

**MINERALS MANAGEMENT SERVICE
DEPARTMENT OF INTERIOR
HERNDON, VIRGINIA**

POST MORTEM PLATFORM FAILURE EVALUATION STUDY

CONTRACT NO. 14-35-0001-30747

FINAL REPORT

93203-FR-01

JANUARY 1995

**INTERNATIONAL DESIGN, ENGINEERING AND ANALYSIS
SERVICES (I.D.E.A.S.), INC.**

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1. INTRODUCTION

1.1 BACKGROUND

In response to a Minerals Management Service (MMS) issued Broad Agency Announcement in 1993 a proposal was submitted to perform a Post Mortem Platform Failure Evaluation Study. International Design, Engineering and Analysis Services (I.D.E.A.S.), Inc. received a contract from MMS to perform the study with the following objectives in mind:

- To understand why some of the structures in the path of Hurricane Andrew were damaged or destroyed while other seemingly similar structures survived.
- To evaluate platform component capacity uncertainties and to develop a practical methodology to determine the likelihood of damage to or destruction of platforms to be subjected to extreme loads associated with future hurricanes.

1.2 SCOPE OF WORK

1.2.1 Original Work Scope

The work scope compatible with project objectives was first defined as:

- Gather available data on platforms subjected to extreme environmental loads.
- Assess platform response and component system (i.e., members, joints and foundations) behavior.
- Develop a parametric study program to evaluate sensitivity and interaction of primary parameters.
- Evaluate potential failure mechanisms and their effect on platform response characteristics and platform reserve strength.
- Prepare figures, charts and engineering aids to facilitate understanding of likely failure mechanisms and determination of corrective measures.
- Select a platform, develop a detailed computer model and perform nonlinear "Push-Over" analysis to validate parametric study results.

1.2.2 Revised and Expanded Work Scope

An API Task Group was active at the beginning of this project and issued a Draft Section 17 of API RP 2A (Reference 1) for platform requalification, necessitating careful scrutiny of the document. Initial failure mechanisms and parametric sensitivity evaluations indicated that interactions between the parameters were sensitive to platform geometry, requiring application of findings on several different platforms. Complexity of parameters and the significant effects of assumptions, modeling and methodology on the outcome

of platform reserve and residual capacities indicated the need to prepare a "Guidelines" document.

Thus, the work scope was revised and expanded to ensure that project objectives were met by:

- (1) Implementing the just-issued Draft Section 17 of API RP 2A for platform requalification on a Gulf of Mexico platform.
- (2) Expanding assessment of the component failure mechanisms to include platforms subjected to seismic loading.
- (3) Selection, modeling and complete nonlinear "Push Over" analysis of not one, but four platforms to validate applicability of findings on platform configurations varying from a simple flexible four-legged structure to a complex structure consisting of two eight legged platforms joined at the deck.
- (4) Developing a separate "Guidelines" document to be used in the assessment of a "Platform Requalification/Analysis Report."

2. SUMMARY

2.1 POST MORTEM FAILURE EVALUATION STUDY

Platforms in the path of Hurricane Andrew were reviewed to first determine why some platforms failed while others did not. Essentially all toppled over or structurally damaged platforms were found to be of first generation (pre-1969) design having inadequate capacity to resist Hurricane Andrew applied loads. The cumulative effect of corrosion, dents and fatigue also contributed to the reduced capability of these platforms to resist applied hurricane-level loads.

This conclusion is validated by those first generation platforms that survived Hurricane Andrew because of mitigative actions previously taken, such as:

- strengthening of leg joints by grouting the annulus between the legs and the piles.
- removing non-functional conductors and relocating lower deck equipment to upper decks minimize applied loading.

Review of damage and failure modes indicated tubular member local and column buckling, tubular member ends collapsing at the joint chord or either tearing away or punching into the chord, weld brittle failures at the joint and foundation pile/soil failures.

These findings allowed finalization of the parametric study work scope and narrowed the range on non-linear platform pushover analyses necessary to validate and/or contradict the findings.

2.2 IN-DEPTH PARAMETRIC STUDIES

Variations in platform ultimate load capacity were assessed by evaluating a number of parameters using nonlinear pushover analyses of specific platforms. While the quantitative results may vary for other platforms, the qualitative conclusions are assumed to hold for typical platform configurations. The parameters studied were:

- **Members**
 - brace effective slenderness ratios
 - brace member post-buckling capacities
 - tension member hydrostatic capacity reduction
- **Joints**
 - joint punching shear capacity
 - joint crushing capacity
 - joint flexibility
- **Foundations**
 - soil lateral, skin-friction and end-bearing resistance

As expected, those parameters affecting member, joint or foundation capacity had the most significant effect on platform ultimate capacity.

