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BENCHMARKING OF COLLAPSE ANALYSIS OF LARGE SCALE ULTIMATE LOAD TESTS ON TUBULAR JACKET FRAME STRUCTURES

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1. INTRODUCTION

Reserve and residual strength, redundancy and collapse are important considerations in the design of offshore structures. Significant progress has been made in modelling and designing offshore structures at component level. The main issue, however, remains with the safety of the overall structure against system (usually catastrophic) failures. In the reassessment of structures there is a need to predict the ultimate response to either withstand 'natural (also known as environmental) hazards' or 'man made (also known as accidental) hazards'. Examples of environmental hazards are extreme wind, wave and earthquake. Examples of accidental hazards are fire, explosion, ship collision and dropped objects. Unforeseen operational changes in the course of the structure's life and long term degradation due to fatigue and corrosion, could also be contributory factors to system failures.

In order to control structural risk, it is necessary to model structural system behaviour in damaged and progressive collapse conditions. This is usually achieved by non-linear structural analysis augmented by component strength models. Traditional design of offshore steel structures has been based on the elastic analyses of the frame using codes which address the design of individual components (joints or members), which are essentially derived from large experimental databases on isolated joints and tubular beam-column members. A combination of loads for a specific design event are usually applied to the frame to determine the internal forces in each component and each member and joint is checked against its allowable strength given in the design code. The structure is considered to meet the design requirement if all individual components satisfy the code requirements. However, improvements in code revisions as a result of new data and/or better understanding of component strength behaviour may mean that older structures fail to meet current design requirements.

However, the potential of non-linear structural interaction between components means that the failure of one component may not necessarily lead to catastrophic collapse. The structure may therefore exhibit reserve and residual strengths beyond the design requirement depending on the conservatisms in the design of individual components, their role within the frame and the level of redundancy available. These analyses are by no means straightforward and some form of benchmarking is necessary.

As a result of the above the HSE funded a major review on the reserve strength of tubular frame structures. The results were presented at the 2nd ERA International Conference on Major Hazards for Offshore Structures in November 1993 (1). The review highlighted the increasing importance that was being placed on the use of non-linear Finite Element software packages, both general purpose and those specifically developed for undertaking ultimate pushover analyses. These are being used to exploit the reserve and residual strength of structures and to identify that a large range of reserve and residual strength values can be obtained. This review has been further supported by recent developments of code provisions such as the draft Section 17.0 for API RP 2A (2) where the role of using non-linear software to demonstrate the structure's adequacy is being considered if other simple and more conservative design checks cannot be satisfied.

However, one of the major concerns highlighted in the study is the lack of benchmarking undertaken when using Finite element programmes. One of the main reasons is the lack of available experimental data for benchmarking. A suitable set of experimental data that could be used for benchmarking existed in the Phase I Frames Test programme (3) which was established in 1987. Phase I was completed in 1990 and provided the first large scale test data on the collapse performance of frames representative of offshore structures. The results of this programme were released from confidentiality in 1993 and were presented at the 2nd ERA International Conference in November 1993 (4) and at the OTC Conference in May 1994 (5).

Prior to the open release of this data HSE considered there was an opportunity to invite a number of organisations who had significant experience in using non-linear finite element (FE) software programs for collapse analyses to participate in benchmarking against this available experimental data.

For the benchmarking exercise to have any significant value it was important that the analyses be undertaken blind (ie no prior knowledge of the actual test results would be provided to each of the organisations except general details of the frames and the member properties would be given). Eleven organisations in the UK, Norway and the US agreed to participate.

This paper provides an overview comparing the general features of the benchmarking exercise using different software programs to model the experimental frame behaviour. The results were presented at an HSE seminar held in September of this year involving the organisations that participated in the benchmarking activity. The results from the benchmarking exercise have been sanitised to ensure that the results obtained from each individual organisation remain confidential.

It should be emphasised that the benchmarking exercise has been limited to a two bay, 2-dimensional frame whereas real structures are more complex than this. However, a number of important lessons have been identified which will assist the Industry when undertaking future collapse analyses of offshore structures using non-linear software to determine the ultimate, reserve and residual strengths of their platforms.

2. PROCEDURE FOR UNDERTAKING BENCHMARKING EXERCISE

The experimental test data put forward for the benchmarking exercise were based on results obtained from the Phase I Frames Project (3) which consisted of four two bay X-braced frames. These tubular frames (see Figure 1) were the largest ever to be tested to collapse in a controlled manner and provided new and important insight regarding the role of redundancy and particularly tubular joint failures within a frame, which have not been investigated in earlier research programmes.

Three of the four test frames undertaken in Phase 1 of the Frames Programme were chosen for the benchmarking as shown in Figure 2 and Table 1. These frames exhibited different modes of failure enabling various aspects of reserve and post failure residual strength to be examined within the validation exercise (see Table 2). It was decided to also include a further frame in the benchmarking exercise as shown in Figures 3a & 3b which included the effects of locked in pre-stresses and initial imperfections. Further details of this frame are given in section 4.4.

2.1 Input Data Package

To enable each of the organisations to undertake analyses a data input package was prepared on behalf of the HSE by BOMEL whose personnel were engaged in the Frames I programme. This report provided details of the frames and member properties required for collapse analyses to be performed. In addition the format for reporting was carefully specified to ensure that the analysis could be directly and unambiguously compared with the experimental results.

Member properties were determined as part of the test programme in detail but to simplify the analyses the test cases were presented without initial imperfections in the data package. In this way the ability of the programs to model the modes and sequences of failure within the structure could be assessed.

It was requested that the output for each test case performed should include the following:

- Organisation.
- Contact person, address, etc.
- Program used and version.
- Brief description of element types used and formulation.
- Man effort and computer time used.

The output for each test case performed to include the following:

- Plot of Global frame load (P) against displacement (delta) and tabulated (P)-(delta) values from which the plot was produced. This would give an immediate visual impression as to whether the correct response characteristic and capacities have been predicted when compared to the experimental data.

- Plot of average brace loads against global frame displacement. This would determine whether the correct component failures, load shedding and redistribution have been predicted when compared to the experimental data.

- Frame deflected shapes at maximum global deflection of the frames at the end of test. This would enable the location of buckling etc to be readily identified. The data package also specified that the FE analyses should be carried out to deflections of 0.3m (ie maximum achieved from experimental tests).

- Statement as to why any analysis was halted (eg less than 300 mm limit achieved, solution failure, etc).

Before releasing the data package to the organisations it was independently checked by personnel within BOMEL who had not been involved in the Frames Project but were experienced in undertaking finite element Push-Over analyses. This Quality Assurance check was important so as to ensure that no errors would be present when circulating the data packages to each of the organisations involved in the benchmarking.

2.2 Distribution of data package and responses received

To ensure confidentiality the HSE requested MaTSU to project manage the distribution of the data packages, receive results and to take up further correspondences where necessary with each organisation. BOMEL were requested to deal with any clarification aspects received from organisations involving the data package.

The following procedure was adopted and undertaken by MaTSU during the course of the exercise:

- Distributed data packages to each of the eleven organisations.
- Receive blind finite element results.
- Compare results with test data and consult HSE.
- Respond on behalf of HSE to each organisation with details of observations and provide actual test data for comparisons.
- Organisations respond to observations and provide further information to assist benchmarking and in some cases undertake further analyses.

As a result of the analyses undertaken it was possible to identify a number of important features. The HSE then invited the organisations to a Seminar to discuss the results which was attended by most of the participants in September of this year.

3. RESERVE AND RESIDUAL STRENGTH OF FRAMES

In assessing the ability of a structure to withstand loads in excess of the design load or to sustain loading in the damaged state some measure of this ability is required. This leads to the terms such as reserve and residual strength and redundancy. It is important therefore to clearly define these terms and their usage. To illustrate these features Figure 4 which is based on Frame I from the Frames programme is used as an example. The reserve and residual strength factors, RES and REF, respectively, are therefore defined as follows:

$$\text{RES} = \frac{\text{Capacity of intact frame}}{\text{Frame design load}}$$

where the capacity of the intact frame is taken to be the peak load sustained by the structure in the test and the design load is calculated to a current design code (eg API RP2A) based on the measured material yield stress and nominal dimensions with the application of a storm condition safety factor. Once the peak load has been sustained the ability of the structure to sustain damage is quantified in terms of the residual strength factor defined by the following relationship:

$$\text{REF} = \frac{\text{Capacity of damaged frame}}{\text{Capacity of intact frame}}$$

where the residual strength depends largely on the structural redundancy within the system. Attention should be paid, in particular, to the margin between the design storm loads and the residual plateau. The combination of reserve and residual strength quantifies the ratio between the residual capacity and the design loads and clearly defines the role of redundancy. If the product of RES and REF exceeds unity then the frame is capable of sustaining the design storm even in the damaged state.

The reserve and residual strengths, and role of redundancy will be examined further when comparing the experimental results with those predicted from finite element analyses.

4. FAILURE FEATURES OF TEST FRAMES USED IN BENCHMARKING EXERCISE

As mentioned previously in Section 2 the Phase 1 Frames exhibited different modes of failure enabling various aspects of reserve and post failure residual strength to be examined within the validation exercise. This section describes the failure features obtained for each frame tested and provides important background information when comparing results with those obtained from the FE analyses. Results showing the key failure components, peak loads achieved and reserve and residual strengths for each of the experimental test frames are given in Table 2 and shown in Figures 5-11. Further information on the failure features can also be found in references (3) and (4).

