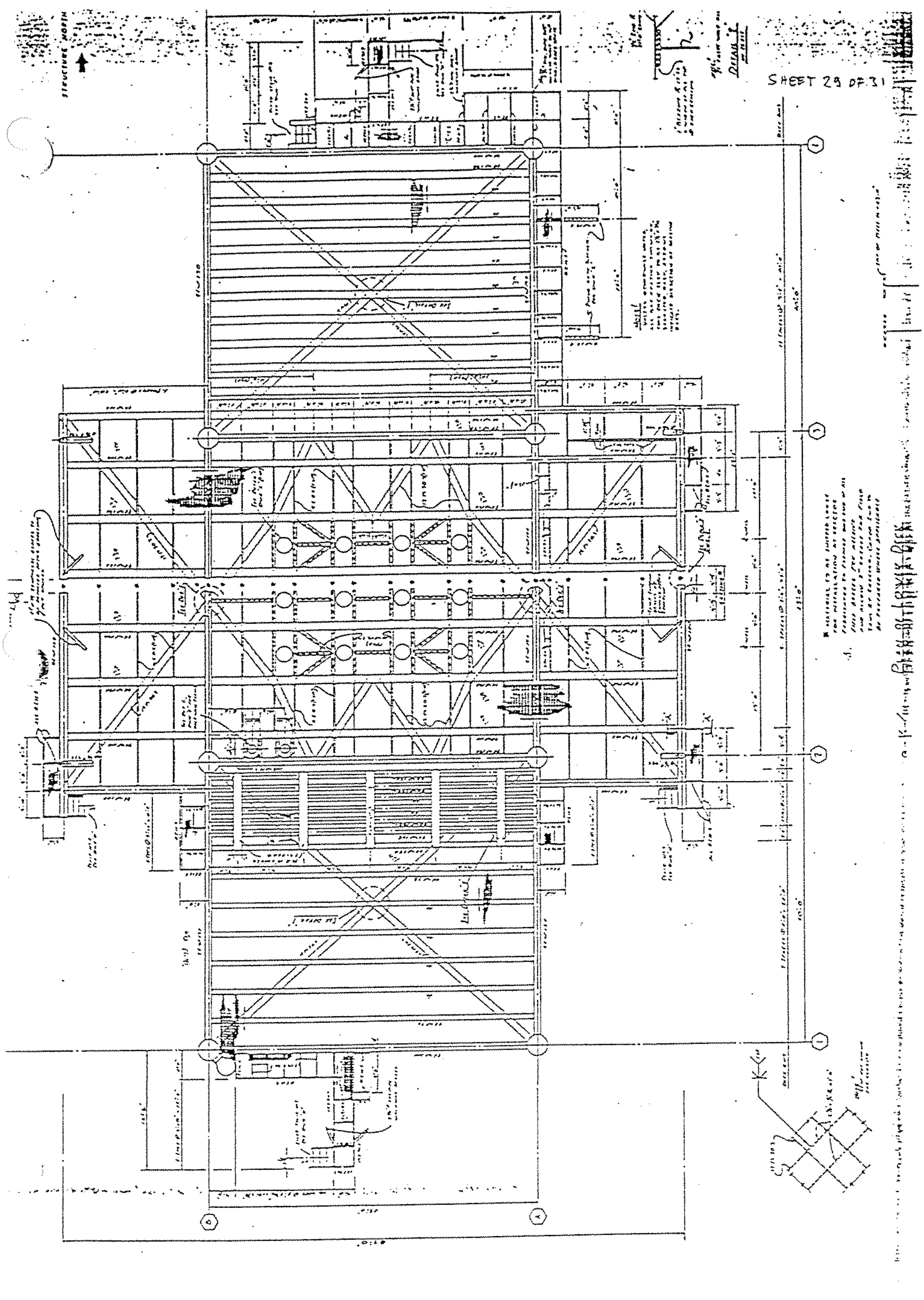
ELEVATION "A"

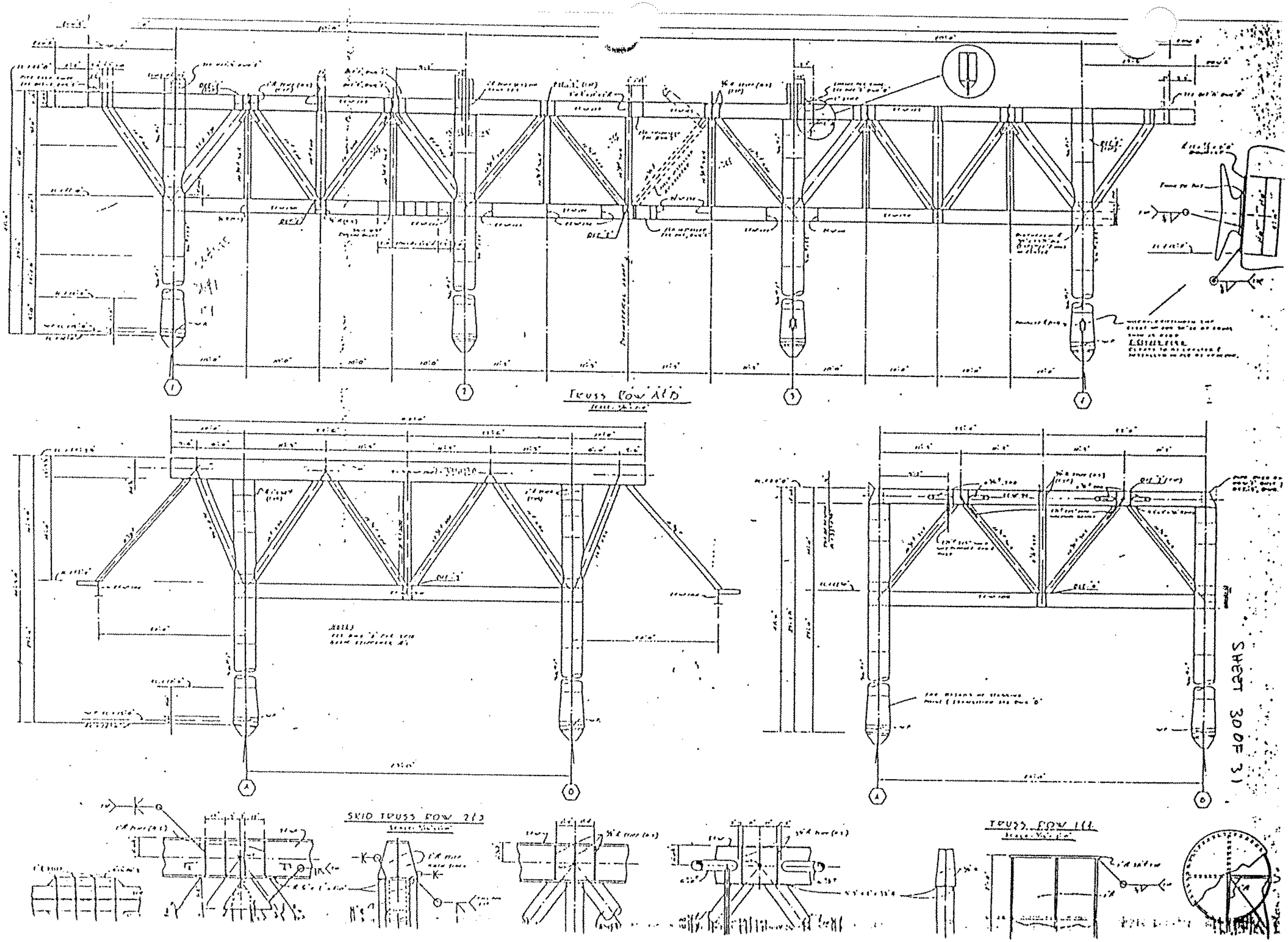
NOTE:
FABRICATOR TO FURNISH 4-1/4" SHIMS & 4-1/8" SHIMS PER LEG. ERECTOR TO INSTALL ALL 1/4" SHIMS IF POSSIBLE. 1/8" SHIMS MAY BE ALLOWED OR SHALL IF IMPOSSIBLE TO USE 1/4" [illegible].

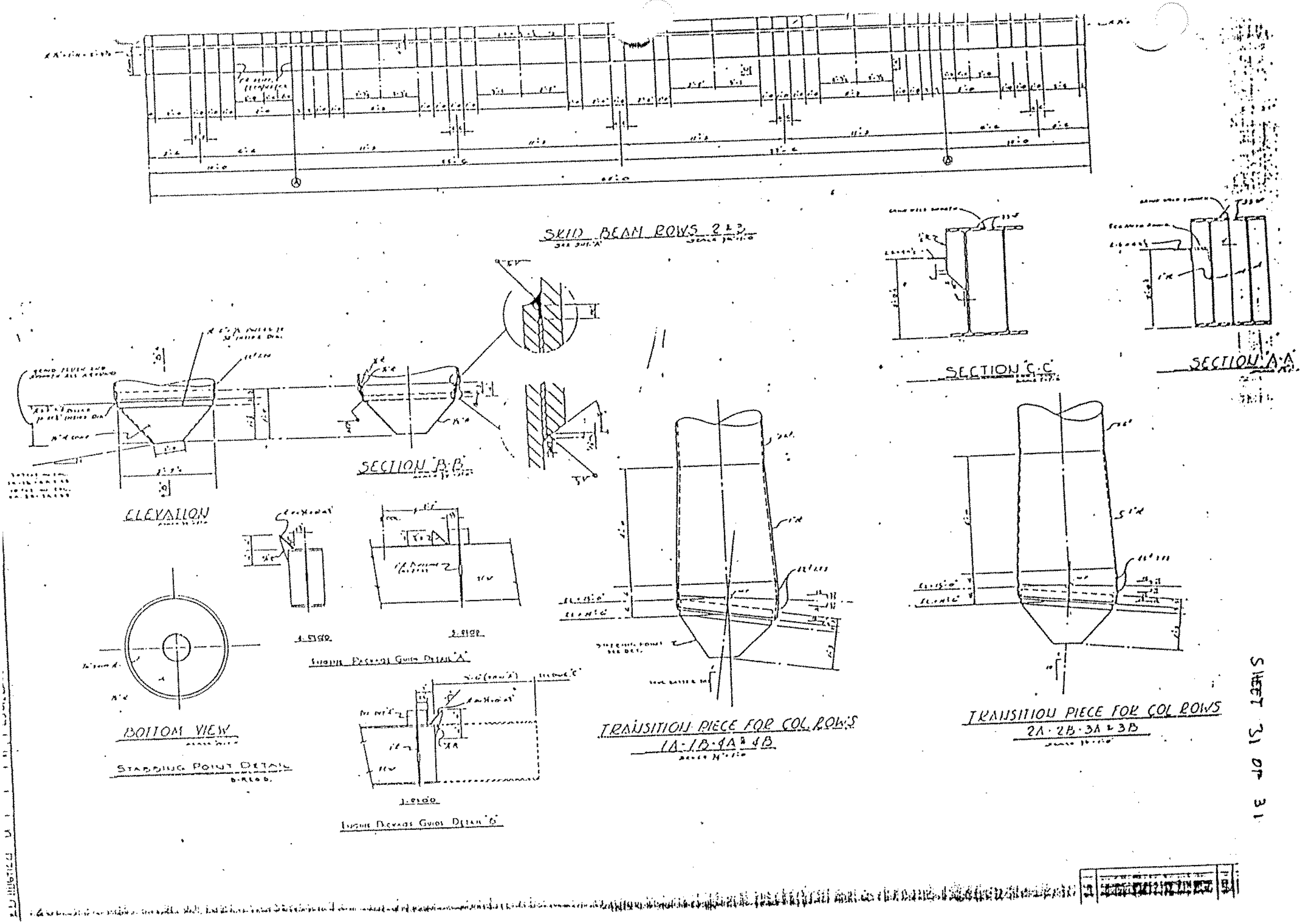


ELEVATIONS


SHEET 28 OF 31
DETAIL '2'
DETAIL '1'
SECTION 'F-F'
DECK FRAMING PLAN @ EL. +58'-0"
SECTION 'D-D'
SECTION 'E-E'




TRUSS ROW A/D
SKID TRUSS ROW 2/3
TRUSS ROW 1/1
SHEET 30 OF 31


SKID BEAM ROWS 2 & 3
SECTION C-C
SECTION A-A
ELEVATION
SECTION B-B
BOTTOM VIEW
STABBING POINT DETAIL
TRANSITION PIECE FOR COL ROWS
1A-1B-4A & 4B
TRANSITION PIECE FOR COL ROWS
2A-2B-3A & 3B

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DESIGN LEVEL ANALYSIS OUTPUT .... BOOK #2

ULTIMATE STRENGTH ANALYSIS OUTPUT .... BOOK #3

PUSHOVER STRENGTH ANALYSIS OUTPUT .... BOOK #4

PERFORMED BY:

W. H. LINDER & ASSOCIATES, INC.
3330 WEST ESPLANADE AVENUE
METAIRIE, LOUISIANA 70002
(504) 835-2577

JULY, 1994

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1. INTRODUCTION

1.1 Scope & History

This study deals with the structural assessment of Shell Oil Co. 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).

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 Shell's 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 Shell Oil Co.'s permission to use it for the assessment exercise.

3. SUMMARY OF RESULTS

Shell's 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'

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 Shell Oil 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 Shell Offshore Inc., New Orleans, LA. Fax from M. E. Celebi of W. H. Linder & Associates, to Mark Hanson of Shell 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 Shell Offshore Inc., "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.

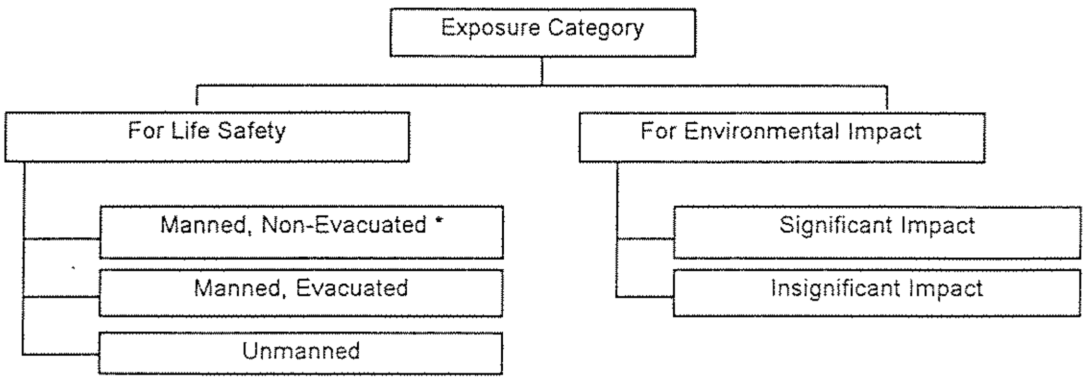
There are six major components used during this process:

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.

Shell 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 Shell 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 <u>Design Basis Check</u>

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).

7. Calculations

This section contains following group of calculations

7.1 Computer Input

- Computer Model / Geometric Description
- Joint Loads on Platform Jacket & Decks
- Wind, Wave and Current Loads (Metoceanic Criteria)
- Soil / Pile interaction

7.2 Design Level Analysis (45°, 67.5°, 90° directions)

7.3 Ultimate Strength Analysis (45°, 67.5°, 90° directions)

7.4 Pushover Analysis I

- for 45° direction only.
- Load redistribution during ultimate strength analysis when Pile 182 fails in bending.

7.5 Pushover Analysis II

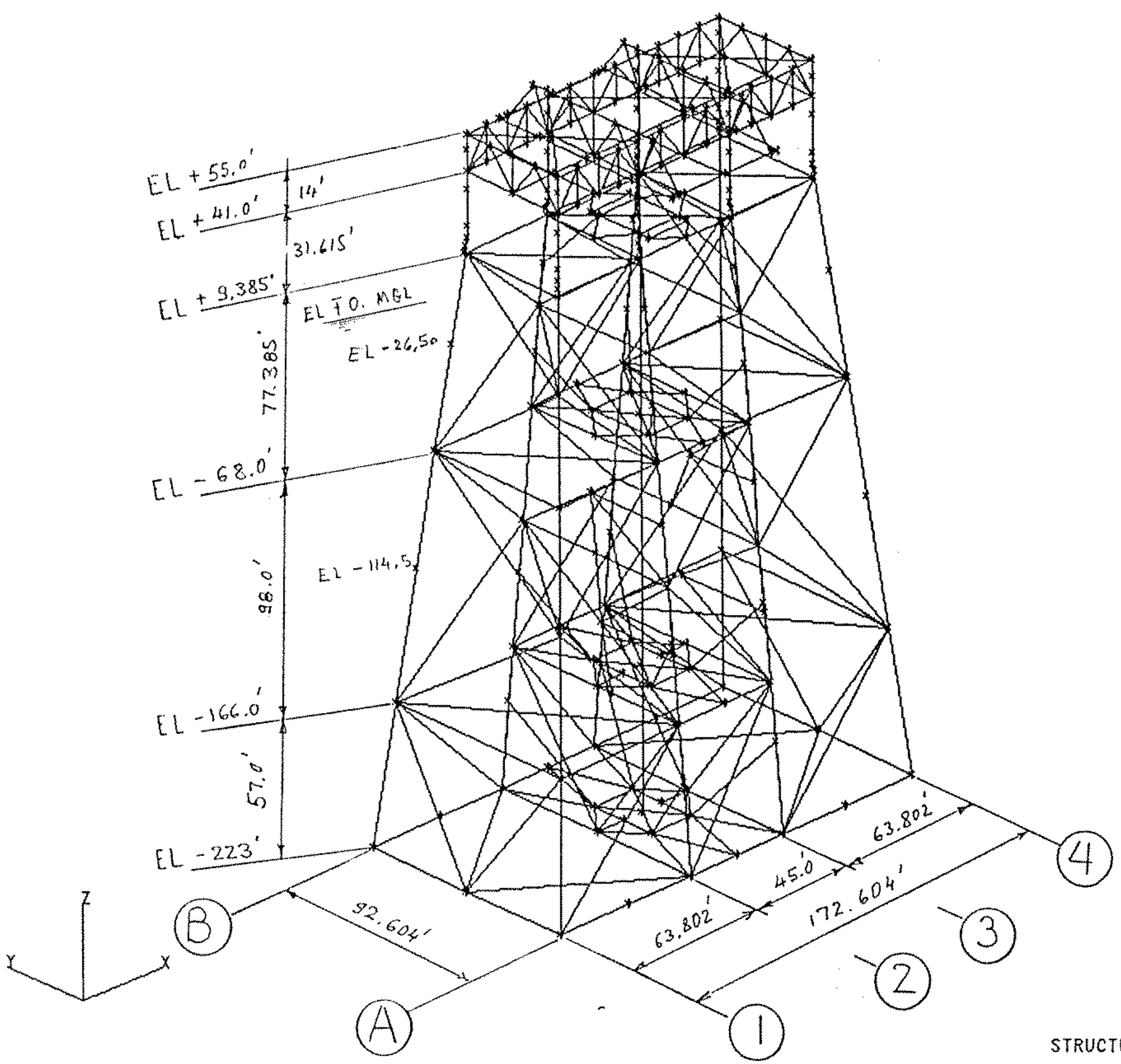
- for 90° direction only
- Failing diagonals in Jacket Plane Rows 1 thru 4 are removed from model, Ultimate Strength Loading in 90° direction is applied.

7.1. COMPUTER INPUT

7.1.1 3-D MODEL

SHELL WEST DELTA BLK 103A 223 FT WATERS

3D VIEW



SHEET 15

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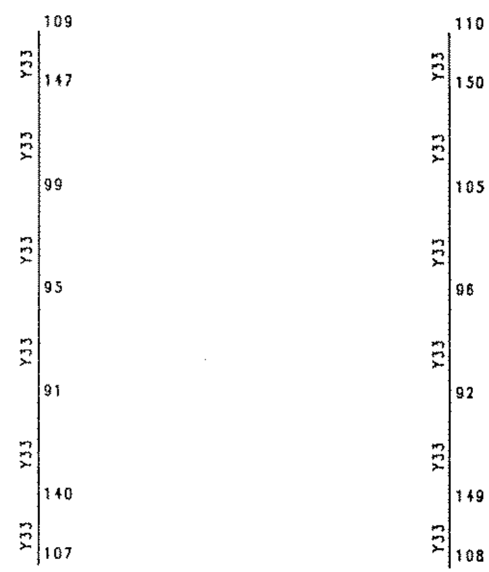
SHEET 16

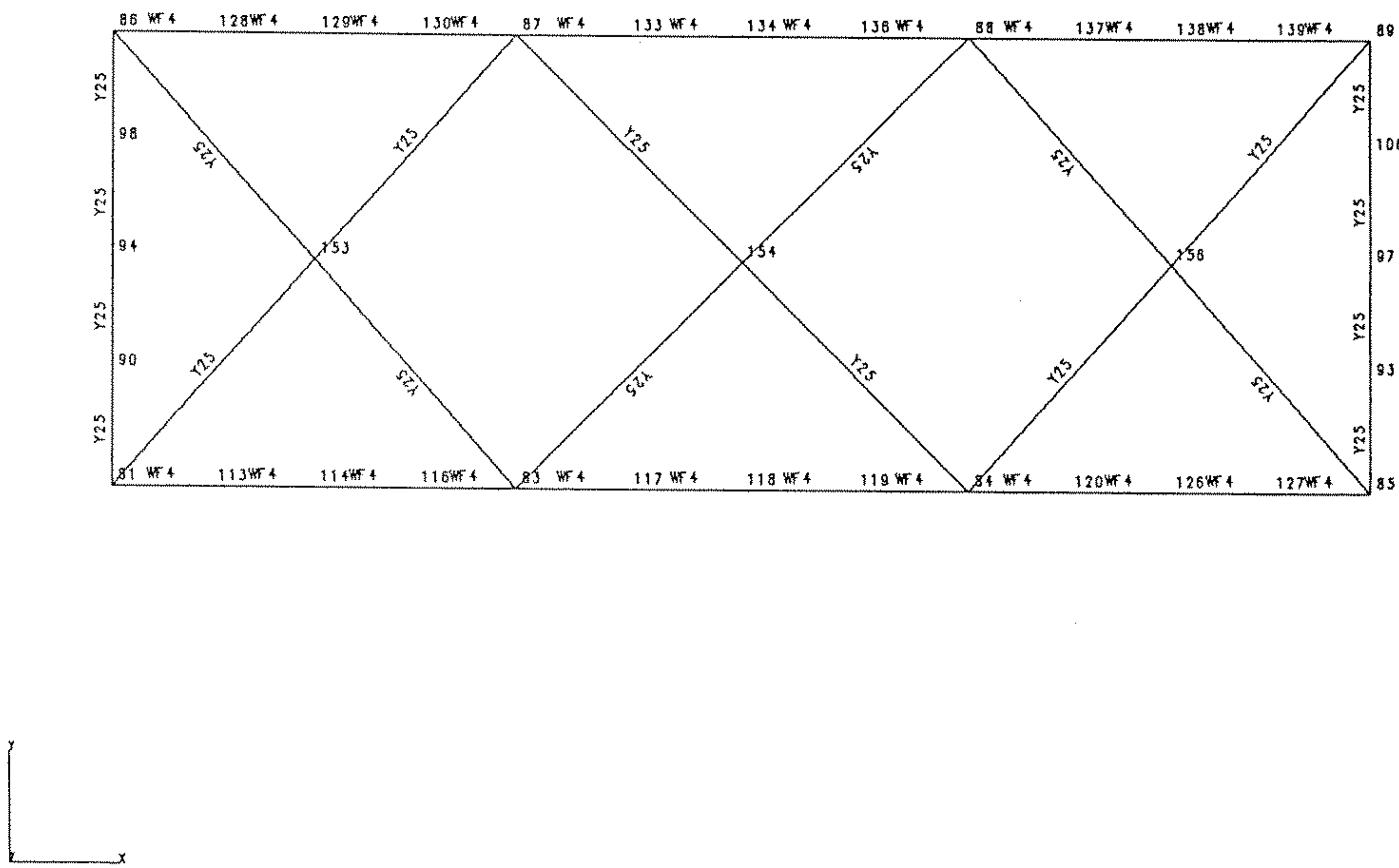
SHELL WEST DELTA BLK 103A 223 FT WATERS

UPPER DECK SKID BEAMS EL +57.38'



SHELL WEST DELTA BLK 103A 223 FT WATERS

UPPER DECK EL +55'



SHEET 17

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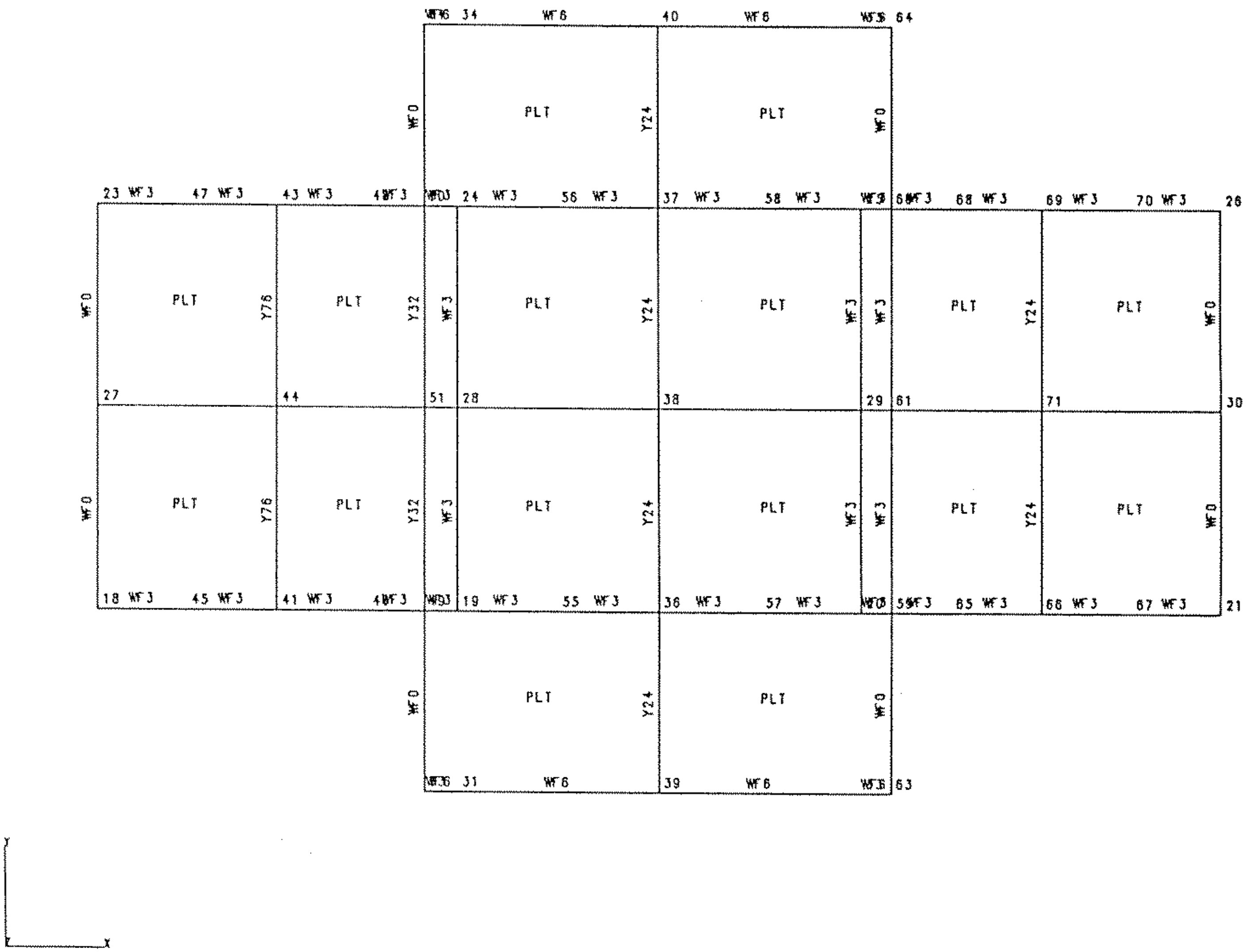
SHELL WEST DELTA BLK 103A 223 FT WATERS

LOWER DECK PLANE +41'



Sheet 18

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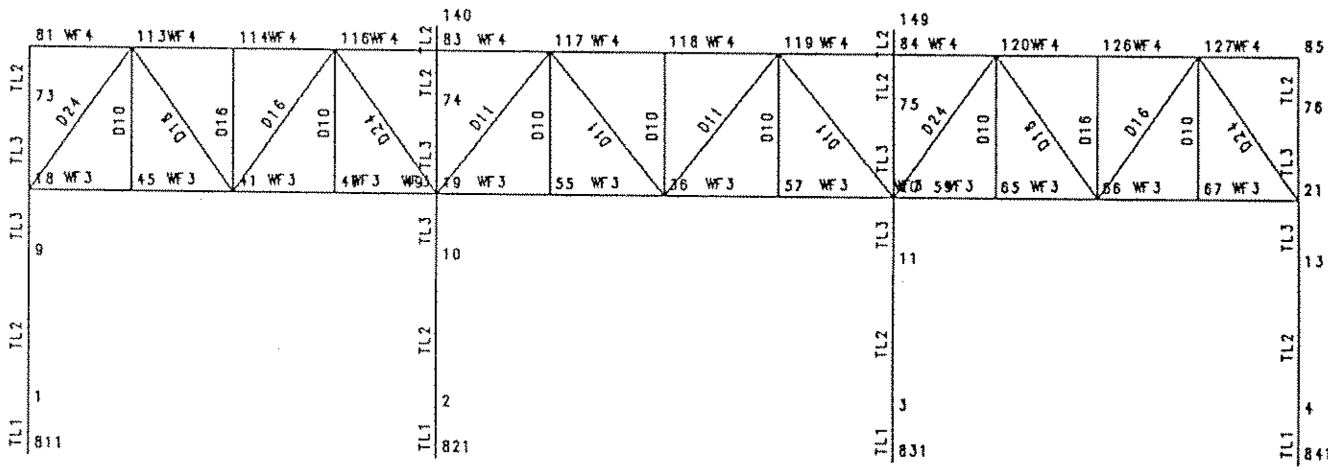
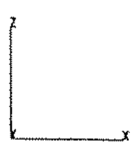
SHEET 19

SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS ROW A





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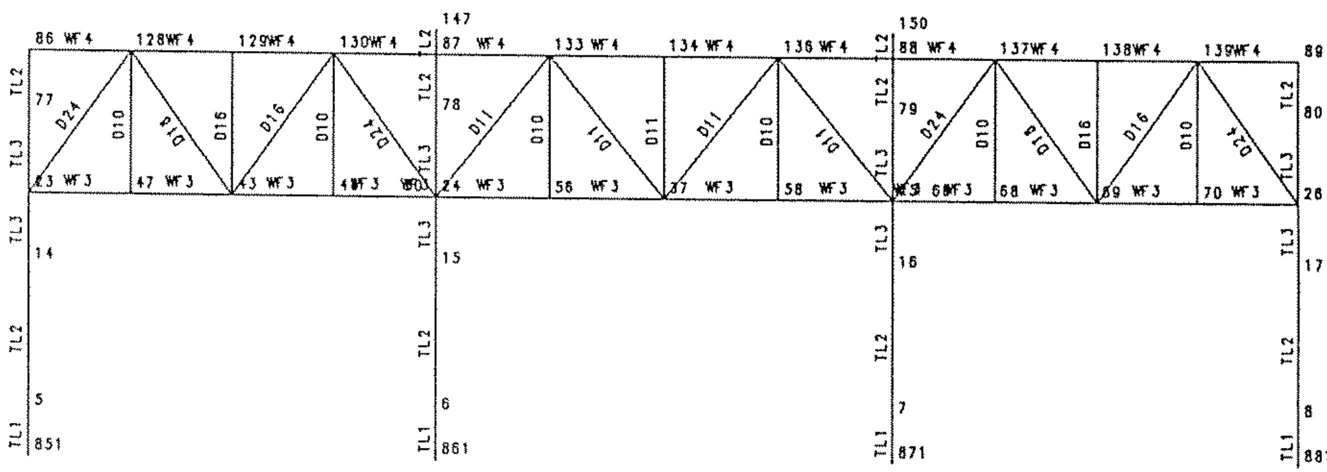
SHEET 20

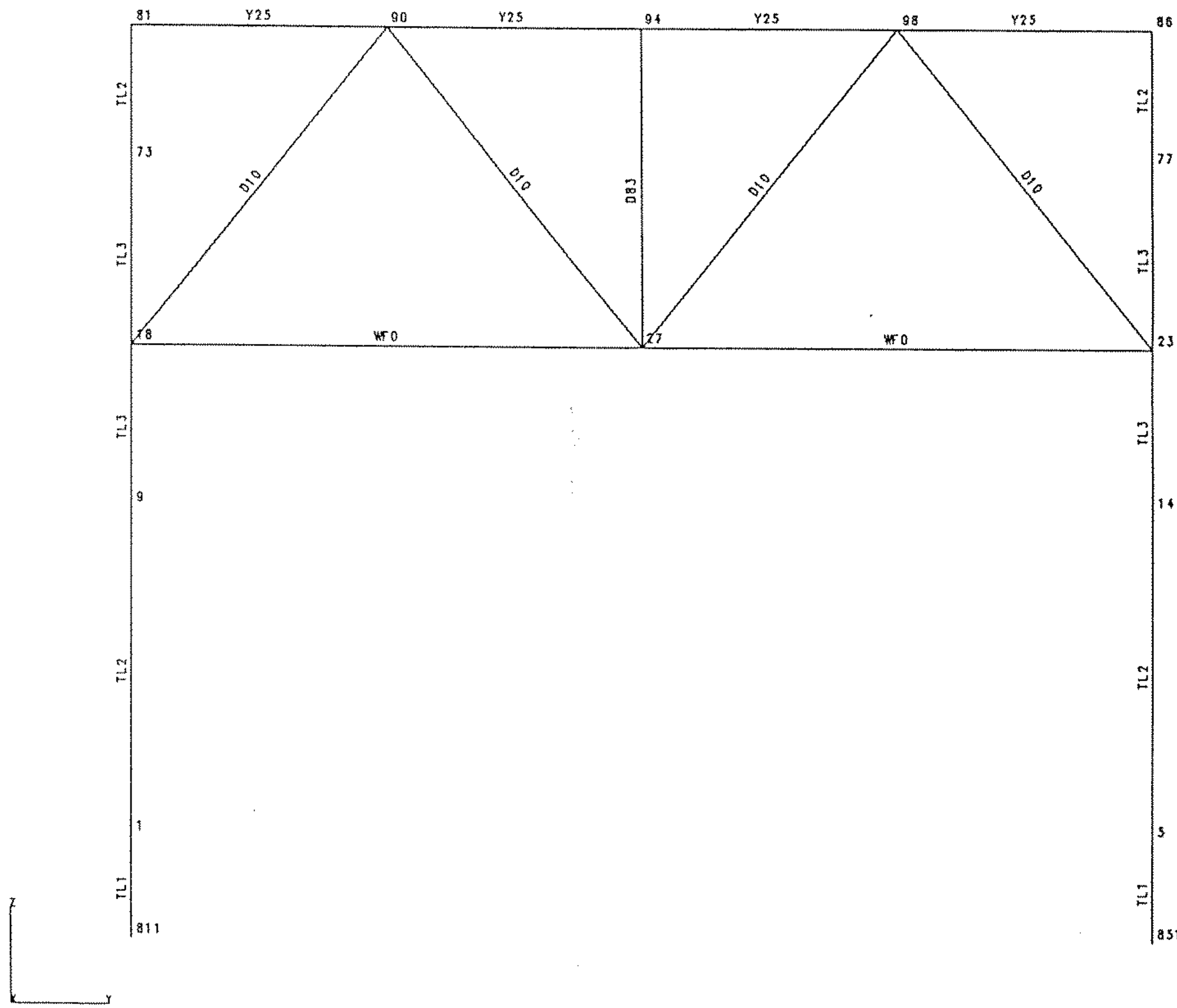
SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS ROW B



SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS FRAME VIEW ROW 1



SHEET 21

STRUCTURAL SOFTWARE, INC

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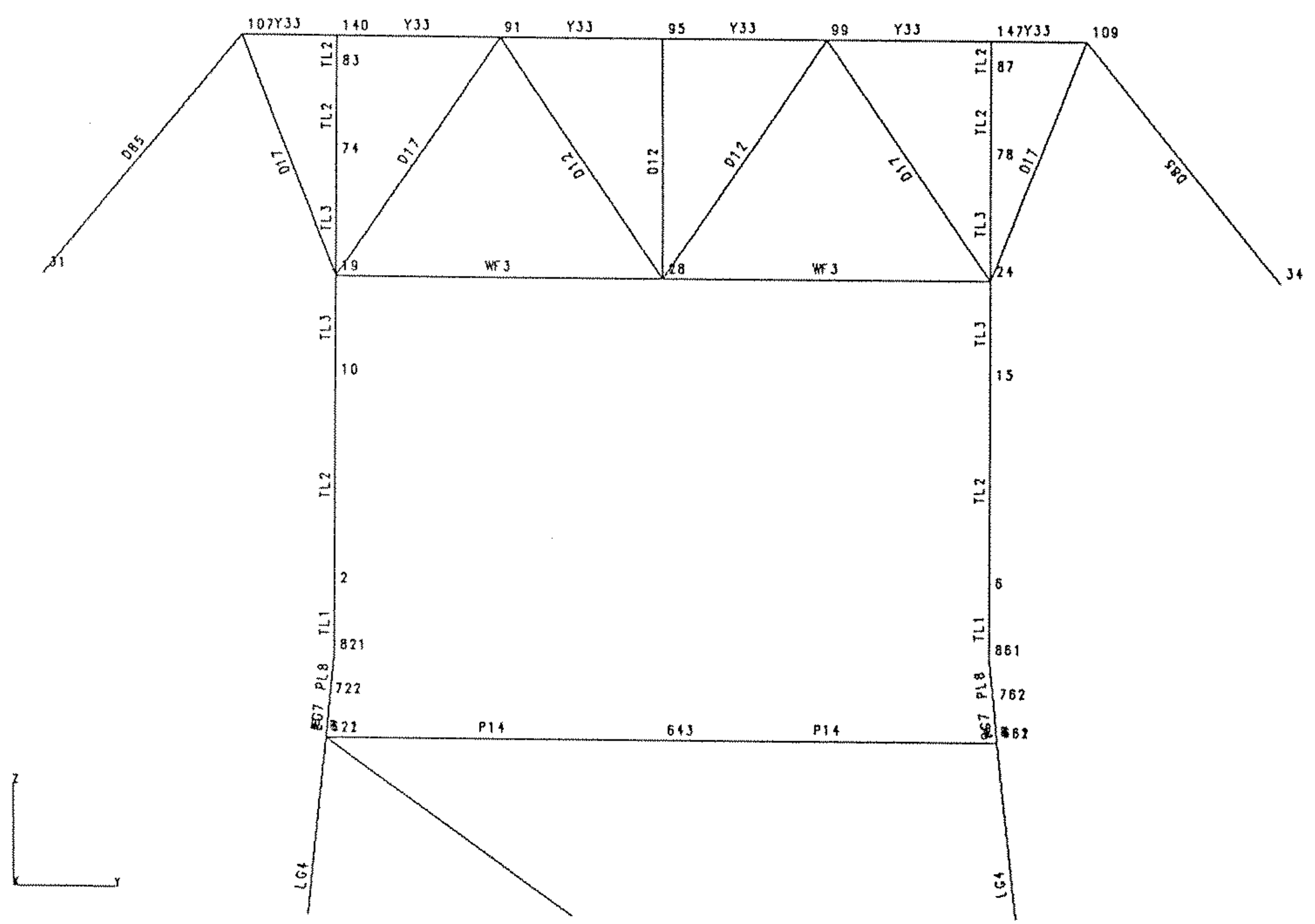
SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS FRAME VIEW ROW 2