The most critical components affecting platform ultimate capacity were the foundation soil properties, the member post-buckling capacity, and the joint punching shear and/or crushing capacity. These parameters directly affect component capacity. In the case of soil properties, the platform ultimate capacity was highly sensitive to variations in assumed soil properties, probably because the number of elements is few with an associated lack of redundancy. In the case of joints, reduced joint capacities may substantially limit the loads carried by higher capacity adjacent braces, thereby limiting the ultimate load carrying capacity of the platform.

Of secondary importance are the tension member hydrostatic capacity reduction and brace effective slenderness ratios. Tension braces are also designed as compression braces with lower capacity so that a reduction in tension capacity does not reduce the ultimate capacity of the platform. Decreasing the brace effective slenderness ratio has only a minor effect on platform ultimate capacity for mild steels and relatively low L/D ratios. The effect would be more pronounced for more slender brace members. In any case, decreasing the brace effective slenderness is not conservative, will not gain much platform capacity, if any, and is better ignored, at least in an initial assessment.

Of little or no importance are joint flexibility considerations, at least with respect to relatively symmetrical platform configurations. Joint flexibility may be important for relatively weak joints and stocky braces or where the load distribution in the platform framing is significantly biased.

For Gulf of Mexico platforms in relatively soft soils, the foundation elements (i.e. piles and soil) can be expected to be the critical component limiting the platform ultimate capacity. The significance of determining the actual soil and pile properties and properly incorporating them into a requalification analysis cannot be overemphasized.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 PLATFORM RESPONSE AND FAILURE MODES

3.1.1 Global Platform Response

A large number of platforms in the path of Hurricane Andrew did not suffer structural damage while a significant number of platforms toppled over and others suffered moderate-to-extensive damage. Some of these structures could not be repaired and, therefore, were removed.

It is important to note that all of the platforms lost were of first generation design (prior to 1969), indicating that the primary reasons for this loss are:

- older platforms were designed to resist smaller wave loads than what is considered appropriate today, often not accounting for combined wave and current action,
- older platforms were designed with inadequate deck height, resulting in wave crests hitting the deck and generating loads unaccounted for in the design, and
- older platforms exhibit degradation in their ability to resist applied loading due to the combined effect of corrosion, dents and fatigue.

Amoco, Chevron and other operators recognized this dilemma in the 1980s and strengthened numerous platforms to meet current requirements. Survival of these repaired structures has validated these conclusions.

3.1.2 Observed Failure Modes

The most common cause of platform toppling is foundation failure. Some failures were initiated due to soil material failure and others were initiated due to formation of multiple plastic hinges on piles supporting the platform.

Another form of platform toppling is due to progressive failure of platform braces and joints until the platform stiffness was reduced to a level initiating system failure.

Observed structural damage includes local and column collapse of tubular braces, tearing of the brace away from the joint chord, punching of the brace through the joint chord, weld brittle fractures and flooding of the buoyant tubulars.

3.1.3 Analytical Study Results

The extensive analytical studies performed as a part of this study were compatible with observed failure modes and exhibited damage. Parametric studies provided adequate data to identify the primary and secondary failure parameters and allowed development of useful formulations and figures to facilitate determination of component capacities and the likelihood of platform failure. Some of the key conclusions are:

- 1) The use of effective length factor ("k") of 0.80 for brace members instead of a more realistic range of 0.55 to 0.65 is conservative. However, the overall effect on the platform capacity is minimal and considering member degradation due to corrosion, dents, etc., a "k" factor of 0.80 is appropriate for design and platform requalification.
- 2) Identification of platform tubular member failures is not adequate. The ability to define post-buckling capacities of members is essential. Removing the failed member (i.e., post-buckling capacity of zero) is too conservative and incorrect. A post-buckling capacity equal to 25 to 50 percent of the original member capacity may be appropriate.
- 3) Joints often do not have adequate capacity and may initiate progressive collapse of the platform. A quick and reliable approach was developed to identify this class of joints.
- 4) Items 1) through 3) above may often be immaterial as the critical factor is often the foundation soil and/or pile failure. Platforms are very sensitive to both the actual foundation material response and the analytical model used to simulate the linear and non-linear foundation response in terms of p-y, t-z and q-z soil resistance and the modeled pile segments.