4.1 Test Case 1

The compression brace was the critical component when analysed in accordance with API RP 2A (6). Figure 5 depicts the overall Global P -delta response of the frame in the test where the peak load (922 kN) coincided with buckling of the top half of the top bay compression brace. Although the test was not continued far into the post-ultimate regime it can be seen that a substantial residual strength remained, attributed to both the transfer of load via the top bay tension brace and horizontal member into the bottom bay, and portal action in the legs which contributed to a gradual increase in capacity at large global deflections.

An interesting feature of this test was the reaching of a yield plateau in the tension brace before buckling of the slender compression brace. Whilst this explains the degradation in Global Stiffness prior to the attainment of the peak load, the result was somewhat unexpected.

The explanation is thought to be found in the presence of locked-in stresses arising from the fabrication procedure. It is postulated that this left a locked-in pre-tension in the braces such that the applied loads transmitted through the compression brace were initially reversing the pre-tension. The net compression force driving the buckling was therefore lower than apparent from the load cell readings which were based on a zero load datum at the start of the test. The converse would apply in the tension chord with a slightly higher stress prevailing at yield than indicated by the load cell readings. The sequence of yield and buckling was therefore governed by locked-in-fabrication stresses in addition to the externally applied loads.

4.2 Test Case 2

Test case 2 was similar to Test case 1 except that the top bay X joint had a substantially reduced chord wall thickness. Consequently the joint was the critical component. The overall global response of the frame is shown in Figure 7. The Beta=1.0 compression X joint gradually softened and chord wall ovalisation became visible at an applied load of only 689 kN (joint failure). Yielding of the tension member and the bottom bay compression member was also noted.

With increasing global deflection yielding of the tension brace increased, and portal action developed in the legs. The joint continued to compress until the braces came into contact across the flattened chord creating a new stiff load path through the panel. The global load sustained by the frame continued until the buckling resistance of the compression brace exceeded 1080 kN (peak) and load was rapidly shed. Ultimate strength was governed by the buckling of the upper compression brace member in the top bay, and not the failure of the top bay joint, albeit at a much higher displacement.

4.3 Test Case 3

Test case 3 was identical to test case 1 except that the mid-height horizontal was omitted. The member carries negligible load in the elastic regime and might be omitted in practice to reduce structural weight. The implications of reduced redundancy on reserve and residual strength were therefore examined in test case 3. The global response for test case 3 (shown in Figure 9), where the plateau following initial failure of the buckling of the lower compression member in the top bay (780 kN) is at a comparable level to test case 1. However in the absence of the horizontal the load from the top bay tension brace passes directly into the bottom bay compression brace. With increasing deflections the load in this compression brace increased until the upper compression brace in the bottom bay buckled (740 kN).

This buckling precipitated a second rapid fall-off in load reducing the residual capacity well below the API RP 2A design storm load ie from Table 2 compare 558 kN (API RP 2A design) against an experimental minimum capacity of 440 kN, from Figure 9.

Unlike test case 1, the relevant locked-in fabrication stresses were compressive as a result of the different fabrication sequence adopted .

Brace buckling in the top bay therefore occurred at a lower applied load than predicted and tensile yielding was significantly delayed and was associated with an apparently higher load than the material properties would suggest. The effect was that, although brace buckling in the top bay occurred, the reduction in capacity was compensated by additional loads carried by the tension chord until yielding occurred. Only at this point did the global capacity fall significantly. Test case 1 and 3 therefore give a useful demonstration of the mechanism of load redistribution attributed to the relative member capacities in response to applied loads and the redundancy within the frames.

4.4 Test case 4

Test case 4 as shown in Figures 3a & 3b, is similar to test case 3 except that explicit locked in pre - stresses and out of straightness values are prescribed. As explained previously in section 2 and demonstrated by the results given above this case was chosen since results from the frame tests had clearly demonstrated that the initial out-of-straightness had reduced buckling loads within the frames, and similarly initial locked-in prestress, influence the loads and sequences of component failures.

To attempt to illustrate the potential effect and identify methods for allowing for the locked in forces in analyses Case 4 was set up as a variant of test case 3. On this basis test case 4 is more representative of the test conditions of test case 3 (ie data for test case 3 was supplied to organisations without knowledge of imperfections and locked in stresses) and therefore should provide closer correlation with the experimental results.

5. COMPARISON BETWEEN PREDICTED FE ANALYSES AND EXPERIMENTAL FRAME TEST CASES

This section compares the predicted FE results with those obtained from the test data. Even though it is not possible to cover all of the many features from the benchmarking activity, by comparing the global load (P) versus displacement (delta) plots it is possible to identify typical types of behaviour and hence assess different modelling uncertainties. Some of the analyses were also undertaken by organisations using similar packages, ie these are referred to as packages A, B and C in this paper therefore enabling the opportunity to also examine user uncertainties. Examples to illustrate the typical type of behaviours identified are shown in Figures 5-11, Figures 12 and 13. Tables 3 and 4 give the range of reserve and residual strengths for each of the test cases examined.

5.1 Modelling and user Uncertainties

When examining the predicted results it was observed for each test case that a number of the analyses could be grouped together as they appeared to demonstrate the same type of behaviour. Three generic types of behaviour were identified from all of the analyses for each test case, namely type 1, type 2 and type 3. Therefore when comparing results in this paper examples of each type of behaviour will be given.

Test Case 1

Three types of behaviour were obtained from the analyses performed for test case 1, shown in Figure 5. Four of the results showed type 1 behaviour, four type 2 and three type 3. All of the analyses predicted buckling of the compression brace although the response of the actual brace failing differed in several cases.

For type 1 behaviour the significant reduction in load as a result of the buckling of the brace member was not captured and instead the global load continued to increase resulting in an overprediction of the reserve strength.

For type 2 behaviour although buckling was captured the reduction in load was not sustained and the load continued to increase resulting in an overprediction as noted above for type 1 behaviour.

For type 3 behaviour buckling and the subsequent reduction in load were generally well captured and the predicted behaviour was close to that observed.

For those organisations using the same FE package A, two different types of behaviour were obtained as shown in Figure 6. The behaviours are similar to types 2 and 3 shown in Figure 5. Similar trends for organisations using packages B and C (not shown) were also observed albeit these consisted of different combinations of either type 1, 2 or 3 behaviours, as shown in Figure 5.

Test case 2

Three types of behaviour were obtained from the analyses performed for Test case 2 (only ten were undertaken for this case) and are shown in Figure 7. Three of the results show type 1 behaviour, five type 2 and two type 3.

Only for type 3 behaviour was modelling of the joint considered and the general features of the joint failure and member buckling were well predicted by the analyses. Type 1 and 2 behaviours did not include joint modelling and the predicted frame pattern responses were generally similar to type 1 and 2 behaviours predicted for test case 1, as seen by comparing results from Figure 5 with Figure 7.

For organisations using the same FE package A, two types of behaviour were obtained as shown in Figure 8. The behaviours are similar to type 2 (no joint modelling) and type 3 (joint modelling) as shown in Figure 7.

The comparisons above between predicted and experimental, clearly demonstrated the large variation in modelling and user uncertainties that occurred as a result of whether joint failure was modelled in the analyses.

Test case 3

Three types of behaviour were obtained from the analyses performed for test case 3 (ten analyses only) as shown in Figure 9. Three of the results show type 1 behaviour, three type 2 and four type 3.

Definition of Residual Strength

For type 3 behaviour the definition of minimum load defined in section 3 and shown in Figure 4 was use to determine the residual strength. Types 1 and 2 behaviours did not display a reduction in frame load after reaching their peak load, therefore residual strengths for these types of behaviour could not be directly obtained .

However, to enable some indirect comparisons to be undertaken between the three different types of behaviour the capacity predicted at the displacement level corresponding to the minimum capacity of the experimental frame was chosen as a basis for determining the residual strength for types 1 and 2 behaviours. This method did have some limitations since many of the type 1 and 2 analyses, especially for test cases 2, 3 and 4 were terminated at displacement levels lower than those corresponding to the experimental minimum capacity.

Table 3 and 4 and Figures 12 and 13 give the range of reserve and residual strengths that were obtained from the analyses undertaken. The design loads were calculated to a current design code in this paper based on the API RP 2A 19th edition (6). As mentioned previously in section 5 some of the analyses were undertaken by organisations using the same FE software package, ie packages A, B and C. Results are presented for these three packages, as well as for other packages but grouped together, and for all analyses together.

The two key issues from the analyses undertaken are accuracy and consistency. The mean ratio between predicted and experimental values ('Bias' in tables 3 and 4) is an indication of the general accuracies. The standard deviation of the ratio ('STDEV' in Tables 3 and 4) is an indication of the consistency of the analyses. Two issues will be discussed as follows:

Results of Reserve Strength Analysis

From Table 3 and Figure 12, for all of the test cases there appears to be an overall bias of the analyses to generally overpredict the experimental reserve strength results.

The maximum overpredictions of reserve strength obtained for each test case were for those analyses that exhibited type 1 behaviour. For these the maximum overprediction ranged from between 39% and 54% for the test cases considered. The minimum overpredictions were generally for those that showed type 3 behaviour for each test case examined. The minimum overpredictions for type 3 behaviour ranged from between 9% and -18% (ie underprediction) for the test cases considered.

The maximum overpredictions were also obtained for test cases which involved more than one component failure, ie case 2 and case 3.

The largest variations in results were obtained from those analyses which exhibited type 1 behaviour, ie standard deviations between 15% and 26%, whilst types 2 and 3 behaviours showed less variation.