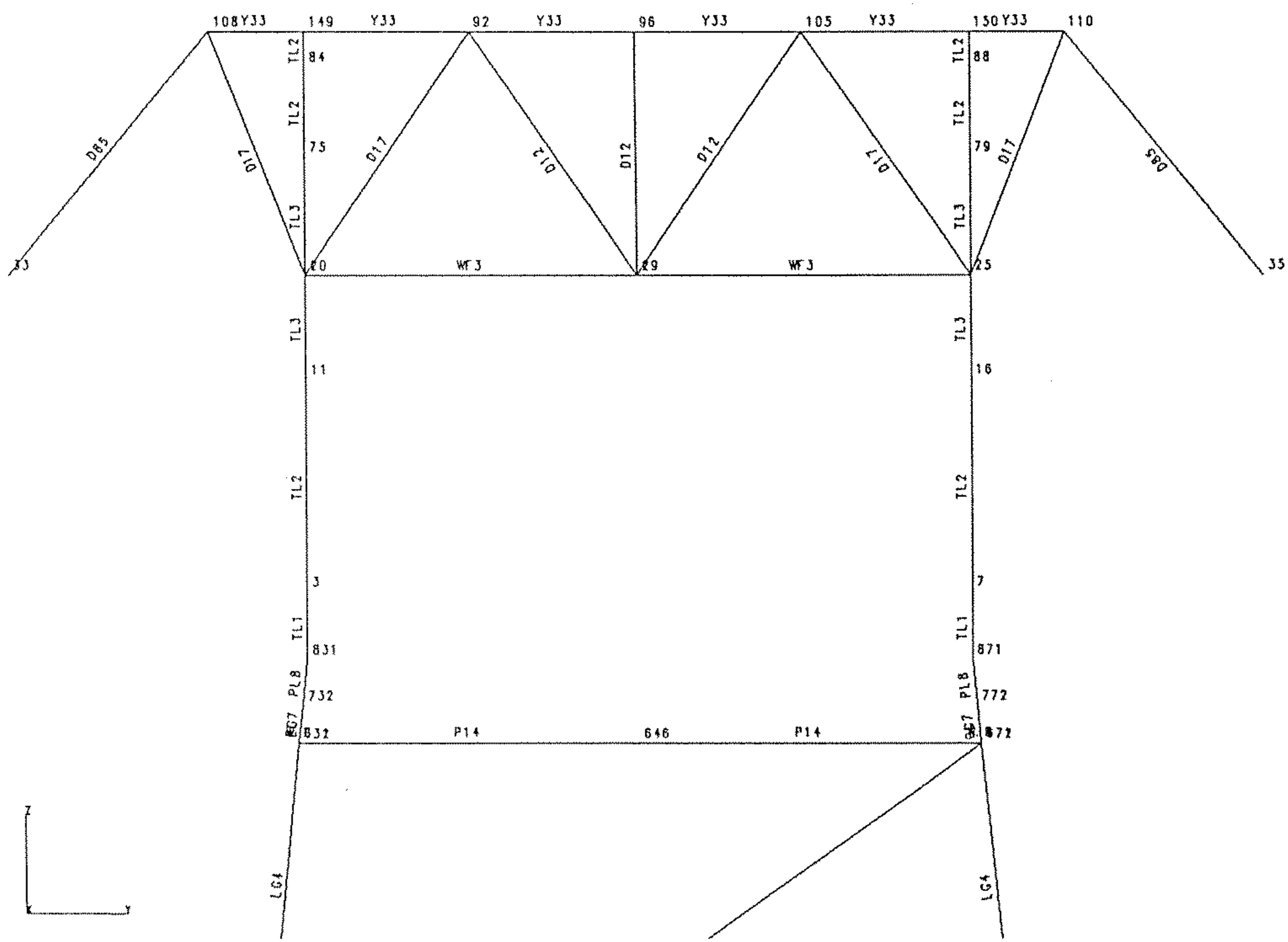


SHEET 22

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS FRAME VIEW ROW 3

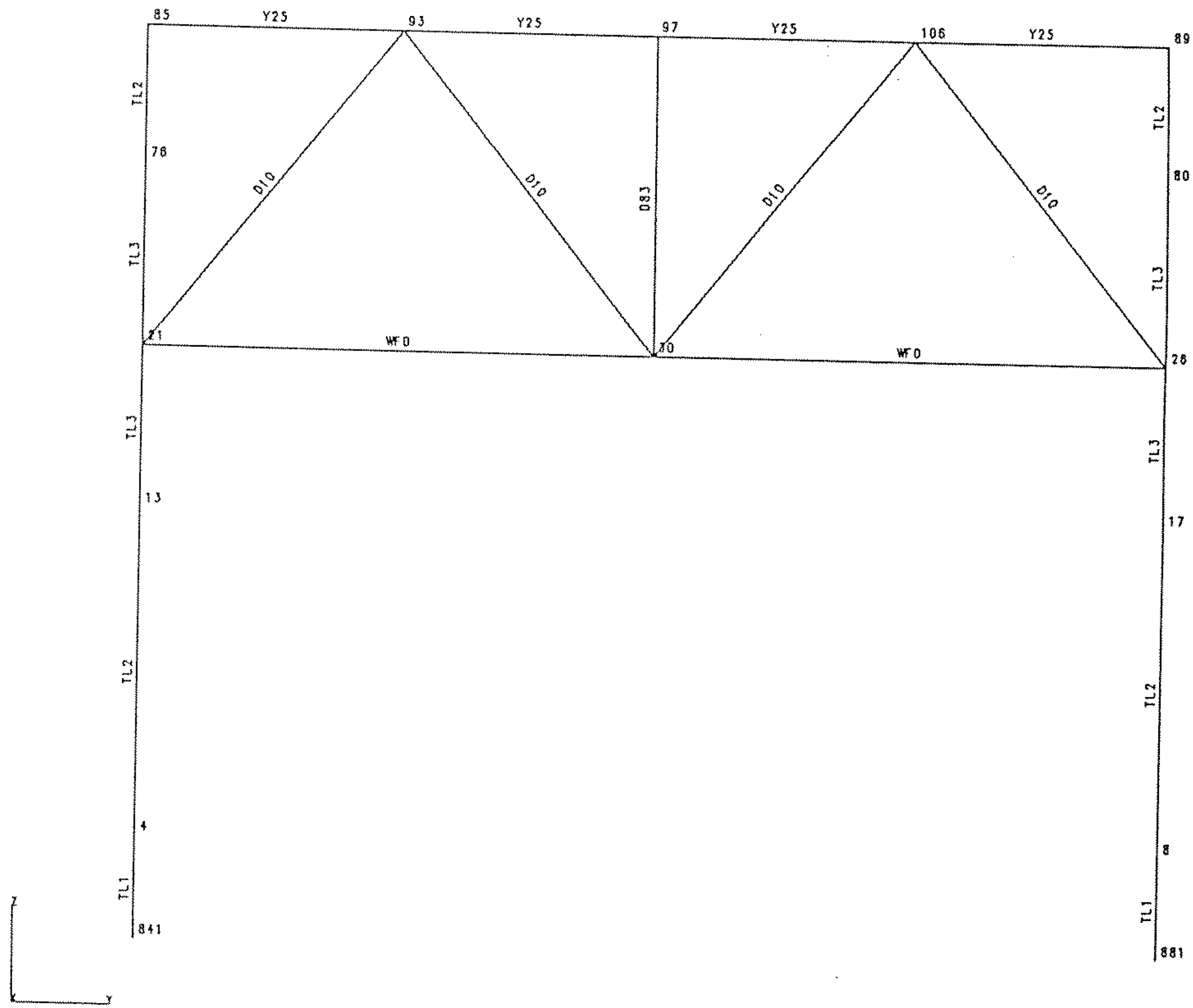


SHEET 23

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

DECK TRUSS FRAME VIEW ROW 4

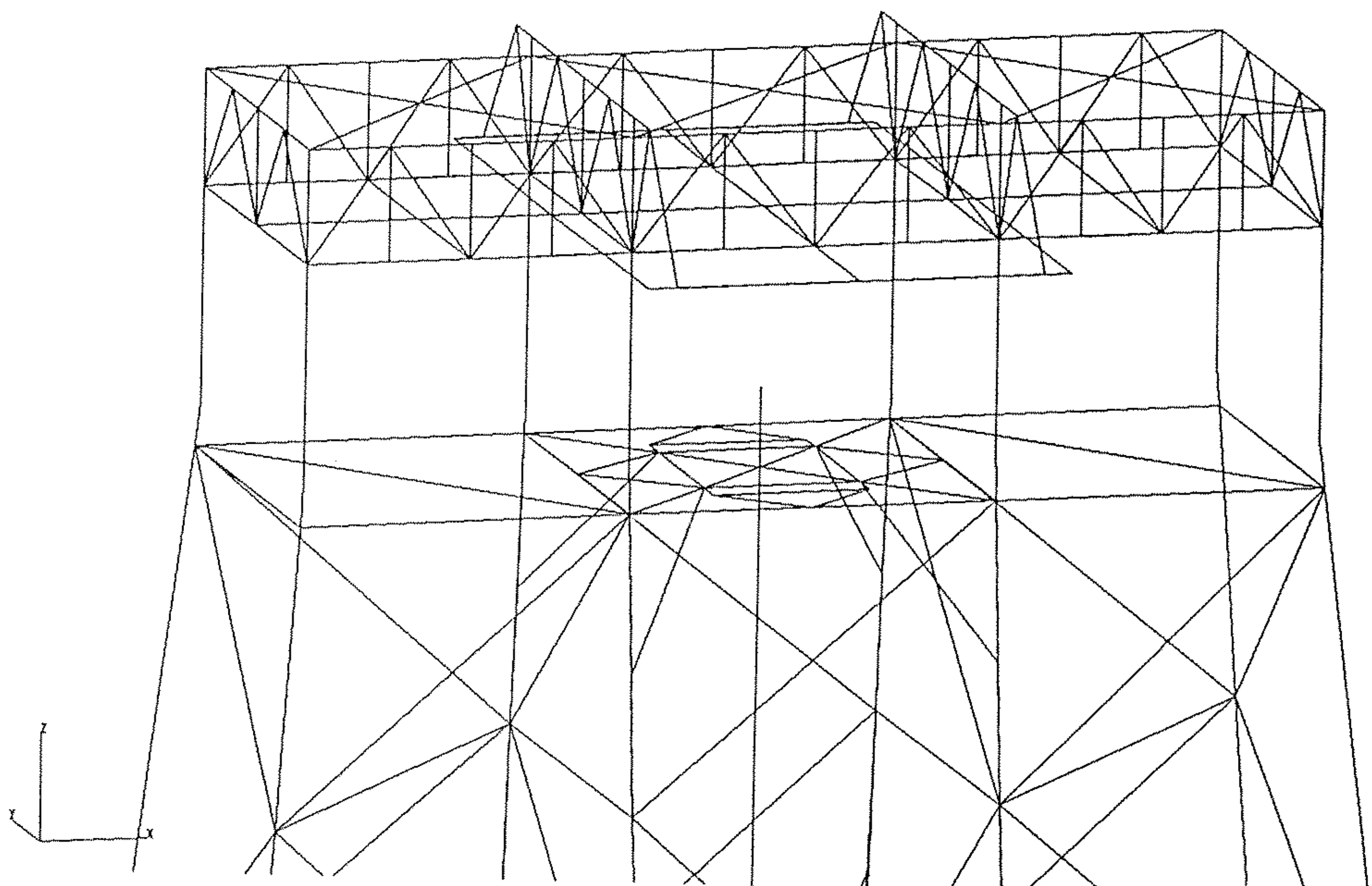


SHEET 24

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

JACKET TOPSIDE & LOWER ; UPPERDECK PERSPECTIVE VIEW

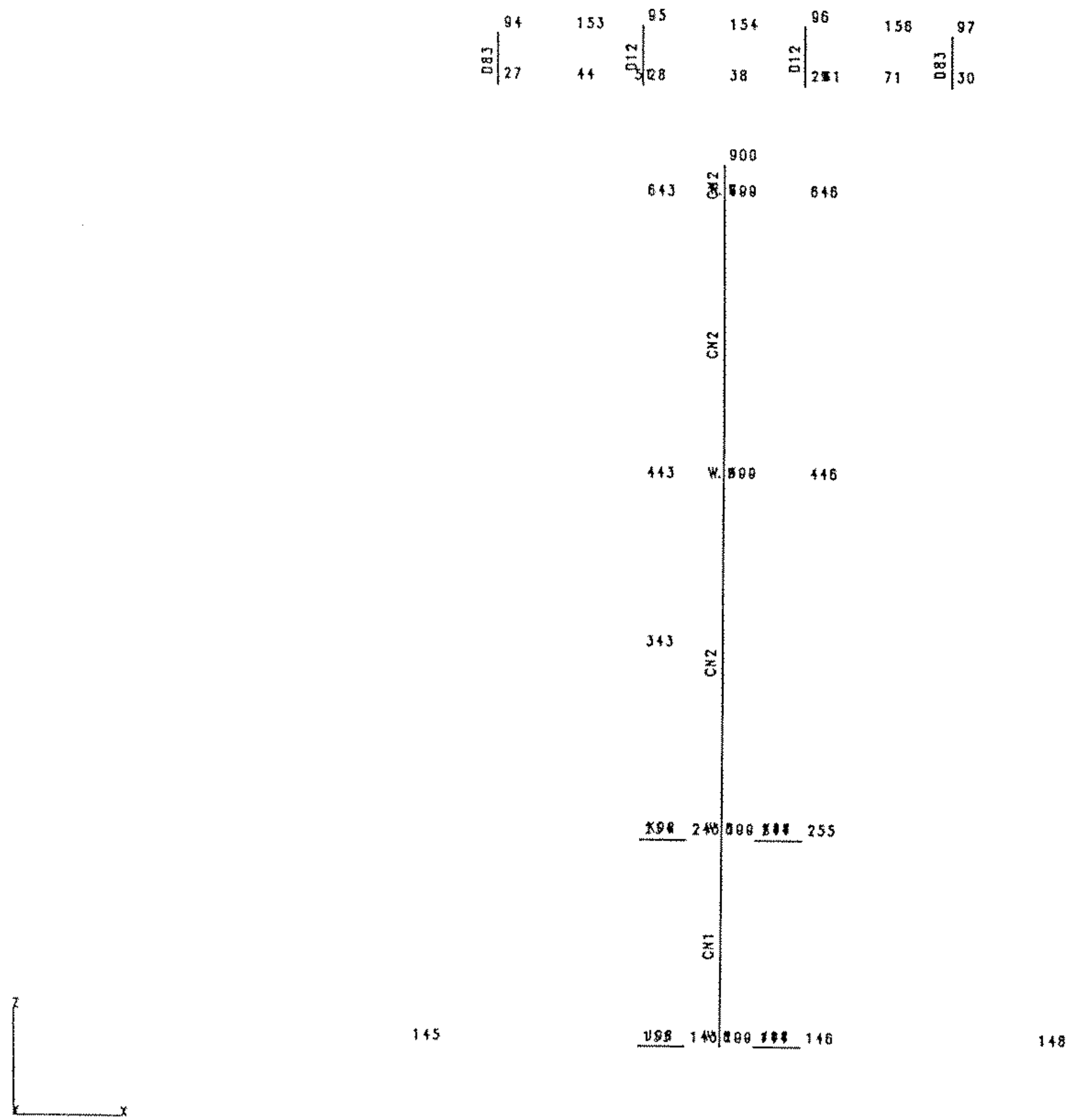


SHEET 25

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

CONDUCTORS & VERT. PLANE Y=0'

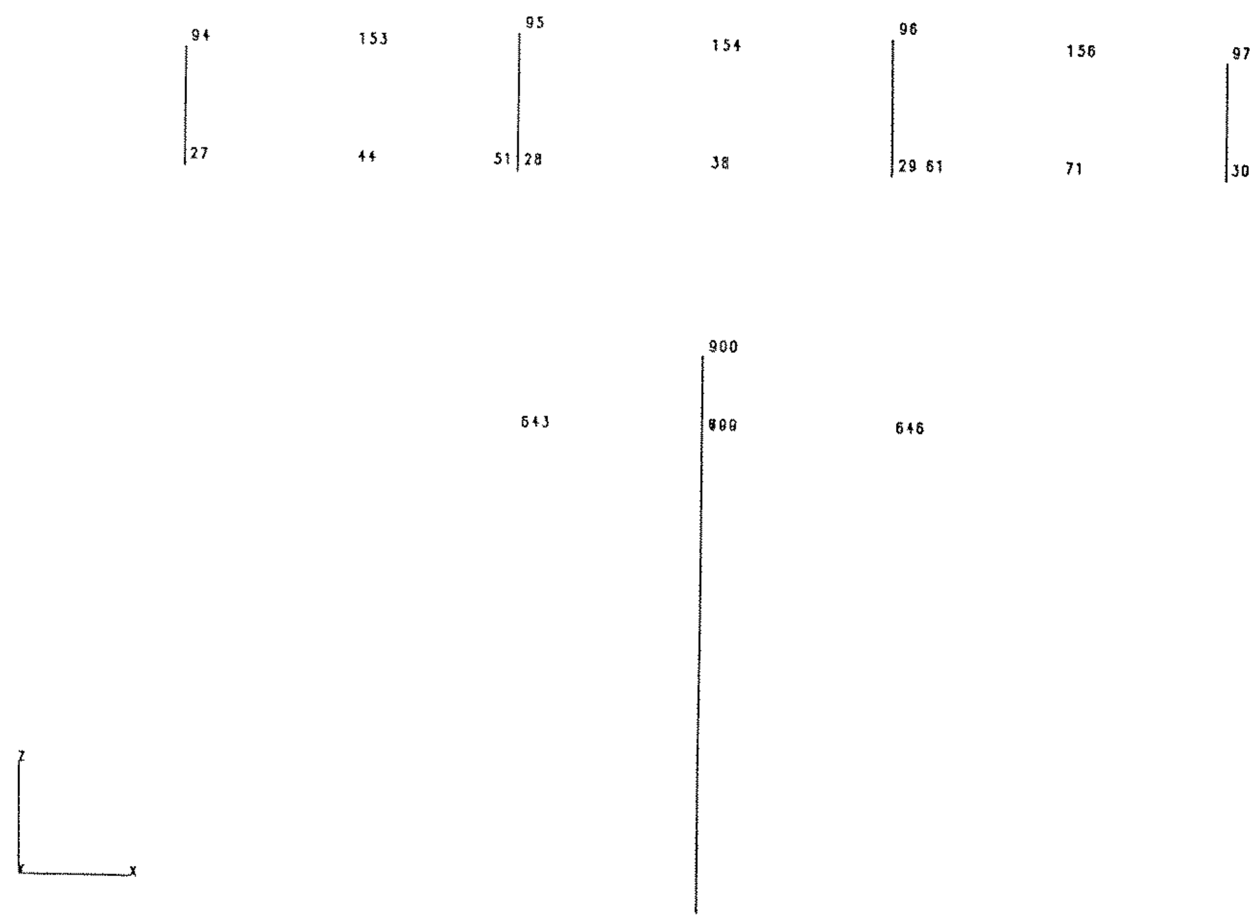


SHEET 26

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

CONDUCTOR TOP DETAIL, VERT. PLANE Y=0'