3.2 DESIGN CRITERIA AND APPLIED LOADS

The platform should be assessed for its sensitivity to metocean criteria. Significant hydrodynamic load collecting structure or appurtenances should be appropriately accounted for in the requalification assessment. Of particular importance are boat landings, fenders, conductors, caissons and sumps. The blockage and diffraction coefficients recommended by API Draft Section 17 should be reviewed for their applicability to such components.

Marginal structures that are governed by metocean criteria should be further assessed as to their sensitivity to slight variations in assumed wave period and wave height.

A requalification analysis of a platform controlled by seismic loading should be based on site specific seismic response spectra. Marginal platforms may require further seismological investigation to establish any increases or decreases in the response spectra as a function of platform location and seismic response direction.

3.3 STRUCTURAL FRAMING SYSTEM AND LOAD PATHS

A platform subject to requalification assessment should be reviewed with respect to its framing system and associated load paths. On a qualitative basis and in approximately decreasing order of importance, the more critical structures will have the following properties:

- relatively soft soil lateral resistance on piles
- three or four platform legs with an associated number of vertical framing bays

- joint designs not in accordance with present-day API RP 2A (Reference 2) recommendations
- non-triangulated or non-trussed framing patterns
- significantly damaged members and/or joints
- significant member corrosion
- non-redundant framing patterns (e.g., k-bracing)

Platforms having one or more of the above properties should be assessed with particular care. Consideration of component reduction parameters as discussed elsewhere in this study should be mandatory.

3.4 AS-DESIGNED AND IN-SERVICE MEMBER AND JOINT CAPACITIES

3.4.1 As-Designed and In-Service Member Capacities

It is concluded that member post-buckling capacity must be considered in any requalification analysis and that the post-buckling reduction factor should preferably be quantified on a member by member basis. If this is not feasible, then the post-buckling reduction factor should be no greater than 0.5, unless a higher factor can somehow be justified. Further work would be desirable to more accurately quantify appropriate member post-buckling reduction factors and their effects on platform ultimate capacity as a function of the member effective length factor ("k"), brace geometry (e.g., D/t and L/D) and P-delta effects.

It is concluded that a reduction in member tension capacity for buoyant members due to hydrostatic pressure must be considered in any requalification analysis, especially for members with D/t exceeding 30 and/or under heads in excess of 200 feet. Reductions in member compression capacity need to be considered if the original design did not consider hydrostatic pressure interaction in accordance with API RP 2A.

For platform capacities controlled by brace member compression, additional capacity may be obtained by using actual "k" factors for determining member effective slenderness ratios. The increase in capacity will be greater for increasing brace member yield strengths and for increasing brace member L/r ratios. The use of API RP 2A recommended brace "k" values (e.g., 0.8) is conservative.

3.4.2 As-Designed and In-Service Joint Capacities

It is concluded that the effect of joint capacity must be considered in any requalification analysis where joint capacity does not exceed the brace member capacity. In these instances, the lower joint capacity effectively limits the maximum brace load and potentially reduces the ultimate capacity of the platform.

It is concluded that the actual joint stiffness for simple tubular joints need only be considered for non-symmetric or relatively non-redundant structures, where stiffness effects could result in a redistribution of forces in the platform, with a resulting variation in platform ultimate capacity. Stiffened or otherwise complex joints should be specifically addressed. Including simple tubular joint stiffness effects for four-legged platforms is

recommended since the failure of any one joint or member would cause a more distinct non-symmetrical redistribution of forces than for more redundant structures.

3.5 AS-DESIGNED AND IN-SERVICE FOUNDATION CHARACTERISTICS

It is concluded that foundation characteristics have a major, and often the most significant, impact on platform response and ultimate capacity, especially for Gulf of Mexico platforms in soft soils that have relatively low lateral and axial stiffness. Marginal platforms should be further assessed as to their sensitivity to assumed soil properties and additional soil data obtained, as necessary, to establish the acceptability of the platform.

Any requalification analysis should be based on actual soil and pile properties where possible. Variations in pile size with depth should also be accounted for; however, averaging or "smearing" techniques are generally insufficient. The nonlinear soil response must be incorporated into the requalification analysis; a linear spring representation will be insufficient for determining platform response and can conservatively or unconservatively predict platform component failures in the vicinity of the foundation.