The variations obtained between those organisations using similar packages tended to be lower, ie less than 10% compared with those obtained from organisations using other packages, ie generally of the order of 20%, as shown in table 3, except for cases involving more than one component failure.

Results of Residual Strength

From Table 4 and Figure 13, it can be seen that for all of the test cases there again appears to be consistent bias of the analyses to overpredict the experimental residual strength results. This bias also appears to be much larger than observed for the estimates of reserve strength, as shown by comparison of the results given in Figures 12 and 13 respectively.

The maximum overpredictions and variations obtained in predicting the residual strength where comparisons could be made between the different types of behaviours (for case 1 only), were again for those which exhibited type 1 behaviours. For test case 2, results from type 2 and 3 behaviours could only be compared, whilst for test cases 3 and 4 only type 3 results and one type 2 behaviour analyses could be compared. As mentioned previously many of the analyses demonstrating types 1 and 2 behaviour (especially for test cases 3 and 4), were terminated at deflections lower than that corresponding to the minimum capacity of the experimental frame. Therefore for these cases comparisons between the different types of behaviour could not be undertaken.

The maximum overpredictions of reserve strength obtained ranged from between 114% to 32% for the test cases considered. The largest overpredictions and uncertainty were obtained for test case 3 which involved the frame with more than one component failure and the largest lateral displacement. The maximum overpredictions for this test case was 114% with a standard deviation of 43%. The minimum overpredictions were again for those analyses which showed type 3 behaviour and ranged between 18% and -17% (underprediction) for the test cases considered .

It was not possible to undertake comparisons between organisations using the same packages and those using others since for a number of cases results were not available.

The above results for reserve and residual strength highlight the significant variation in accuracy and consistency obtained. However one could argue that a better measure of accuracy and consistency is to combine the two values of BIAS and STDEV. A measure analogue to the reliability index in structural reliability analyses can be defined as:

$$\text{Beta}' = (1 - \text{mean})/\text{STDEV}$$

If the bias ratio is distributed normally, the probability of underprediction (thus non-conservative) is:

$$\text{probability of underprediction} = \Phi\,(-\text{Beta}')$$

where Φ (), is the cumulative standard Normal Distribution function. The student's t distribution should be used instead of Φ in the case of small samples. In other words 'statistically correct analyses affected only by random statistical errors will always have a beta' value near to 0.0. If an analyse is more likely to give conservative results, beta' will tend to be larger (that is more positive).

To illustrate the above, results presented in for the residual strength in Table 4 show a range of beta' values between -3.8 and -0.5. The beta' values consistently show negative values and this clearly illustrates the consistent over-prediction as noted earlier. Further treatment of the above will be given in a later paper.

6. ADDITIONAL ANALYSES

As mentioned previously in section 2.2 some organisations, as a result of comparing their predicted blind analyses with the experimental results, undertook further analyses to improve their prediction. In general it was noted by many of the organisations that information provided after undertaking the blind analyses on the local and global failure features of each test enabled the user to undertake further modelling to try and capture the correct behaviour. At the time of writing this paper these additional analyses were still being assessed. The results of these will be presented at a later date.

7. CONCLUSIONS

A benchmarking exercise has been completed involving 11 organisations from both the UK, Norway and the US. The test data was supplied 'blind' and the results obtained included both modelling and user uncertainties.

I. Three different generic types of modelling behaviours (namely type 1, 2 and 3) were obtained for each of the test cases examined. The range of predicted versus experimental reserve and residual strength found varied depending on the type of behaviour. For each test case at least one program gave a good prediction of overall performance. The results for type 3 behaviour generally compared well with the experimental results, whilst for type 1 and some type 2 behaviours fits were less good and the highest overpredictions and largest variations were observed for these types of behaviours.

II. For the case of frame 1, failure involving member buckling, all of the analyses predicted buckling of the compression brace although the actual response of the brace failing differed in several cases. Seven of the analyses showed behaviour types with a peak in the load-displacement curve, as in the test case (types 2 and 3), with three of these also capturing the subsequent reduction in load (type 3), as in the actual test. Four of the results showed type 1 behaviour in which the load continued to increase with increasing displacement although an inflection occurred.

III. Joint failure dominated in test case frame 2 and many packages failed to predict this as a result of not modelling joint failure. Hence behaviour types observed were similar to those predicted for case 1. This led to a wider range in predicted performance.

IV. For test cases 3 and 4 with the mid-height horizontal removed initial imperfections and locked in residual stresses appeared to play a significant role in determining the reduced peak load, correct load redistribution and sequence of component failure. These were not generally modelled well by any of the packages.

V. The uncertainties in prediction also increased with increasing complexity of the failure mode. In addition the prediction of residual strength was less certain in most cases, than estimates of the reserve strength, as the former involved higher levels of displacement and non-linear response. It is likely that this trend could be greater (ie higher uncertainty) in analyses of a more complex model structure where there is the greater likelihood of non-linear interaction, resulting in larger displacement, yielding and failure of more than one component.

VI. Residual strengths could only be predicted for those analyses that mainly exhibited type 3 behaviour. For many of the type 1 and 2 behaviours residual strengths could not be determined either as a result of the analyses showing no reduction in the frame load after reaching the peak load or being terminated at deflections lower than that obtained from the experiment at minimum capacity.

VII. For those organisations that used the same computer package the consistency of results was generally better. However, some packages did show both types 1 and 2 behaviour (ie high overpredictions) and standard deviations similar to those obtained from other organisations, (ie greater than 20%). It is likely that these would be greater in analyses of a more complex structure where there are more opportunities for user variation in the modelling.

The benchmarking has also identified the following problems which require further attention when undertaking system collapse analyses.

VIII. Inconsistent modelling of component behaviour. There are a number of valid ways to model the failure of components during system collapse. However in order to select the most appropriate model in different circumstances, better understanding is needed for the behaviour of components within a system (as opposed to components in isolation).

IX. Choices and decisions of the analyst have a large effect on the results. Therefore, some form of quality control is needed in order to achieve more consistent results.

X. Collapse failure sequences vary. This is an issue important to structural system reliability analyses. It is because system reliability is calculated from the sequential failure probability of components. If this sequence is not of the correct order, or contains the wrong list of components, the system reliability value could be affected significantly.

It is recommended that a benchmarking exercise is undertaken for a more complex structure, using a comparison of different computer models and similar programs using different users.

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TEST CASE	DETAILS OF FRAME
1	AS SHOWN IN FIGURE 2
2	AS FRAME 1 BUT WITH UPPER BAY X JOINT CAN REMOVED
3	AS FRAME 1 BUT WITH MID HORIZONTAL REMOVED
4	AS FRAME 3 BUT WITH INITIAL PRE STRESS AND INITIAL IMPERFECTIONS INTRODUCED AS SHOWN IN FIGURE 3A & 3B

Table 1: Frame Details

Test Case	Critical Component	Frame Load (Y) (kN)	Design Storm Load (X) (kN)	Reserve Strength Factor RES=(Y/X)	Residual Strength Factor REF=(Z/Y)	(RES) x (REF)
1	Buckling of Lower Half of Top Bay Compression Brace	922	579	1.59	0.79	1.25
2	Compression Failure of X Joint	689	276	2.5		
	Buckling of Upper Half of Top Bay Compression Brace	1080		3.91	0.81	3.16
3	Buckling of Lower Half of Top Bay Compression Brace	780	558	1.4	0.94	1.32
	Buckling of Upper Half of Bottom Bay Compression Brace	740			0.56	0.79

Note: For definitions for reserve and residual strength factors see figure 4

Table 2: Summary of Experimental Reserve and Residual Strength Factors

Frame Test Case	Design Storm Load (kN)	Lateral Deflection Ratio	(Reserve x Residual Strength) Factors										
			Experimental	Predicted - Finite Element Analyses									
				Software Package	Mean	Maximum	Minimum	Bias	Max/Exp	Min/Exp	STDEV	β′
1	579	0.32	1.25	A	1.4	1.47	1.35	1.12	1.18	1.08	0.05	-2.4*
				B	1.73	1.82	1.63	1.38	1.46	1.3	0.1	-3.8*
				C	1.68	1.8	1.56	1.35	1.44	1.17	0.14	-2.5*
				OTHERS	1.52	1.71	1.34	1.21	1.37	1.07	0.14	-1.5*
2	276	0.7	3.16	ALL	3.42	4.18	2.64	1.08	1.32	0.83	0.16	-0.5
3	558	0.84	0.79	ALL	1.21	1.69	0.93	1.54	2.14	1.18	0.43	-1.26
4	558	0.84	0.79	ALL	1.11	1.32	0.92	1.41	1.67	1.16	0.23	-1.78

* Note result based on sample size of less than 5 data points

Table 4: Comparison of Predicted v Experimental Reserve and Residual Strength Factors