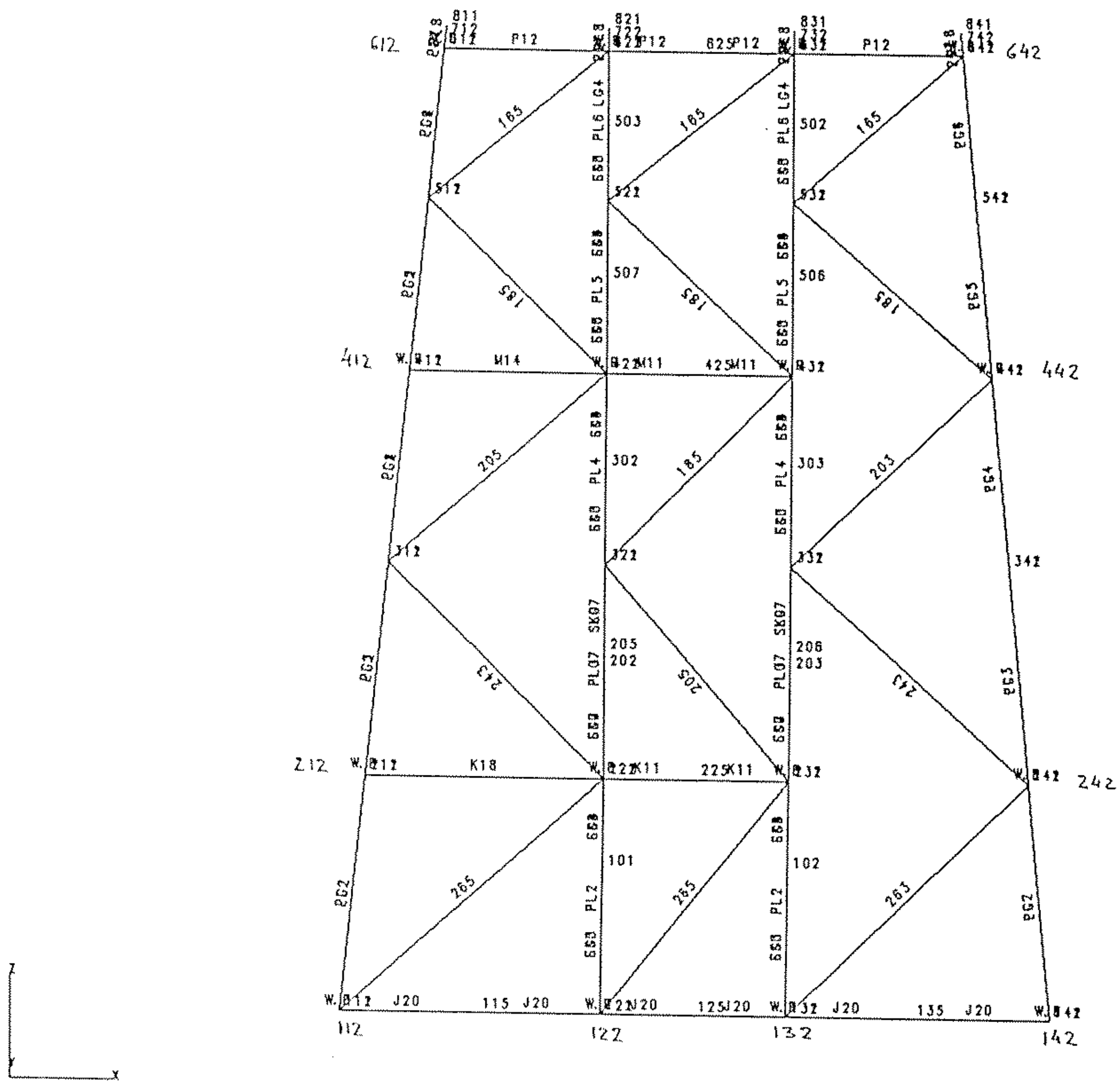


SHEET 27

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

JACKET BROADSIDE VIEW ROW A

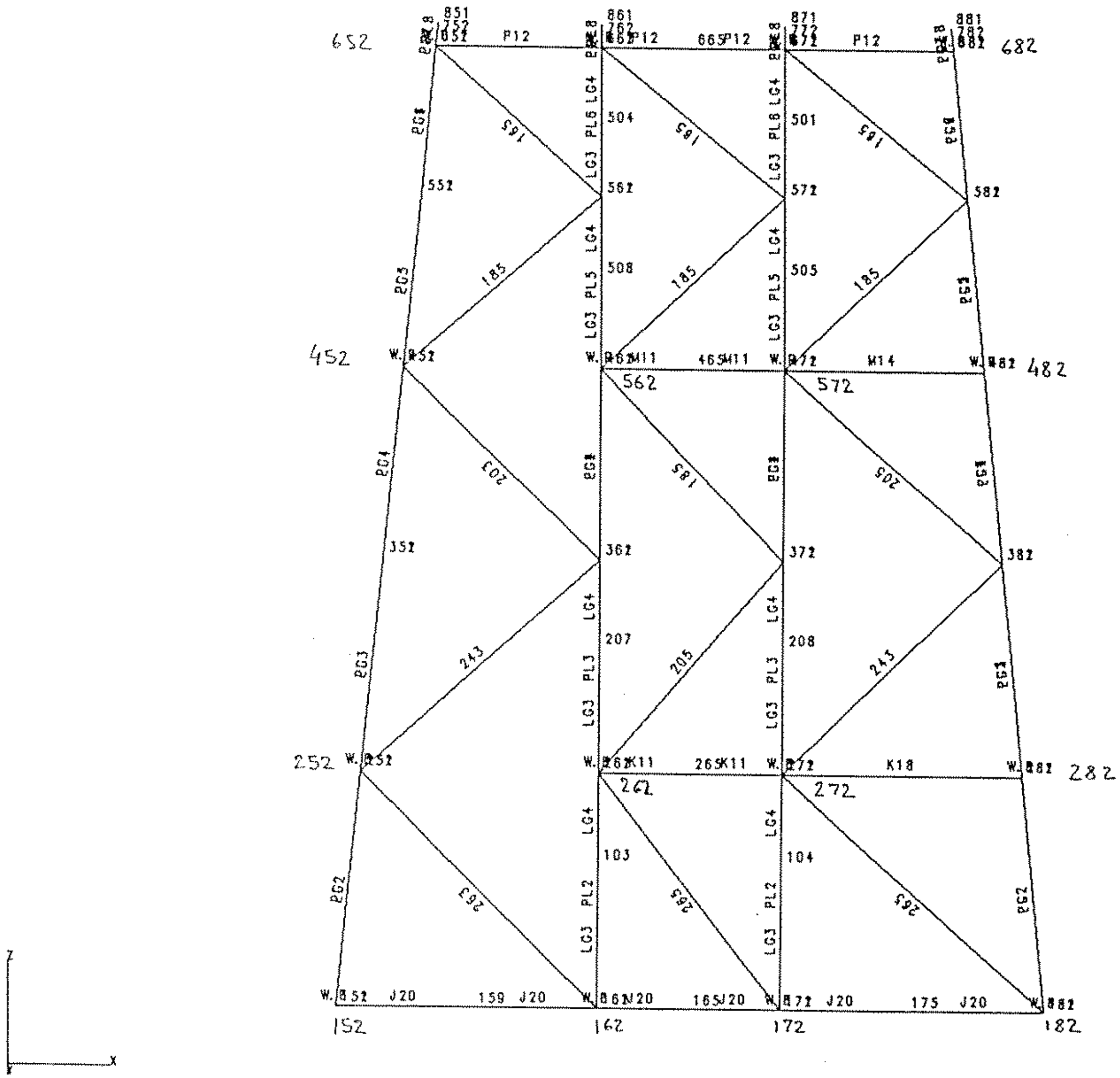


SHEET 28

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

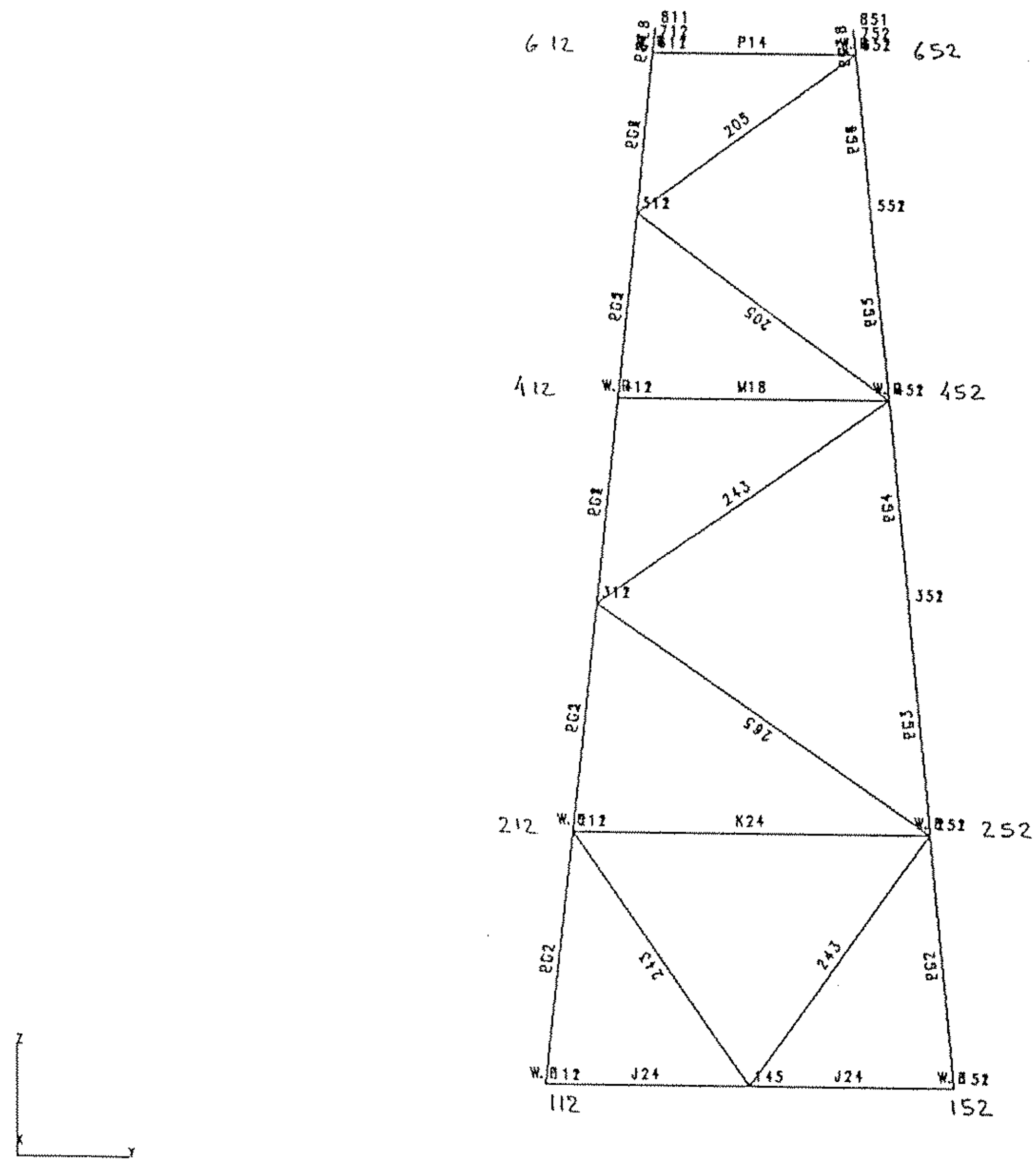
JACKET BROADSIDE VIEW ROW B



SHEET 29

SHELL WEST DELTA BLK 103A 223 FT WATERS

JACKET SIDEVIEW ROW 1

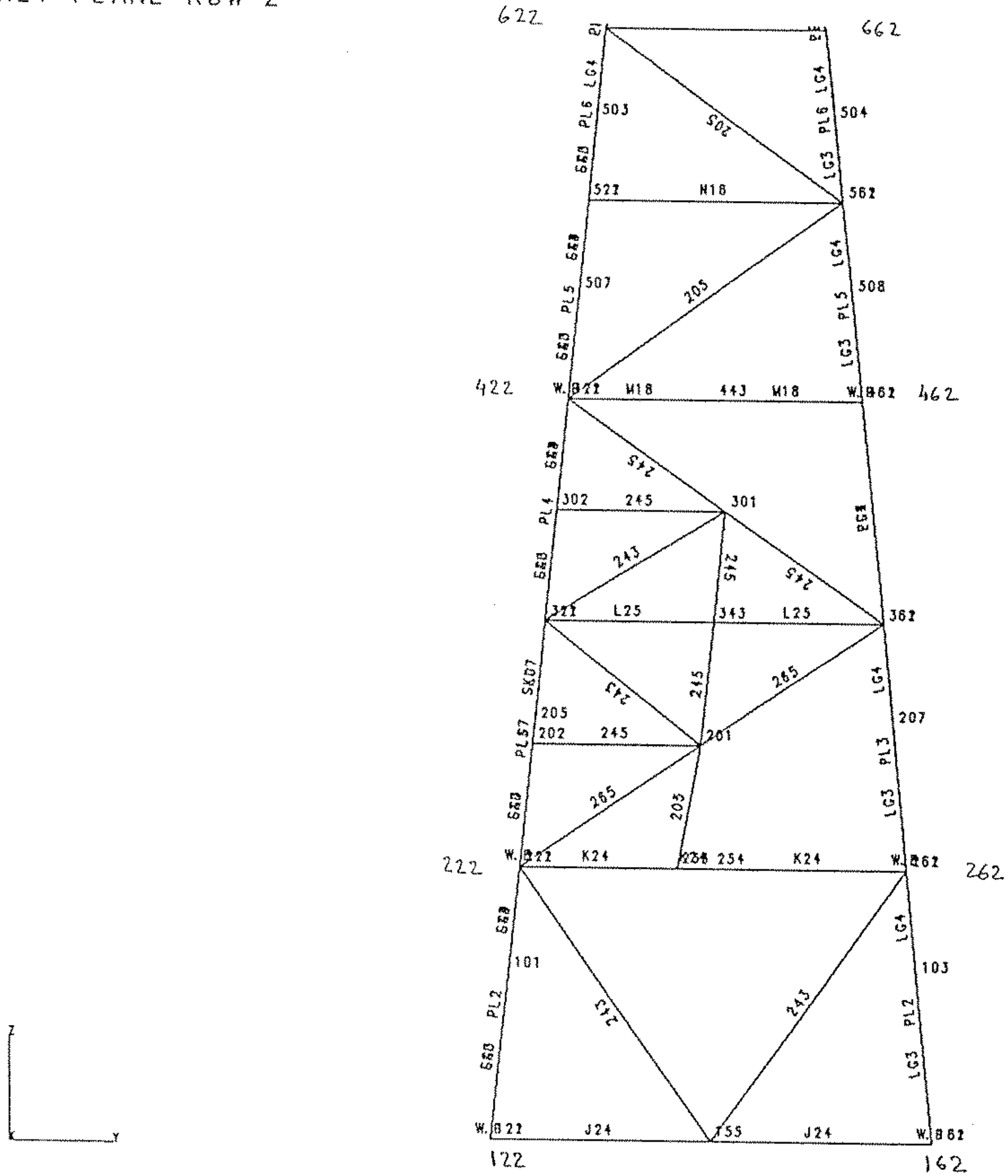


SHEET 3D

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

JACKET PLANE ROW 2

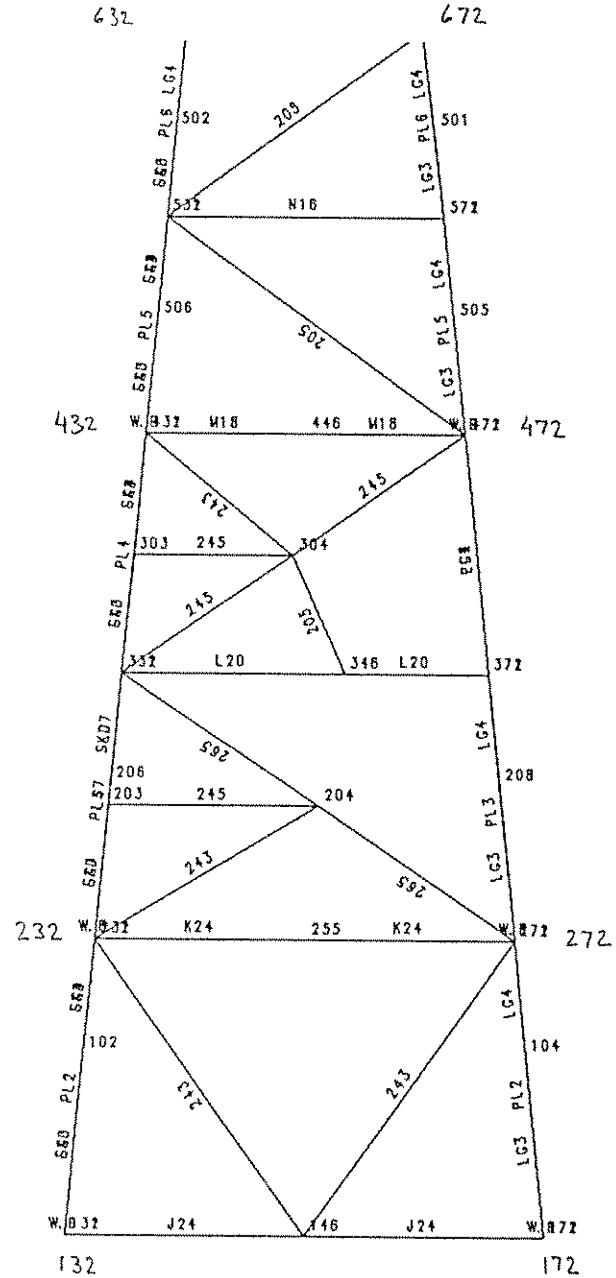


Sheet 31

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

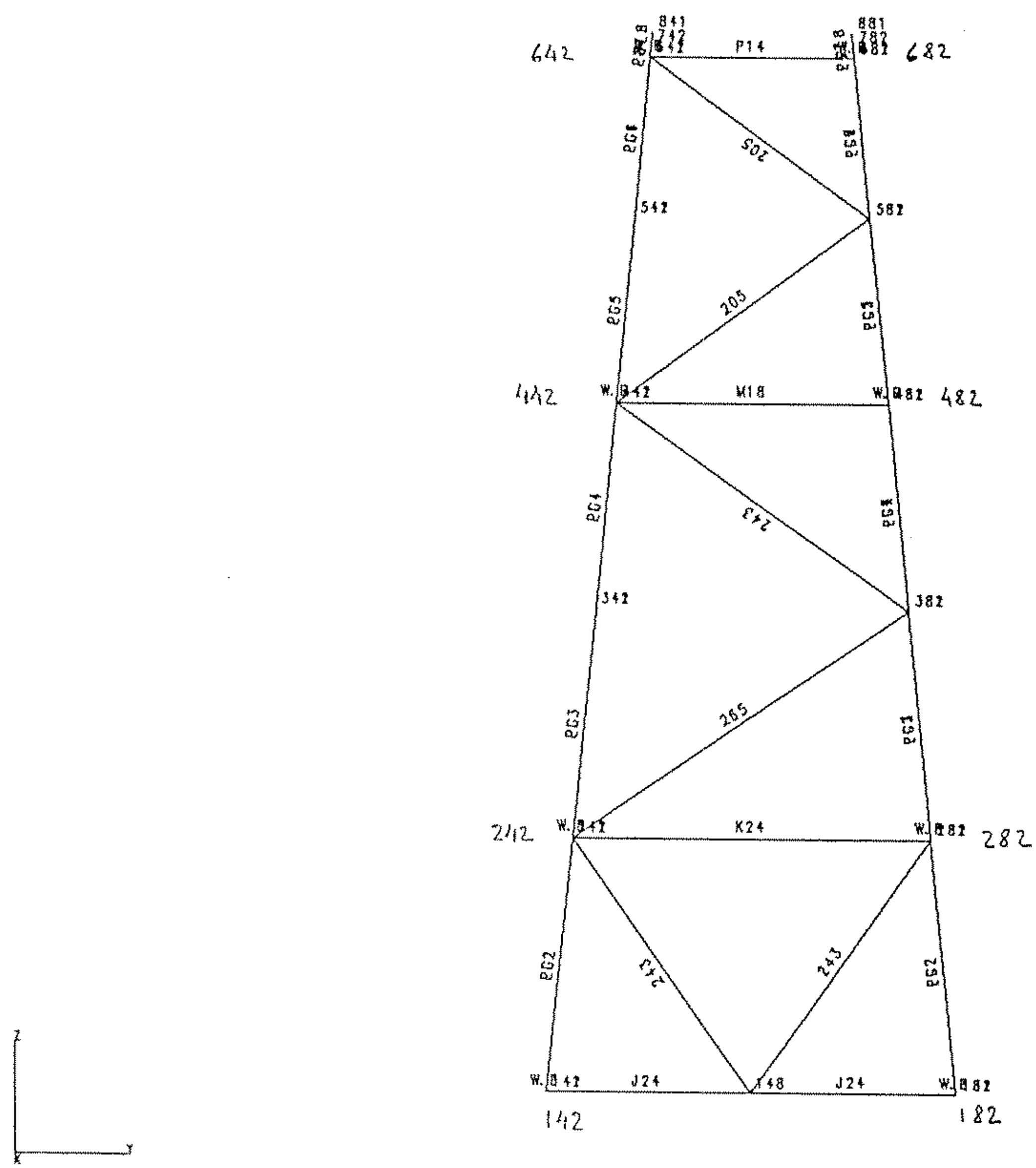
JACKET PLANE ROW 3



SHEET 31

SHELL WEST DELTA BLK 103A 223 FT WATERS

JACKET SIDEVIEW ROW 4



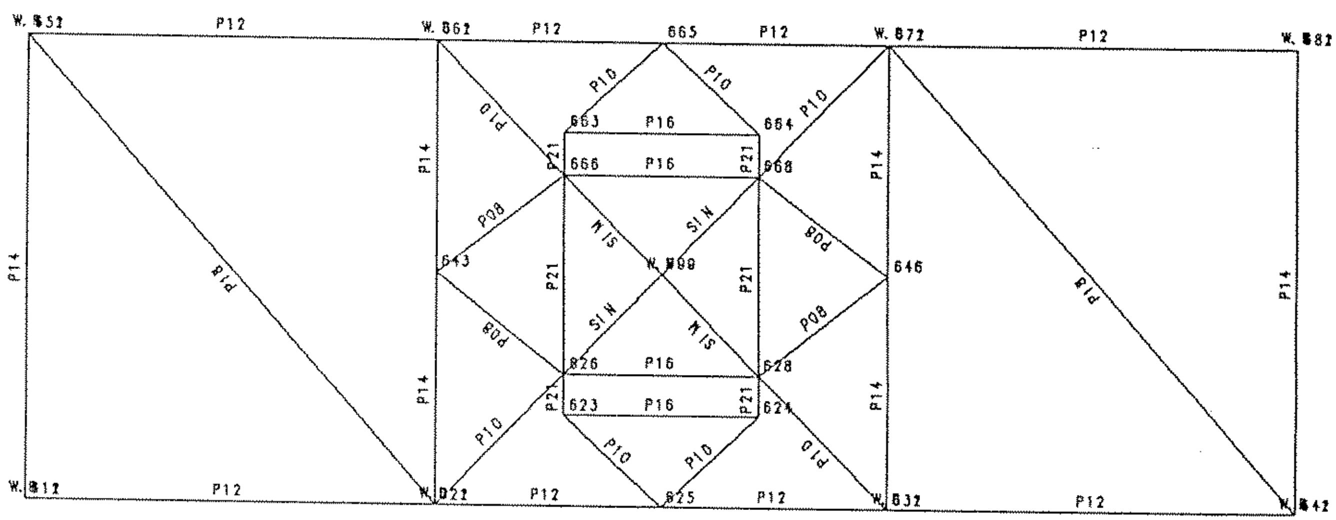
SHEET 32

STRUCTURAL SOFTWARE, INC

SHEET 33

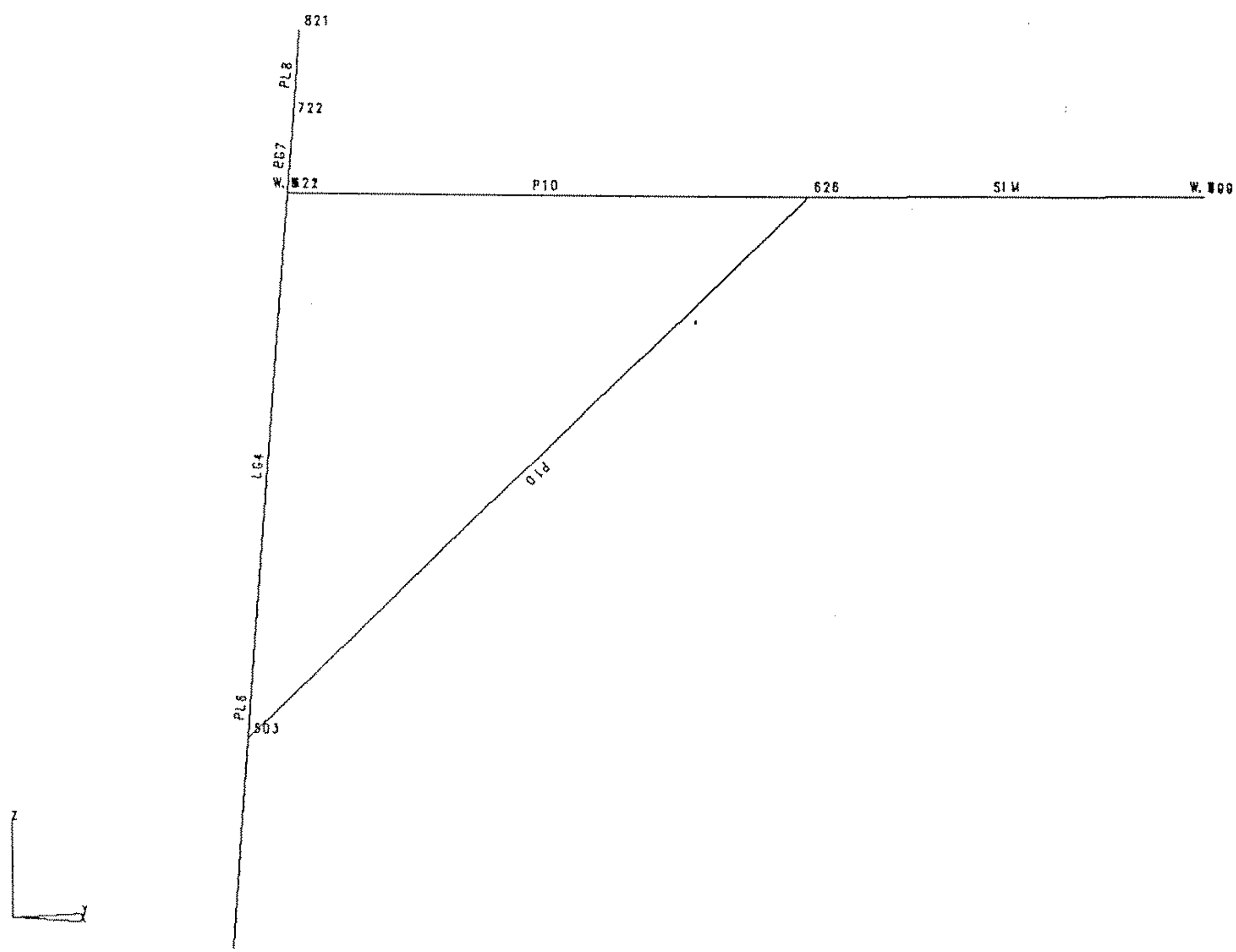
SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL +9.385'



SHELL WEST DELTA BLK 103A 223 FT WATERS

BRACE TO SUPPORT EL +9.385' PLANE

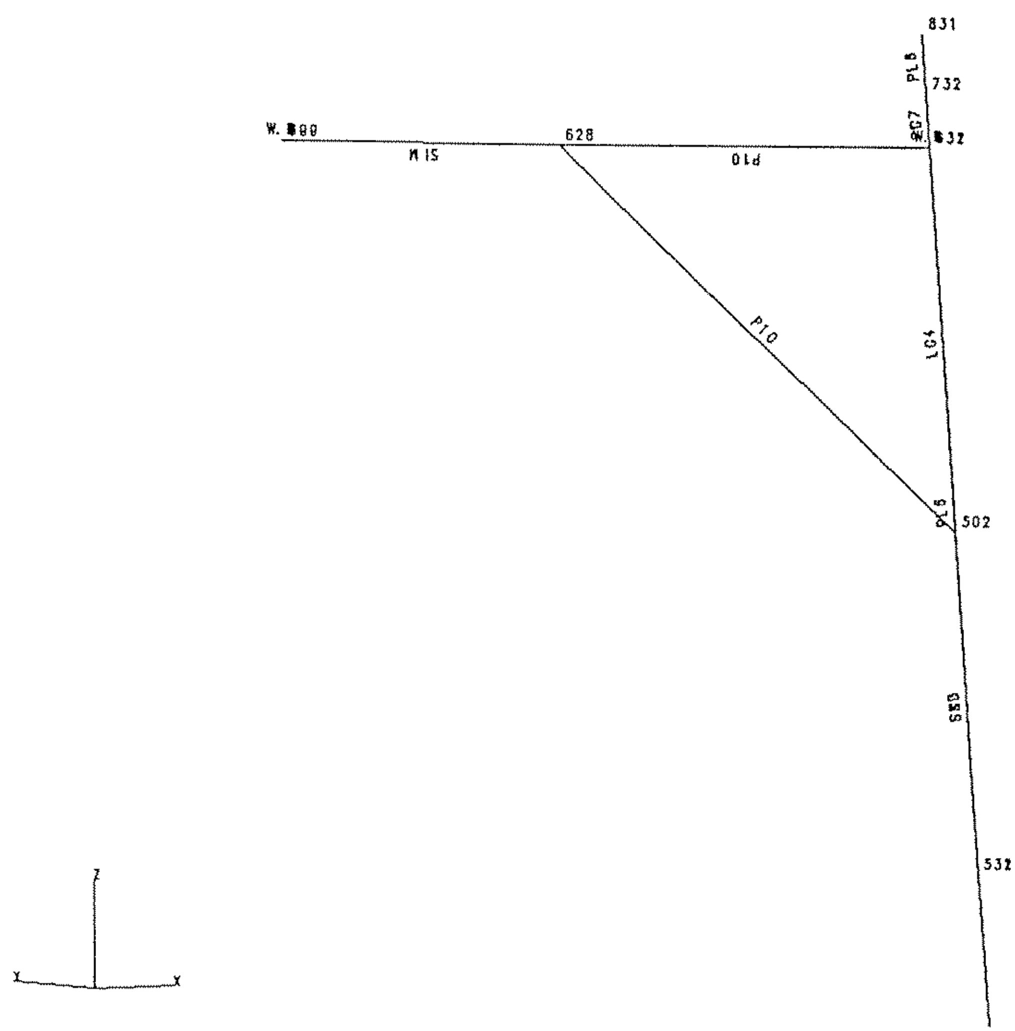


SHEET 34

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

BRACE TO SUPPORT EL +9.385' PLANE

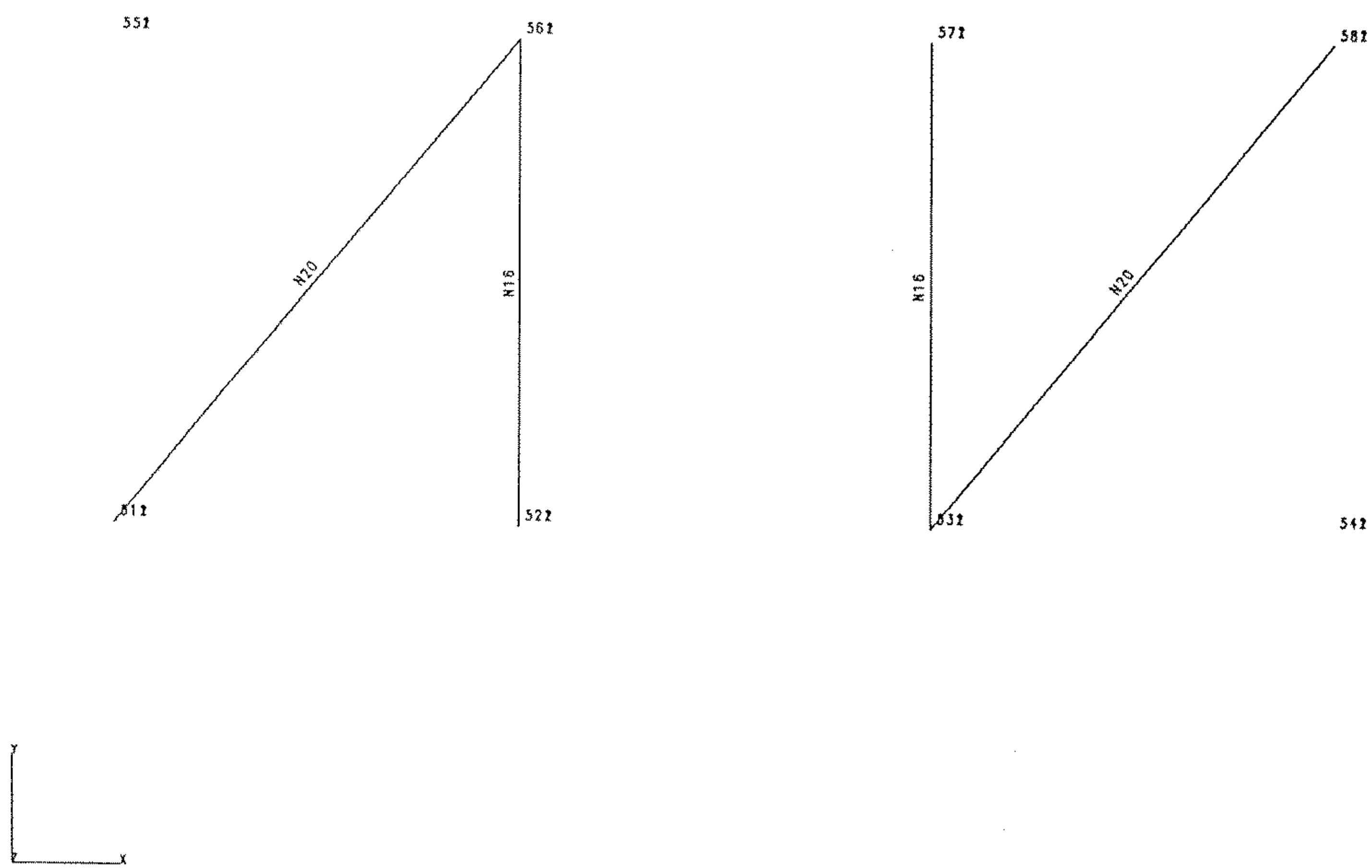


SHEET 35

SHEET 36

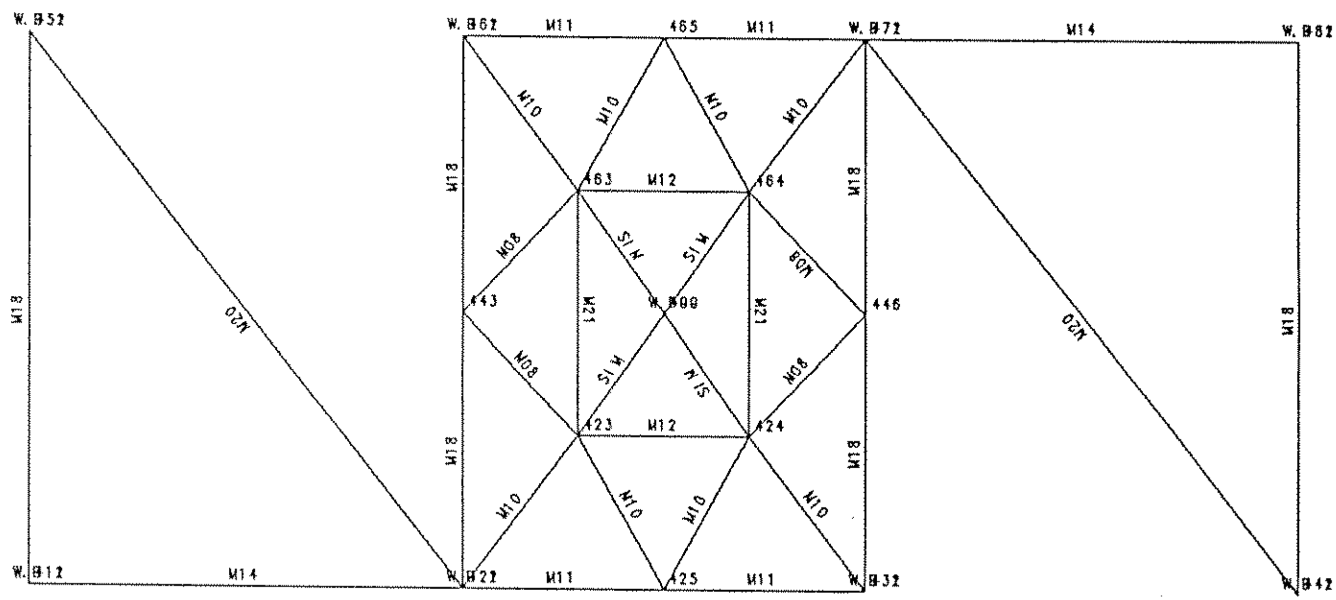
SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -26.5'



SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -68'

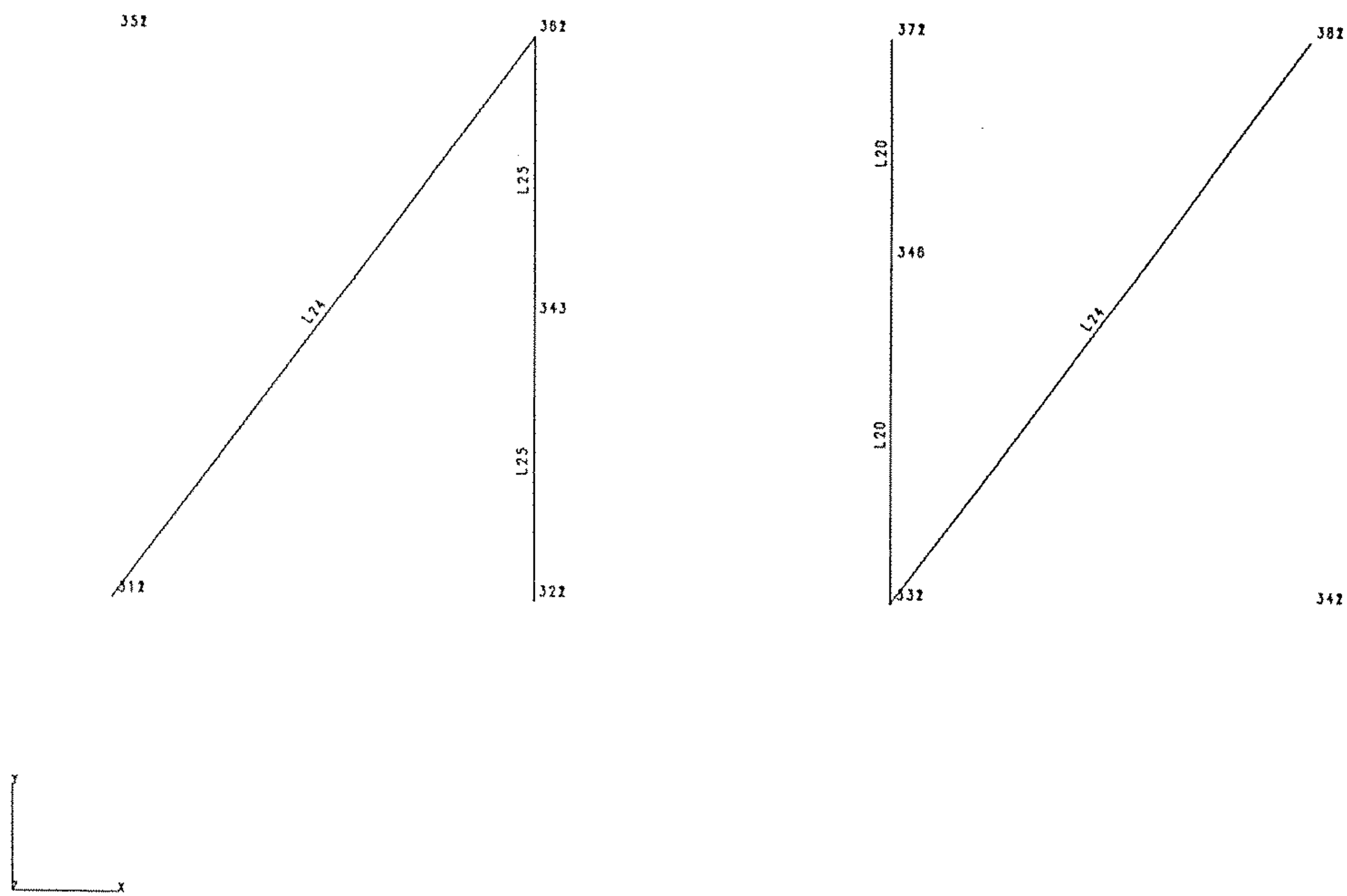


SHEET 37

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -114.5'



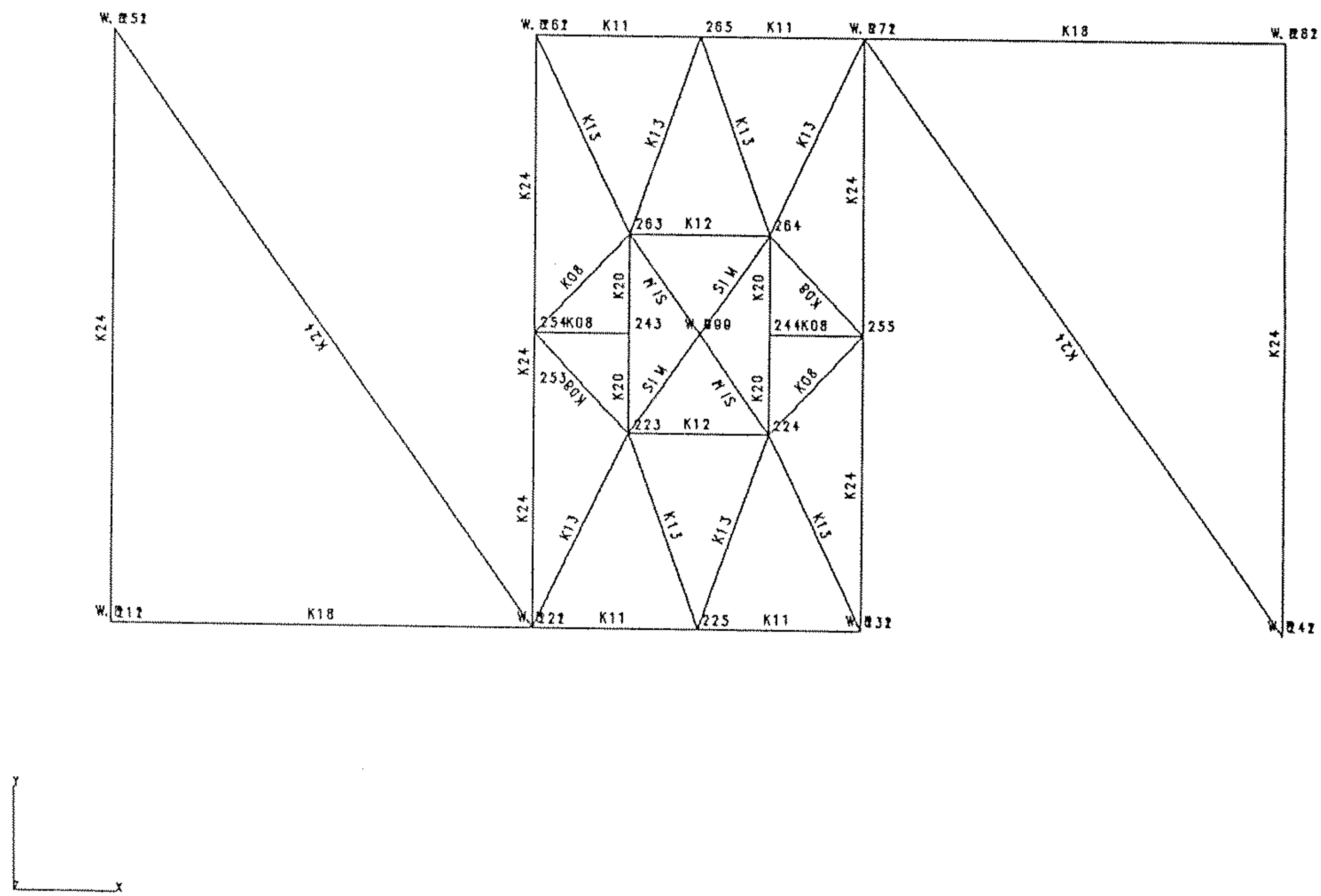
SHEET 38

STRUCTURAL SOFTWARE, INC

Up18h0s4102I

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -166'



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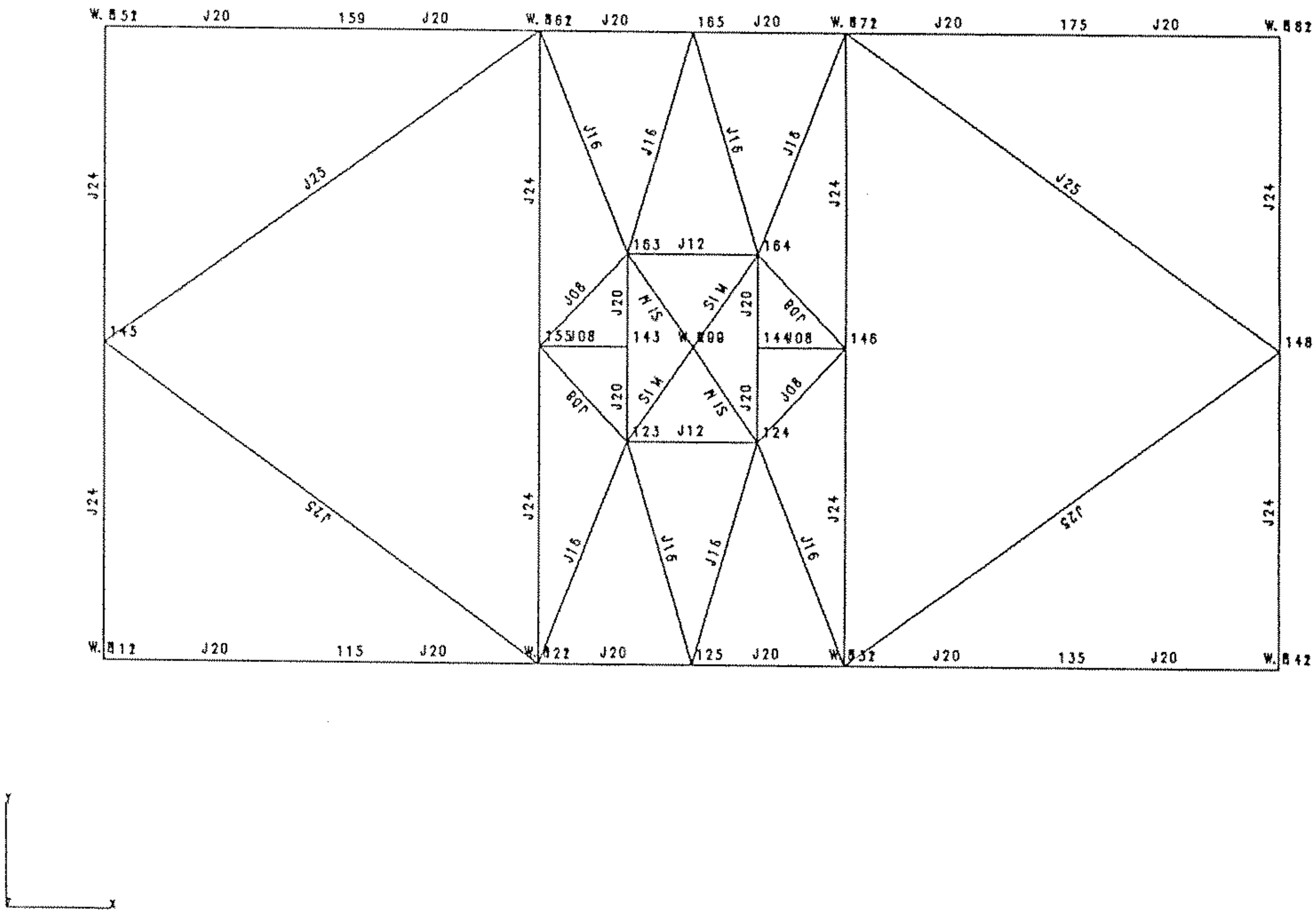
SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -223'



SHEET 39

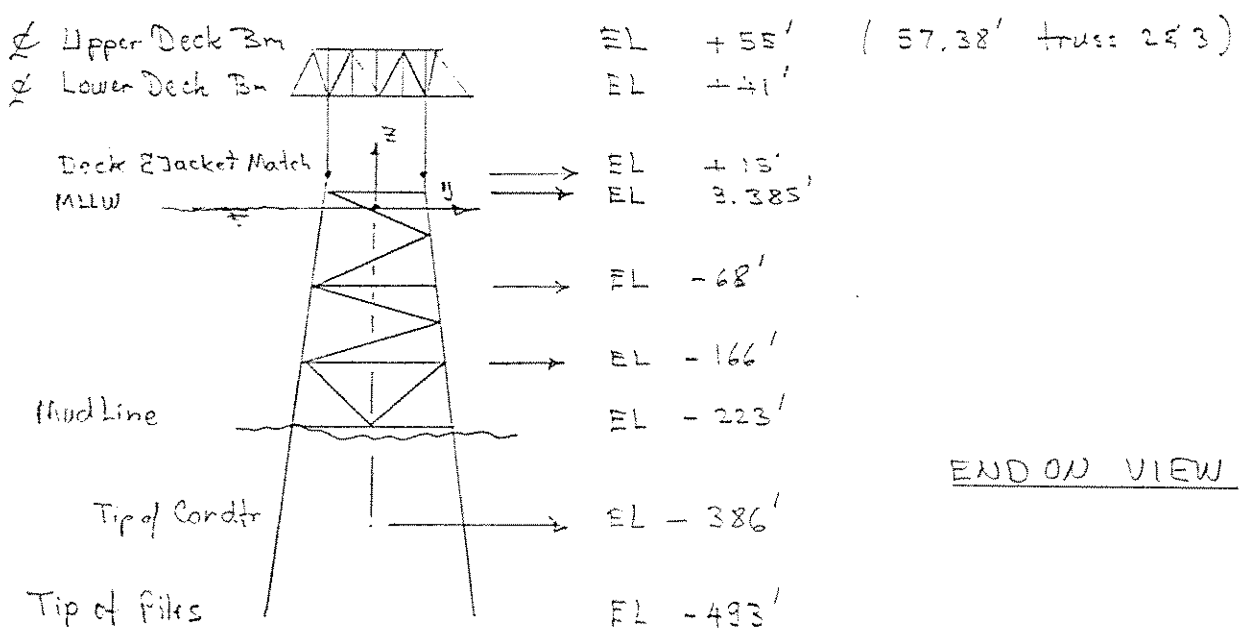
STRUCTURAL SOFTWARE, INC

7.1.2. Geometric Description

Computer model is prepared using graphic capabilities of Strucad * 3D program. The plots of the model are in section 7.1.1. These shows the joint numbers and member group ID. The platform decks, framing, the jacket structure the conductors and piles are described in the same model.

Reference Elevation of the model is at MLLW. Coordinate Origin is at intersection of symetry axes X, Y of platform deck planes. Z measures positively upwards from MLLW.

In the following some important elevations are shown, for detail informations see computer echos sections 8.1, 9.1, 10.1



END ON VIEW

(7.1.2. Geometric Description cont.)

All of jacket members, piles and conductors have tubular sections. Deck truss diagonals, verticals and colums of deck truss frames tieing into jacket are also tubular members. All deck beams including the ones in truss planes are Wide Flange members.

In order to keep overall number of joints and members to a minimum the model described the jacket and deck truss frames as is but Lumped some of the intermediate deck beams together.

For this reason deck Loads are hand calculated and applied to the joints of frame column at that deck Level.

Upper deck horizontal rigidity is provided by idealized horizontal braced members.
Lower deck horizontal rigidity is provided by 3/8" thk plate elements with no offset from the center line Lower deck beams.

12 - 24" O.D Conductors are idealized by one conductor. The jacket provide only horizontal support to the conductor at elevations +9.383', -68', -166', -223'.
All the other forces (i.e moments) are carried by the conductor and by its continuation as a pile below mudline.

(7.1.2. Geometric Description cont.)

The jacket and platform topside framing are supported by 8 battered piles.
The rigid connection between pile and jacket joints are at EL + 12.0'. The piles continues upwards to EL +15 to mate with topside framing structure and downward up to EL -223' inside jacketlegs and up to EL - 493' below mudline.
Inside jacketLeg at EL +9.385', -68', -166', -223' the pile touches the jacket Leg to only transfer horizontal forces.

Pile joints at mudline are refered as pile head joints and restrained by soil springs specified by 222222 coding in the joint input.

7.1.3 LOADS

The platform structure is analyzed to support the following Loads.

7.1.3.1 Dead & Buoyancy Loads

7.1.3.1.1 Dead Loads

a) Dead Loads of Non coded Jacket Members
(Barge Bumpers, Boat Landing, Walkway & Stairs etc)

These weights are hand calculated and applied as joint Loads after considering the buoyancy. (see 7.1.3.3 Joint Loads)

b) Dead Loads of Coded Members

Dead Loads of structurally coded members are calculated by the computer utilizing the member related input (cross-section area, member or segment Length, material density).

7.1.3.1.2 Buoyancy Loads

a) Buoyancy Loads of Non Coded Jacket Members

Hand calculated (See 7.1.3.3 Joint Loads)

b) Buoyancy Loads of Coded Members (for buoyant Members only)

Computer calculated using member input and water profil.