It is particularly important that nonlinear elements used to model piled foundations correctly account for the following:

- variations in pile cross-section with depth
- variations in soil lateral resistance with loading and depth
- variations in soil skin friction and end-bearing resistance with loading and depth
- pile loading and displacement response with depth

This implies the use of multiple elements to model each pile, if a single element does not capture pile load, resistance and response at multiple locations within the element. The pile non-linear lateral response will be particularly sensitive to these effects within about 10 pile diameters of mudline, so special care should be taken to accurately model the pile response in this area. Pile axial response is distributed along the entire pile length, so distributed modeling of these effects is also required.

3.6 RECOMMENDATIONS

3.6.1 Requalification Analyses

A proper nonlinear analysis used in a platform requalification assessment needs to account for a number of member, joint and foundation considerations. The following recommendations are made with respect to requalification assessment and associated nonlinear analysis procedures:

- Incorporation of reduced joint capacities, where applicable, should be required of all nonlinear analysis assessments, unless it can be demonstrated that the resultant minimum RSR, excluding foundation effects, exceeds a specified value that is significantly higher than that required to requalify a platform. The incorporation of reduced joint capacities should definitely be required in the

assessment of marginal platforms.

- Incorporation of reduced member post-buckling capacities should be required in all nonlinear analysis assessments, unless it can be demonstrated that the resultant minimum RSR, excluding foundation effects, exceeds a specified value that is significantly higher than that required to requalify a platform. Including post-buckling capacities should definitely be required in the assessment of marginal platforms. A standardized methodology for developing post-buckling capacities should be developed for industry use.
- Incorporation of the actual detailed nonlinear response characteristics of the foundation, including actual soil p-y, t-z, and q-z data, and pile member variation with length, should be required in all nonlinear analysis assessments, unless it can be demonstrated that the resultant minimum RSR, excluding platform member and joint effects, exceeds a specified value that is significantly higher than that required to requalify a platform. Significant detail in both modeling and response should be required along the entire pile length, with increasing detail for the upper portion of the piled foundation.

3.6.2 Further Work

This study has demonstrated that predicted platform ultimate capacity is highly sensitive to assumed soil/pile properties, and joint and member capacities. Work by others has also demonstrated that predicted platform ultimate capacity and displacement response may vary significantly among various operators, engineering contractors and consultants. Since a requalification assessment using nonlinear analysis techniques is most likely to be used for marginal platforms, it is important that the significant parameters identified in this study be further quantified and standardized as much as possible for industry use. In some cases, this will involve a literature search of published research on these topics. In other cases, this will involve further analytical work to establish RSR values at which these significant parameters would have little or no effect on the conclusion of a requalification assessment.

The following specific recommendations are made with respect to further work on this subject:

- Joint capacities can be reasonably established based on the joint punching shear and crushing resistance capacity established by API RP 2A. However, applicable joint capacities subsequent to attaining these limits need to be quantified and standardized for industry use. It is recommended that all research data on this topic, including those funded by operators and government agencies, be gathered and analyzed for their impact on a requalification assessment. An industry standard should then be developed, perhaps in conjunction with the API Task Group on Platform Requalification.
- Member post-buckling capacities are a function of brace geometry and "P-delta" effects. A standard post-buckling assessment procedure should be developed and standardized for industry use. It is recommended that all research data on this topic, including those funded by operators and government agencies, be gathered and analyzed for their impact on a requalification assessment. An

Industry standard should then be developed, perhaps in conjunction with the API Task Group on Platform Requalification.

- The requalification assessment of marginal platforms will require the detailed nonlinear assessment of some or all of the significant component parameters discussed in this study. RSR levels which preclude such consideration should be established to facilitate the requalification process. It is recommended that further parametric analysis be done to establish RSR levels at which such detailed consideration can be neglected without changing the conclusion of a platform requalification assessment.

4. POST MORTEM PLATFORM FAILURE ASSESSMENT

4.1 PLATFORM SELECTION BASIS

Numerous platforms in the path of Hurricane Andrew were not damaged. Yet, a substantial number of platforms either toppled over or suffered appreciable damage. An initial review of available data indicated that all of the platforms that toppled over or damaged were of first generation design (prior to 1969), indicating that:

- Platforms were designed to meet environmental loads smaller than what is considered appropriate today.
- Platforms may have exhibited reduced load carrying capacities due to the cumulative effects of component degradation due to in-service corrosion, dents and fatigue.