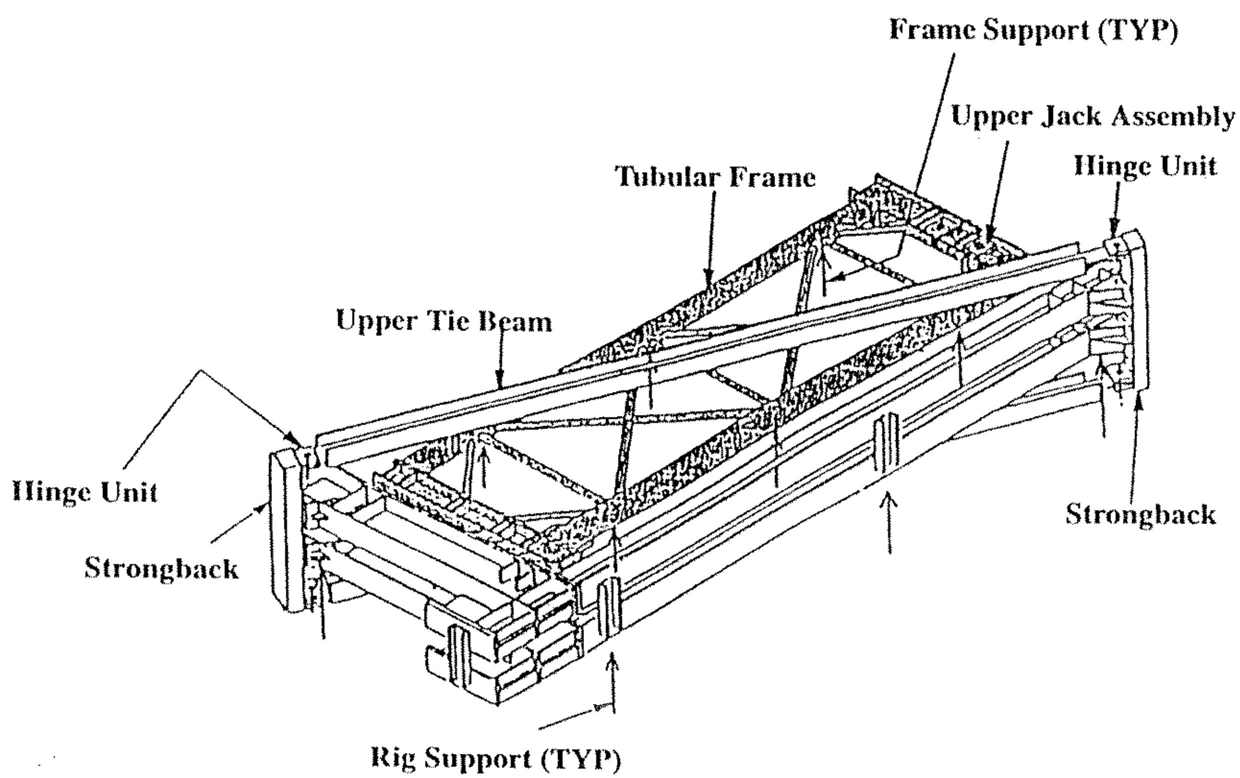


Figure 1: Schematic View of Test Frame in Triangulated Rig

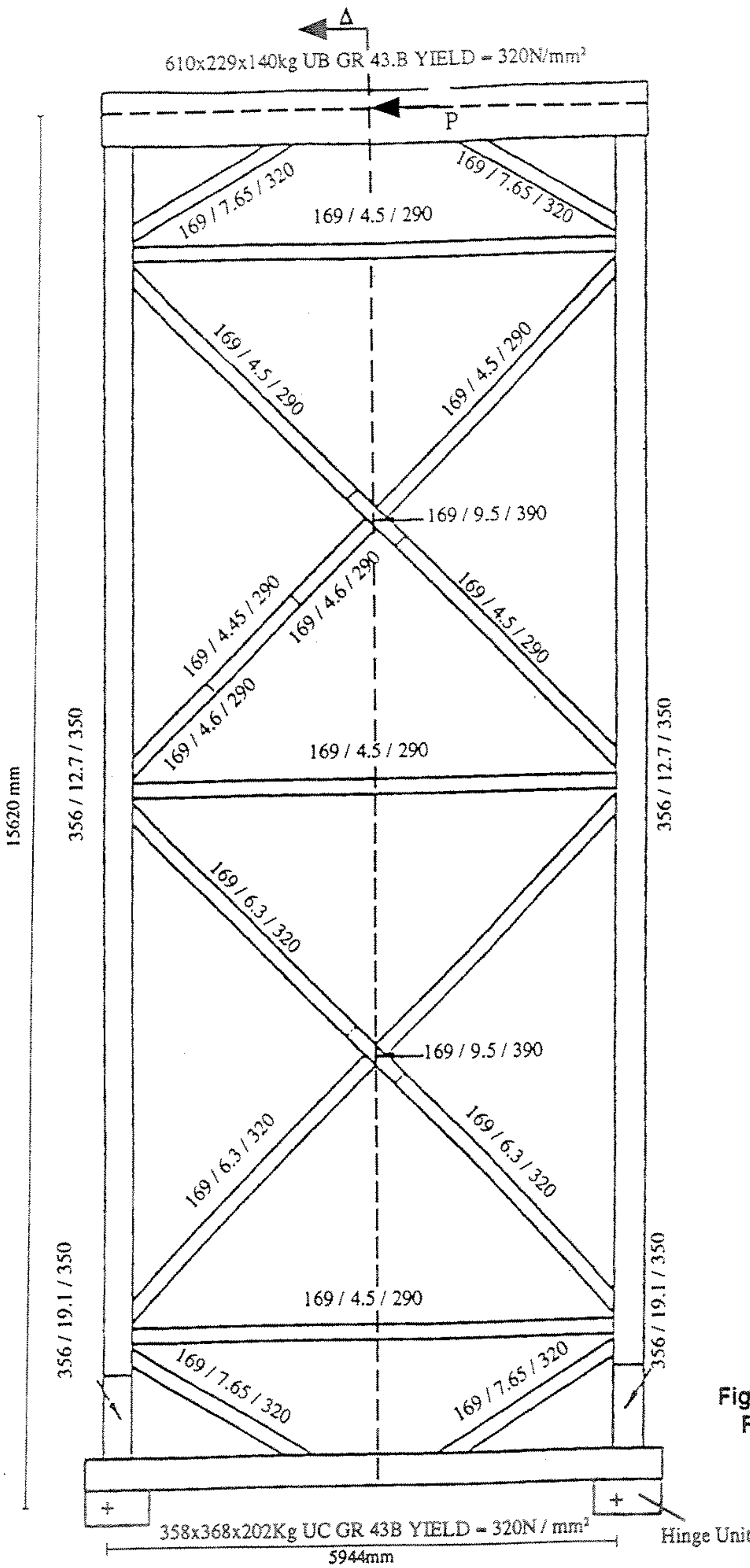


Figure 2: Test Case 1 Frame Properties

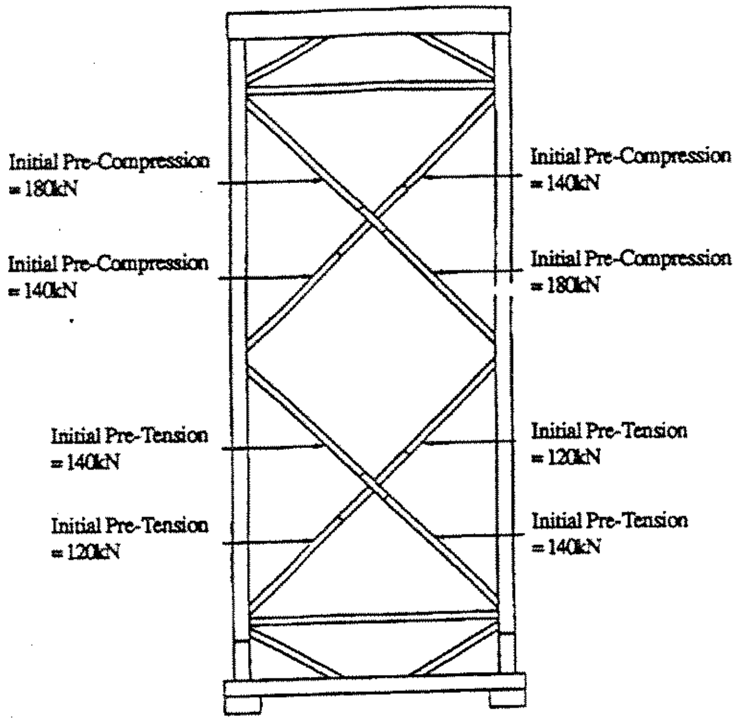


Figure 3a: Test Case 4: Initial Pre-Load

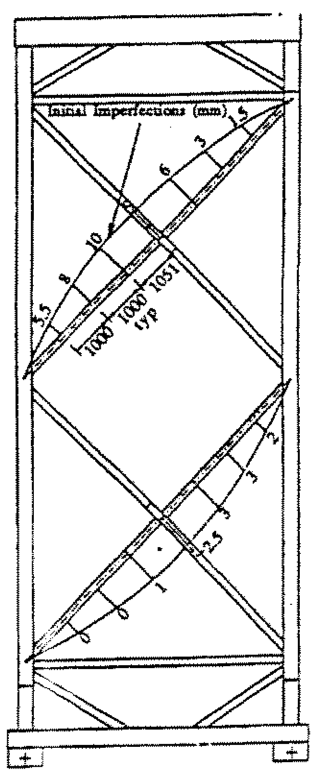
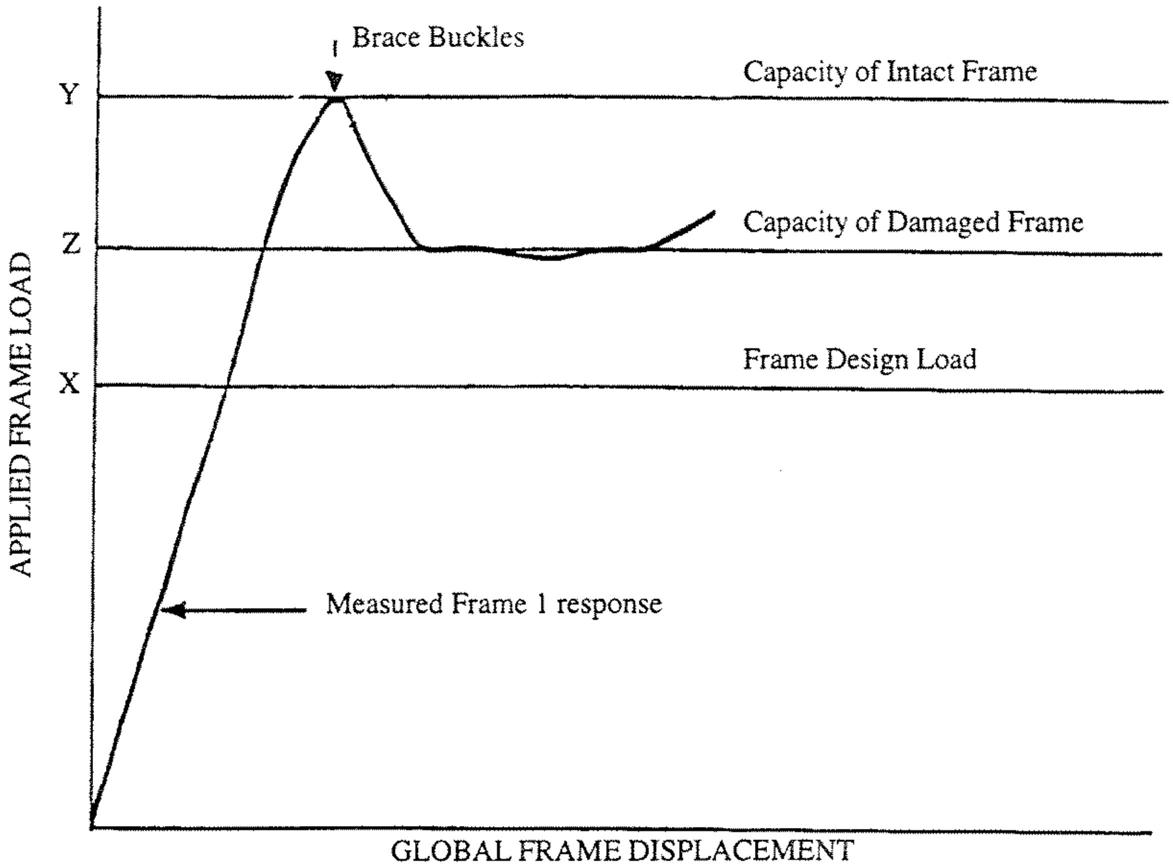


Figure 3b: Test Case 4: Initial Imperfection