7.1.3.2. Deck Lds & Equipment Loads

Intensity for uniform deck / equipment Loads are assumed by Linder Associates after reviewing Ref 4.7

Using the intensity of Loads and the contributary areas the concentrated joint Loads are calculated by hand. See 7.1.3.3 Joint loads

7.1.3.3 JOINT LOADS ON PLATFORM JACKET & DECKS

(7.1.3.3 Continued)

DEAD WEIGHT AND BUOYANCY LOADS ON NON CODED MEMBERS OF THE JACKET

1) BOAT LANDINGS
2) BARGE BUMPERS
3) WALKWAY @ (+) 10'-0"

1) BOAT LANDING (1- REQ'D)

12¾"φ x 0.500	242'	x 65.4 #/ft	= 15827 #
8⅝"φ x 0.500	127'	x 43.4 #/ft	= 5512 #
10¾"φ x 0.500	147'	x 54.7 #/ft	= 8041 #
CONRETE FILLED 10¾φ		13 x 11.25' x 0.52 ft² x 150 #/ft³	= 11403 #

TOTAL WGT = 40.787 KIPS x 1.10 {10% INCREASE FOR MISC. STEEL}
= 45 KIPS

Buoy ≈ 5 [illegible]

DISTRIBUTE AS JT. LOAD @ JOINTS 631, 641
LD = 22.5 K @ EACH JOINT
Buoy = 2.5 K @ EACH JOINT
NET APPLIED 20 K @ EA JOINT

2) BARGE BUMPERS (6 - REQ'D) (see next page)
EACH WEIGHS ≈ 11.0 K
BUOYANCY ≈ 2.4 K
} 11.0 - 2.4 ≅ 9 K

NET APPLY AS JOINT LOAD @ JOINT 611, 621, 651, 661, 671, 681
9 KIPS

(7.1.3.3 Continued)

BARGE BUMPER WEIGHT (see 2) on previous page)

	Long	Unit Weight (Lbs)	Weight (Kips)
TOP BRACKET 21 WF 81	4'	81	0,32
1/2" PL R=10" half circle	4'	$\pi \times \frac{10''}{12} \times 20{,}4 = 53{,}4$	0,21
1/2" PL (BOTTOM)	4'	$21/12 \times 20{,}4 = 35{,}7$	0,14
3/4" PL (FLAT BAR)	~ 10'	$7/12 \times 30{,}6 = 17{,}86$	0.18
3/4" PL (FACE REINF)	3,67'	$2{,}25 \times 30{,}6 = 68{,}85$	0,25
19" OD - 75 CENTER POST	17,33'	146,3	2,54
3/4" PL BOTTOM BRACKET	2,67'	$2' \times 30{,}6 = 61.2$	0.16
16 ϕ Lower Support	~ 5,0'	108	0,54
3/4" PL (FACE REINF)	4,0'	$56{,}5''/12 \times 30{,}6 = 144$	0,58
		subtotal	4,92
1/2" PL Outside (43" O.D)	14'-0	227	3,18
FILL SPACE TRUCK TIRES	14'-0	$\left[\frac{\pi(42^2-19^2)}{4} \times \frac{94\ \text{lbs/cb/ft}}{144} \times 25\%\right]$ (fill ratio assumed)	2.52
Chain + misc			0,38
			11.00 K

Bouyancy

	7'-0	$\frac{\pi}{4}(43^2-42^2) \times \frac{64}{144}$	0,21
	7'-0	$\frac{\pi}{4} \times 19^2 \times \frac{64}{144}$	0,88
	5'-0	$\frac{\pi}{4} \times 16^2 \times \frac{64}{144}$	0,45
	7'-0	$\left[\frac{\pi}{4}(42^2-19^2) \times \frac{64}{144} \times 25\%\right]$	0,86
			2,40 K

(7.1.3.3 continued)

3) WALKWAY @ (+)10'

10¾"φ x 0.365	344'	x	40.5 #/ft	=	13932 #
3½"φ x 0.216	360'	x	7.6 #/ft	=	2736 #
1½" GRTG	1584 ft^2	x	10 #/ft^2	=	15840 #

TOTAL = 32.5 KIPS x 1.15 ≃ 38 KIPS {15% INCREASE FOR HANDRAILS & MISC. STEEL}

Buoy ≃ 15 KIPS

DISTRIBUTE @ JOINTS 611, 621, 631, 641, 651, 661, 671, 681

LD = -4.75 KIPS
Buoy = 1.875 KIPS
2.875 SAY 3K

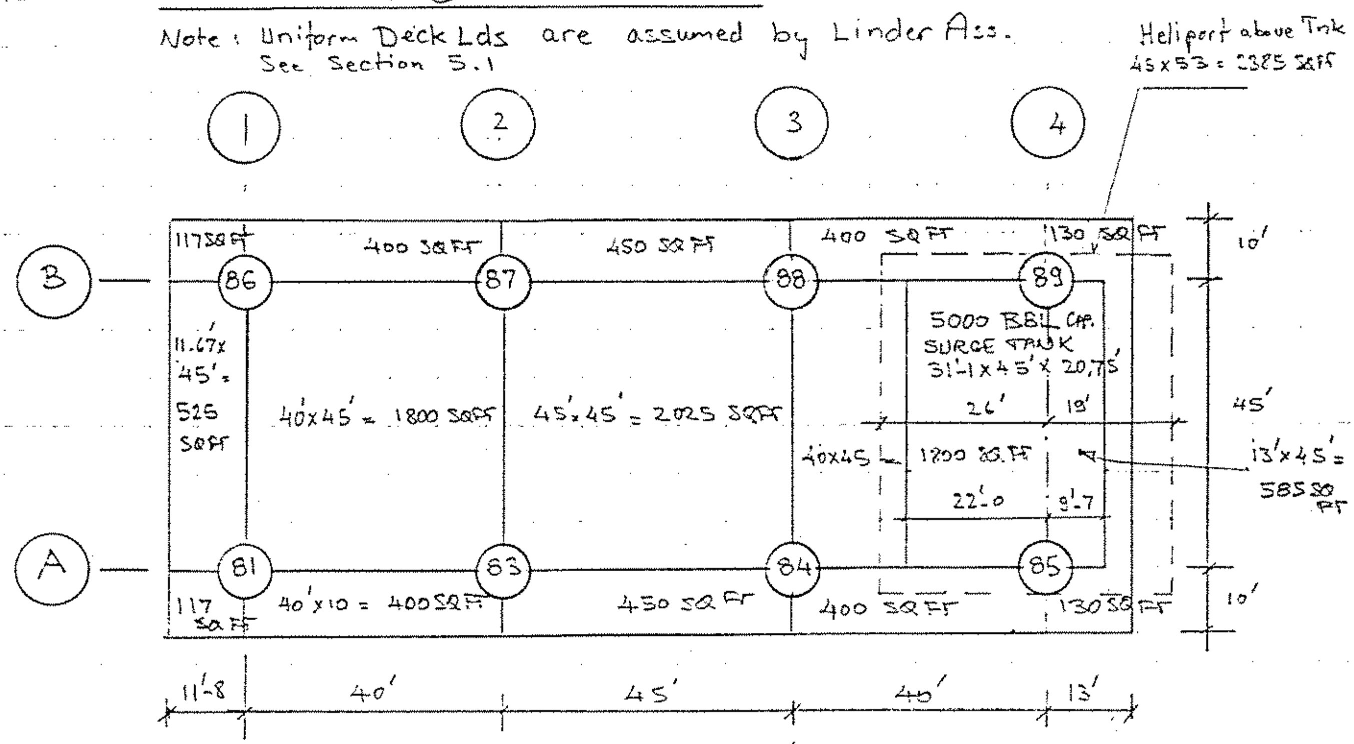
} NET APPLY 3 K

(7.1.3.3 continued)

CALCULATE JOINT LOADS

UPPER DECK @ EL. +58'-0¼" (REF 4.7)

Note: Uniform Deck Lds are assumed by Linder Ass.
See Section 5.1



USE D.L ← 50 PSF → ← 50 PSF → ← 50 PSF →

EQUIP. L ← 300 PSF → ← 150 PSF → ← 300 PSF →

SURGE TANK AREA 20.75' x 0.064 = 1.328 KSF
USE 1.35 KSF

HELIDECK 50 PSF DL + HELICOPTER

AREA CONTRIBUTIONS & CONCENTRATED JOINT LOADS

JOINT # 81 OR 86

$117 + 525/2 + 400/2 + 1800/4 = 1029.5$ SQ FT

DL: $0.05 \times 1029.5 \rightarrow 51.5$ K

E.Q.: $0.30 \times 1029.5 \rightarrow 308.9$ K

360.4 K

Subtract Modeled Bm Wts — 7.4 K → USE 353 K

JOINT # 83 OR 87

$400/2 + 450/2 + 1800/4 + 2025/4 = 1381.25$ SQ FT

DL: $0.05 \times 1381.25 \rightarrow 69$ K

EQ: $0.30 \times (200 + \frac{1800}{4}) \rightarrow 195$ K

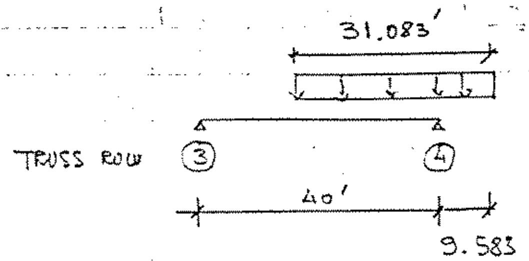
$0.30 \times (225 + \frac{2025}{4}) \rightarrow 219.4$ K

483.4 K
− 14.8 K

Subtract Modeled Bm Wts → −14.8 K USE 470 K

(7.1.3.3 Continued)

TOTAL SURGE TANK LOAD



5000 BBL x 42 US GAL x 0.1337 CB FT x 0.064 K ≅ 1800 K

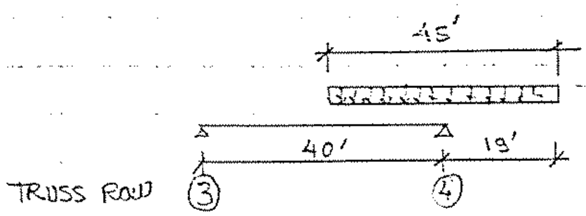
Reactions:

$R_{④} = \frac{1}{40} \times \left[1800 \left(49.583 - \frac{31.083}{2} \right) \right] = 1531.87^K$ say 1530^K

$R_{③} = 1800 - 1530 = 270$ K

HELIDECK + HELICOPTOR LD

0.050 x 2385 = 119.25 K → USE 200 K

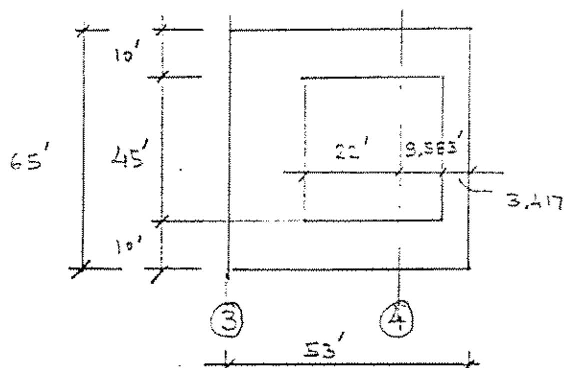


Reactions:

$R_{④} = \frac{1}{40} \times \left[200 \left(59' - 45/2' \right) \right] = 182.5$ USE 180 K

$R_{③} = 200 - 180 = 20$ K

CALCULATE THE C.G OF DECK LD. OUTSIDE SURGE TANK

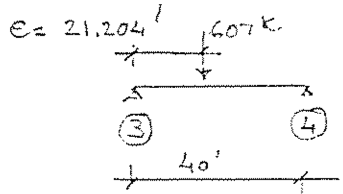


	AREA		1ST AREA MOMENT
	65 x 53 = 3445 SQ FT	x 53/2 =	91292.5 CB FT
	31.583 x 45 = −1421.235 "	x (49.583 − 31.083/2) =	48380.97
SUM:	2023.765	SUM:	42911.53

e = 42911.53 / 2023.765 = 21.204'

Load outside Surge Tank

2023.765 x 0.3 = 607 K



Reactions

$R_{④} = 607 \times \frac{21.204}{40} = 321.84$ SAY 322^K

$R_{③} = 607 - 322 = 285^K$

(7.1.3.3 continued)

F.W.K.O
NEAR JOINT # 84

Cylindrical Tank 14'-0" O.D 24 FT High.

Assume 3/8" PL is used (15.31 psf)
Calculate Tank Weight;

$(\pi \times 14' \times 24 + \pi \times 14 \times 14/4 \times 2) \times 0.0153 = 20.86^k \rightarrow$ 21 Kips

content (say filled with water)

$\pi \times 14 \times 14/4 \times 24 \times 0.064 = 9.85^k \rightarrow$ 10 Kips

Tank + Content $\Rightarrow$ 31 Kips

W. H. LINDER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHELL W.D. BLK 103 A
PLATFORM ASSESSMENT

PROJECT NO. 1830MO'
BY MEC DATE 6/27/91
CHK. ______ DATE ______
SHEET 52 OF ______

(7.1.3.3 continued)

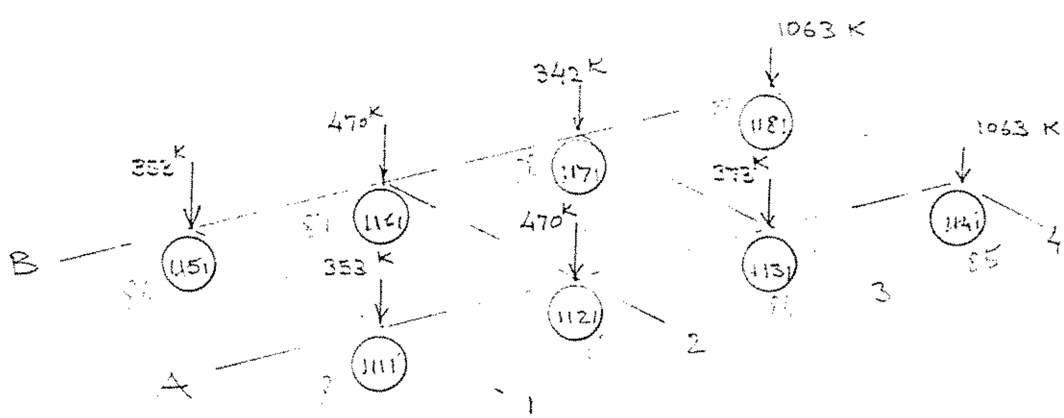
JOINT # 1171

DL: SIMILAR TO JNT# 1121 ⟹ 69 K
EQ: OUTSIDE SURGE TNK ⟹ 285/2 = 142.5 K
SURGE TANK ⟹ 270/2 = 135 K
HELIDECK ⟹ 20/2 = 10 K
356.5
Subtract Modeled Bm Wts − 14.8 K
341.7 K USE 342 K

JOINT # 1131 Add FWKO Wt ⟶ 31 K USE 373 K

JOINT # 1141 OR 1181

DL: $0.05 \times \frac{65'}{2} \times (49/2 + 13') \stackrel{\sim}{\Rightarrow}$ 54 K
EQ: OUTSIDE SURGE TANK ⟹ 322/2 = 161 K
SURGE TANK ⟹ 1530/2 = 765 K
HELIDECK ⟹ 180/2 = 90 K
1070 K
Subtract Modeled Bm Wt − 7.4 K
1062.4 K USE 1063 K



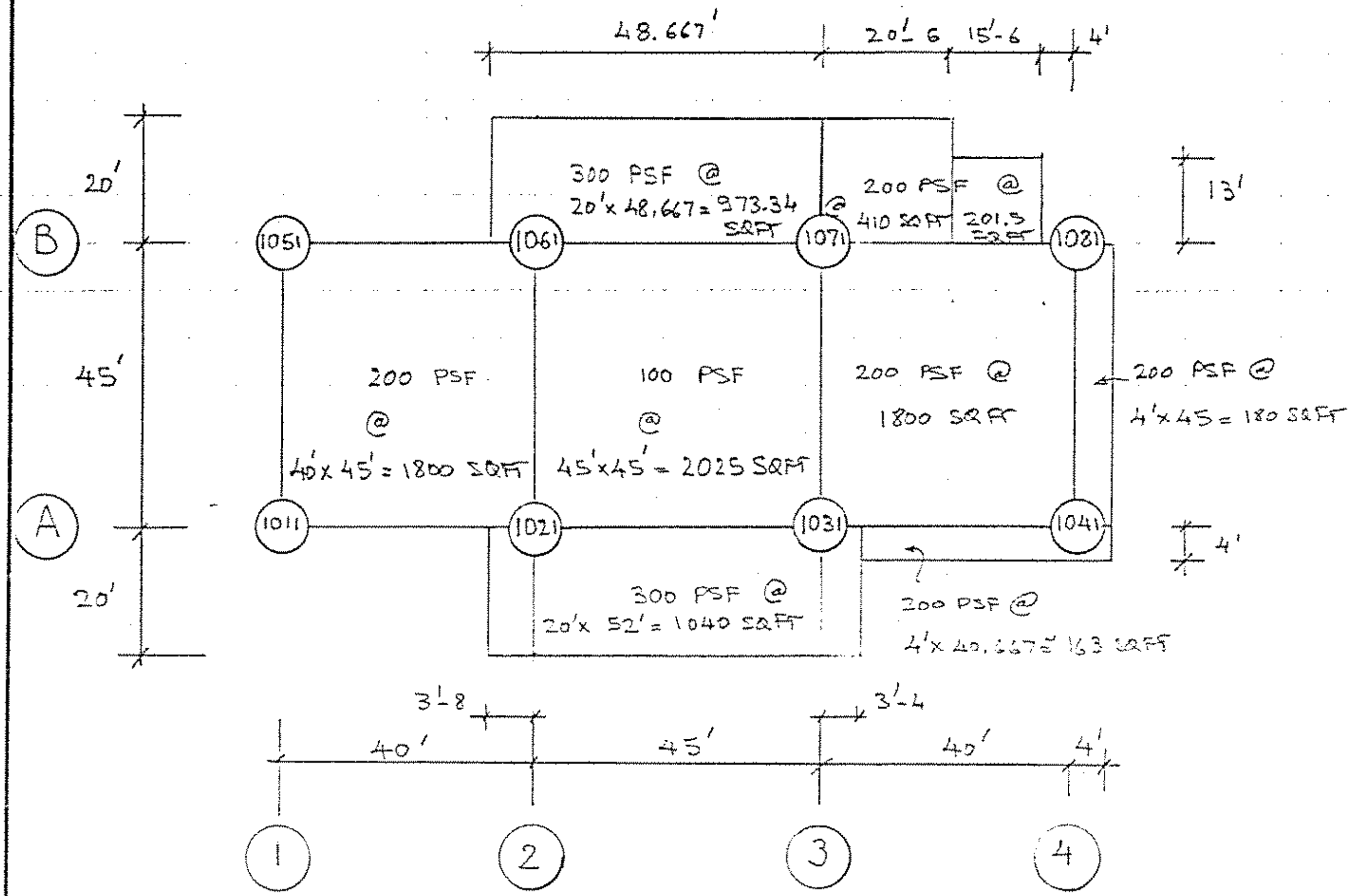
SUMMARY OF UPPER DECK LOADS (DL + EQ)

TOTAL 2x(353 + 470 + 1063) + 342 + 373 = 4487 K

(7.1.3.3 CONTINUED)

LOWER DECK @ EL + 42'-0

Note: Uniform Deck Loads are assumed by Linder Asso.
See Section 5.1



DEAD LOAD OF BEAMS & PLATING ARE GENERATED WITHIN THE PROGRAM IN LOAD CASE 1

EQUIPMENT & DECK LOADS are shown above and will be put in as concentrated joint Loads in LOAD CASE 2

(7.1.3.3 CONTINUED)

CALCULATE JOINT LOADS @ LOWER DECK LEVEL DUE TO EQ + DECK LDS

JOINT 1011 & 1051

0,200 x 1800/4 = 90.0 K   USE → 90 K

JOINT 1021 & 1061

similar to JNT 1011	→	90,0 K
0,100 x 2025/4	→	50,6 K
0,300 x 1040/2	→	156.0 K
		296.6 K USE → 300 K

JOINT 1031

similar to JNT 1021	→	296.6 K
0,200 x 163 x 40%	→	13.0 K
		309.6 USE → 310 K

JOINT 1071

similar to JNT 1061	→	296.6 K
subtract 0,3 x 20' x 3.33'	→	− 20,0 K
0,20 x 410 x (40' − 20.5/2) x 1/40'	→	61,0 K
0,20 x 201,5 x (15.5'/2 + 4') x 1/40'	→	11.8 K
		349.4 K USE → 350 K

JOINT 1041

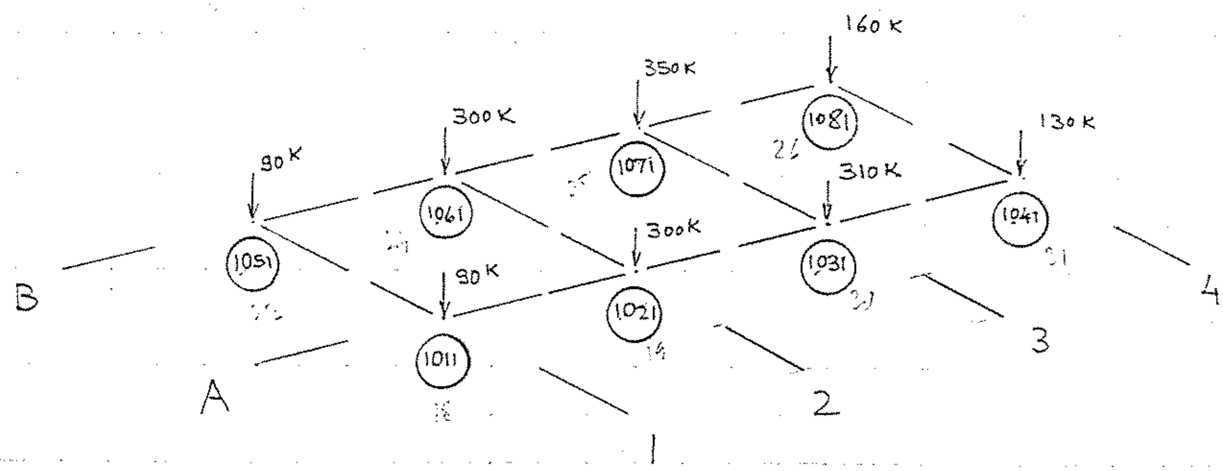
	similar to JNT 1011	→	30,0 K
see JNT 1031 →	0,200 x 163 x 60%	→	20.0 K
	0,200 x 180 x 1/2	→	18.0 K
			128,0 K USE → 130 K

JOINT 1081

	similar to JNT 1051	→	90.0 K
	0,200 x 180 x 1/2	→	18,0 K
	0,20 x 410 x (20.5/2) x 1/40	→	21,0 K
see JNT 1071	0,20 x 201,5 − 11,8	→	28,5 K
			157,5 K USE 160 K

W. H. LINDER & ASSOCIATES, INC. CONSULTING ENGINEERS	SHELL W.D. BLK 103A	PROJECT NO. 1830M01
	PLATFORM ASSESSMENT	BY MEC DATE 6/28/94
		CHK. ____ DATE ____
		SHEET 55 OF ____

(7.1.3.3 CONTINUED)



SUMMARY OF LOWER DECK LOADS (EQ + DK LDS)

TOTAL: $2 \times (90 + 300) + 310 + 350 + 130 + 160 = 1730\ K$

7.1.3.4 <u>WIND LOADS</u> Calculated by Computer

a) <u>Wind Speed</u> See Section 7.1.3.6

This is a reference value $V(1\,hr, z_R)$; a one hour mean speed at reference elevation z_R of 33'. (Ref. 4.1 Sect 2.3.2b)
For different elevation wind speed is calculated by the formula 2.3.2-1 in (Ref 4.1 p. 23).

b) <u>Wind Velocity / Force Relation</u>

$F = (w/2g) \times V^2 \times C_S \times A$ (Equation 2.3.2-8 Ref 4.1 p. 23)

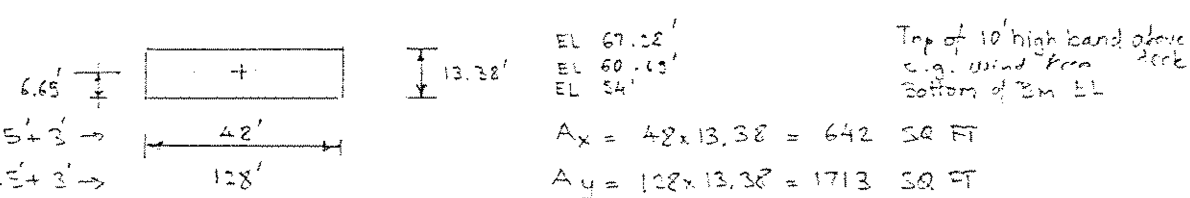
w = weight density of air (0.0756 lb/ft³ standard temp & press.)
$g = 32.2\ ft/sec^2$
(When V in MPH use $\frac{w}{2g} = 0.00256$)

C_S = Shape Coef from Sect 2.3.2e Ref 4.1 p. 24

A = Wind Area in ft^2

c) <u>Wind Areas</u>

c1) <u>Above Upper Deck</u> → AREAW2 (Computer input)

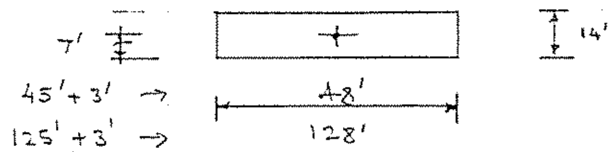


$A_x = 48 \times 13.38 = 642$ SQ FT
$A_y = 128 \times 13.38 = 1713$ SQ FT

c.g. coordinates @ 0, 0, 60.65'
Wind force to be applied to the Upper Deck
Joint No: 83, 84, 87, 88

(7.1.3.4 Continued)

c2) Between Lower and Upper Deck → AREAW1 (Computer Input)



EL + 54' Bottom of Upper Dk Bms
EL + 47' c.g. Wind Area
EL + 40 Bottom of Lower Dk Bms

$A_x = 48' \times 14' = 672$ SQ FT

$A_y = 128' \times 14' = 1792$ SQ FT

c.g coordinates @ 0, 0, 47'

Wind force to be applied to the Lower Deck
Joint No: 19, 20, 24, 25

c3) Top side Structure Column Surface Areas
Between EL + 30' and EL + 40' with $C_s = 0.5$ for tubular sect.
↳ conservative Crest EL (see Sect. 7.1.1)

$A_x = A_y = 8 \times 3 \times 10 \times 0.5 = 120$ SQ FT

for 55 Knot wind $F_{@C3} \cong 0.00256 \times 55^2 \times 120 \times 10^{-3} = 0.9$ K

for 70 Knot wind $F_{@C3} \cong 0.00256 \times 70^2 \times 120 \times \frac{4'}{10'} \times 10^{-3} = 0.6$ K

Area reduced due to higher crest

Note: 1) Wind on C3) Areas are not considered in computer input.

2) Member Wind surfaces within C1 & C2 Areas are suppressed in computer calculations so that only C1 & C2 areas are considered.

7.1.3.5 WAVE/CURRENT Calculated by Computer

WAVE velocity & accellerations are calculated using STOKE 5 theory.
CURRENT velocities see Section 7.1.3.5.4

Force exerted by waves on submerged members of the structure are calculated using Morrison Equation

$$F = F_D + F_I = C_d \frac{w}{2g} A\, U|U| + C_m \frac{w}{g} V \frac{\delta U}{dt}$$ (Ref 4.1 p.20 Equa. 2.3.1-1)

F = Hydrodynamic force per unit Length acting normal to member axis
F_D = Drag component of force F (Lbs/ft)
F_I = Inertia " " " " "

w = weight density of water (Lbs/ft³)
A = projected area normal to the axis of the cylindrical member per unit Length
For circular cylinder $A = D$ diameter of cylinder (ft)
V = displaced volume of cylinder per unit Length ($ft^3/ft \rightarrow ft^2$)
D = effective diameter of circular cylindrical member including marine growth (ft)

U = Component of the velocity vector (due to wave or current) of the water normal to the member axis (ft/sec)

$|U|$ = absolute value of U (ft/sec)

$\frac{\delta U}{dt}$ = Component of the Local acceleration vector (ft/sec^2) of the water normal to the member axis
C_D = Drag Coefficient
C_M = Inertia Coefficient

Computer determines the submersed portion of members passing the wave profile, divides it to segments and calculates for

(7.1.3.5 continued)

each segment the hydrodynamic force F then for the member and properly applies the wave/current loads to the connected member.

The wave theory produces the wave profile, the components of velocity and acceleration vectors of water particals at required location of the wave profile.

A, V, D are determined from member property input.
When Marine Growth is considered (See 7.13.5.1) A, V, D are modied by the computer.

Marine Growth also effects the surface roughness which effects C_D, C_M

Strucad program has a library for C_D and C_M coefficients for clean surfaced circular members with various diameters.

For cases listed below new C_D and C_M coefficients and/or equivalent strength tubular member sizes have to be hand calculated and supplied with the input to the program.

a) When member is not circular (Like in the case of Deck beams or Launch Skid beams welded on some Jacket Legs) (See Section 7.1.3.5.2)
b) When several members are represented by one (Like 12 - 24" O.D conductors are idealized by one conductor) (See Section 7.1.3.5.2)

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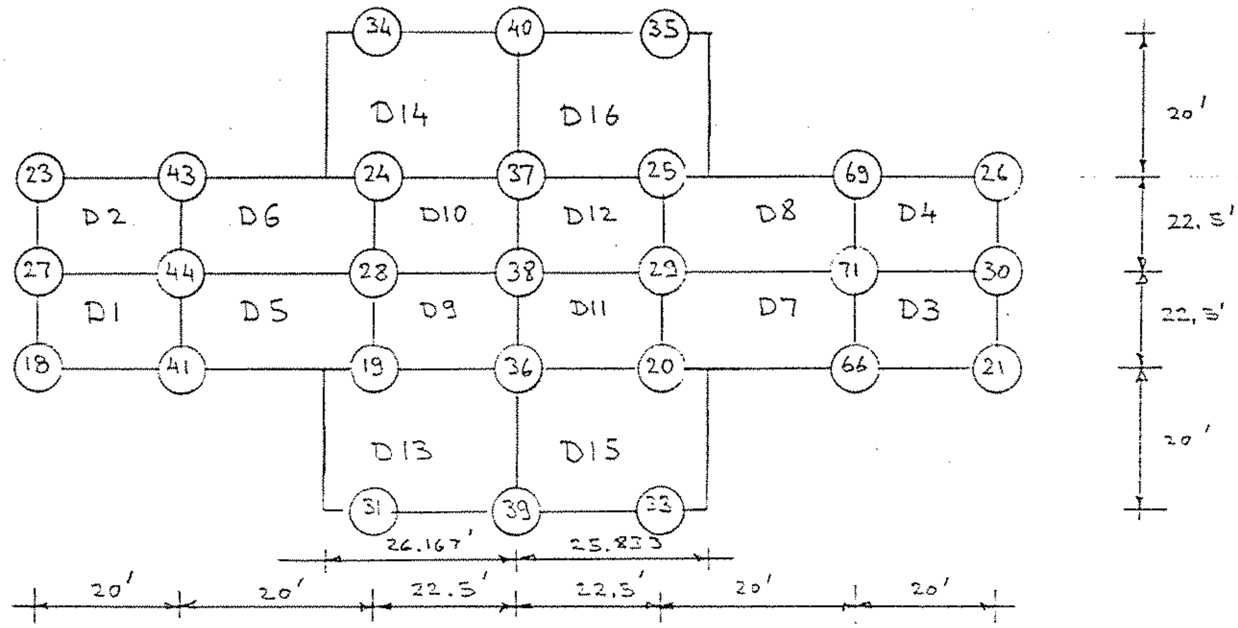
(7.1.3.5 Continued)

c) To consider Wave inundation in Lower deck horizontal drag area its c.g and joint numbers that are effected by the wave force perpendicular to the drag area have to be given in the input. (This process is similar to wind area discription in section 7.1.3.4 c)

For Lower Deck Plane following Drag Area Information is prepared and input. (See Sect. 7.1.3.5.3)



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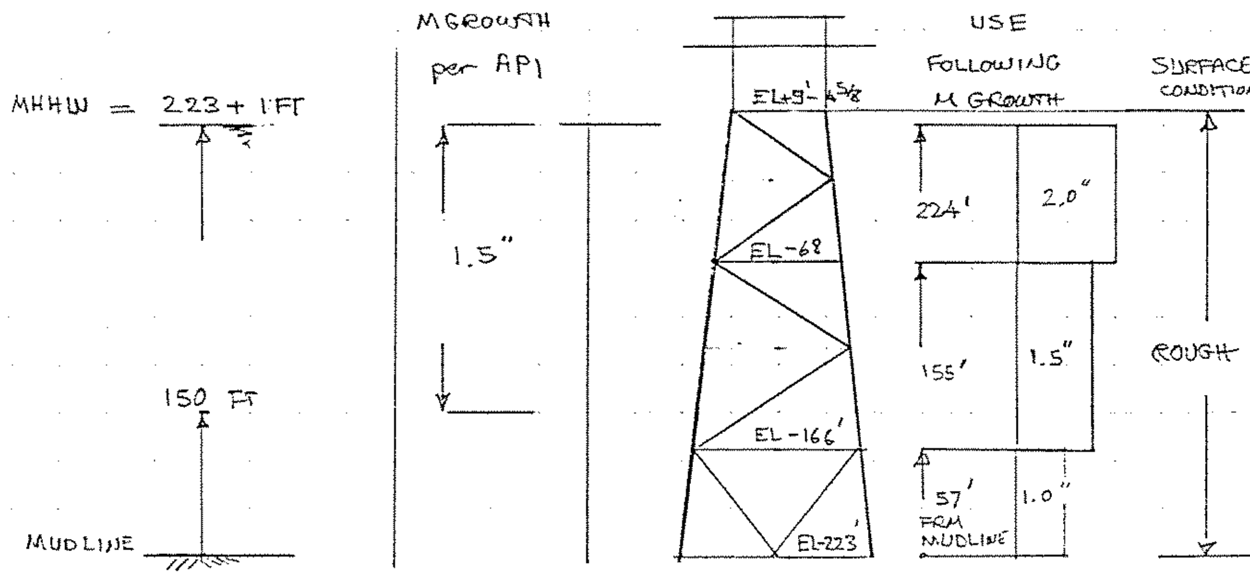
7.1.3.5.1 MARINE GROWTH REF 4.1 SECT. 2.3.4d.2



This information is input in Strucad program by MGROW cards.

7.1.3.5.2 DRAG & INERTIA COEF. CALCULATION

FOR	Idealized Ø	Flooded	Not Flooded
a) CONDUCTORS	29.5" & 33"		x
b) LAUNCH SKID LEGS A2 & A3	12.75"	x	

c) SUMMARY OF EQUIVALENT DECK BEAMS

C1) EQUIVALENT LOWER DECK MEMBER PROPERTIES

C2) " UPPER DECK " "

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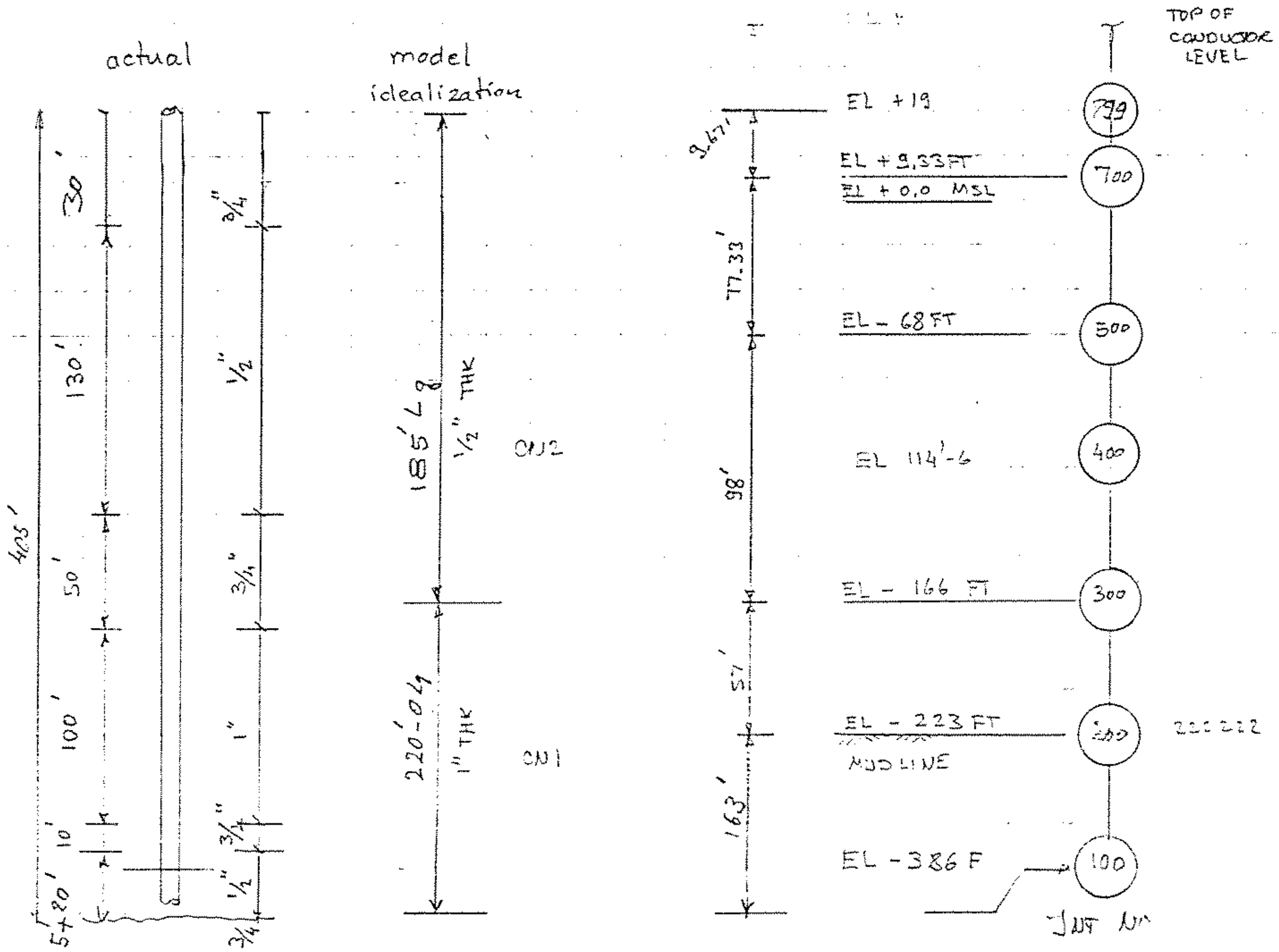
(7.1.3.5.2 Continued)

a) CONDUCTOR SIMILATION

There are (12) 24" O.D. Conductors (with thicknesses 1/2", 3/4" and 1") which are Lumped to one cross-section in structural idealization



GRUP I.D.	IDEALIZED CONDUCTOR PROPERTIES	ROUGH SURFACE COEF.
CN2	24" OD x 1/2" THK A = 36.9 IN² I = 2550 IN⁴ 12 A = 442.8 IN² 12 I = 30600 IN⁴ USE 29.5" OD x 2" THK A = 442.? IN² I = 30??? IN⁴	for equivalent C_D, C_M calc. see next page. $C_{D_{NEW}} = 10.25$ & $C_{M_{NEW}} = 9.53$
CN1	24" OD x 1" THK A = 72.3 IN² I = 4787 IN⁴ 12 A = 867.6 IN² 12 I = 57444 IN⁴ USE 33" OD x 16.43" THK A = 854.9 IN I = 58228 IN⁴	see next page $C_{D_{NEW}} = 9.16$ & $C_{M_{NEW}} = 7.62$

(7.1.3.5.2 continued)

Calculate Modified C_D/C_m for idealized conductor

Smooth C_D = 0.65 C_m = 1.6
rough C_D = 1,05 C_m = 1,2
average C_D = 0,85 C_m = 1.4

} REF 4.1. p. 19 Sect. 2.3.1.6-7
C_D = 1,05 C_M = 1,2
USE ROUGH SURFACE

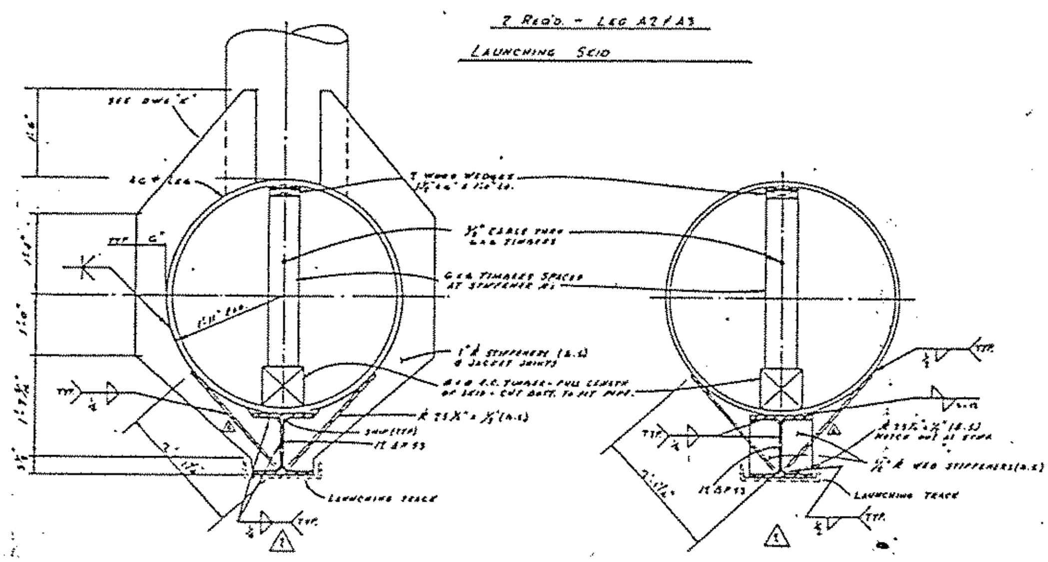
Spacing between conductors S = 8 FT D = 24" S/D = 4
Shielding Factor 1,0 Table 2.3.1.4 Ref 4.1

		C_D; C_M VALUES
EQUIVALENT DRAG COEF:		
for 12 24"Ø conductor	$n \times C_d \times D \rightarrow 12 \times 0{,}85 \times 24'' \rightarrow 244.8$	302,4
shall be equivalent to		
(for 1/2" thk sections) D_{equiv} = 29,5"	$1 \times C_{d_{new}} \times D_{equiv} \rightarrow 1 \times C_{d_{new}} \times 29{,}5 \rightarrow 244.8$	"
FOR GRUP ID CN2 ⟹	or $C_{d_{new}} = 244.8/29.5 = 8.30$	10,25
(for 1" thick section) D_{equiv} = 33"	$\rightarrow 1 \times C_{d_{new}} \times 33 \rightarrow 244.8$	302,4
FOR GRUP ID CN1 ⟹	$C_{d_{new}} = 244.8/33 = 7{,}42$	9,16
EQUIVALENT INERTIA COEF		
for 12 24"Ø conductor	$n \times C_M \times D^2 \rightarrow 12 \times 1{,}4 \times 24^2 \rightarrow 9676{,}8$	8294.4
(for 1/2" thk section) D_{equiv} = 29.5"	$1 \times C_{M_{new}} \times D_{equiv}^2 \rightarrow 1 \times C_{M_{new}} \times 29{,}5^2 \rightarrow 9676{,}8$	"
FOR GRUP ID CN2	$C_{M_{NEW}} = 9676.8/29{,}5^2 = 11.12$	9.53
(for 1" thk section) $D_{equivalent}$ = 33"	$1 \times C_{M_{NEW}} \times 33^2 \rightarrow 9676{,}8$	8294,4
FOR GRUP ID CN1	$C_{M_{NEW}} = 9676.8/33^2 = 8{,}89$	7,62

(7.1.3.5.2 Continued)

b) LAUNCH SKID LEGS A2 & A3 From EL -7 to EL -223'

USE JOINT 502 & 503 @ EL -9'-8
" " 121 & 131 @ EL -223'



SECTION "A-A" TYPICAL STIFFENERS @ JOINTS

SECTION "B-B" TYPICAL WEB STIFFENERS

EL	JNT # (A2 Leg)	JNT # (A3 Leg)
+9'-4.625"	621	631
-9'-8"	503	502
-26'-6"	521	531
[illegible]	507	506
-62'	421	431
-93'-3"	302	303
-114'-6"	321	331
-140'-3	205	206
-142'-0	202	203
-166'	221	231
-183	101	102
-223	121	131

SKID STIFFENER

A2 Leg A3 Leg

LAUNCH SKID LEGS ARE REINFORCED WITH 12BP53 AS SHOWN IN SECTIONS A-A & B-B FROM EL -9'-8 TO THE BASE

ADD 12.75" Ø .500 TUBULAR MEMBER (65.42 Lbs/FT)
(WITH GLOBAL OFFSET Y -20" @ BOTH ENDS
MEMBER GROUP NAME SKD $A = 19.22$ SQ IN. STEEL

C_D multiplier Cylinder versus Open Sect

ROUGH SURFACE C_D cyld Ø → 1.05 C_D open SECTION → 2.3

CLEAN SURFACE $C_{D_{cyl}}$ → 0.60

$$C_{D_{NEW}} = 2.3 \times \frac{1.05}{0.6} = 4.025 \rightarrow \text{use } 4.0$$

Use inertia multiplier $C_M = 1.4$
(REF 4.6 Manual p. 3-12-3)

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(7.1.3.5.2 continued)

c) SUMMARY OF EQUIVALENT DECK BEAMS.

DECK	MEMBER	GROUP I.D.	Equivalent New Member	Flooded	Cd Multiplier	or New C_D (multipl. x 0.6)
Upper Deck						
	2-33WF130	Y33	38"ϕ .65	Yes	3.33	2.00
	Endon horiz truss	Y25	24"ϕ .625	"	3.83	2.30
Lower Deck						
	4-24W130	Y24	30"ϕ 1.75	Yes	3.07	1.84
	7-24W130	Z24	32"ϕ 2.875	"	2.88	1.73
	3-30W132	Y32	35"ϕ 1.125	"	3.29	1.97
	7-24W76	Y76	30"ϕ 1.75	"	3.07	1.84
	24W76	W76	W24x76	Yes		2.3
	24W100	WF0	W24x104	"		↓
	24W120	WF2	W24x117	"		↓
	24W130	WF3	W24x131	"		↓
	24W145	WF4	W24x146	"		↓
	24W160	WF6	W24x162	"		↓

Truss Diagonals

8"ϕ .328	D83
8"ϕ .625	D85
10¾ϕ .365	D10
10¾ϕ .500	D11
12¾ϕ .500	D12
16 ϕ .500	D16
16 ϕ .656	D17
24 ϕ .750	D24

Frame Legs

Tapered 42"ϕ1 to 36"ϕ1	TL1
36"ϕ1	TL2
36"ϕ1.25	TL3

Note: Truss Diagonals & Frame Legs are not flooded (incld. bouyancy)

(7.1.3.5.2 Continued)

C1) EQUIVALENT LOWER DECK MEMBER PROPERTIES

Group I.D. Y.76 to represent 7 - 24WF76 by a tubular member

Single 24WF76 $A = 22.4\ in^2 \times 7 \rightarrow 156.8\ in^2$

$I_x = 2100\ in^4 \times 7 \rightarrow 14700\ in^4$

The beams are 4'-0 spaced, therefore wave force is shieldied by the first outside 24WF.
For open section $C_D = 2.3$ versus for cylindrical section $C_D = 0.6$

Try 30" ∅ 1.75" $A = 155.2\ in^2 \leq 156.8\ in^2$ ok

$I = 15557\ in^4 \geq 14700\ in^4$ ok

Use multiplier $\frac{24^4}{30^4} \times \frac{2.3}{0.6} \doteq 3.07$

or

$C_D = 3.07 \times 0.6 = 1.84$ with 30" ∅ 1.75

(7.1.3.5.2 Continued)

Group I.D. Y.32 to represent 3-30WF132 by a tubular member

Single 30 WF 132 $A = 38.9\ in^2 \times 3 = 116.7\ in^2$

$I_x = 5760\ in^4 \times 3 = 17280\ in^4$

Try 35" ø 1.125 $A = 119.7\ in^2 > 116.7\ in^2$ OK ✓

$I = 17196.5 \leq 17280\ in^4$ OK ✓

For open section $C_D = 2.3$ versus $C_D = 0.6$ for cylinder secti.

Use C_D multiplier $\frac{30"}{35"} \times \frac{2.3}{0.6} = 3.29$

or $C_D = 3.29 \times 0.6 = 1.97$ with 35" ø 1.125"

Group I.D. Y24 to represent 4 - 24 WF 130 by a tubular member

single 24 WF 130 $A = 38.3\ in^2 \times 4 = 153.2\ in^2$

$I_x = 4020\ in^4 \times 4 = 16080\ in^4$

Try 30" ø 1.75 $A = 155.2\ in^2 \geq 153.2\ in^2$ OK ✓

$I = 15557\ in^4 \leq 16080\ in^4$ OK ✓

use C_D multiplier $24"/30" \times \frac{2.3}{0.6} \cong 3.07$ or

$C_D = 3.07 \times 0.6 = 1.84$ with 30" ø 1.75

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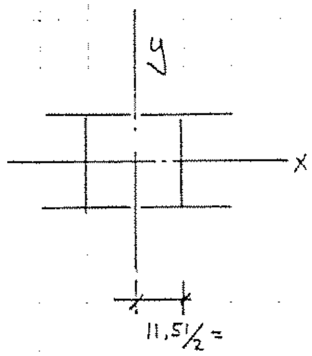
(7.1.3.5.2 , Continued).

EQUIVALENT UPPER DECK MEMBER

Group I.D Y33 — to represent 2 – 33WF130 by a tubular member



$A = 2 \times 38.3 = 76.6 \ in^2$

$I_x = 2 \times 6710 = 13420 \ in^4$

$I_y = 2\left(218 + 38.3 \times \left(\frac{11.51}{2}\right)^2\right) = 2973 \ in^4$ (but braced against other deck bms)

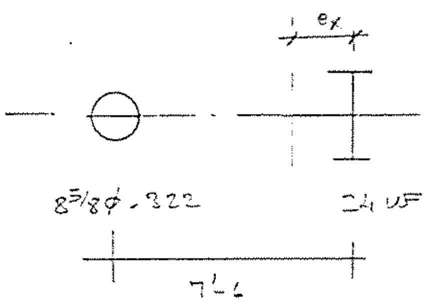
TRY 38"φ .650

$A = 76.23 \ in^2 \leq 76.6 \ in^2$ ok

$I = 13307 \ in^4 < 13420 \ in^4$ ok

C_d multiplier $\frac{33}{38} \times \frac{2.3}{0.6} = 3.33$

GROUP I.D Y25 — to represent horizontal end on truss



8 5/8φ 24WF (for brace wt)

$A = 8,40 + 27,7 + \approx 1,5 \times 8,40 = 48,7 \ in^2$

$I_x = 72,5 + 2690 + \approx 72,5 = 2835 \ in^4$

$e_y = 8,40 \times 7.5' / (8,40 + 27,70) = 1,745'$

$e_y' = 7,5 - 1,745 = 5,755'$

$I_y = 72,5 + 108 + 8,40 \times 5.755^2 + 27.7 \times 1,745^2$
$= 543 \ in^4$

TRY 24φ .625

$A = 45,9 \ in^2 < 48.7 \ in^2$ ok

$I = 3140 > 2835 \ in^4$ ok

C_d multiplier $\frac{24"}{24} \times \frac{2,3}{0,6} = 3,83$

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7.1.3.5.3 DRAG AREA INPUT DATA
FOR WAVE INUNDATION ON LOWER DECK

DRAG AREA I.D.	AREA SQFT.	C.g. X	C.g. Y	C.g. Z	JOINT NUMBERS TO WHICH WAVE FORCE IS APPLIED			
DD 1	20'x22.5' = 450	-52.5	-11.25	41.0	18	27	41	44
" 2		"	11.25		23	27	43	44
" 3		52.5	-11.25		21	30	66	71
" 4		"	11.25		26	30	69	71
" 5		-32.5	-11.25		19	28	41	44
" 6		"	11.25		24	28	43	44
" 7		32.5	-11.25		20	29	66	71
" 8		"	11.25		25	29	69	71
" 9	22.5'x22.5' = 506.25	-11.25	-11.25	41.0	19	28	36	38
" 10		"	11.25		24	28	37	38
" 11		11.25	-11.25		20	29	36	38
" 12		"	11.25		25	29	37	38
" 13	26.17'x20 = 523.4	-13.085	-32.50		19	31	36	39
" 14	"	"	32.50		24	34	37	40
" 15	25.833x20 = 516.7	13.085	-32.50		20	33	36	39
" 16	"	"	32.50		25	35	37	40

Drag Areas and Volumes for Boat Landing & Bumpers are calculated in following pages.

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(7.1.3.5.3 Continued)

C_D / C_M Input for Boat Landing (One Req'd)

DRAG AREA ID : BL

Area (SQ FT) X Projection use $\frac{1}{2}(4'+2') \times (8'+3) = 33$ SQ FT REF 4.2 DWG 19N

Area (SQ FT) $12^{3/4}\ \phi \times 242' \rightarrow$ 257.125
$8^{5/8}\ \phi \times 127' \rightarrow$ 91.281
$10^{3/4}\ \phi \times 147' \rightarrow$ 131.688 } see Sect. 7.1.3.2

Y Projection 480.009 SQ FT use 480 SQ FT
Z Projection $\frac{1}{2}(4'+2') \times 40 = 120$ SQ FT

C.g of Drag Area BL $X = 22.5 + 20 = 42.5'$

$y = -\left[\frac{1}{2}(45'-7\frac{1}{4}) + 3.5'\right] = -26.3'$

$Z = +2.5'$

Wave Force to be distributed to JNT: 631 & 641

Specify D in column 80 of input card to indicate DRAG
$C_D = 1.05$ on Column 46 to 50

INERTIA VOLUME ID : BL

Volume (Cb Ft) 480 × (~3') = 1440.00 Cb FT

c.g of Inertia Volume VL is same as for BL

Wave force to be distributed to the JNTS: 631 & 641

Specify $C_M = 1.4$ on Column 46 to 50
I on " 80 to indicate INERTIA

(7.1.3.5.3 continued)

C_D, C_M Input for BUMPERS (6 Required)

DRAG AREA ID ; BB

X & Y Projection Area (SQ FT) $(43''/12) \times (17.33') \approx 62.0$ SQFT (see Sect 7.1.3.2)

Z Projection " (SQ FT) $\pi \times (43/12)^2 \times 1/4 \times 1/0.7 = 14.4$ SQFT

(F.S flat area) $C_D = 1$

c.g of Drag Area ; X and Y varies see below

$Z \cong +1.0$

ID	BB ①	BB ②	BB ③	BB ④	BB ⑤	BB ⑥
c.g X	−63.064	−22.50	−63.064	−22.50	22.50	63.064
c.g Y ($\approx 23.064 + 2' + 3'$)	−28.064	−28.064	28.064	28.064	28.064	28.064
c.g Z	+1.0	1.0	1.0	1.0	1.0	1.00
Wave force @ JOINT #	611	621	651	661	671	681

Specify $C_D = 0.7$ on Col 46 to 50 and D on Column 80

INERTIA VOL ID ; BB

Volume (CBFT) $\pi \times \left(43/12\right)^2 \times \frac{1}{4} \times \approx 15' = 151.27$ ← (Thks & pile)

cg and Joint info as in B1 thru B6 above

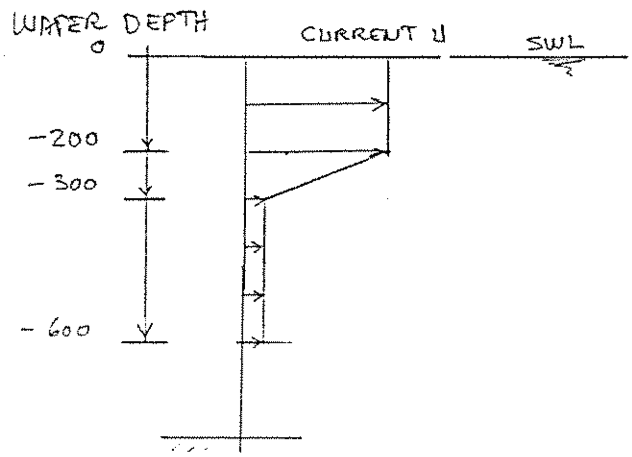
Specify $C_M = 1.45$ on Col 46 to 5 and I on Col 80

7.1.3.5.4 CURRENT

Current Associated with the Wave Height is discussed in RP-2A WSD (Ref 4.1. Sect 2.3.4c.4.).

For all water depths (Shallow < 150', Deep > 300' intermediate from 150' to 300') max current is given as 2.1 Knots.

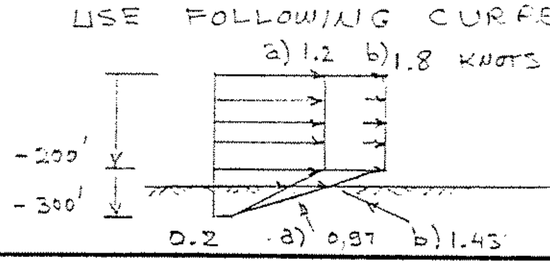
Design current profile is given in Ref 4.1 p. 29 fig. 2.3.4-6.



Max Current reduces for deepwaters with Wave direction measured clockwise from North (see Fig. 2.3.4-4 p. 28)
for shallow waters design current direction is shown on Fig. 2.3.4.5 p. 28)

For intermediate water depths for each wave direction Current values will be interpolated between Shallow and Deep water values (see Section 2.3.4c4)

Draft section 17 of Ref 4.1 gives new values for Current Speed in Table 17.6.2-1.
See also Section 6.5.1 of this calculations.
USE FOLLOWING CURRENT PROFILS

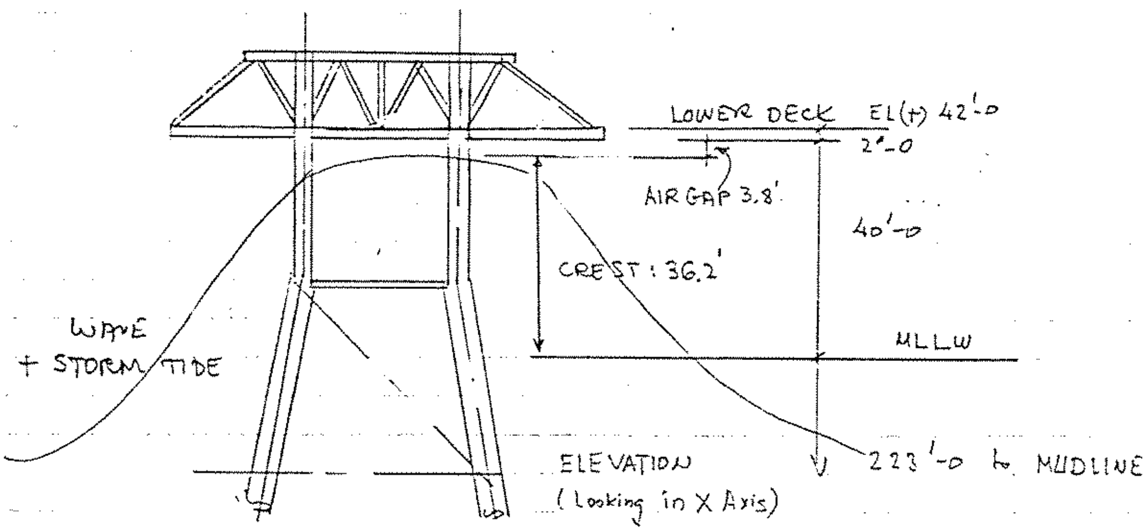


a) FOR DESIGN LEVEL ANALYSIS
MAX CURRENT 1.2 KNOTS

b) FOR ULTIMATE STRENGTH
MAX CURRENT 1,8 KNOTS

7.1.3.6 SUMMARY OF METOCEAN CRITERIA FOR WD 103A PLTF

REF 4.1 SECT 17.6.2.1



From Table 17.6.2-1 GULF OF MEXICO METOCEAN CRITERIA — REF 4.1 P. 13

for Insignificant Environmental Impact / Manned - Evacuated
SUDDEN HURRICANE WAVES

Read for Water depth 223'

Fig:		Design Level Analysis	Ultimate Strength Analysis
17.6.2-3a	Wave Height	46.3 ft	59.9 ft
	Storm tide	3.0 ft	2.5
		49.3	62.4
17.6.2-3b	Minimum Deck Height: → CREST (WAVE + TIDE) → 36.2' < 40' design		
	Wave direction	Omni Direction	Fig. 17.6.2-4
	Current Speed	1.2 Knots	1.8 Knots
	Wave Period	11.3 Second	12.5 Second
	Wind	55 knots	70 Knots

As minimum deck height criteria is satisfied 36.2' < 40'

7.1.3.7 SOIL / PILE INTERACTION

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(7.1.3.7 continued)

SOIL DATA FOR SOL1

BELOW MUD LINE	SOIL	SOIL TEST RESULTS SAMPL #	PENETR. FT	DRY WT LB/FT3	SHEAR STRENGTH C T/FT2	FAILURE STRAIN %
SEE NOTES	①	②				
0' to 33'	very soft gray clay	4	13'	52	0,04	22.1
33' to 100'	soft clay w/shell frag	10	73'	60	0,21	22.1
100' to 150'	gray silty fine sand	13	132' (2a)	103	0,50 (2a) φ = 38°	
150' to 210	gray sandy silt	15	172' (2a)	81	0,50 (2a) φ = 40°	
210' to 270°	Firm gray clay w/silt parting	19	233'	75	0,41	16.6

NOTES:

① REF 4.3 Plate 4 & 4a

② REF 4.3 Plate 2 ; (2a) Plate B-1

(7.1.3.7 Continued)

SOIL DATA INPUT PREPARATION FOR STRUCAD

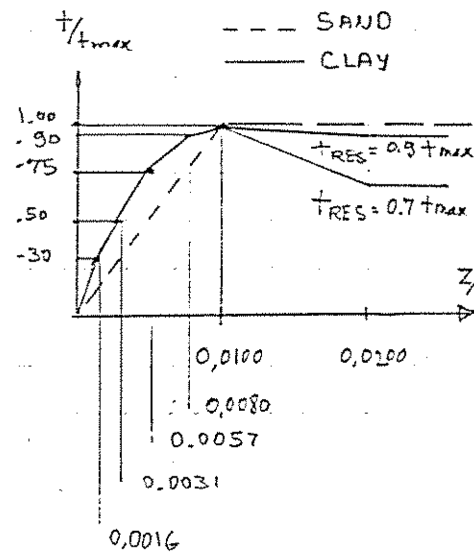
Note: Although T-Z and TB-Z curves are described for soil layer at a time computer input contains T-Z data from pile head to pile tip followed by TB-Z data same way for each layers along the pile length.

DIMENSIONLESS PILE FRICTION (T-Z) CURVES

REF: 4.1 Fig 6.7.2-1 IS USED

FOR PILE D = 42"

*Z/D	.0016	.0031	.0057	.0080	.0100	.0200	
Z (in)	.07	.13	.24	.34	.42	.84	
t/t_{max}	.30	.50	.75	.90	1.00	1.00	SAND
						0.90 TO .70	CLAY

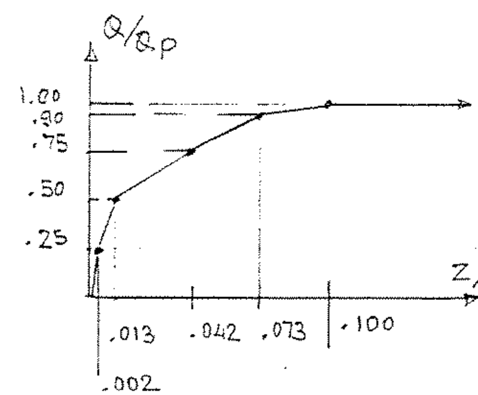


DIMENSIONLESS PILE ENDBEARING (TB-Z) CURVE

REF: 4.1 Fig 6.7.3-1 IS USED

FOR PILE D = 42"

*Z/D	.0020	.0130	.0420	.0730	.1000
Z (in)	0,08	0,55	1,76	3,07	4,20
Q/Q_p	0,25	0,50	.75	.90	1,00



* Note: IN STRUCAD INPUT FOR ZVAL USE Z/D VALUES; GIVE D AS MULTIPLIER

(7.1.3.7 Continued)

SOIL DATA INPUT FOR STRUCAD

Soil type SOL1 (Between 0' to 33' below Mudline)
Very soft Clay

Prepare T-Z Curve Values for PILE FRICTION (see sheet)

Ref: 4.1 Fig 6.7.2-1 $t_{max} = f$

Ref: 4.1 p.59 (6.4.2-1) $f = \alpha \times C$

$C = 0{,}04\ T/FT^2$ (Sheet) Undrain shear strength test result

α = dimensionless factor (Ref: 4.1 p.59 (6.4.2-2)
for under consolidated clay $\alpha = 1$ per Ref: 4.1 p.59

therefore

$$t_{max} = f = 1 \times C = 0{,}04\,(T/FT^2) \times 2\,(K/T) \times \frac{1}{144}\left(\frac{FT^2}{IN^2}\right)$$

$t_{max} = 0{,}000556$ KSI $t \rightarrow$ TVAL $z \rightarrow$ ZVAL

(SEE SH:) (SEE SH:)

DATA POINTS	t/t_{max}	TVAL KSI	ZVAL IN
1	0	0	0.0
2	0,30	0,00017	0,0016
3	0,50	0,00028	0.0031
4	0,75	0,00042	0,0057
5	0,90	0,00050	0,0080
6	1,00	0,00056	0,0100
7	0,70	0,00039	0,0200

USE MULTIPLIER D=42. IN

INPUT FOR TVAL w/MULTIPLIER

$t_{max} \times \pi \times D = 0{,}000556 \times \pi \times 42.0 = 0{,}07336$
(KSI) x (-) x (IN) K/IN

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(7.1.3.7 continued)

(Soil Data between 0' to 33' below mudline continues)

Prepare T-B Curve Values for END BEARING (See SHT ...)

Ref: 4.1 Fig. 6.7.3-1 and p.59 (Equ. 6.4.2-3)

$q = 9c$ $c = 0.04\ T/FT^2 = 0.000556$ KSI (SHT

$q = 9 \times 0.000556 = 0.005$ KSI

Calculate TBVAL $= Q/Q_p \times q$ ZVAL from SHT

DATA POINTS	Q/Q_p	TBVAL (KSI)	ZVAL (IN)
1	0.00	0	0.0
2	0.25	0.00125	0.0020
3	0.50	0.00250	0.0130
4	0.75	0.00375	0.0420
5	0.90	0.00450	0.0730
6	1.00	0.00500	0.1000

USE
↑ MULTIPLIER D = 42. IN

* INPUT FOR
TBVAL MULTIPLIER $(q \times \pi \times D^2/4) = 6.93$ Kips

(7.1.3.7 continued)

Soil Data

Soil type SOL1 (Between 33.1' to 100.' below mudline)

Soft clay w/shell fragments

Prepare T-Z Curve Values for PILE FRICTION (see SHT)

Ref: 4.1 Fig. 6.7.2-1 $t_{max} = f$ $f = \alpha \times C$

for underconsolidated clays $\alpha = 1$ (see SHT)

from Soil test Results SHT $C = 0.21\ T/FT^2 = 0.42$ KSF

$t_{max} = c = 0.42/144 = 0.00292$ KSI

similar to SHT $TVAL = t/t_{max} \times t_{max}$

DATA POINTS	t/t_{max}	TVAL KSI	ZVAL IN
1	0	0	0
2	0.30	0.00088	↑
3	0.50	0.00146	
4	0.75	0.00219	
5	0.90	0.00263	SEE SHT:
6	1.00	0.00292	
7	0.80	0.00233	↓
8	0.80	0.00233	1.00

↑ MULTIPLIED D = 42.0 in

↑ w/MULTIPLIER $(0.00292 \times \pi \times D) = 0.38528$ K/IN

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(7.1.3.7 continued)

Soil Type SOL L (continues) (Between 33.1' to 100.0' below mudline)
(Cohesive Soil)

Prepare T-B Curve Values for END BEARING (See Sht:)

Ref: 4.1 Fig 6.7.3-1 and p. 59 (Equ. 6.4.2-3)

$q = 9 \times C$ $C = 0.21\ T/FT^2 = 0.0029167$ KSI

$q = 9 \times 0.0029167 = 0.02625$ KSI

Calculate $TBVAL = Q/Q_p \times q$ ZVAL from SHT:

DATA POINTS	Q/Q_p	TB VAL (KSI)	Z VAL (IN)
1	0.00	0.0	0.0
2	0.25	0.00656	↑
3	0.50	0.01313	
4	0.75	0.01969	SEE
5	0.90	0.02363	SHT:
6	1.00	0.02625	↓
7	1.00	0.02625	1.00

└ MULTIPLIER D = 42.0

└ W/MULTIPLIER $q \times \pi \times D^2/4 = 36.37$ K

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(7.1.3.7 continued)

Soil type SOL 1 (Between 100.1' to 150.' below mudline)

Gray silty fine sand (cohesionless soil)

Prepare T-Z Curve Values for PILE FRICTION (see SHT.)

Ref: 4.1 Fig. 6.7.2-1 $t_{max} = f$

Ref: 4.1 (Equ 6.4.3-1) $f = K p_o \tan \delta$ $K = 0.8$ to 1.0

overburden pressure at mid level depth of soil 3 → 125'

$p_o = 10^{-3} \times (52 \times 33' + 60 \times 67' + 103 \times 25') = 8,311$ KSF

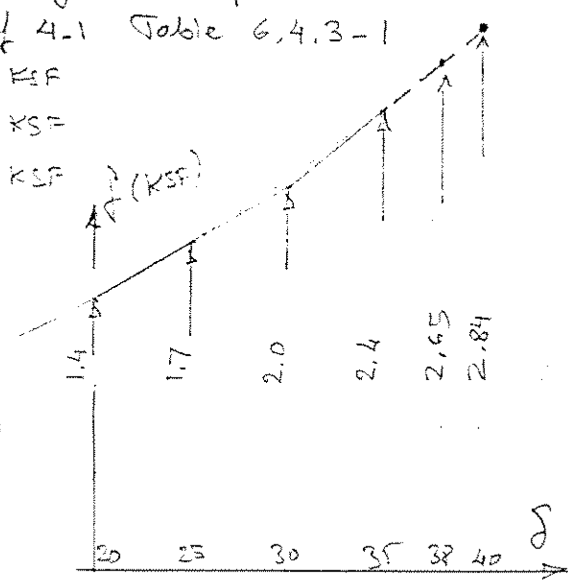
$\delta = 38°$ (SHT.) $f = 0,8 \times 8,311 \times \tan 38° = 5,2$ KSF

(this value compares with Limiting Values of Ref 4.1 Table 6,4.3-1

for $\delta = 25$ $f = 1,7$ KSF

" $\delta = 30$ $f = 2,0$ KSF

$\delta = 35$ $f = 2,4$ KSF



USE $t_{max} = 2,65^{KSF}$ (Extrapolated)

$= 2,65 \times 1/144 = 0,0184$ KSI

$TVAL = t/t_{max} \times t_{max}$

DATA POINT	t/t_{max}	TVAL KSI	Z VAL IN
1	0.0	0.0	0.0
2	1.00	0.0184	0.01
3	1.00	0,0184	1.00

← MULTIPLIED D = 42.0 IN

W/MULTIPLIER $t_{max} \times \pi \times D = 0,0184 \times 131,947 = 2.4278$ K/IN

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(7.1.3.7 continued.)

Soil type SOL 1 (Between 100.1' to 150' below mudline)

Prepare T-B Curve Values for END BEARING

Ref: 4.1 Fig 6.7.3-1 and p.60 Equ. 6.4.3-2

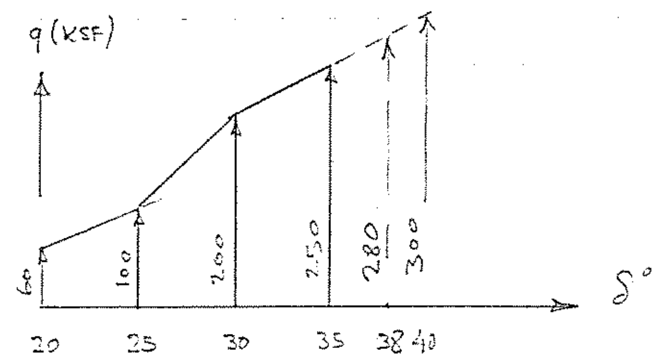
Limiting Unit End Bearing Values extrapolated from Last column in Ref 4.1 Table 6.4.3-1

$q = 280 \text{ KSF} \times \frac{1}{144} = 1.944 \text{ KSI}$

$TBVAL = Q/Q_p \times q$

$ZVAL = Z$

SEE SHT 1



DATA POINTS	Q/Q_p	TBVAL KSI	ZVAL IN
1	0	0	0
2	0,25	0,486	↑
3	0,50	0,972	SEE
4	0,75	1,458	SHT 1
5	0,90	1,750	↓
6	1,00	1,944	
7	1,00	1,944	1.0

MULTIPLIER D = 42.0 IN

W/MULTIPLIER $q \times \pi \times D^2/4 = 1.944 \times 1385.44 = 2693.3$ K

(7.1.3.7 Continued)

Soil type SOL 1 (Between 150.1' TO 210.0' below Mudline)

gray sandy silt (cohesionless soil)

Prepare T-Z Curve Values FOR PILE FRICTION

use API Table 6.4.3-1 (Limiting Values for $\delta = 40°$ extrapolated see SHT.)

$t_{max} = 2.84\ KSF/144 = 0.019722\ KSI$

$TVAL = t/t_{max} \times t_{max}$

DATA POINT	t/t_{max}	TVAL KSI	ZVAL IN
1	0.00	0.0	0.0
2	1.00	0.0197	0.01
3	1.00	0.0197	1.00

MULTIPLIER D = 42"

MULTIPLIER $t_{max} \times \pi \times D = 2.60226\ K/IN$

(7.1.3.7 continued)

Soil type SOL1 Between 150.1' TO 210.0' below Mudline

Prepare T-B Curve Values for END BEARING

use API Table 6.4.3-1 limiting values for $\delta = 40°$ extrapolated see SHT.

$q = 300 \times 1/144 = 2.0833$ KSI

TBVAL = $Q/Q_p \times q$ ZVAL = Z

DATA POINTS	Q/Q_p	TBVAL KSI	ZVAL IN
1	0.0	0.0	0.0
2	0.25	0.521	↑
3	0.50	1.042	SEE
4	0.75	1.562	SHT.
5	0.90	1.875	
6	1.00	2.083	↓
7	1.00	2.083	1.00

ZVAL: USE MULTIPLIER D = 42.0"

Q/Q_p: USE MULTIPLIER $q \times \pi \times D^2/4 = 2886.3$ K

(7.1.3.7 continued)

Soil type SOL1 (Between 210.1' to 270.0' below mudline)

Firm gray clay w/silt parting (cohesive soil)

Prepare T-Z Curve Values for Pile Friction

Ref: 4.1 Fig 6.7.2-1 $t_{max} = f = \alpha \times C$ (Eq. 6.4.2-1)
Ref: 4.1 Eq 6.4.2-2
dimensionless factor $\alpha = 0.5 \times \psi^{-0.5}$ when $\psi \leq 1.0$
$\alpha = 0.5 \times \psi^{-0.25}$ " $\psi > 1.0$

$\psi = c/p_o'$ for point in question

C = 0.50 T/SQFT (SHT:) test result
C = 1.00 KSF = 1000 PSF

p_o' = effective overburden pressure at the point in PSF
Calculate p_o' for mid depth of SOL5 → ½(210+270) = 240'

$p_o' = (52 \times 33' + 60 \times 67' + 103 \times 50' + 81 \times 60' + 75 \times 60'/2) = 17996$ PSF

$\psi = c/p_o' = 1000/17996 = 0.0556 < 1$ therefore $\alpha = 0.5 \times (\psi)^{-0.5} = 2.12$

$t_{max} = 2.12 \times 1.0 \times 1/144 = 0.01472$ KSI

USE MULTIPLIER $t_{max} \times \pi \times D = 1.94226$

DATA POINTS	t/t_{max}	TVAL KSI	ZVAL IN
1	0.0	0.0	0.0
2	0.30	0.00442	↑
3	0.50	0.00736	
4	0.75	0.01104	
5	0.90	0.01325	SEE
6	1.00	0.01472	SHT:
7	0.90	0.01325	
8	0.90	0.01325	↓ 1.00

USE MULTIPLIER D = 42.0'

(7.1.3.7 continued)

Soil type SOL1 (Between 210.1' to 270.0' below mudline)

Firm gray clay w/silt partings (Cohesive Soil)

Prepare T-B Curve Values for END BEARING

Ref: 4.1 Fig. 6.7.3-1 and p.59 Equ. 6.4.2-3

$q = 9C$ $C = 0{,}41$ T/SQ FT (SHT)

$C = 0{,}41 \times 2 \times 1/144 = 0{,}0056944$ KSI

$q = 9 \times 0{,}0056944 = 0{,}05125$

Calculate TBVAL similar to SOL 1

DATA POINTS	Q/Q_p	TBVAL KSI	ZVAL IN
1	0,00	0,0	0,0
2	0,25	0,01281	↑
3	0,50	0,02563	SEE
4	0,75	0,03843	SHT
5	0,90	0,04613	↓
6	1.00	0,05125	↓
7	1,00	0,05125	1.00

ZVAL: USE MULTIPLIER D = 42 IN

Q/Q_p: USE MULTIPLIER $q \times \pi \times D^2/4 = 71.0$ K

(7.1.3.7 continued)

CALCULATE ULTIMATE & ALLOWABLE PILE AXIAL CAPACITIES
using "Input Soil Curve" data
in WD103DL.OT1 p. 36 thru 39

Ultimate Axial Capacity of 42" O.D pile at 270' below Mudline:

Skin friction or tension Capacity (see T-Z Curves)

Depth below Mudline	h ft	Force Kips/ft	Force Multiplier	Ultimate Tension KIPS
0' to 33'	33'	.70 x 12	.073	20
33' to 100'	67'	.70 x 12	.385	217
100' to 150'	50'	1.00 x 12	2.428	1457
150' to 210'	60'	1.000 x 12	2,602	1873
210' to 270	60'	.70 x 12	1.942	979
	Σ = 270'		Total ⇒	4546 K

End Bearing Capacity (see TB-Z curves)

@ Depth 270' = 71 K

Compression Capacity (Tension + End Bearing) = 4617 K

Factor of Safety (Ref 4.1. p. 6,3,4.1 or 3or5)
Design environmental Conditions F.S = 1.5

Allowables: Tension: 4546 / 1.5 = 3031 K
Compression: 4617 / 1.5 = 3078 K

7.2 Design Level Analysis

Design Level Analysis is performed according to Ref 4.1 Section 17.7.2

Assessment criteria for the platform in Gulf of Mexico is shown in Ref 4.1 Table 17.5.2a.

Shell West Delta Blk 103A platform is (Manned, evacuated during storm and has insignificant environmental impact) assessed for "Sudden hurricane design Level analysis Loading see Ref 4.1 p.18,19 Figure 17.6.2-3. See also Table 17.6.2-1 on p.13 of Ref 4.1 This table Leads to the figures from which Wave Ht, Deck Ht, Wave & Current direction are established. The current speed, wave period and wind speed are directly read from Table 17.6.2-1.

For Shell W.D BLK 103A 223' WATERS platform these information are summarized in Section 7.1.3.6

Design Level analysis is performed on three dimensional idealized model of Pile/Conductor/Jacket/Top Platform using STRUCAD*3D program (Ref 4.6).