Residual strength factor (REF)	=	Z/Y
	=	0.79 for Frame 1
Reserve strength factor (RES)	=	Y/X
	=	1.59 for Frame 1

Figure 4: Definitions of Reserve and Residual Strength

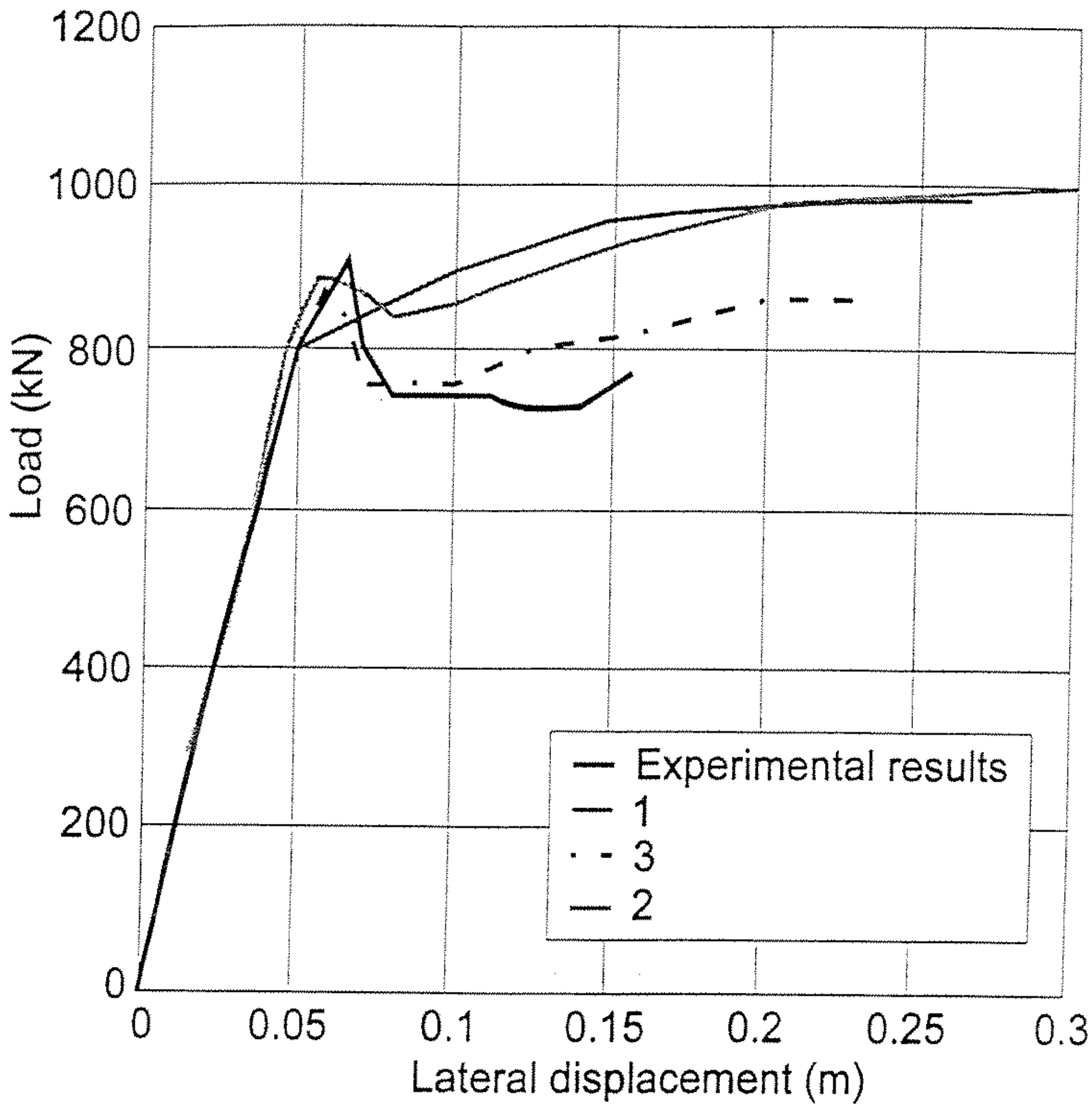


Figure 5: Typical Global-Displacement Features for Test Case 1

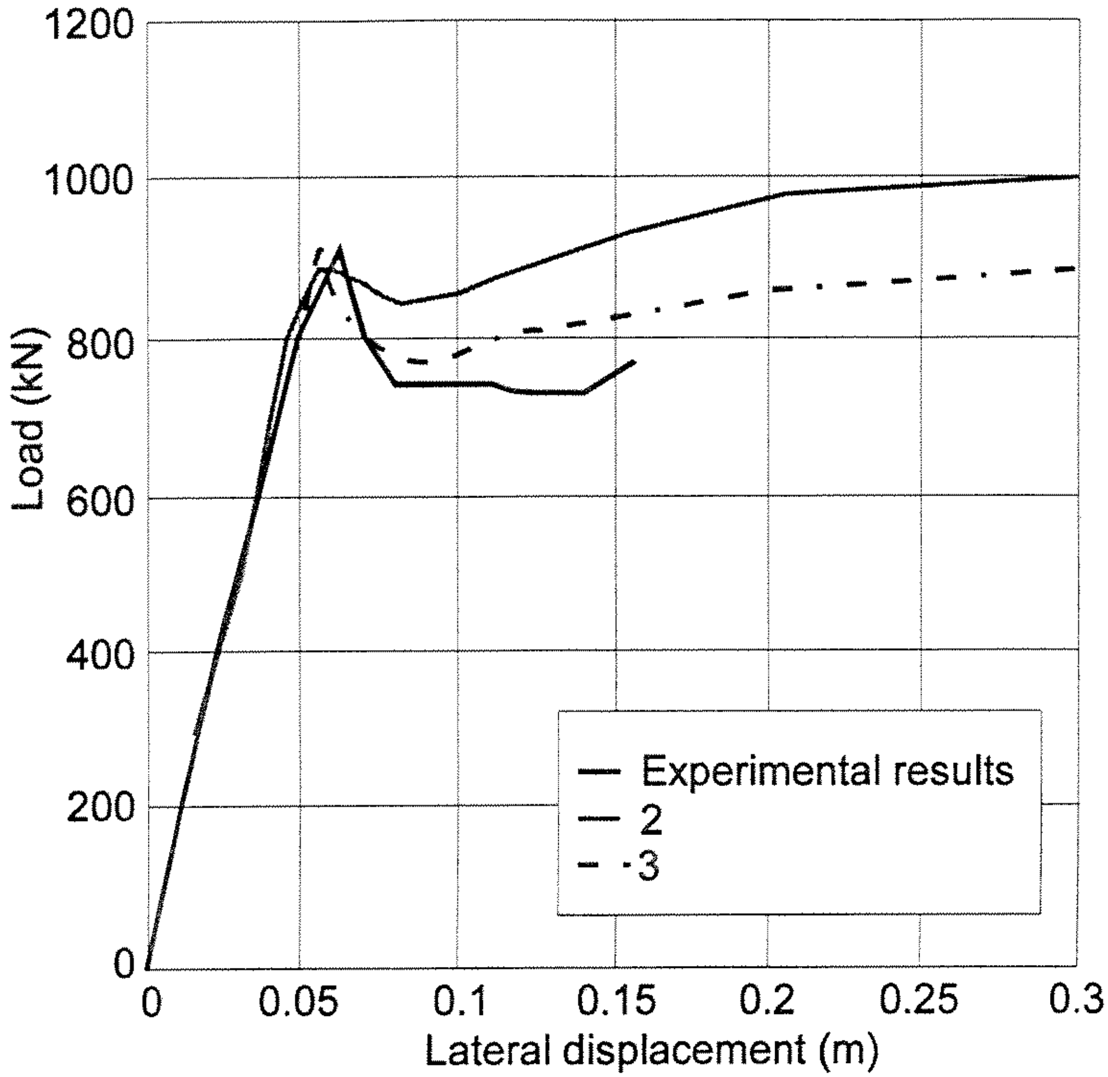


Figure 6: Typical Global-Displacement Features using Similar FE Software for Test Case 1

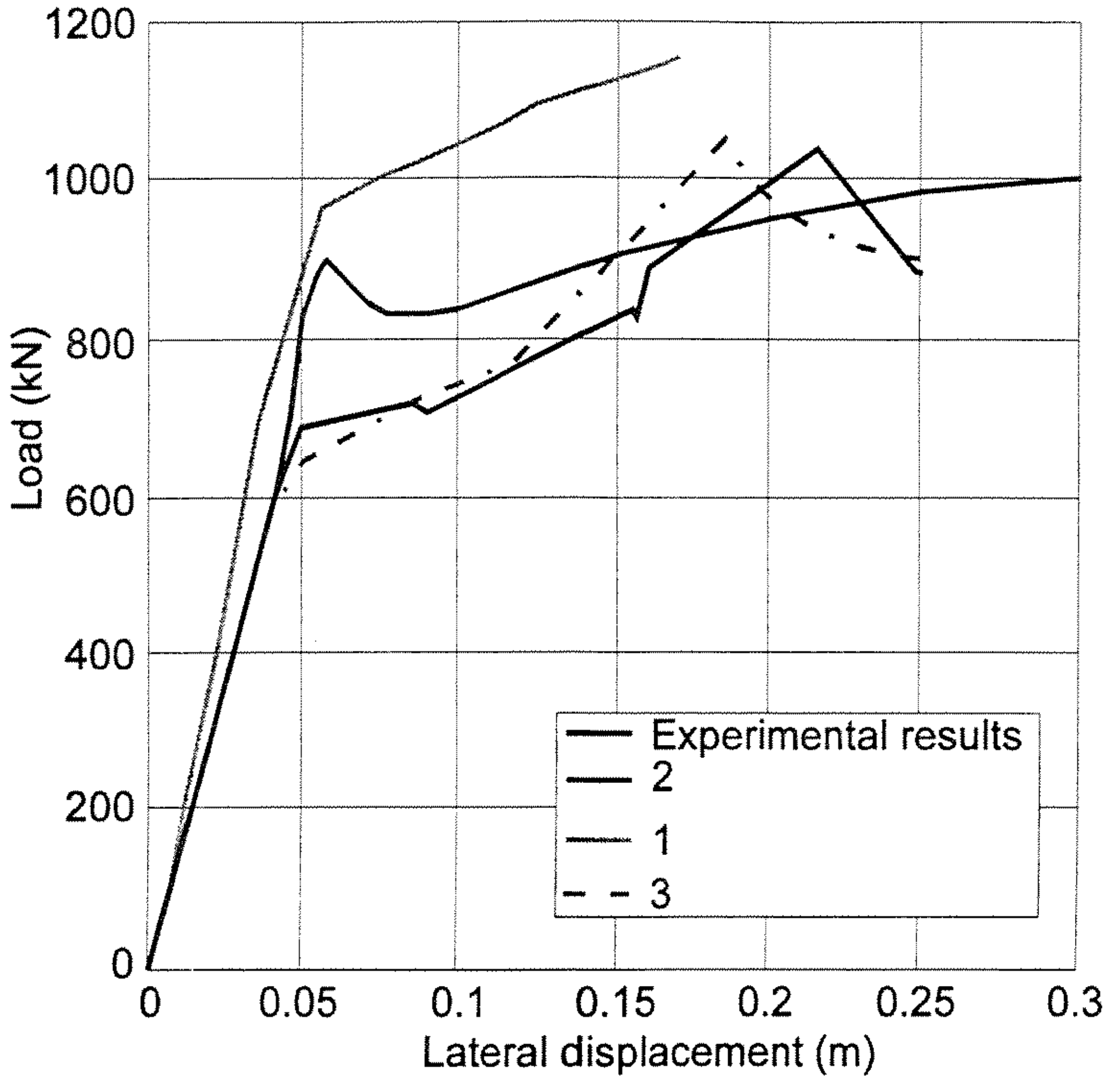


Figure 7: Typical Global-Displacement Features for Test Case 2

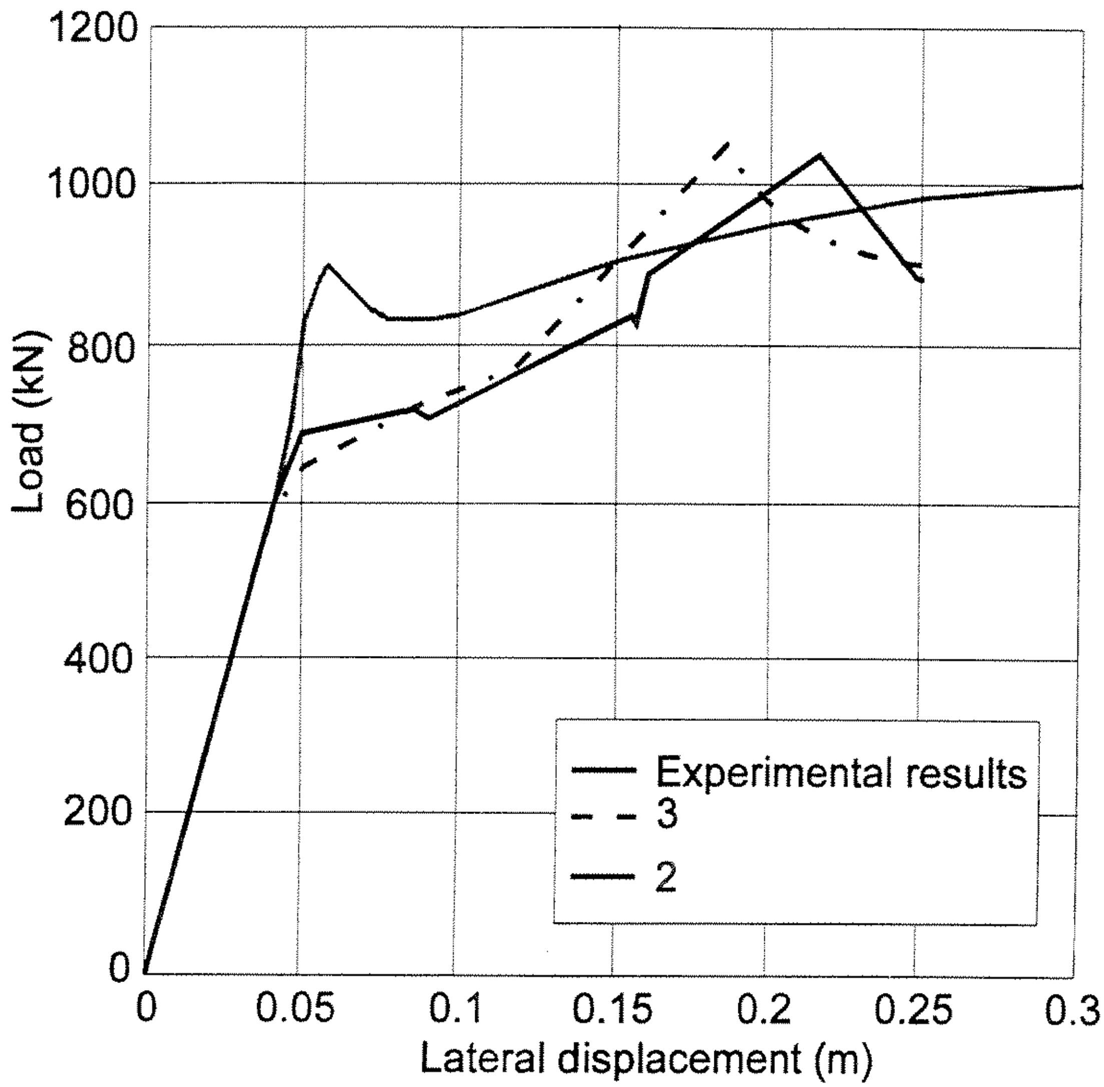


Figure 8: Typical Global-Displacement Features using Similar FE Software for Test Case 2