Design Level analysis results are in sect. 7.2.1

7.2.1. DESIGN LEVEL ANALYSIS RESULTS

Design Level Analysis fails due to following reasons:

7.2.1.1. No soil failure occures. (see 7.2.3.1)
7.2.1.2. Pile failure (axial & bending) (see 7.2.3.2)
7.2.1.3. Jacket diagonal member failure in buckling (see 7.2.4)
7.2.1.4. No punching shear failure in joint cans (see 7.2.5.1)

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(7.2 Continued)

7.2.2. Input Data Summary

For Total Input Echo see Section 8.1.1 COMPUTER MODEL

7.2.2.1 Loading Cases

Basic Loads:
1) DL
2) Deck Lds
3) 45° Metocean Loads
4) 67.5° "
5) 90° "

(Loads 3–5:) WIND: 55 Knots; WAVE: 49.3 FT/11.3 SEC; CURRENT: 1.2 Knots

Combined Loads:
6) 100% DL + 100% Dck Ld + 45° Metoceanic
7) " + " + 67.5° "
8) " + " + 90° "

7.2.2.2 Allowable Increase: AMOD 1.33 on Load Cases 6 thru 8

7.2.2.3 Applied Load Summary (see Sect. 8.8.1 WD103DL.OT1 output p. 96, 97)

Combined Loads	Vertical Z KIPS	Horiz. X KIPS	Horiz. Y KIPS	Resultant Horiz. $(x^2+y^2)^{1/2}$
L.C: 6 (45°)	-8262	1580	1579	2234
L.C: 7 (67.5°)	-8267	869	2096	2269
L.C: 8 (90°)	-8277	30	2251	2251

Note: Vertical Loads are basicly the result of

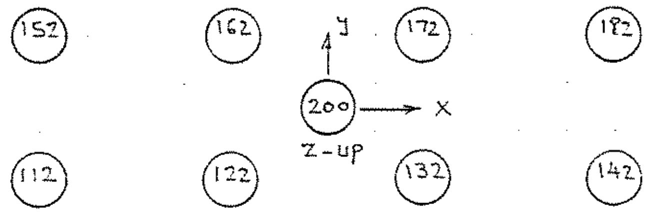
Net Gravity loads	-2209 K	L.C. 1
Deck & Equipment	-6067 K	L.C. 2
	-8276 K	

Metoceanic Loads (+42ᴷ; +38ᴷ; 25ᴷ) L.C. 3, 4, 5 for different directions.

(7.2 Design Level Results continued)

7.2.3 Soil and Pile Results

Pile Head Joints "joints at mudline elevation", describing the transition of jacket structure to piles in soil, are shown for reference below



Soil and Pile results are in WD103DL.OT2 output file see Sect. 8.1.2

7.2.3.1 Soil Results

Soil data is contained in section 7.1.3.6. Computer utilize this data to calculate soil spring characteristics. The pile is supported by soil springs at its segmental joints. The platform jacket is supported by piles at pile head joints (7.2.3). The solution of jacket and pile deformations (i.e internal forces) is a non-Linear iteration process which starts with the assumption that jacket support springs at pile head joints are rigid. The calculated reactions are then applied to the pile at its pile head which produce deformation on soil supported pile. Deformations at pile head allows computer to calculate a new spring stiffness which is no longer rigid as initially assumed. The jacket supported on springs at pile head with new rigidities produce new support reactions.

This process of calculating old and new Pile Head Forces and Displacements continues until the difference between old and new displacement is smaller than a tolerable value.
If in any iteration stage any of the pile head displacement values exceeds the maximum displacement of the corresponding soil curve computer prints out Warning message at Iteration Report.
If the Iteration Report was not selected in Pile Option "PILOPT" card, the Warning Message is printed on (default pile output) Pile Head Load and Deformation Report.
The Warning Line appears as many times as the iterations in which soil curve max deflection is exceeded.

In this computer run soil / pile related computer output in file WD103DL.OT2 (Sect. 8.1.2) contains several Lines of Warnings that for pile head joint 182 in all combined Load cases (6, 7, 8) the axial deflections exceeded the T-Z curves. (See Section 8.1.2 p. 4, 5, 6)

As Long as calculated pile axial force is Less than the Ultimate axial pile capacities (in Compression: 4617 K; in tension: 4546 Kips see section 7.1.3.6) the Warning does not represent any soil failure and will be ignored.

<u>No soil failure exists</u> for Pile Axial Load calculation at Design Level analysis

(7.2. Design Level Results continued)

7.2.3.2 Pile Results (refere output file WD103DL.OT2) Section 8.1.2

7.2.3.2.1 Max Axial Pile Force (see Sect. 8.1.2 p.232, 233)

* Note: Allowable Pile Axial Load (see Sect. 7.1.3.7)

a) Max Tension 863 K < 3031 * OK

@ Pile Joint 112 L.C. 6 see p. 233

b) Max Compression -2867 K 3078 * O.K

@ Pile Joint 182 L.C. 6 see p. 233

7.2.3.2.2 Max Pile Stress Unity Check (AMOD: 1.33)

Pile Stress failure U.C > 1 occures only in Pile Joint 182 (Sect. 8.1.2 p. 233) for

Load Cases	6 (45°)	7 (67.5°)	8 (90°)
U.C	1.17 > 1	1.11 > 1	1.04 > 1

U.C > 1 Values are Located in the pile head region. Overstress condition dies out 2' to 8' below pile head (U.C ≤ 1) until 20' depth and start rising up again until 50' - 60' upto u.c = 0.95 but never exceeds U.C = 1.0 (See Section 8.1.2 p. 245, 248 and 251).

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7.2.3.3 Max Base Shear at Pile Head Joints (WD103DL.OT2 output.) Section 8.1.2

541 K @ JNT 182 L.C. 6

SECT. 8.1.2 PAGES	COMBINED LOAD CASES		JOINT #	LOCAL PILE AXIAL Kips		GLOBAL SHEAR FORCES X Kip	Y Kips	GLOBAL RESULTANT BASE SHEAR $(X^2+Y^2)^{1/2}$ KIPS
233, 254	LC: 6	(45°)	112	863	TENSION	238	187	303
" "			182	-2867	COMPR.	383	382	541
233, 255	LC: 7	(67.5°)	112	723	TENSION	154	213	263
", 255			182	-2749	COMPR.	317	400	510
" "	LC: 8	(90°)	112	334	TENSION	30	189	191
" "		Conductor	200	-375	COMP.	-9	165	165
", 256			182	-2366	COMPR	219	372	432

7.2.3.4. Max Pile Head Joint Deflections

Sect. 8.1.2 WD103DL.OT2 OUTPUT PAGE NO.	COMBINED LOAD CASES		JOINT #		(WD103DL.OT2 OUTPUT) Pile Head Axial Defl (IN)	(in Sect. (WD103DL.OT3 p. 8 and 13) OUTPUT) JOINT DEFLECTIONS (IN) GLOBAL COORDINATES Z	X	Y	$(x^2+y^2)^{1/2}$
14	LC: 6	(45°)	112	DURING max Pull	-.2373	-1.533	10.581	7.144	12.767
63			182	" Down	.3934	0.843	9.342	9.120	13.056
80	LC: 7	(67.5°)	112	DURING max Pull	-.1935	-1.308	5.506	9.531	11.007
129			182	max Down	.9487	0.618	4.262	11.503	12.267
150	LC: 8	(90°)	112	DURING max Pull	-.0757	-0.964	-.134	10.541	10.542
199			182	max Down	.8047	0.273	-1.097	11.957	12.007

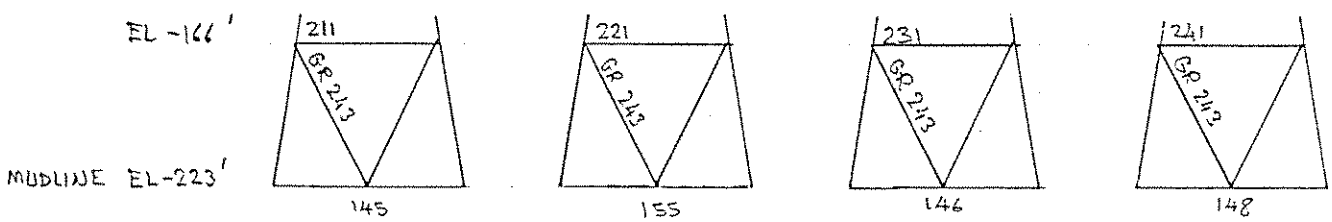
(7.2. Design Level Results Continued)

7.2.4 Jacket Member Results

Failing Jacket members are Listed in WD103DL.OT3 Section 8.1.3 p. 83 & 84 under "Member Group Summary Report" Group I Unity Check > 1.33 and Group II " " > 1.0.

These members are displayed below:

Truss Plane 1	Truss Plane 2	Truss Plane 3	Truss Plane 4



Report Page					
		ALL failures for L.C. 8 (90° Direction)			KL/r = 99.6
83	max U.C. > 1	1.066	1.609	1.652	1.284
30	f_a/F_a	.771	.947	.968	.830
"	f_{by}/F_{by}	.285	.662	.684	.437
"	f_{bz}/F_{bz}	.071	.025	.024	.126
83	f_a	-13.25 KSI	-16.32 KSI	-16.68 KSI	-14.26 KSI
"	f_{by}	- 3.77 "	- 4.85 "	- 4.53 "	- 4.89 "
"	f_{bz}	0.94 "	- 0.19 "	- 0.16 "	-1.41 "

Buckling Failure.

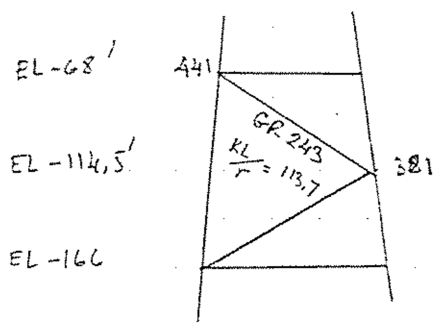
* REF 4.6 (Manual) Sect 4.6-4 (Compression + Bending)

(7.2. Continued)

Similar Buckling Failure occures for Jacket K-Brace diagonals at higher elevations.

Truss Plane 4 — U.C. 1.711 > 1 — L.C. 8 (90°)

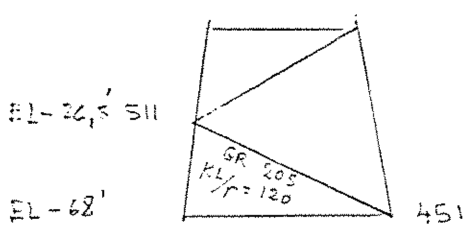
Sect. 8.1.3 p.83 — file .OT3 report p. 30



		U.C. Components	
f_a = -12.92	KSI	.873	.873
f_{b_y} = -4.63	KSI	.740 } SRSS	
f_{b_z} = -2.47	KSI	.394 }	.838
			1.711

Truss Plane 1 — UC : 1.00 = 1 — L.C. 8 (90°)

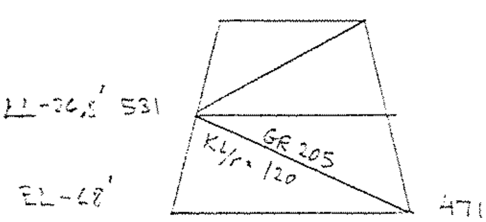
P. 84 — file .OT3 p. 25



		U.C. Components	
f_a = -8.98	KSI	0.659	.659
f_{b_y} = -3.51	"	0.240 } SRSS	
f_{b_z} = +3.55	"	0.243 }	.342
			1.001

Truss Plane 3 — UC : 1.153 > 1 — L.C. 8 (90°)

Section 8.13 P. 84 — file .OT3 p. 30



	U.C. Component	
f_a = -10.17	0.744	0.744
f_{b_y} = -4.15	0.376 } SRSS	
f_{b_z} = 1.52	0.161 }	0.409
		1.153

(7.2 Continued)

7.2.5 Joint Can Analysis Results

(See Section 8.1.4 WD103DL.OT4)

Pertinent joint can analysis information are reported in Joint Can Summary in WD103DL.OT4 p. 84 thru 87 as followed:

a) Joint numbers (Common Jnt, Chord, Brace)

b) Chord & Brace member sizes and material yield stress.

c) Unity Check for actual brace Load punching shear

d Unity check for 50% brace strength punching shear

In design level joint can analysis allowable stresses are increased by a factor AMOD = 1.33.

Unity Check ≥ 1 signals that total joint capacity has reached and exceeded the factored allowable stress Levels.

7.2.5.1 Failing Joint Can

A joint can is declared failed when u.c ≥ 1 occured due to actual brace Loading. (see Ref 4.1 Section 17.7.2c Connections)
According this criteria NONE of the Joint Can of Shell West Delta Blk 103A fails in Design Level Analysis.

7.3. ULTIMATE STRENGTH ANALYSIS

Since in previous sections the Design Basis Check is by passed (see Sect. 6.4) and the Design Level Analysis has failed (see Sect 7.2.1) Ref 4.1 Section 17.7.3 requires that platform must demonstrate adequate strength and stability to survive the ultimate strength Loading criteria set forth in Sections 17.5 and 17.6 of Ref 4.1 so that current and extended use of the platform is insured.

Ultimate Strength Analysis of Shell West Delta Blk 103A platform is performed using STRUCAD * 3D program (Ref 4.6) which uses the method of Linear elastic analysis. This approach is permitted in Section 17.7.3a of Ref 4.1.

For this purpose safety factors will be removed by using AMOD card feature of STRUCAD program. AMOD is a user selected multiplication factor effective only on basic Load cases and typically applied on environmental Load cases.

Strucad program allows to use different AMOD cards for Member Stress analysis and for Joint Can punching shear calculations.

(7.3 continued)

Ref 4.1 Section 17.7.3 c allows further that Ultimate Strength Analysis may use a mean yield stress instead of nominal yield stress of the material. For ASTM A36 steel nominal yield stress is 36 KSI and mean yield stress is 42 KSI. The effect of higher yield stress will be incorporated into calculated AMOD value, although the computer input/output will still show yield as 36 KSI.

7.3.1 Calculate AMOD multiplier to approximately remove the factor of safety (F.S) and incorporate effect of mean yield stress.

7.3.1.1 Factor of Safety for different Stress types

Stress Type	Allowable Stress	F.S.	Ref 4.1 Section
a) Tension	$0.6\ F_y$	$1/0.6 = 1.67$	(3.2.1)
b) Compression			
Column buckling Elastic	$\alpha\ F_c'$	$1/\alpha = 1.92$	(3.2.2a)
" Inelastic	$\alpha\ F_c$	$1/\alpha$ varies from 1.67 to 1.92	
Local buckling of Tubular Members		2	(3.2.2b) (Commentary C.3.2.2)
c) Bending	$0.75 F_y$	2 (against ultimate)	C.3.2.4
d) Shear	$0.40\ F_y$	$1/\frac{2}{3} = 1.5$	(4.1.5)

(7.3. continued)

7.3.1.2 Effect of using mean Fy

$\text{mean } F_y / \text{nominal } F_y = \frac{42}{36} = 1.17$

7.3.1.3 AMOD selection

7.3.1.3.1 To use for Combined Load cases in Member Stress U.C.

Most probable failure of Jacket Member Stress is expected to be Local buckling of tubular members. For this case F.S = 2.0

For slender member column buckling in the overlap range from inelastic to elastic is the next probable failure senerio with F.S = 1.92

The most conservative F.S = 1.67 is for tension

Therefore for AMOD calc. remove safety against tension

AMOD = 1.17 x 1.67 = 1.9539 ⟹ USE 2.0

7.3.1.3.2 To use for Combined Load Cases in Punching Shear U.C

Remove safety against Shear

AMOD = 1.17 x 1.5 = 1.755 ⟹ USE 1.75

(7.3 Ultimate Strength Analysis Continued

7.3.2 Loads

7.3.2.1 Dead & Buoyancy Loads

Same as in Design level analysis (See 7.1.3.1)

7.3.2.2 Deck Loads, Equipment Loads

Same as in Design Level analysis (see 7.1.3.2)

7.3.2.3 Metocean Loads (see 7.1.1)

7.3.2.3.1 Wind 70 Knots

7.3.2.3.2 Current 1.8 Knots

7.3.2.3.3 Wave 62.4 FT / 12.5 SEC.

(7.3. Ultimate Strength Analisis Continued)

7.3.3 ULTIMATE STRENGTH ANALYSIS RESULTS

Ultimate Strength Analysis fails due to following reasons:

7.3.3.1 Soil failure
None (see section 7.3.5.1)

7.3.3.2 Pile failure
Pile 182 collaps by forming full plastic hinges at two Levels (see sections 7.3.5.2.4 and 7.3.5.2.5

7.3.3.3 Jacket failure
Several diagonals buckles (see Section 7.3.6)

7.3.3.4 Jacket Leg Punching Shear (Sect. 7.3.7)

Jacket Leg Chord have punching shear failure at mud line Level joints (thickness 1"), as well as at higher elevation joints with thickness ½".

(7.3. Continued)

7.3.4 Input Data Summary

(for input echo refer to section 8.2.1 in Book 3 of 4)

7.3.4.1 Loading Cases

Basic Loads:

1) DL
2) Deck Loads
3) 45° Metocean Loads
4) 67.5° "
5) 90° "

} WIND : 70 Knots; WAVE : 62.4 FT/12.5 SEC.; CURRENT : 1.8 Knots

Combined Loads:

6) 100% DL + 100 Deck Ld + 45° direction Metoceanic Load
7) " + " + 67.5° " "
8) " + " + 90° " "

7.3.4.2 Allowable increase : AMOD 2.0 on Load Cases 6 thru 8 for Deflections & Stress
AMOD 1.75 for Joint Can Punching Shear

7.3.4.3 Applied Load Summary (See WD103U3.OT1 output p. 96 and 97)

Combined Loads	Vertical Z Kips	Horiz. X Kips	Horiz. Y Kips	Result. Horiz. $(x^2+y^2)^{1/2}$ Kips
L.C: 6 (45°)	-8234	2908	2864	4082
L.C: 7 (67.5°)	-8238	1588	3786	4106
L.C: 8 (90°)	-8252	58	4072	4072

7.3 Ultimate Strength continued.

7.3.5 Soil and Pile Results (WD103US.OT2 output)
(Section 8-2.2 in Book 3 of 4)

For Pile Head Joint Numbering
refer to section 7.2.3 (Design Level Results)

7.3.5.1 Soil Results

Jacket / Pile / and soil nonlinear interaction was explained in 7.2.3.1 Soil Results (for design Level analysis).
Ultimate strength analysis is carried out for more severe environmental Loading same way as design level analysis was carried out except factor of safety for member stress calculation was removed by using AMOD 2.0 for Member stress Levels and AMOD 1.75 for Joint Can stresses.
None of the AMOD is effective on Soil Pile behaviour or deformation calculations
Therefore AMOD 2.0 appearing on Pile Head Load and Deformation Report pages shall be ignored.

Similar to Design Level calculations computer flags Warning messages during the iteration process of non Linear solution of Ultimate Strength calculations, that some pile head joint deformation exceeds the T-Z curve axial deflection limit.

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(7.3.5 continued)

Warning message is not critical as long as the calculated pile axial Load does not exceed ultimate pile axial capacities.

Ultimate Pile Axial Capacities are: (sect. 7.1.3.6)
Compression : -4617 K
Tension : 4546 K

Max Pile Axial Loads are: (see p.234 WD103US.OT2)

Compression :

Pile Head JNT	Load Case	Axial force		Pile Axial Ultimate	
182	6	-4212	<	-4617 K	OK
"	7	-3963	<	"	O.K
"	8	-3270	<	"	O.K

Tension

Pile Head JNT	Load Case	Axial force		Pile Axial Ultimate	
112	6	2170 K	<	4546 K	OK
"	7	1879 K	<	"	
"	8	1163 K	<	"	

Soil Result Conclusion

There is no soil failure in skin friction and end bearing. All calculated pile axial Loads remain below the ultimate axial capacity Level of the piles.

(7.3.5. ... continued)

7.3.5.2 Pile Results (see Section 8.2.2 in Book 3 of 4)

Pile Failure : UC > 1 see 7.3.6.2

7.3.5.2.1 Max Axial Pile Force

a) Tension 2170 K @ Jnt 112 L.C.6 45° direction
see (WD103US.OT2 p.234 also previous page)

b) Compression + 4212 K @ Jnt 182 L.C.6 45° direction
see (WD103US.OT2 p.234 also previous page)

7.3.5.2.2 Max Pile Stress Unity Checks

Pile Head Unity Check Report (WD103.OT2 p.234)
Failure : U.C. > 1

	U.C.	GRID	L.C.	PILE HEAD JNT#	STRESS (KSI) f_a	STRESS (KSI) f_b	SHEAR FORCE KIPS
Conductors	0.74 < 1 OK	P33	6	200	-.44	39.25	350
Piles	1.23 > 1	P42	6	112	9.81	54.39	395
↓	1.09 > 1	"	"	122	1.18	57.20	350
	1.15 > 1	"	"	132	3.11	58.18	368
	1.26 > 1	"	"	142	-6.26	60.32	307
	1.14 > 1	"	8	152	-11.93	46.61	218 (306^K L.C.6)
	1.31 > 1	"	6	162	-11.61	56.26	253
	1.29 > 1	"	6	172	-10.51	56.78	263
	1.54 > 1	"	6	182	-19.03	59.12	206 (221^K L.C.8)

All piles except conductors fail in bending forming plastic hinge at pile head joint.

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(7.3. Ultimate Strength Continued)

7.3.5.2.3 Comparative Pile Unity Check for Pile Joint 182 L.C. 6

Ultimate Strength Capacities of 42" O.D 1.75 Pile Cross Section

Section Properties:

$A = \frac{\pi}{4}\left[42^2 - (42 - 2\times1.75)^2\right] = 221.3 \text{ in}^2$

$I = \frac{\pi}{64}\left[42^4 - 38.5^4\right] = 44\,896.8 \text{ in}^4$

$S = 44896.8 \times 2/42 = 2137.9 \text{ in}^3$

$Z = \frac{42^3}{6} - \frac{38.5^3}{6} = 2836.9 \text{ in}^3$

F_y	36 KSI	42 KSI
$P_y = F_y \times A$	7967 Kips	9295 Kips
$M_y = F_y \times S$	76 964 K-in	89 792 K-in.
$M_p = F_y \times Z$	102 128 K-in	119 150 K-in.

For comparison with computer unity check

Check Pile Joint 182 L.C. 6 by hand using (WD103US.OT2 p.234 P & M values)

Axial	$P = -4212$		
	$P/P_y = 4212/7967 \rightarrow$ 0.53	$4212/9295 \rightarrow$ 0.45	
Moment	$M = 126391$		
	$M/M_y = 126391/102128 \rightarrow$ 1.24	$126391/119150 \rightarrow$ 1.06	
	1.77	1.51	

Computer U.C: 1.54 compares with hand calculated 1.51 close enough

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(7.3.5.2 ... continued)

7.3.5.2.4 Check U.C. > 1 Along Pile Depth

Load Case: 6 (45° Direction) "Critical Load Case"

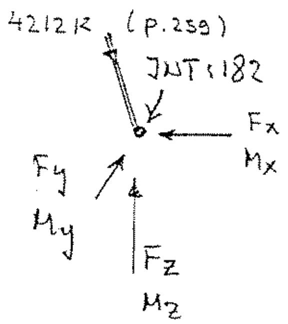
PAGE NO IN WD103US.OT2 →	208	210	214	216	219	222	225	228	231
Distance Along Pile FEET	Conduct.	PILE JOINTS							
	200	112	122	132	142	152	162	172	182
0.0	0.74	1.23	0.58	1.15	1.26	1.14	1.31	1.29	1.54
2.0	↑	1.16	↑	1.08	1.19	1.08	1.25	1.23	1.47
4.0		1.09		1.00	1.12	1.03	1.18	1.16	1.40
6.0		1.05		0.93	1.05	0.97	1.11	1.02	1.34
8.0		0.95		↑	0.98	↑	1.04	0.96	1.27
10.0		↑			↑		0.98	↑	1.20
12.0							↑		1.13
14.0									1.10
16.0									0.98
									↓
30.00									0.47
									↓
50.0	U.C. < 1	U.C. < 1	U.C. < 1	U.C. < 1	U.C. < 1	U.C. < 1	U.C. < 1	U.C. < 1	0.98
52.0									1.02
54.0									1.05
56.0									1.08
58.0									1.10
60.0									1.12
62.0									1.14
66.0									1.16
↓									1.16
74.0									1.16
									↓
90.0	↓	↓	↓	↓	↓	↓	↓	↓	1.00
92.0									.97

(7.3.5.2 continued)

From above results it is seen that pile No. 182 exhibits U.C > 1 not in the pile head joint region but also at depth 50' to 90' below mudline also. In such a condition pile section from 0 ft to 90 ft below mudline becomes unstable and fails.

From p. 231 of WD103US.OT2 output pile depths 15 ft and 51 ft it can be seen the U.C = 1 is reached at about
Pile axial Load : 4200 K and
Pile bending Moment : 65000 K-in.

In order to analyze platform Ultimate Strength a push over type analysis shall be performed only for 45° direction on a model & Loading just like Ultimate Strength Analysis case except pile 182 be removed and Joint 182 be Loaded by following reactions.



Forces (WD103US.OT2 p-256)
(SECT 8.2.2 Book 3 of 4)

$F_x = -567.715$
$F_y = -555.258$
$F_z = +4141.899$

moments are calculated by following approximation using p. 256 values $M_x = 85520$ $M_y = -93062$ $(M_x^2 + M_y^2)^{1/2} = 126389$

use $M_x = -\left(\frac{65000}{126389}\right) \times 85520 = -43982.$

$M_y = +\left(\frac{65000}{126389}\right) \times 93062 = +47860.$

$M_z = \longrightarrow = +754.$

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(7.3.5.2 ... Continued.)

7.3.5.2.5 Max Base Shear at Pile Head Joints (WD103US.OT2 see Sect 8.2.2 Book 3 OF 4)

794 K @ JNT 182 LC: 6 (45° direction)

Comb. Load Case	WIND/WAVE Direction	JOINT #	PILE AXIAL LD Kips (p. 234 LOCAL AXIS)	GLOBAL SHEAR FORCES X (p. 256, 257, 258)	Y	GLOBAL RESULTANT BASE SHEAR K $(X^2+Y^2)^{1/2}$
L.C: 6	(45°)	112	2170 Tension	538	437	693
		182	-4212 Comp	568	555	794
L.C: 7	(67.5°)	112	1879 Tension	358	483	601
		182	-3963 Comp	469	581	747
L.C: 8	(90°)	112	1163 Tension	117	427	443
	Conductor →	200	- 375 Comp	8	315	315
		182	-3270 Comp	308	544	625

7.3.5.2.6 Max Pile Head Joint Deflections

Local Axial Defl -0.6746" @ JNT 112 LC. 6 (TENSION)
1.5030" " 182 " (COMPRESSION)
Global Max Sideway 33.807" JNT 182 LC. 6 (WD103US, OT3) p. 8, 13 OUTPUT

* WD103US.OT2 pages	Comb. Load Case	WIND/WAVE Direction	JOINT #	* LOCAL AXIS PILE HD AXIAL DEFL (IN)	GLOBAL AXES PILE HEAD JOINT Z	X	Y	DEFLECT(IN) $(x^2+y^2)^{1/2}$
16	L.C: 6	(45°)	112 max pull	- .6746	-3.918	27.269	18.728	33.081
65			182 max down	1.5030	3.256	25.169	22.571	33.807
82	L.C: 7	(67.5°)	112 max pull	- .5716	-3.299	14.279	24.484	28.343
131			182 max down	1.4084	2.632	12.010	28.530	30.955
152	L.C: 8	(90°)	112 max pull	- .3333	-2.340	0.233	26.536	26.537
201			182 max down	1.1456	1.658	-1.787	29.939	29.992

(7.3.5.2 continued)

7.3.5.2.7 Pile Results Conclusion

a) All pile unity checks except conductors exceed unity. U.C > 1

b) This mean all piles form plastic hinge at pile head joint when U.C = 1 (see 7.3.6.2.)

c) In critical Load Case L.C. 6 U.C > 1 continues below pile head up to 8 ft below mudline for all piles (see 7.3.6.4)

d) For Pile 182 : L.C. > 1 plastic hinge formation occures in depths (0 to 14 ft) and (50 to 90 ft) causing a total pile failure.

e) An improved Ultimate Strength Analysis with push over concept is required to consider total failure of pile 182 mentioned in paragraph d) before it occures (see suggested approach in Section 7.3.6.4).

f) Max Base Shear (Global Coord.) 794 K (Sect. 7.3.6.5)

g) Max Pile Head Deflections : (Section 7.3.6.6)

Local Axial (in Tension) : −0.6746"
Local Axial (in Compression): 1.5030"

Global Lateral (due to Shear) : 33.807"

(7.3 Ultimate Strength Continued)

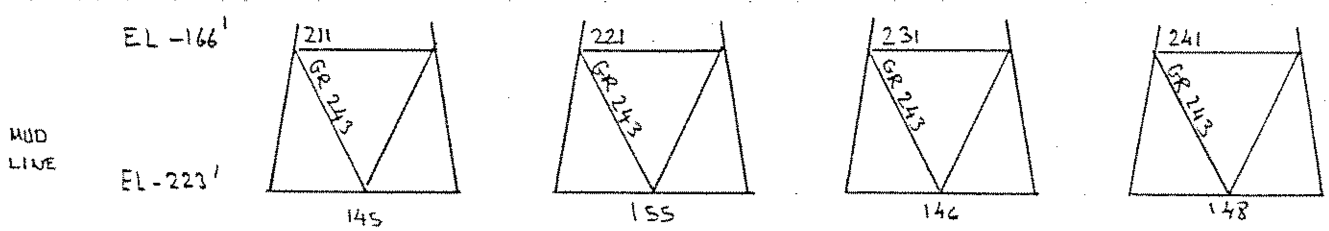
7.3.6 Jacket Member Results

Book 3 of 4
Section 8.2.3

Failing Jacket members are Listed in WD103US.OT3 output p. 83 and 84 under "Member Group Summary Report" Group I Unity Check > 1.33 and Group II Unity Check > 1.0

These members are displayed below:

	Truss Plane 1	Truss Plane 2	Truss Plane 3	Truss Plane 4



		Plane 1	Plane 2	Plane 3	Plane 4
	L.C →	8	7	8	8
	U.C →	1.768	215.791 *	142.681 *	2.814
Report Page # 30	fa/Fa	.992	1.162	1.260	1.065
	fby/Fby	.768	209.947	100.000	1.723
	fbz/Fbz	.115	44.591	100.000	.304
83	fa	-25.63 KSI	-30.11 KSI	-32.66	-27.51
	fby	6.55 "	-7.46 "	-2.43	-8.28
	fbz	.98 "	1.58 "	-1.43	-1.46

All buckling failure

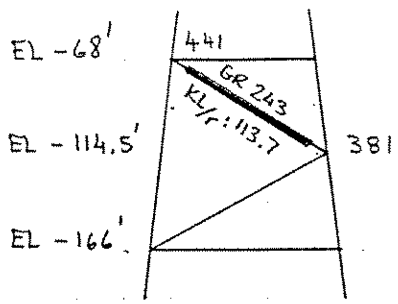
* 4.6 (Manual) Sect 4.6-4 (Compression + bending)

(7.3.6 Continued)

Similar Buckling Failure occures for Jacket K-Brace Diagonals at higher elevations

Truss Plane 4 U.C : 2.722 > 1 L.C: 8 90° direction

WD103US.OT3 p. 83 WD103US.OT3 p. 30

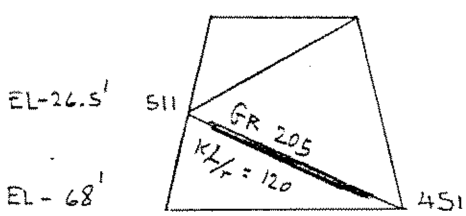


U.C Components

fa =	- 21.37	KSI	0.961	0.961
fby =	- 6.11	"	1.387 } SRSS →	
fbz =	- 4.79	"	1.087	1.762
				2.723 > 1

Truss Plane 1 U.C : 1.221 > 1 L.C: 7 67.5° direction

WD103US.OT3 p. 84 WD103US.OT3 p. 29



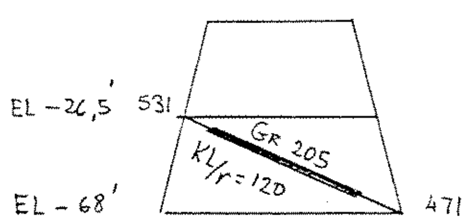
U.C Components

fa =	-14.86	KSI	0.725	0.725
fby =	- 4.01	"	0.225 } SRSS →	
fbz =	7.87	"	0.442	0.496
				1.221

Truss Plane 3 U.C : 1.284 > 1 L.C: 8 90° direction

WD103US.OT3 p. 84 WD103US.OT3 p. 30

fa =	-15.99	KSI	0.778	0.778
fby =	- 6.83	"	0.471 } SRSS →	
fbz =	- 2.65	"	0.183	0.505
				1.283



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PROJECT NO. 1836M01.
SHELL WEST DELTA BLK 103 A
PLATFORM ASSESSMENT
BY MEC DATE
CHK. DATE
SHEET 115 OF

(7.3.6 Continued)

SHELL 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

f_{by} = 6.92 — .137

f_{bz} = −1.74 — .035

KL/r = 26.7, U.C. 1.158

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

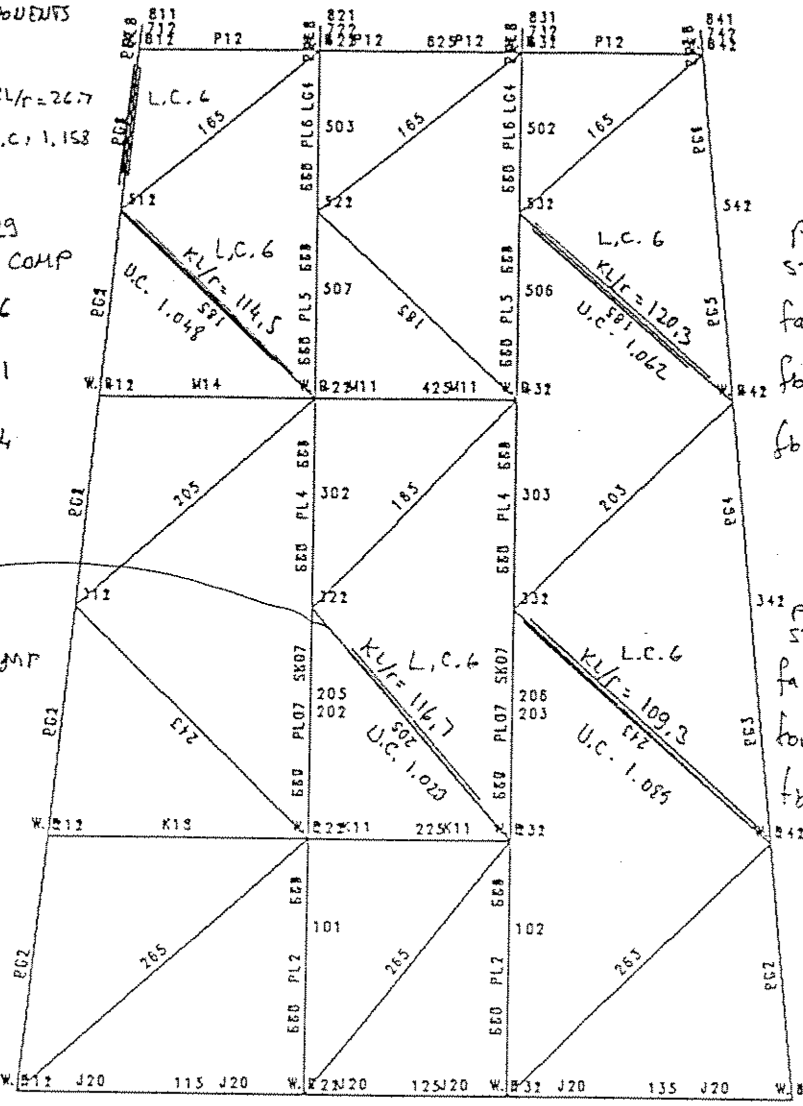
f_{bz} = −5.32 — .281

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)

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PLATFORM ASSESSMENT

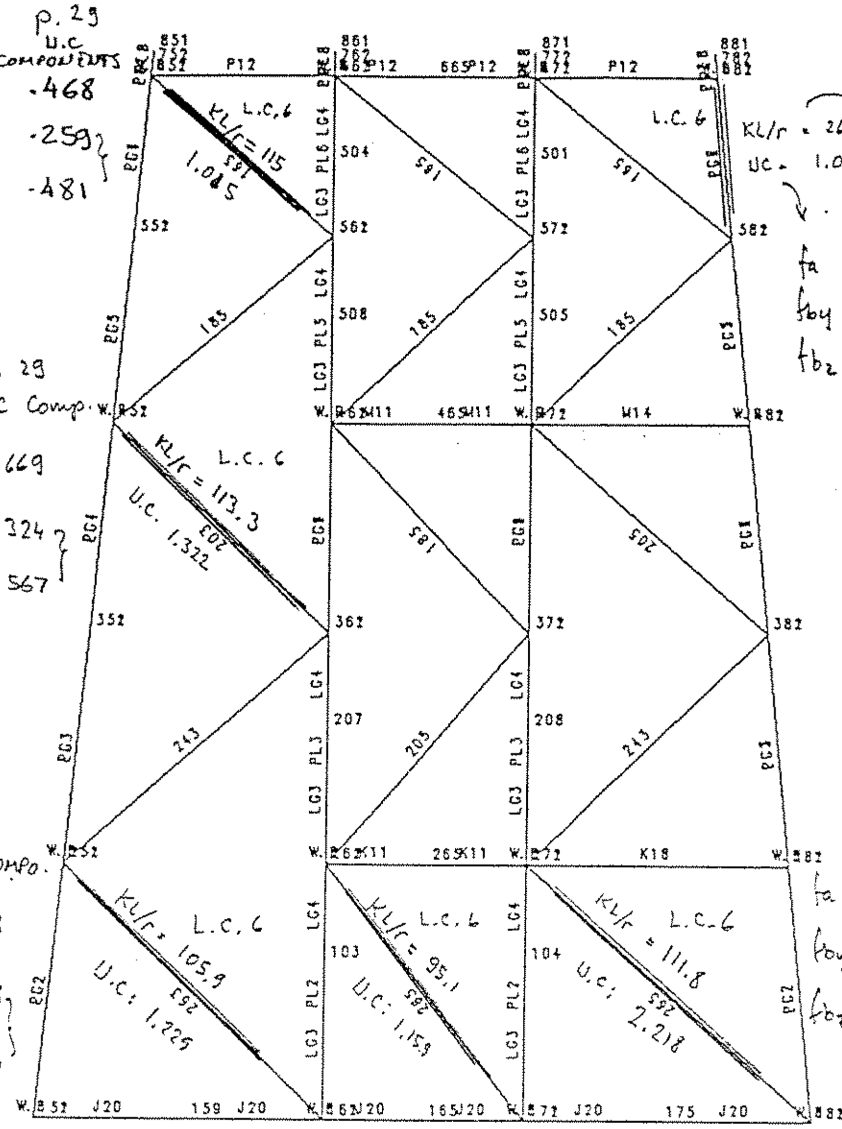
PROJECT NO. 1830M01
BY MEC DATE 7/23/96
CHK. ______ DATE ______
SHEET 116 OF ______

(7.3.6 Continued)

SHELL WEST DELTA BLK 103A 223 FT WATERS

BROADSIDE VIEW ROW B

NOTE: STRESS & U.C COMPONENTS FROM WD103US.OT3 output



Upper left member — OUTPUT p.84 STRESS / p.29 U.C COMPONENTS:

STRESS	U.C
fa = −10.28 KSI	.468
fby = −8.97 KSI	.259
fbz = −16.64 KSI	.481

Upper right member: KL/r = 26.7, UC = 1.053

p.36 U.C Components: Axial .924; Y-Axis .119; Z-Axis .035

p.84: fa = 40.12 KSI; fby = −5.61 "; fbz = 1.64 "

Middle member — p.84 STRESS / p.29 U.C Comp.:

STRESS	U.C
fa = −15.00 KSI	.669
fby = −7.06 "	.324
fbz = −12.34 "	.567

Lower left member — p.84 STRESS / p.31 U.C. COMPO.:

STRESS	U.C
fa = −19.30 KSI	.798
fby = −4.64 "	.288
fbz = −5.16 "	.320

Lower right member — p.83 STRESS / p.31 U.C. COMP.:

STRESS	U.C
fa = −21.84	.956
fby = −5.60	1.059
fbz = −3.63	.686

Lower middle member — P.84 STRESS / P.31 U.C COMPONENTS:

STRESS	U.C	
fa = −22.56 KSI	0.830	0.830
fby = −6.26 "	0.320	0.330
fbz = −1.54 "	0.079	
		1.160

FAILURES

LEG MEMBER ; INELASTIC BUCKLING

TRUSS DIAGONALS; BUCKLING + BENDING

(7.3. continued)

7.3.7 Joint Can Analysis Results :

(See Section 8.2.4 WD103US.OT4)

Pertinent joint can analysis information are reported in joint can Summary in WD103US.OT4 p.84 thru 87 as followed:

a) Unity check for punching shear due to actual brace Loading
b) Unity Check for punching shear due to 50% Brace Strength
d) Chord and Brace member size and material yield strength
e) the Loading case #.
f) Common, Chord and Brace joint #.