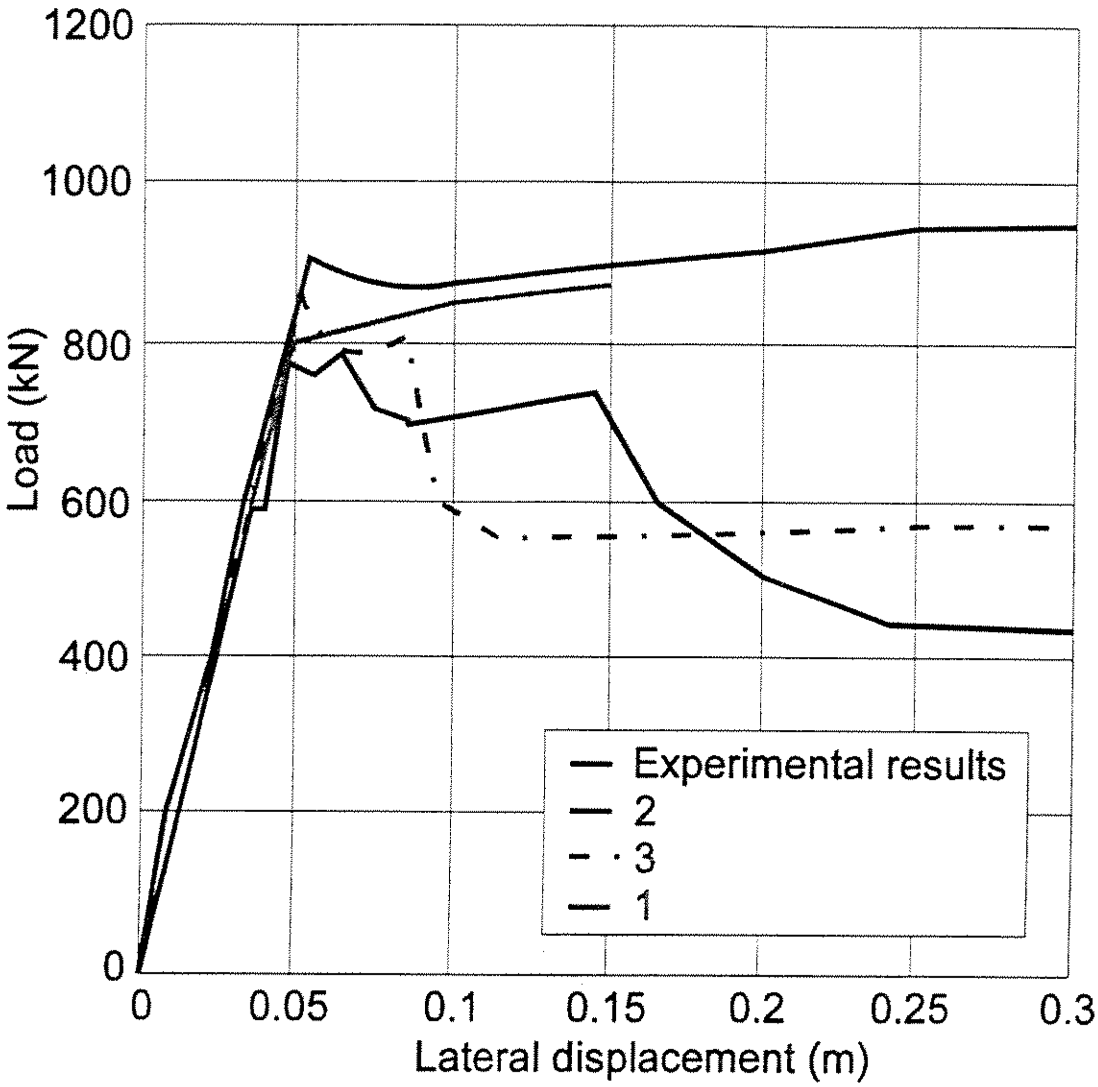


Figure 9: Typical Global-Displacement Features for Test Case 3

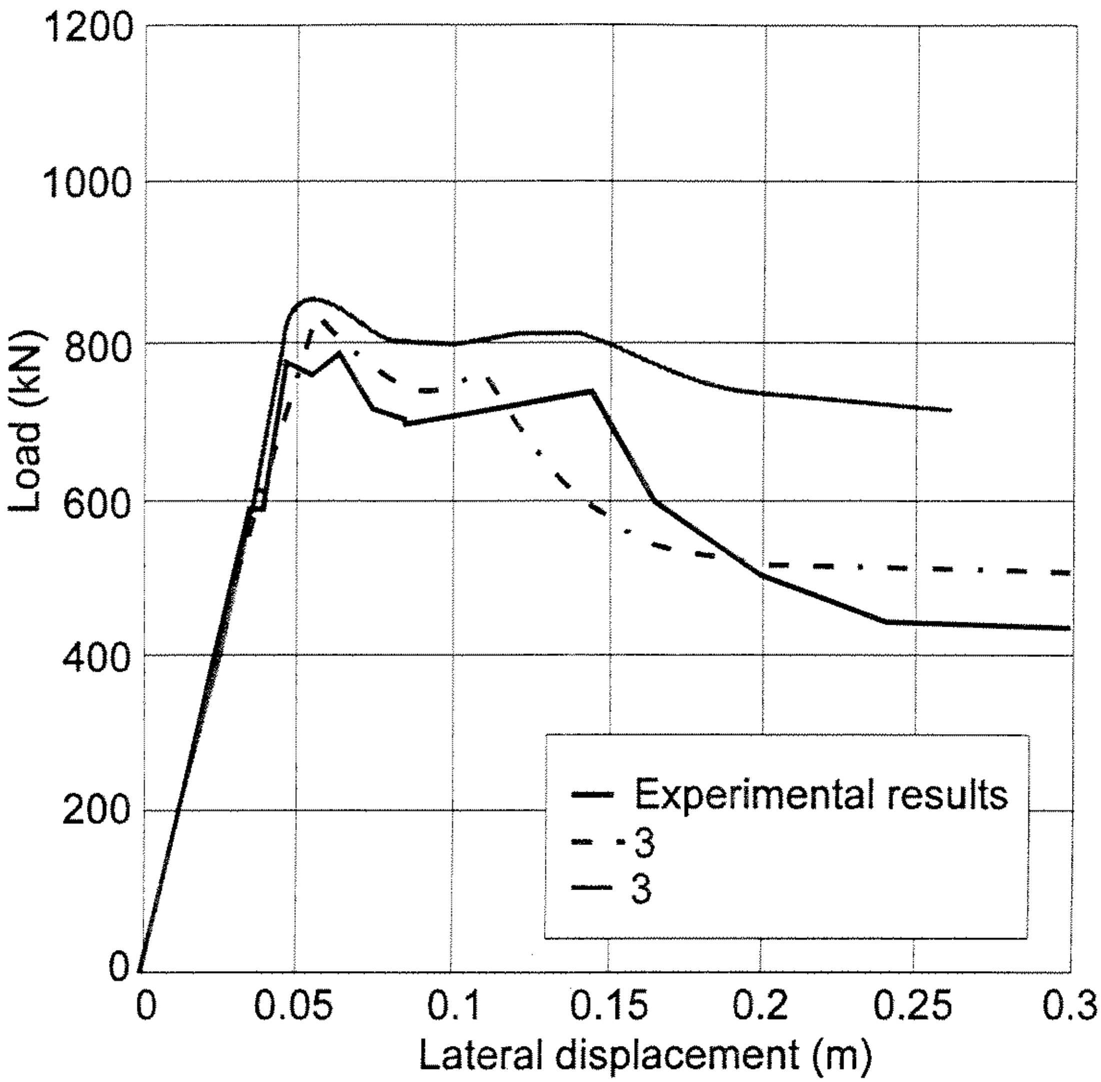


Figure 10: Typical Global-Displacement Features using Same FE Software for Test Case 3

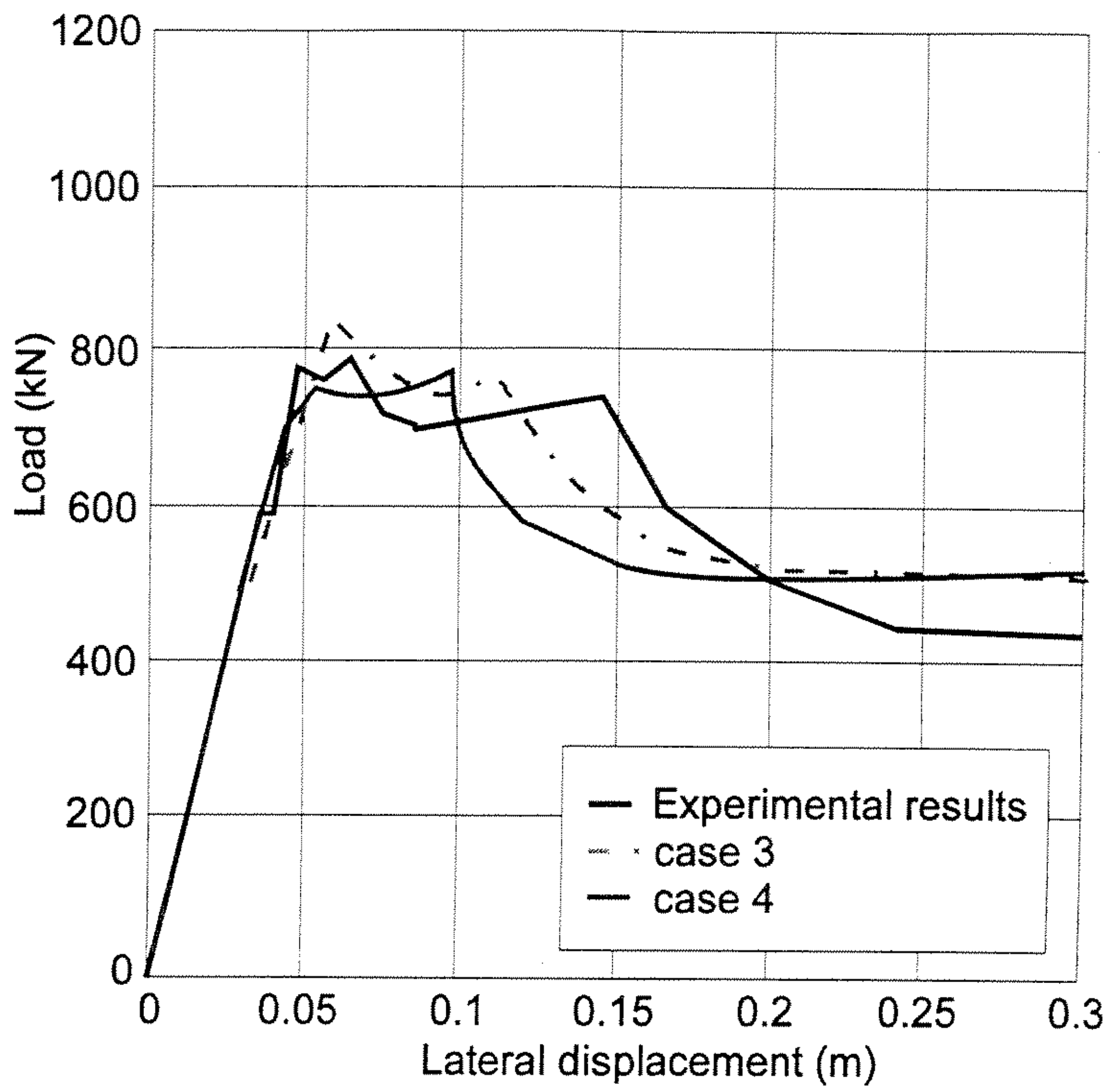


Figure 11: Typical Global-Displacement Features of Test Case 3 vs Test Case 4

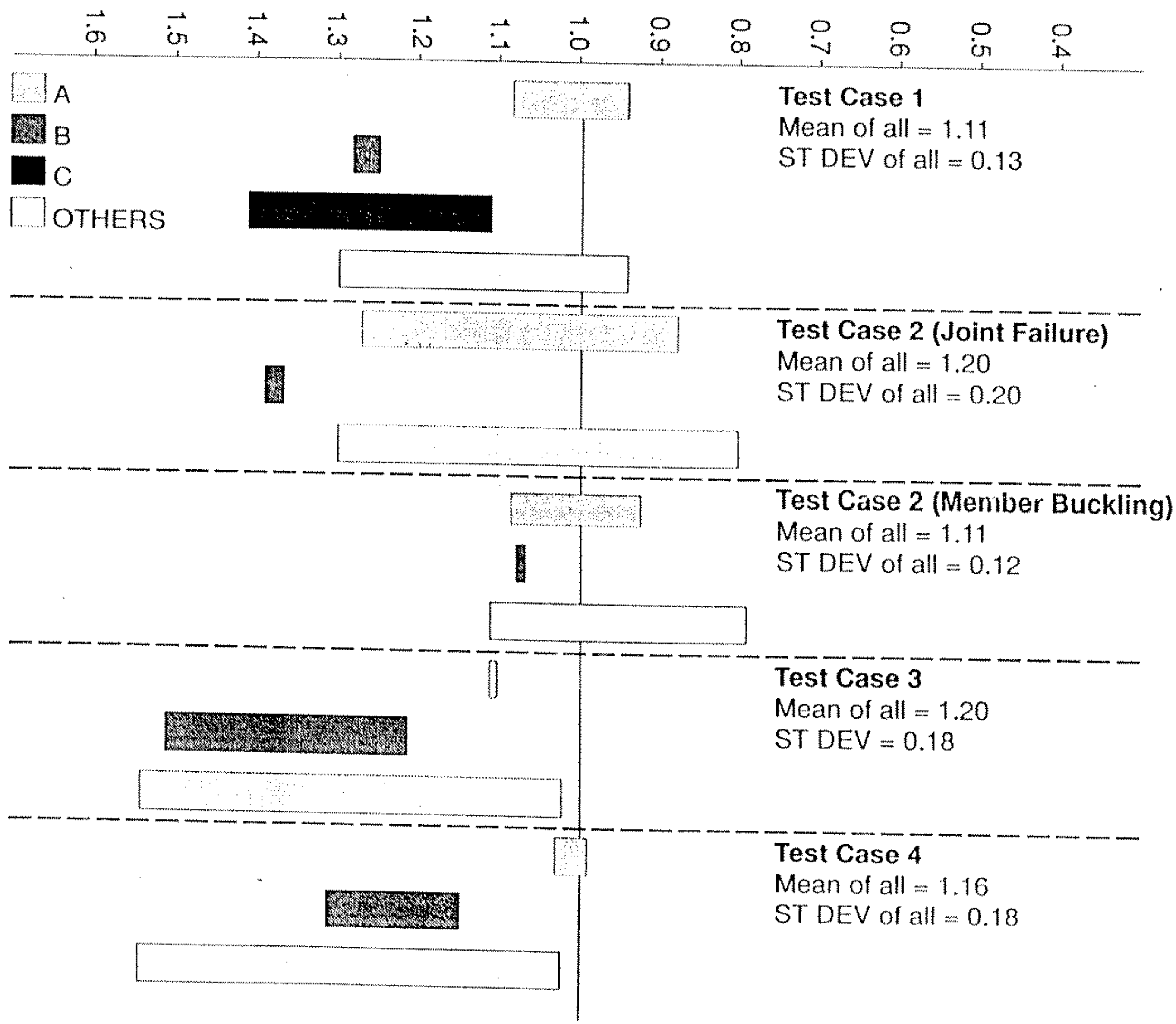


FIGURE 12 : Ratio of Predicted V Experimental Reserve Strength Factors

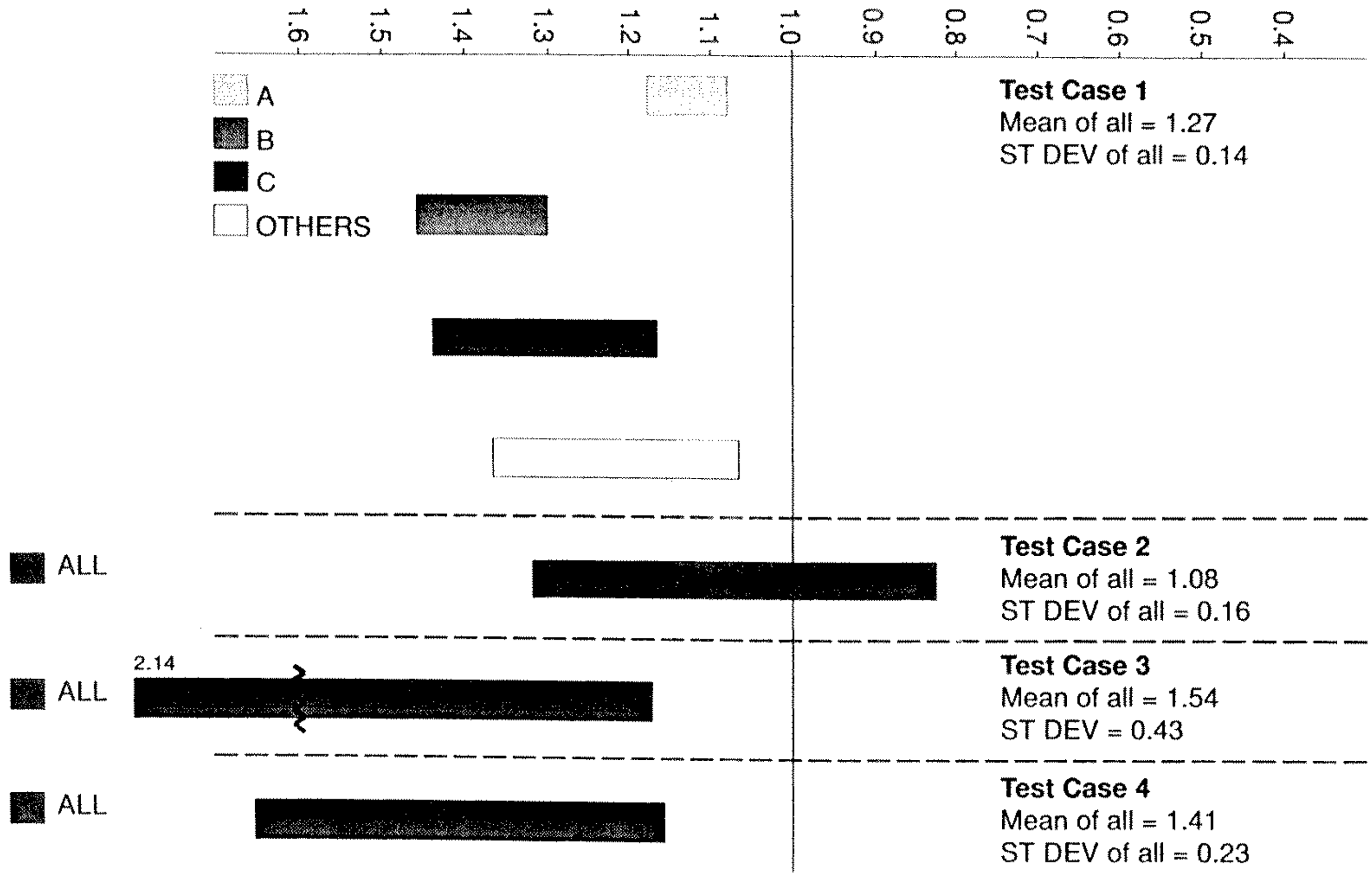


Figure 13 : Ratio of Predicted V Experimental (Reserve x Residual) Strength Factors