If any Unity Check value is greater than 1, then the punching shear strength of the joint is not adequate. In Ultimate Strength Analysis unity check U.C = 1 represents shear yield failure with no factor of safety.

7.3.7.1 Failing Joint Cans
A joint can is declared failed when U.C ≥ 1 occured due to actual brace Loading.

a) Following Leg Joints @ EL -223' (mudline) exhibits U.C >1

Joint No: 121, 131, 141, 151, 161, 171 Load Case 6

Joint No: 111 Load Case 8

(7.4.5 continued)

b) Further Leg Joints with U.C > 1

@ EL −166'	Joint No 241	Load Case: 6
@ EL −91.25'	Joint No 302	Load Case: 8
"	Joint No 303	" ": 8
@ EL −9.667'	Joint No 502	Load Case: 7
"	Joint No 503	" ": 7

7.3.7.2 Stresses for Failing Joint Cans

Selective pages of WD103US.OT4 file are printed (see Section 8.2.4) for Joint Cans with U.C > 1 which contain following informations:

a) Common, Chord and Brace Joint #
b) Chord member sizes and Fy
c) Computed Alpha to Classify Joints
   Alpha = 1 for perfect "K" joints
   Alpha = 1.7 " " "T" "
   Alpha = 2.4 " " "X" "
   } see Ref 4.6 Sect. 4.6.2
d) Gap
e) Brace Member size
f) Chord / Brace Angle
g) Load Case
h) Actual Stresses Chord fa + fb
   Brace fa
   " out of plane fb
   " in plane fb
i) Punching Shear Allowable Stresses for Fa and Bending
j) Unity Check

7.4 PUSHOVER STRENGTH ANALYSIS I (45° Direction only)

Ultimate Strength Analysis determined the weakest Link in this platform is the foundations; in specific the pile No: 182 during Loadcase L.C. 6.

The Ultimate Strength Analysis did not pass because of overloaded Local members and joints as well as overloaded foundation members.

The intend of this analysis is to recognize the remaining strength of other piles and Load distribution capacity of the structure just before pile 182 reaches its collaps Load.

The model and Loading used for push over strength analysis is almost same as in Ultimate Strength analysis except following changes;

a) Pile Head definition (222 222) at joint 182 is removed.

b) The pile head reactions at joint 182 (as determined in Section 7.3.6.4) are applied as Load Case 4 global joint Loads.

c) Pushover Strength analysis is performed only for the critical direction 45° found in the Ultimate Strength Analysis.

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PLATFORM ASSESSMENT

PROJECT NO. 1830 M01
BY MEC DATE 7/27/94
CHK. ______ DATE ______
SHEET 120 OF ______

(7.4.3.2 Continued)

7.4.3.2.3 CHECK U.C > 1 Along Pile Depth

Unity Check for stations along each pile are reported in "Pile Critical Load Case Report" Section 8.3.2 p.62 thru 84 in Book 4 of 4.

All U.C > 1 values are shown below

Page No of Report →	62	64	67	70	73	76	79	82
Distance Along Pile Feet	CONDUCTOR	PILE			JOINTS			
	200	112	122	132	142	152	162	172
.00	.74	1.24	1.09	1.15	1.27	1.05	1.31	1.30
2.00	↓	1.16	1.02	1.08	1.19	.98	1.25	1.23
4.00	↓	1.09	.95	1.01	1.12	↓	1.18	1.16
6.00	↓	1.02	↓	.93	1.05	↓	1.11	1.09
8.00	↓	0.95	↓	↓	.98	↓	1.05	1.03
	↓	↓	↓	↓	↓	↓	.98	.96
	U.C<1	U.C<1	U.C<1	U.C<1	U.C<1	U.C<1	U.C<1	U.C<1

Pile failures U.C>1 are all in the pile head region.
Plastic hinges form in the pile head region.
Foundation of the platform is stabil; i.e no secondary plastic hinge along the piles.

(7.4. Continued)

7.4.1 PUSHOVER STRENGTH ANALYSIS I RESULTS

7.4.1.1. Soil Failure

None (see Section 7.4.3.1)

7.4.1.2 Pile Failure

All piles fail in the pile head region forming full plastic hinge. (See Sect. 7.4.3.2.3)
These failures are of Local nature.
The overall stability is not violated.

7.4.1.3 Jacket Member Failure (see 7.4.4)

Several diagonals buckles in Pushover Analysis but Ultimate Strength results are more critical since 90° direction is considered there.

7.4.1.4 Leg Punching Shear (see 7.4.5)

There are 7 joint can failures with U.C>1 under actual brace Load condition.

(7.4 continued)

7.4.2 Input Data Summary
(for input echo refere to Section 8.3.1 in Book 4 of 4)

7.4.2.1 Loading Cases

Basic Loads:

1) DL + BUOYANCY
2) Deck Loads
3) 45° Metoceanic Loads; Wind: 70 Knots
   Wave: 62,4 FT/12.5 SEC.
   Current: 1,8 Knots
4) Reaction of pile No 182 before collaps in Ultimate Strength Analysis applied as global joint Load at Joint No: 182

Combined Loads:

5) 100% of (LC: 1, 2, 3 AND 4)

7.4.2.2 Allowable increase: AMOD 2.0 on L.C. 5 for Deflections and Stress
AMOD 1.75 for Joint Can Punching Shear

7.4.2.3 Applied Load Summary: (See WDPUSH.OT1 output p. 76

Combined Loads	Vertical Z KIPS	Horiz. X KIPS	Horiz. Y KIPS	Resultant $(x^2+y^2)^{1/2}$
L.C: 5	-4092	2341	2308	
subtract Jnt. 182 Loading (L.C: 4	-4142	568	555)	
	-8234	2909	2863	4082
this compares with Ultimate Strength L.C: 6	-8234	2908	2864	4082

(7.4. continued)

7.4.3 Soil and Pile Results (See Section 8.3.2 in Book 4 of 4)

7.4.3.1 Soil Results

Max Pile axial forces calculated are;

a) Tension - 2145 K @ JNT 112 L.C.5 (45° direction)
(see Section 8.3.2 p.85 WDPUSH.OTZ)

b) Compression - 2557 K @ JNT 162 L.C.5 (45° direction)
(see Section 8.3.2 p.85)

These values compared to the ultimate pile axial capacities in skin friction and bearing are;

for tension $2145^{K} < 4546$ K

for compression $2557^{K} < 4617$ K

Warning messages encountered in "Pile Head Forces and Displacement in pile coordinates during iterations" (Section 8.3.2 in Book 4 of 4 p. 3 thru 8) as "Warning: Axial Deflections Are off T-Z Curves Pile Joint 162" are of Local nature along the pile and do not represent a soil failure as long as the calculated axial pile Load does not exceed ultimate pile capacity.

[The max pile forces are even less than pile allowables

for tension $2145^{K} < 3035^{K}$ } see Section
for compression $2557 < 3078^{K}$ } 7.1.3.7]

There is no SOIL FAILURE in axial direction.

(7.4 Continued)

7.4.3.2 Pile Results (See Section 8.3.2 in Book 4of4)

Pile Failure

7.4.3.2.1 Max Axial Pile Force (see also 7.4.3.1)

a) Tension → 2145 K @ Joint 112 L.C.5 (45° Direct.)

b) Compression → -2557 K @ Joint 162 L.C.5 (45° Direct.)

7.4.3.2.2 Max Pile Stress Unity Checks

For all piles except conductors
U.C > 1

see Section 8.3.2 p. 85
Pile Head Unity Check Report

U.C	Gr. ID	L.C.	Pile Head Joint#	fa KSI	fb KSI	Pile Shear Force K
0.74 < 1	P33	5	200	-.44	39.42	352
1.24 > 1	P42	↓	112	9.20	54.58	396
1.09 > 1	↓	↓	122	1.12	57.40	350
1.15 > 1	↓	↓	132	3.08	58.38	368
1.27 > 1	↓	↓	142	-6.22	60.58	308
1.05 > 1	↓	↓	152	-2.33	53.55	307
1.31 > 1	↓	↓	162	-11.56	56.51	254
1.30 > 1	P42	5	172	-10.39	57.07	265

All pile head joints fail forming plastic hinge.

(7.4.32 Continued)

7.4.3.2.4. Max Base Shear at Pile Head Joints
(see Section 8.3.2 Book 4 of 4 for refered pages)

LOAD CONDITION	PILE HEAD JOINT #		p.104 Local PILE AXIAL KIPS	p.103 Global SHEAR X	Global SHEAR Y	Global Resultant BASE SHEAR KIPS $(x^2+y^2)^{1/2}$
L.C. 5 (45°)	112	max Pull	-2146 K	536	435	690 K
	162	max Comp	+2557 K	197	416	460 K
(see 7.3.5.2)	182	max Comp	+4200 K	568	555	794 K

7.4.3.2.5 Max Pile Head Joint Deflections
(See Section 8.3.3 Book 4 of 4 for Refered pages)
max Local Axial -.6658" (Tension) 1.5030" (Compression)
max Global Sidesway 33.948"

(see Section 8.3.2 p. *) p. 4, 6, 7

	LOAD CONDITION	PILE HEAD JOINT #		Local Pile Hd Axial Deflect IN	Global Axes Z IN	X IN	Y IN	Deflection $(x^2+y^2)^{1/2}$ IN
* p.13	L.C.5 (45°)	112	max Pull	-.6658	-3.939	27.299	18.814	33.154
p.48		162	max down	.8718	1.191	25,386	20,671	32.737
	(See 7.3.5.2.6)	182	max down	1.5030	3,337	25,259	22,682	33.948

(7.4 Continued)

7.4.4. Jacket Member Results (Section 8.3.3 Book 4 of 4)

Push over analysis I is performed only in 45° direction combined Loads with the intent that platform overall stability is intact.

Failing Local members of Pushover analysis with U.C. > 1 are Listed in WDPUSH. OT3 output in Book 4 of 4 p. 71 and 72.

Ultimate strength analysis Load case 8 in 90° direction produce more critical Jacket member failure (see section 7.3.6.

7.4.5 Joint Can Analysis Results (Section 8.3.4 Book 4 of 4)

For failing joint cans with U.C > 1 under actual brace Load refer to p. 64, 65 in Section 8.3.4 WDPUSH.OT4 in Book 4 of 4. Failing cans are at Leg Member Joints with Joint No: 121, 131, 141, 151, 161, 171 at EL-223' mudline Level

and

Joint No: 303 at -31.25'

Ultimate Strength Analysis failures are more critical see Section 7.3.7.1

7.5. PUSHOVER STRENGTH ANALYSIS II (90° Direction only)

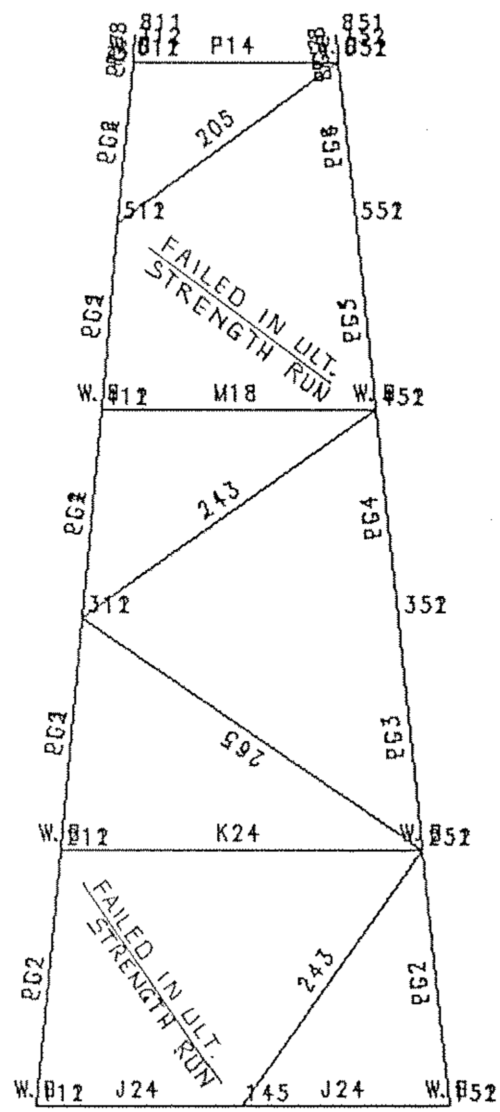
Pushover Strength Analysis I demonstrated that before Pile 182 failed deflection and Load distribution took place thru jacket other piles and overall foundation failure was not reached.
But just as in Ultimate strength analysis the Local diagonal members and joints still failed.

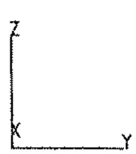
In order to establish the reserve strength of the jacket when jacket diagonals failed the 90° direction Ultimate Strength case is repeated as Pushover Strength Analysis II after removing the failing diagonals totally from the model. Revised Truss Planes Row 1 thru 4 is on following pages.

If the pushover analysis II results show that the jacket Legs "do not buckle or collapse" then it will have some reserve strength.
But if they do fail or piles collapse then the platform do not pass assessment requirement of Ref 4.1 draft section 17.

PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 1





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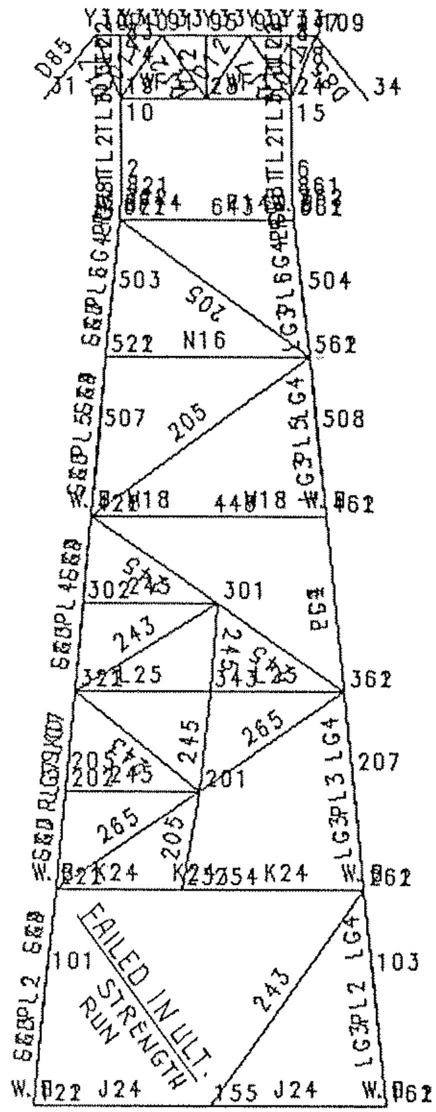
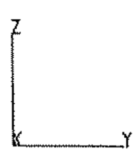
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 2





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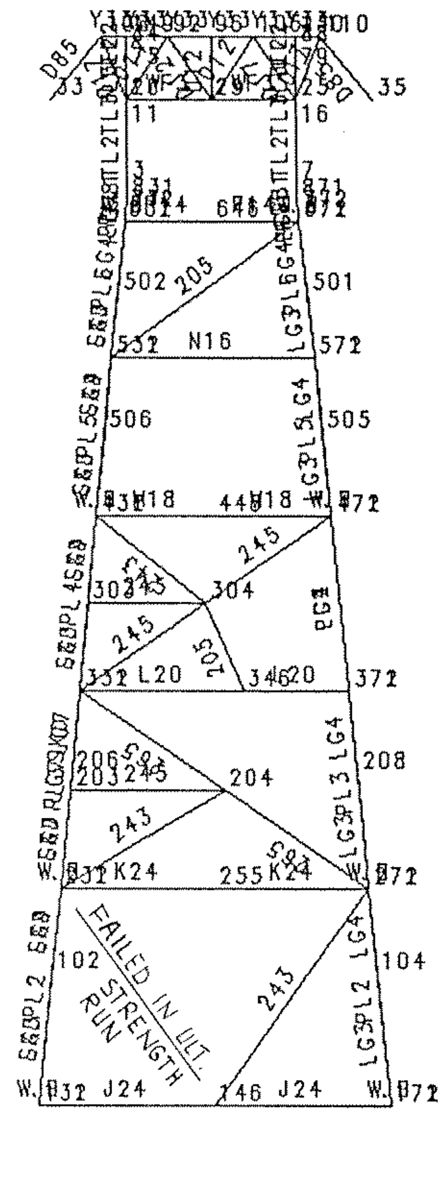
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 3



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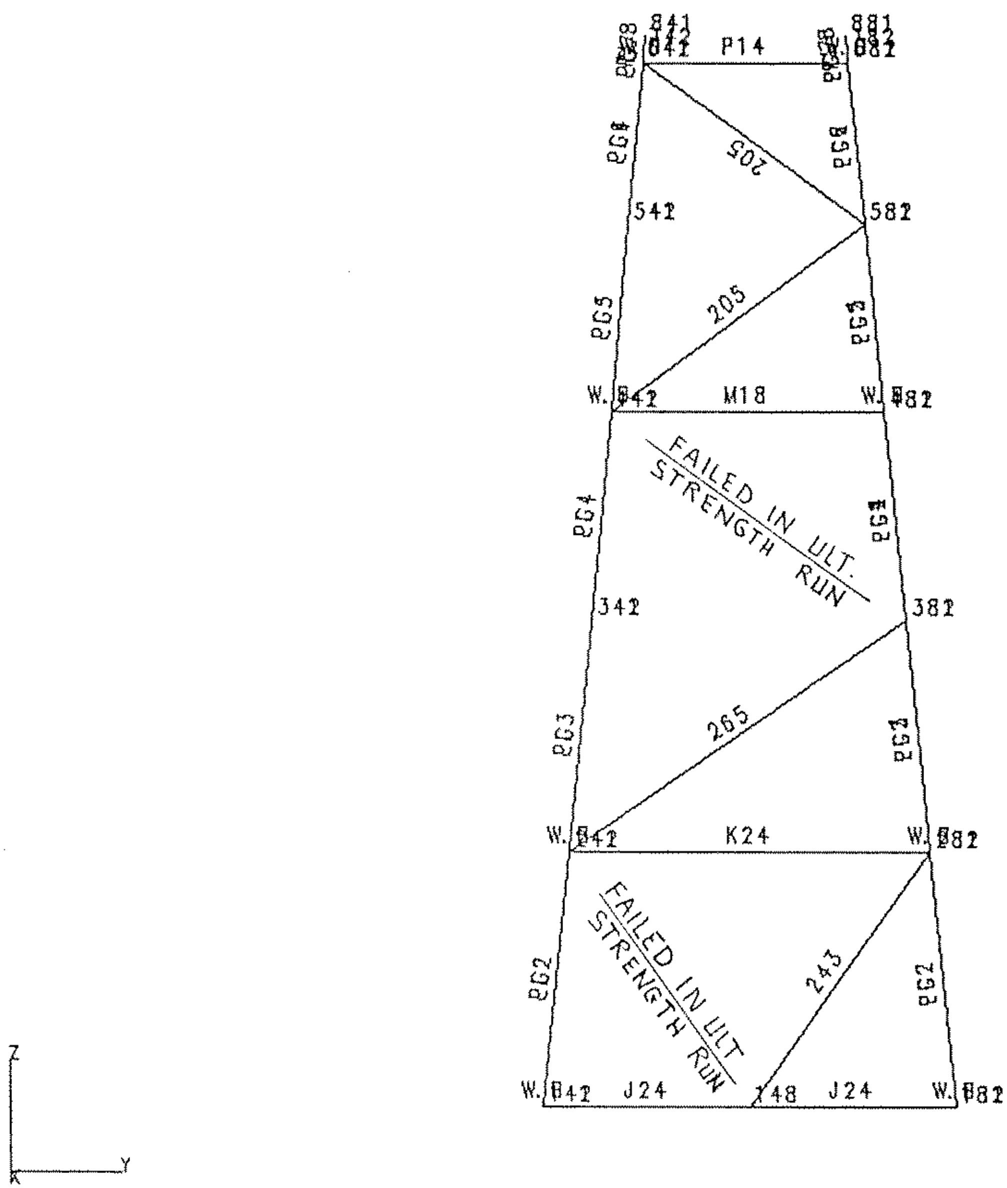
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 4



SHEET 131

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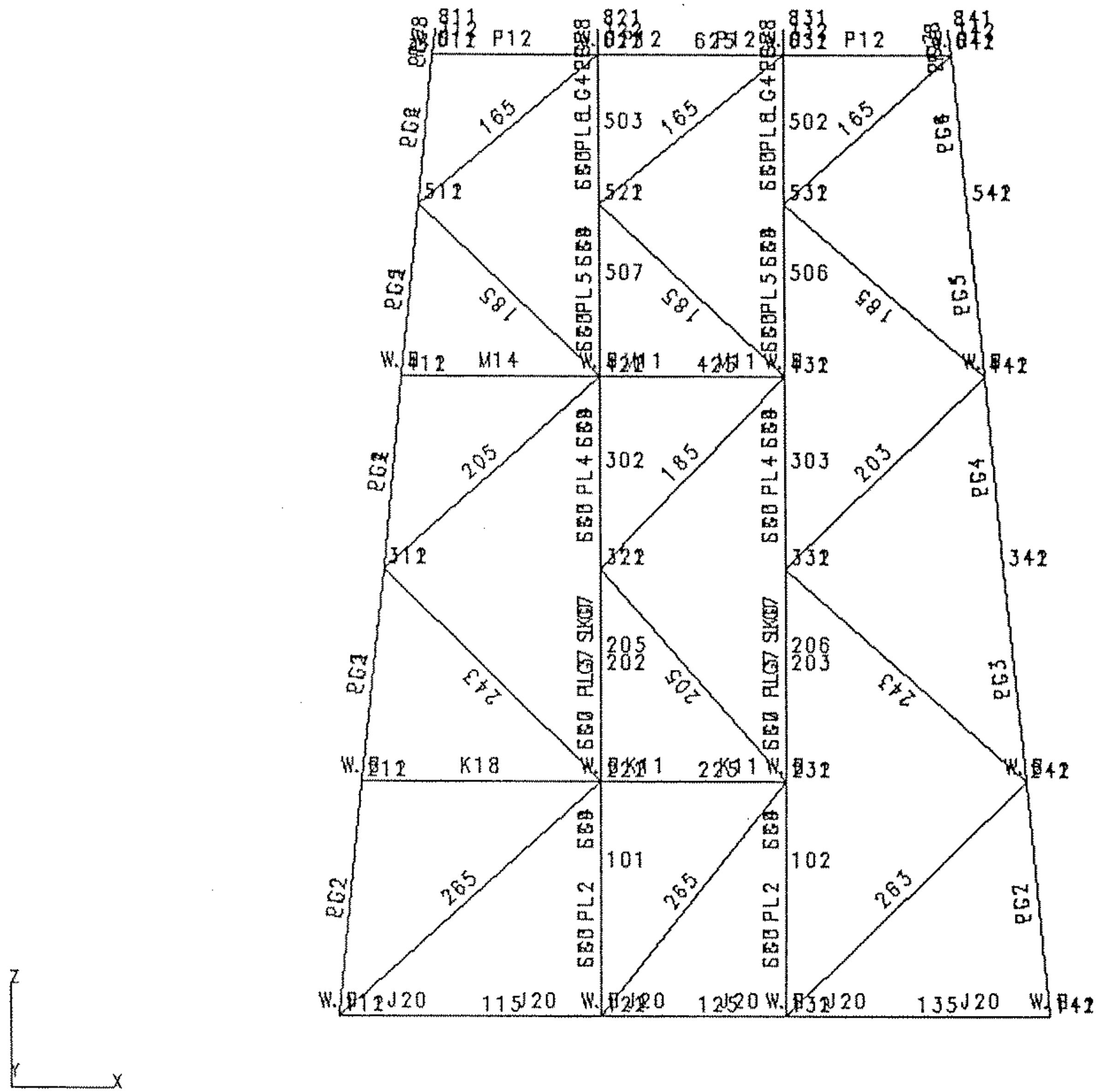
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET BROADSIDE VIEW ROW A



SHEET 132

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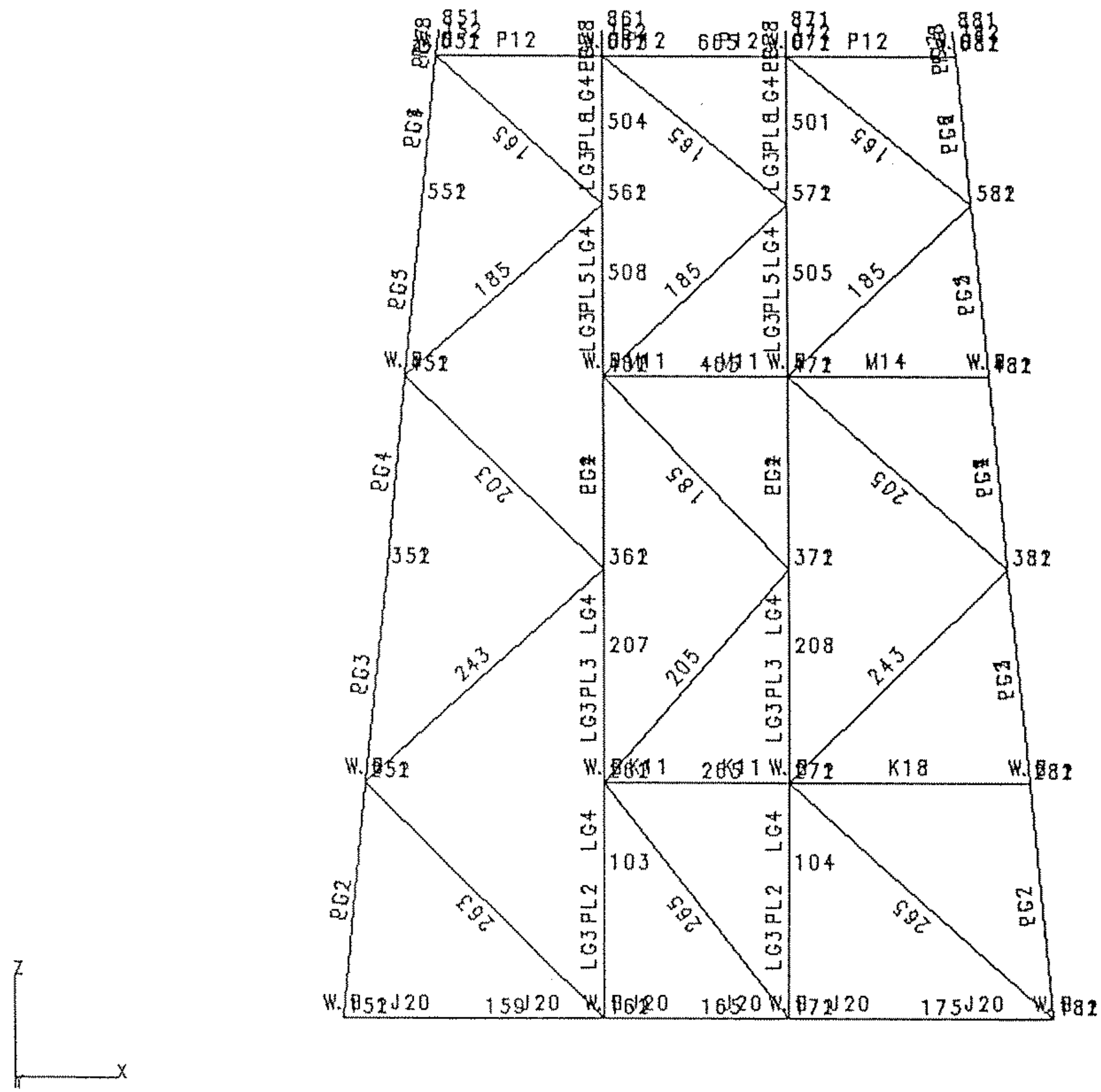
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET BROADSIDE VIEW ROW B



7.5.1 PUSHOVER STRENGTH ANALYSIS II RESULTS

7.5.1.1. Soil Failure

There is no soil failure in skin friction and end bearing (see 7.5.3.1)

7.5.1.2 Pile Failure

Piles 162, 172, 182 Collapse with double plastic hinge formation (see 7.5.3.2.3)

7.5.1.3 Jacket Member Failure

Jacket collapses due to failure of Legs, pile inside Leg and diagonals in jacket planes at row 1, 2, 3 and 4.

Jacket horizontal plane truss at mud Line Level also greatly fails.

7.5.1.4 Joint Can Failure

There are 63 joint can failures with U.C. > 1 under actual brace Load conditions

7.5.2 Input Data Summary

For Input Echo refere to Section 8.4.1 in Book 4 of 4

7.5.2.1 Loading Cases

Basic Loads:

1) D L + BUOYANCY
2) Deck Loads
3) 90° Metoceanic Loads: Wind: 70 Knots
   Wave: 62.4 FT/12.5 SEC.
   Current: 1.8 Knots

Combined Loads:

4) 100% of (LC: 1, 2, 3)

7.5.2.2 Applied Load Summary; (See WDCOLLP. OT1 p. 76

		(Wet Gravity) Vertical Z KIPS	Horiz. X KIPS	Horiz Y KIPS	Resultant $(X^2+Y^2)^{1/2}$
Combined Loads					
90°	L.C. 4	−8338	57	4029	4029
Ultimate Str. 90° (see 7.3.4.3)	as compared to same Loading in L.C. 8	−8252	58	4072	4072
	Difference	−86.	−1	−53	−53

due to removal of diagonals.

7.5.3 Soil and Pile Results (see Section 8.4.2 in Book 4 of 4

7.5.3.1 Soil Results

Calculated Max Pile Axial forces do not exceed the Ultimate Pile Axial capacities. Therefore there is no Soil failure in Skin friction and End Bearing.

Calculated max Pile Axial force (see Section 8.4.2 p.118 in Book 4 of 4)
For the Ultimate Pile Axial Capacities See Sect. 7.1.3.7

	Joint #	Calculated		Ultimate
max Tension @	122	1104 K	<	4546 K
max Compression @	182	3151 K	<	4617 K

Warning messages encountered in "Pile Head Forces and Displacement in pile coordinates during iterations" (see Section 8.4.2 Book 4 of 4 p. 3 thru 11) as "Warning: Axial Deflections Are Off T-Z Curves Pile Joint Number" are of Local nature along the pile and do not represent a soil failure as long as the calculated axial pile Load does not exceed the Ultimate Pile capacity.

(7.5 continued)

7.5.3.2 Pile Results (See Section 8.4.2 Book 4 of 4)

Pile failure due to collapse of pile No: 162, 172, 182

7.5.3.2.1 Max Axial Pile Force (see also 7.5.3.1)

a) Tension 1104 K @ Joint 122 L.C.4 (50°)

b) Compression 3151 K @ Joint 182 L.C.4 (90°)

7.5.3.2.2 Max Pile Stress Unity Checks (see 8.4.2 p. 99 p. 100 Book 4 of 4)

U.C. > 1 (Means full plastic hinge forms)

	U.C	Group ID	L.C.	Pile Head Joint #	fa KSI	fb KSI	Pile Shear Force K
at critical pile sections	1.10 > 1	P42	4	152	-11.17	45.21	72
	1.18 > 1	↓	↓	162	-12.97	47.73	88
	1.23 > 1	↓	↓	172	-13.65	49.21	97
	1.25 > 1	↓	↓	182	-13.91	50.22	99
at pile heads	1.06 > 1	P42	4	162	-13.29	40.86	195
	1.09 > 1			172	-13.99	41.36	190
	1.13 > 1			182	-14.24	43.29	192

Pile Head Joints (162, 172, 182) fail forming plastic hinges.

(7,5.3.2 Continued)

7,5.3.2,3 CHECK U.C>1 Along Pile Depth

As seen in Section 7,5,3.2.2 U.C.>1 occures not only in pile head but also at a critical section other than the pile head of the same pile.
This indicate double hinge formation and collapse of these piles.
Spread of the U.C>1 along the pile Length is shown below. This data is reported in Section 8.3.2 Book 4 of 4 pages are indicated below.

Report page number	46	53	60	67
Distance Along Pile (Feet)	PILE JOINTS			
	152	162	172	182
0,00	.91	1,06	1,09	1,13
2.00		.99	1,02	1,06
4.00			.95	.98
	.30			
22,00	.35	.34	.36	.36
42,00	.90		.98	.99
44.00	.94	.99	1.03	1.04
46,00	.97	1,02	1,07	1.08
48,00	1,00			
60,00 to 68,00	1,10	1,18	1,23	1,25
78.0	1,00			
84,00	.67	1,02	1,06	1.08
86,00	.64	.99	1,02	1,05
			.99	1,01
				.98

PILE NO; 162, 172, 182 COLLAPSES WITH DOUBLE PLASTIC HINGE FORMATION !

(Note: There was no pile collapse in same direction Ultimate Strength run See Sect. 8.2.2 Book 3 of 4)

(7.5.3.2 Continued)

7.5.3.2.4 MAX BASE SHEAR at Pile Head Joints

(See Sect. 8.4.2. Book 4 of 4 for refered pages)

Load Condition	Pile Head Joint #		p. 118 Local Pile Axial KIPS	p. 117 Global Shear X	Global Shear Y	Global Resultant Base SHEAR $(x^2+y^2)^{1/2}$
LC.4	122	max Pull	-1104	8	462	462
	182	max Compr.	3151	296	503	584

7.5.3.2.5 MAX PILE HEAD JOINT DEFLECTION

(See Section 8.4.3 Book 4 of 4 for refered pages)

max Local axial defl. (Tension) (Compression)
max Global Sidesway

(See Section 8.4.2 p. *) Page: 5,7

	Load Condition	Pile Head Joint #		Local Axial Defl. @ Head	Global Axes Z (IN)	X (IN)	Y (IN)	Deflection $(x^2+y^2)^{1/2}$ (IN)
p. 24	L.C.4	122	max Pull	-.3130	-4.071	.701	43,859	43,865
p. 66		182	max Down	1.1006	3,311	2,232	46.458	46.512

7.5.4. Jacket Member Results Section 8.4.3 Book 4 of 4

Pushover Analysis II is performed only in 90° direction.

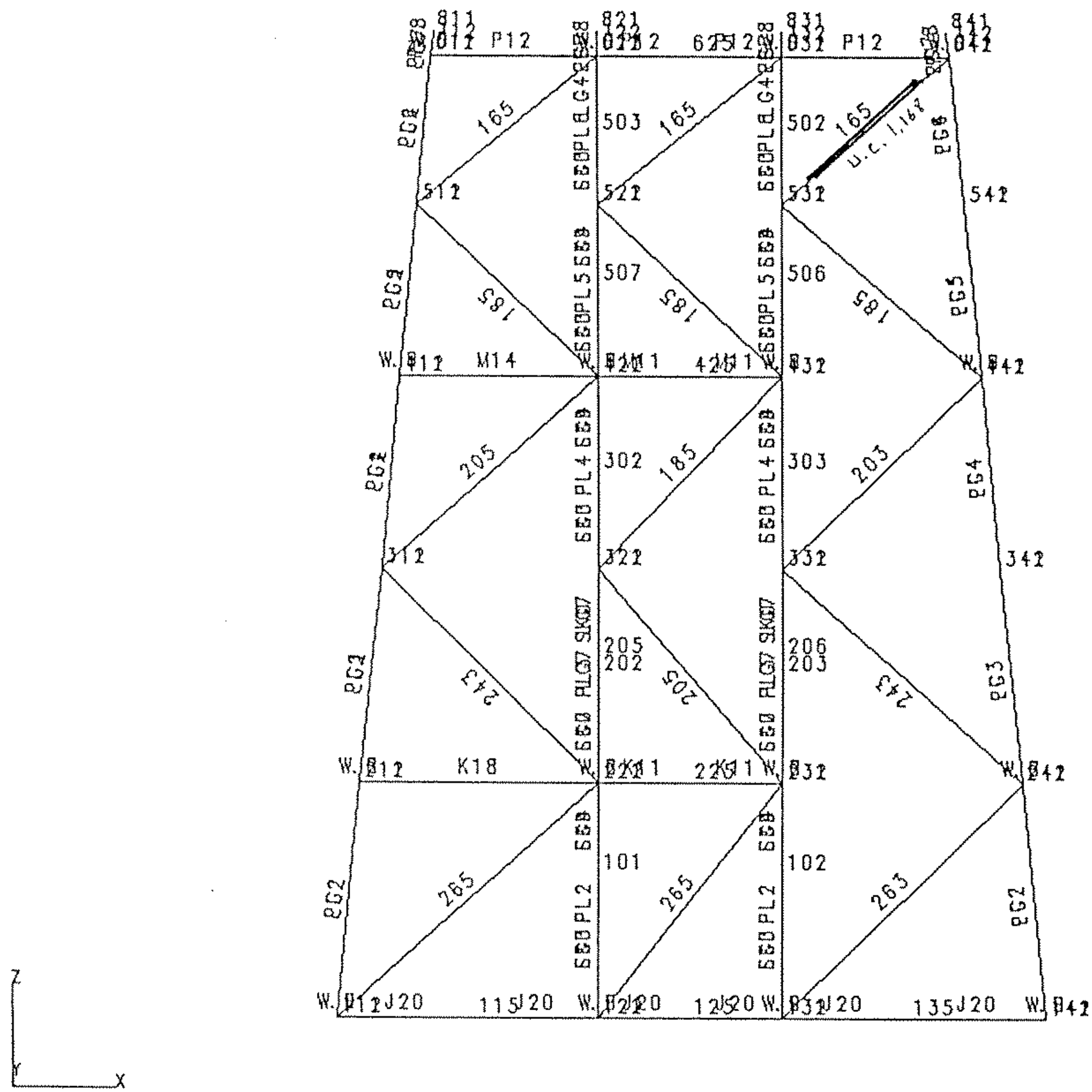
Failing 58 members U.C > 1 are Listed in WDCOLLP. OT3 output in Book 4 of 4 Section 8.4.3 p. 68 thru 72. These are also Marked with dark Lines on plots following this page with their U.C. Values. Among the failing members there are 13 Leg Members Listed below with calculated combined U.C. and Stresses:

Elevation Between ft	Member JA - JB	Group ID	Max Comb. U.C.	f_a KSI	f_{by} KSI	f_{bz} KSI	KL/r
-223' & -166'	111 - 211	LG2	1,579	-3.23	-49.69	-49,35	42.7
" "	141 - 241	"	1,395	-2,42	-47,16	40,94	"
" "	181 - 281	"	1,340	0,30	44.26	44.50	"
-166 & -114.5	211 - 311	"	1,474	-3,80	-45,62	-45,50	38.6
" "	281 - 381	"	1,376	+3.83	42,83	42,85	"
-166' & -136	261 - 207	LG3	1,550	7.38	64.73	4.59	22.3
" "	271 - 208	"	1,538	13,76	57.09	-6.22	"
-188' & -166'	101 - 221	LG4	2,759	9.66	-119.25	5,23	"
"	102 - 231	"	2,745	10,23	-119.01	-3,07	"
"	103 - 261	"	1,337	-.57	62,14	3,40	"
"	104 - 271	"	1,699	-.26	79.45	5,56	"
-166' & -140.25'	221 - 202	LG7	1,407	-3,67	-70,09	5,37	19,5
" "	231 - 203	LG7	1,561	.01	-83.10	-4,27	"

All these Leg members fail in plastic bending.

PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103 A

JACKET BROADSIDE VIEW ROW A



SHEET 141

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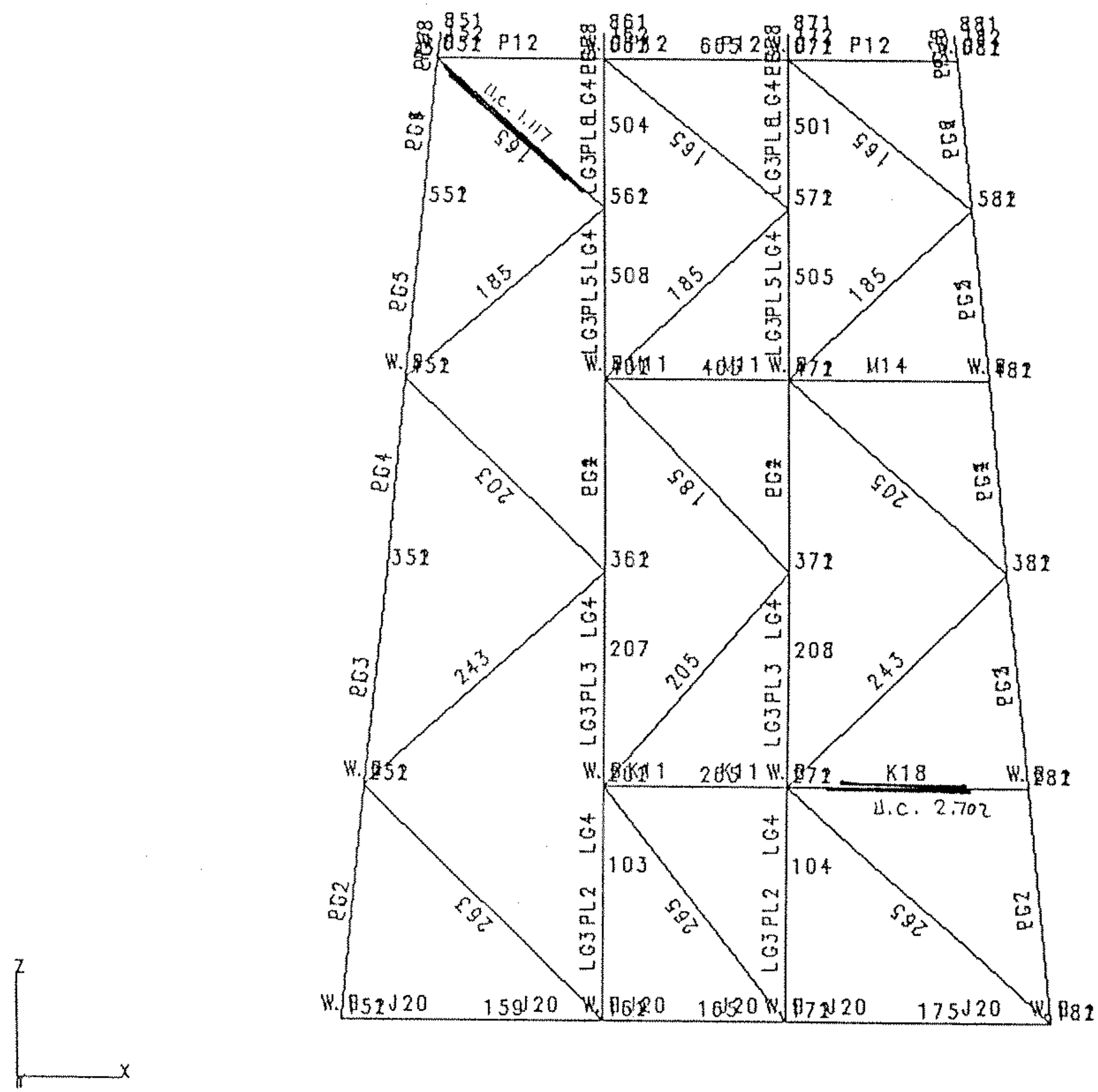
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

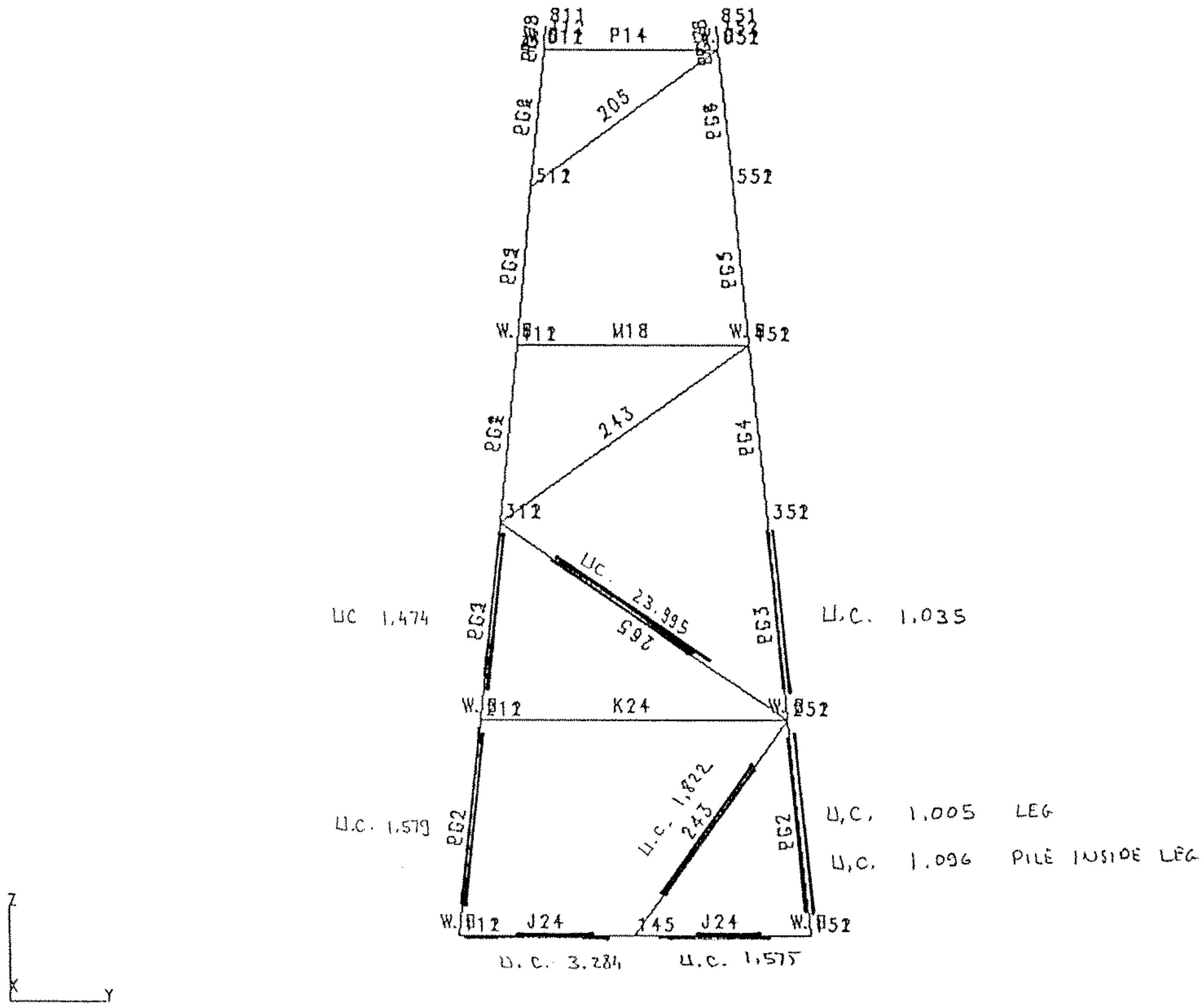
JACKET BROADSIDE VIEW ROW B



SHEET 142

PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 1



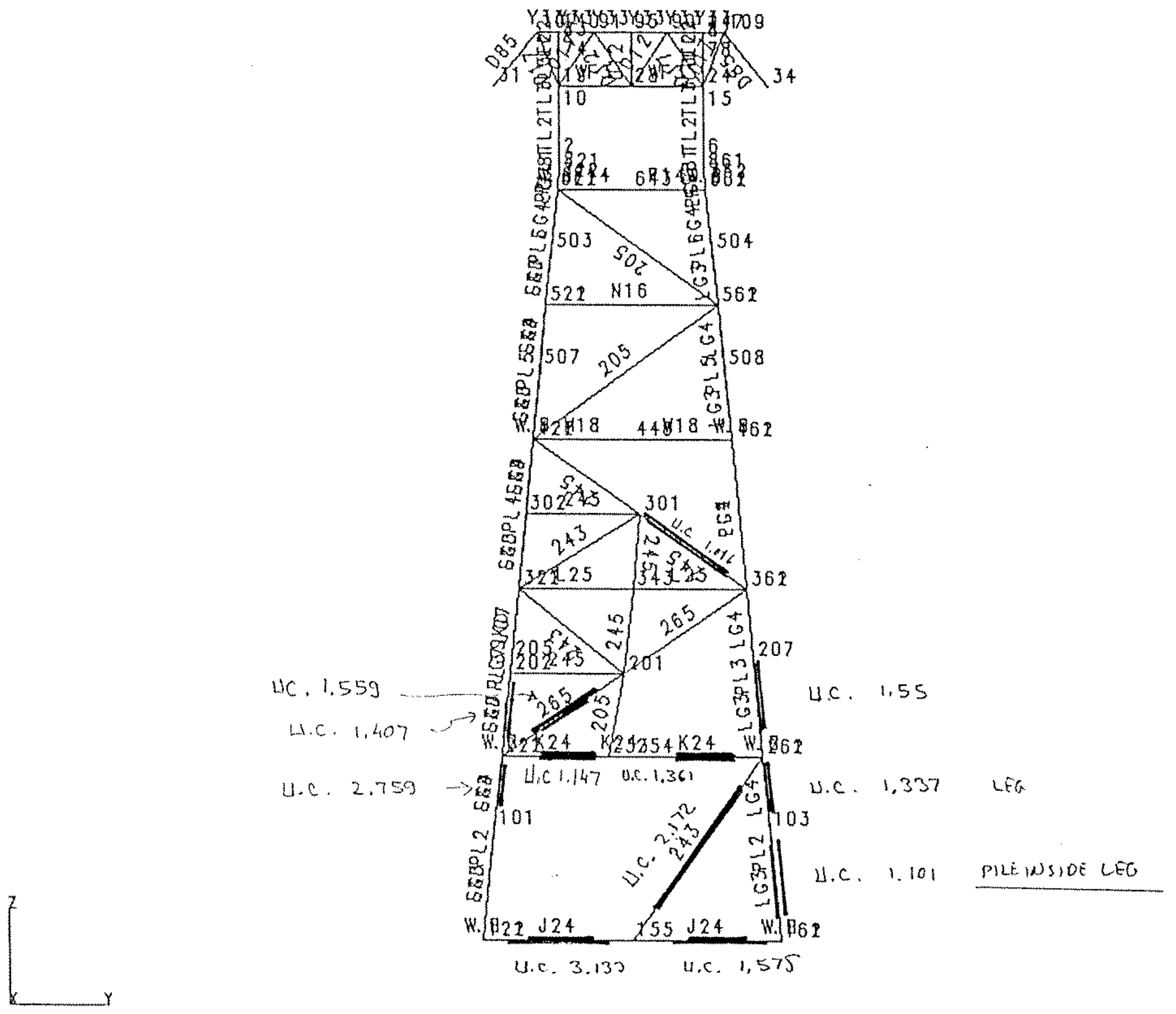
SHEET 143

STRUCTURAL SOFTWARE, INC

SHEET 144

PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103 A

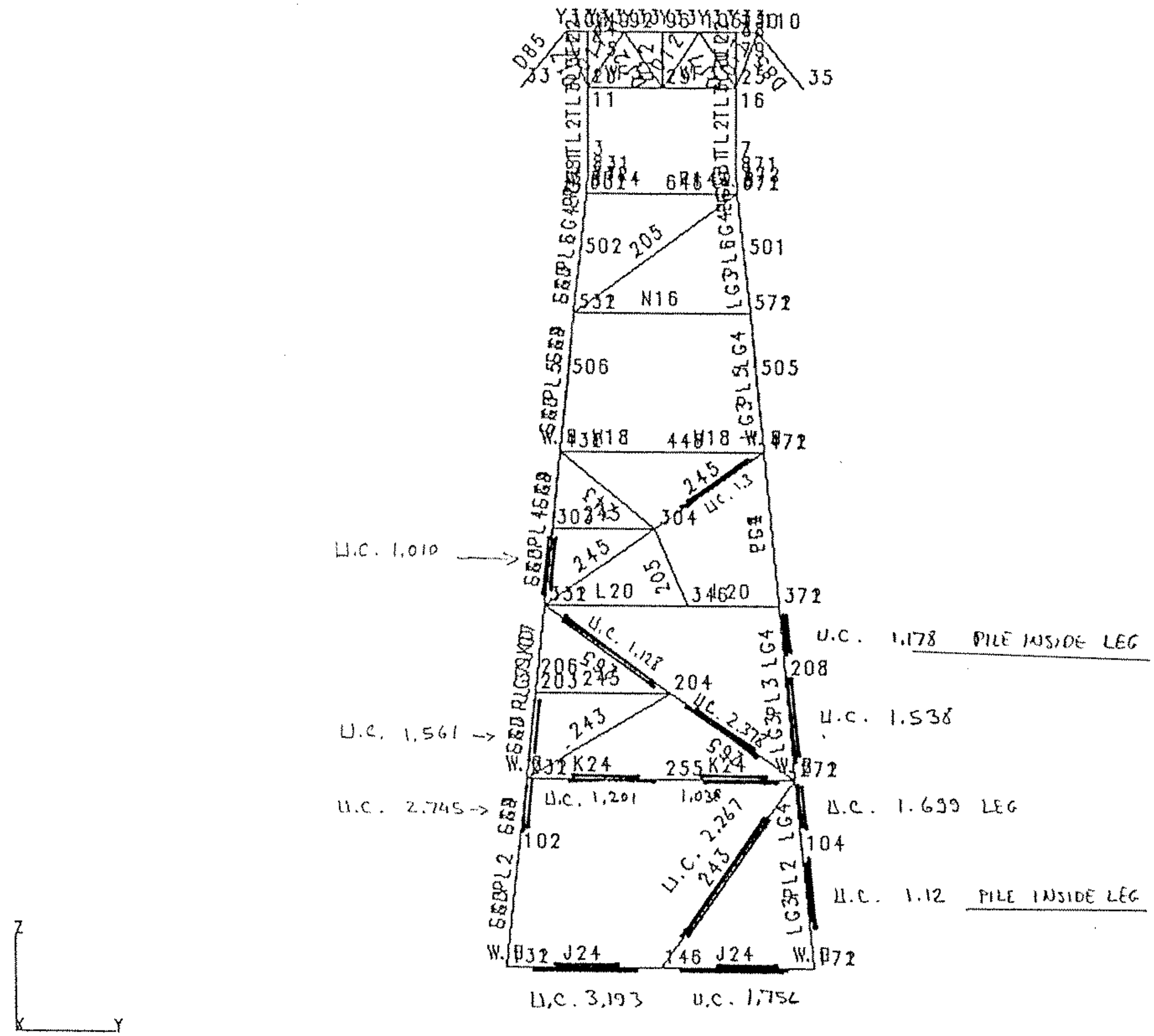
JACKET SIDE VIEW ROW 2



STRUCTURAL SOFTWARE, INC

PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 3



SHEET 145

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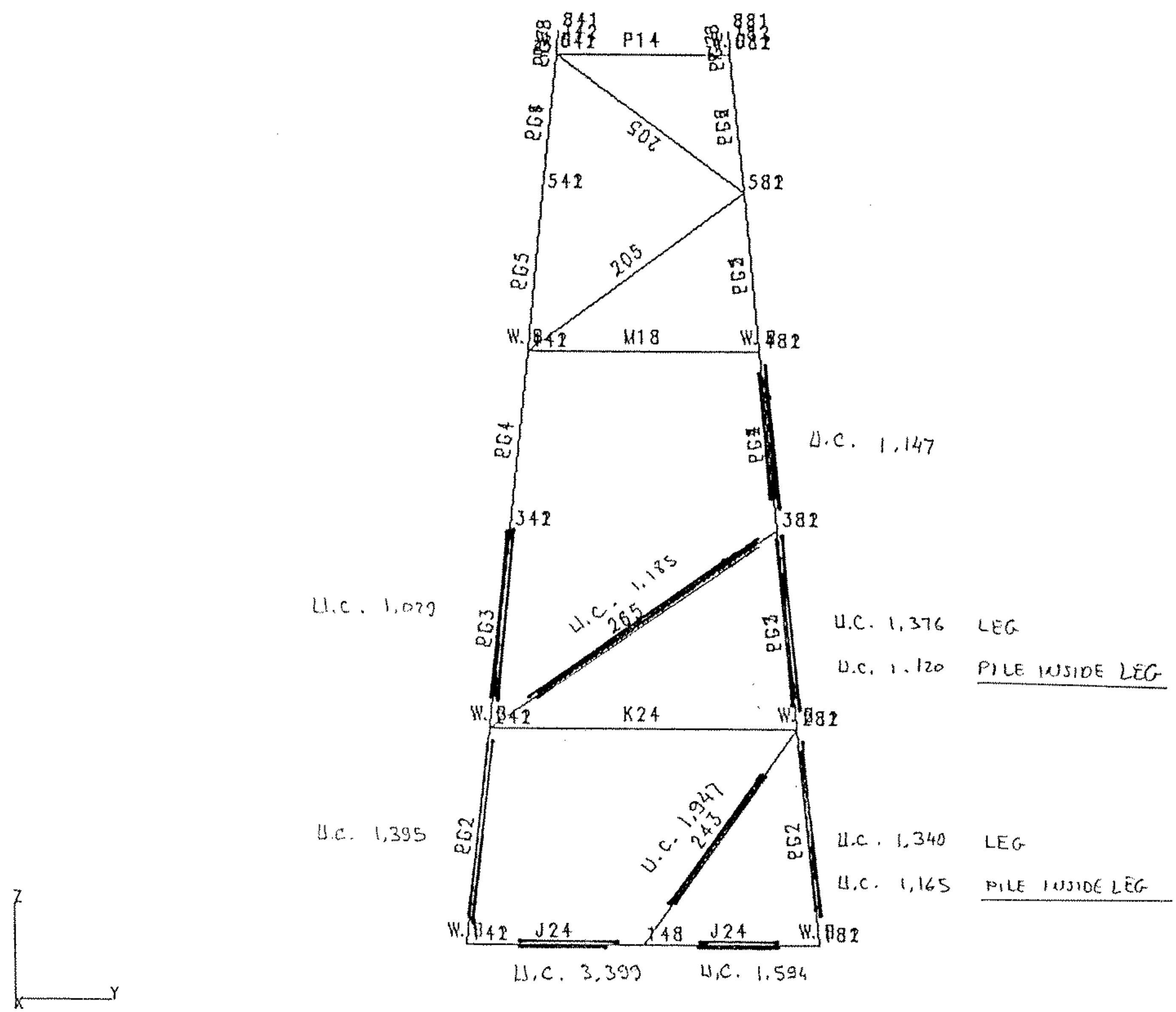
PUSHOVER ANALYSIS MODEL II for SHELL WEST DELTA BLK 103A

JACKET SIDE VIEW ROW 4

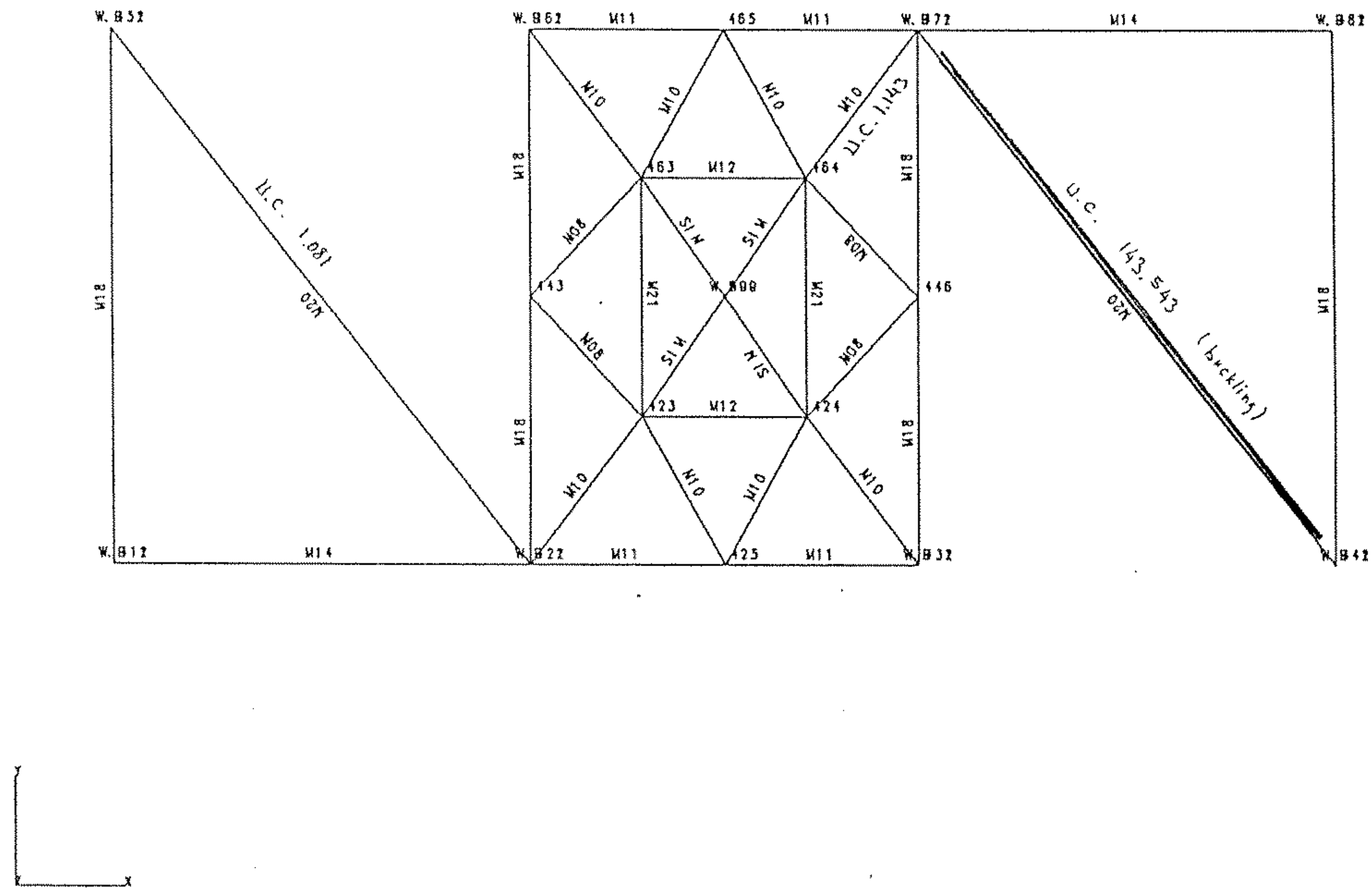


SHEET 146

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -68'

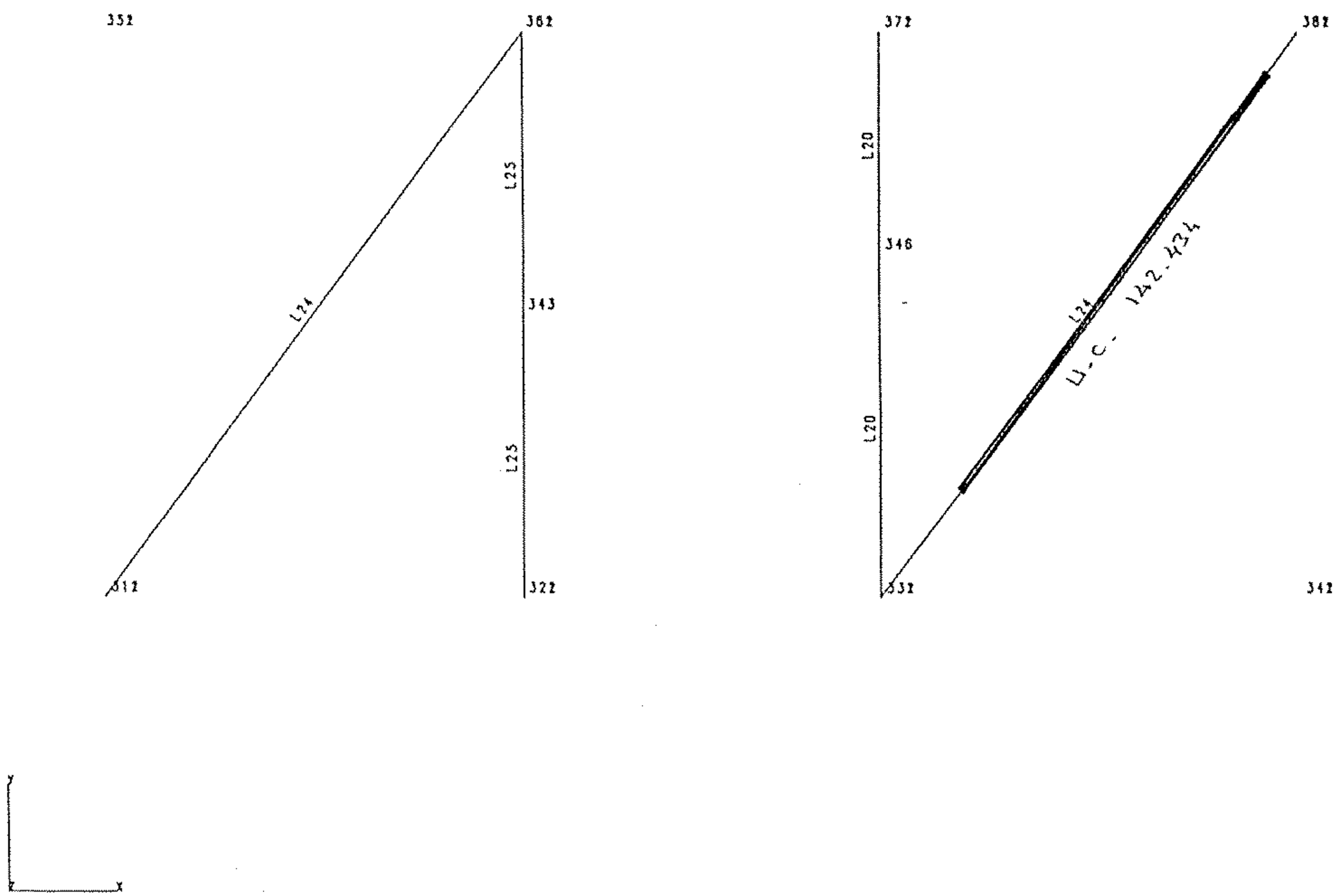


SHEET 147

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -114.5'



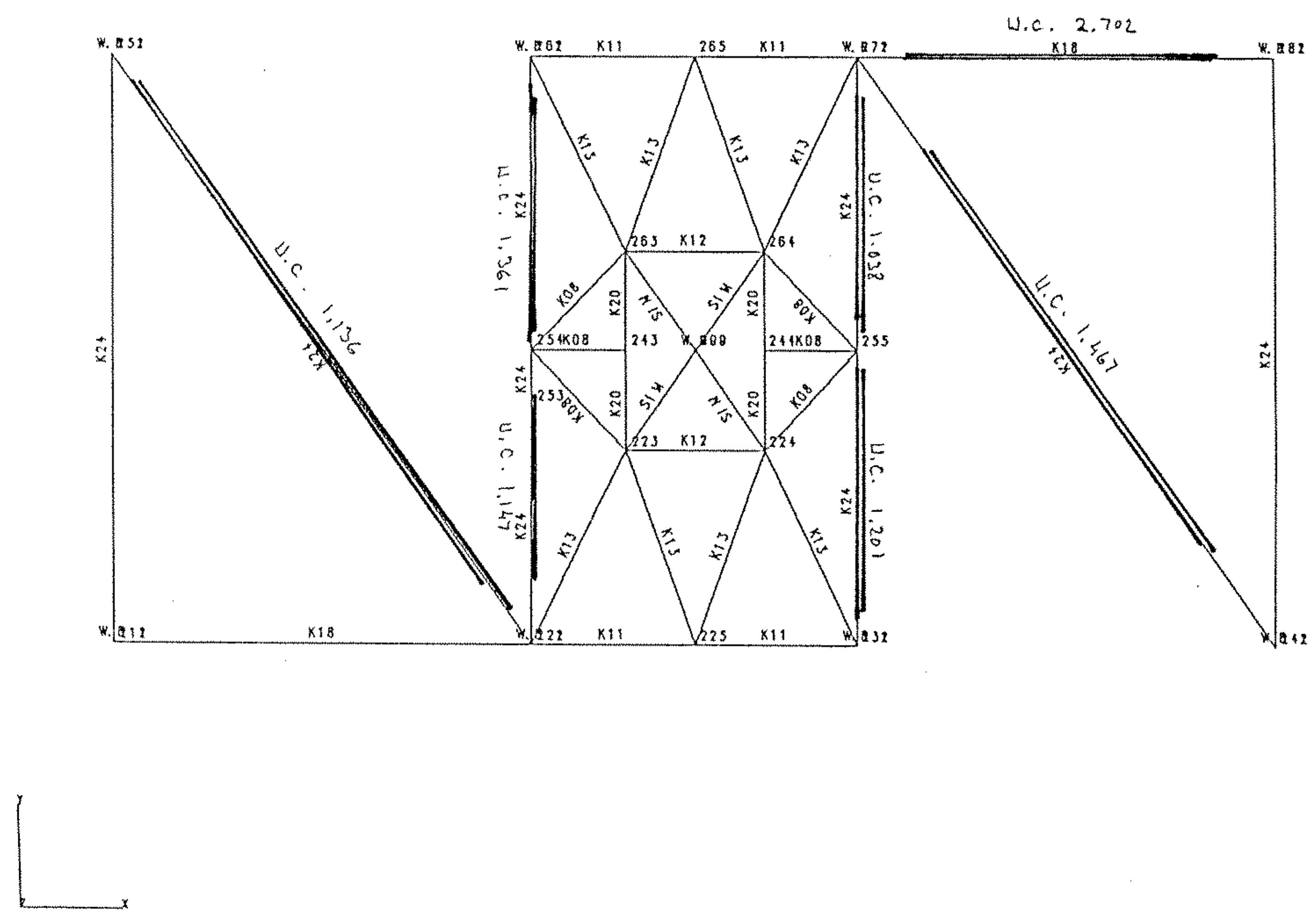
SHEET 148

STRUCTURAL SOFTWARE, INC

p18H0*4102I

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -166'

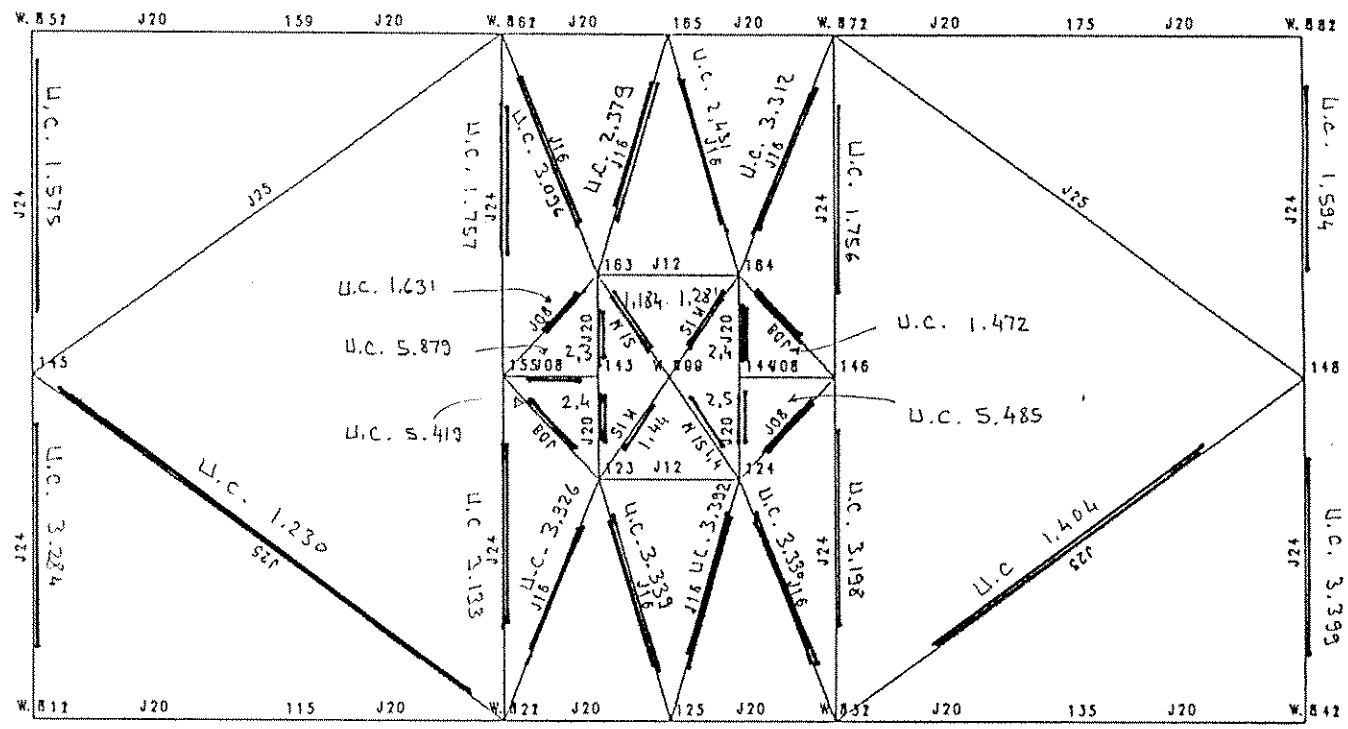


SHEET 147

STRUCTURAL SOFTWARE, INC

SHELL WEST DELTA BLK 103A 223 FT WATERS

PLAN EL -223'



SHEET 150

(7.5.4 continued)

The other failing members are the diagonals in Jacket vertical planes 1 thru 4 and also in horizontal planes at Elevations −223' (Mudline), −114.5', −68'.

The U.C.>1 Marked on Vertical Truss Planes 1 thru 4 and also dark lines for failing member clearly indicates of the total collapse of Jacket at first tier above mud-line level.

7.5.5 Joint Can Analysis (Section 8.4.4 in Book 4 of 4)

For failing joint cans with U.C>1 under actual brace Load refer to p. 62 thru 65 in Section 8.4.4 WDCOLLP. OT4 in Book 4 of 4.

There are 63 can failures in Pushover Analysis II versus the 3 can failures in Ultimate Strength Analysis run for 90° direction.

The failures occure on Jacket Legs as well as diagonal members.

June 17 94

MARK HANSON
Project Engineer
Shell Offshore Inc.
N.O.L.A
Phone: 588 6121 Fax: 5886092

Dear Mr Hanson

As Mr. Barry Reed discussed with you on the phone this morning we would appreciate receiving from you the follow information about Shell West Delta Blk 103 A 270' Water Platform :

1) Deck drawings (Structural & Major Equip. Location)
2) Deck Loads , Equip. Loads
3) To determine the EXPOSURE CATEGORY by API RP 2A 20TH ED). SECT 1
   a) Is this platform
      . Manned, non evacuated during major storm ?
      ✓ . Manned, evacuated " " " ?
      . Unmanned ?
   b) Environmental Impact (if platform fails)
      . Significant environmental impact ?
      ✓ . Insignificant environmental impact ?

Craig or Simon
I've already asked for a copy.

Thanks.

M. Ekrem Celebi (for Barry Reed)

[illegible] 835 2577

W.H. LINDER & ASSOCIATES, INC.
CONSULTING ENGINEERS

FACSIMILE COVER SHEET

DATE: 6-17-94 NO. OF PAGES: COVER + 1

LINDER PROJECT NO. 1830M01 LINDER PROPOSAL NO.:

MESSAGE TO: NAME: MARK HANSON

COMPANY: SHELL OFFSHORE INC. N.O.LA.

TELEPHONE NO: 588 6121 FAX MACHINE NO.: 588 6092

MESSAGE FROM: M. EKREM CELEBI / BARRY REED

TELEPHONE NO.: (504) 835-2577 FAX MACHINE NO.: (504) 837-5924