Since it is difficult to determine potential component degradation without extensive inspection, emphasis was placed on analytical effort to determine the expected behavior of platforms in "pristine" condition. To ensure that a realistic range of platforms was covered effectively, the following criteria were used to select the platforms for this study:

- Both damaged and undamaged platforms.
- Platforms designed to meet pre-1977 API criteria.
- Substantially different platform configurations.
- Operator's willingness to provide platform data.

Several Operators with platforms in the path of Hurricane Andrew and having characteristics considered compatible with the study criteria were contacted to solicit their support. All of the Operators contacted were extremely helpful in discussing the characteristics of their platforms and in their willingness to provide data. Drawings, design criteria and other pertinent data were obtained for the platforms listed on Table 4.1-1 and these platforms were selected for further assessment of platform response and failure modes.

Table 4.1-1 Platforms Selected for Assessment

OPERATOR	PLATFORM	COMMENTS
AMOCO Production Co.	ST 161A WD 90A	South Timbalier West Delta
CHEVRON U.S.A.	ST 130A	South Timbalier
CONOCO Inc.	GI 47C	Grand Isle
PACIFIC OPERATORS OFFSHORE	Hogan	
TRUNKLINE GAS COMPANY	ST 72 (T-21) ST 52 (T-23) SS 139 (T-25)	South Timbalier South Timbalier Ship Shoal

It should be noted that although Hogan, an offshore California platform, was nearly two thousand miles away from the path of Hurricane Andrew it was added to this study to ensure that findings would be equally applicable to structures designed and installed primarily to resist earthquake loads.

We appreciate the cooperation received from the following personnel and wish to acknowledge their support of this project:

- Mr. James M. Light - AMOCO Production Co., Tulsa, Oklahoma
Mr. Gary M. Imm - AMOCO Production Co., Tulsa, Oklahoma
- Mr. William F. Krieger - CHEVRON U.S.A., Inc., San Ramon, California
Dr. Jen-Hwa Chen - CHEVRON Petroleum Technology Co., La Habra, California
- Mr. P.H. Kwok - CONOCO Inc., Houston, Texas
Dr. H. Chong Rhee - CONOCO Inc, Houston, Texas
- Mr. Scott Hulsey - PACIFIC OPERATORS OFFSHORE Inc., Ventura, California
- Mr. Kris A. Digre - SHELL OIL Co., Houston, Texas
- Mr. J.E. Meyer -TRUNKLINE GAS Co., Houston, Texas

4.2 ASSESSMENT OF PLATFORM RESPONSE AND FAILURE MODES

Some of the platforms in the direct path of Hurricane Andrew either collapsed or were heavily damaged while some of the platforms did not suffer any structural damage. Platform collapse or heavy damage is reasonable as the applied environmental loads on these older platforms were substantially higher than the loads they were originally designed for. Other platforms at some distance from Hurricane Andrew's path suffered either minor damage or no damage at all. This outcome was expected as the applied environmental loads would be equal to or less than the design loads.

A brief discussion of undamaged and damaged/collapsed platforms follows.

4.2.1 Undamaged Platforms

Platforms suffering no structural damage, with or without nonstructural damage, are identified as "Undamaged Platforms." Some of the platforms in the direct path of Hurricane Andrew and expected to be damaged and/or toppled survived. Generally, two explanations are available for their performance:

- (1) Platforms had adequate deck height and the environmental loads applied, though significantly greater than design loads, did not exceed the design loads times the safety factors used in the design.
- (2) Platforms did not have adequate deck height and the environmental loads applied on the platform and a part of the deck were greater than design loads times the safety factors used in design. However, these structures escaped damage

because such damage was expected and the structures were strengthened prior to Hurricane Andrew.

Platform strengthening (i.e., increasing its capacity to resist environmental loads) was typically achieved by performing one or more of the following:

- Strengthening of leg joints by grouting of the annulus in-between platform legs and piles.
- Strengthening of above-water joints by introducing external ring-stiffeners.
- Enhancing platform capacity by removing all inactive wells to reduce applied loads.
- Enhancing platform capacity by relocating lower deck equipment to the upper deck to reduce applied loads.

4.2.2 Damaged/Collapsed Platforms

Platforms toppling over or suffering structural damage are identified as "Damaged Platforms." Minor damage observed on some of the strengthened platform joints indicate environmental load levels at joint capacity. Typically, unstiffened and stiffened joint damage falls into several categories:

- Tubular brace member punching through the leg chord (i.e., punching shear).
- Tubular brace member separating from leg chord (i.e., tear).
- Joint deformation and local buckling (i.e., collapse).
- Fatigue crack growth resulting in flooding of buoyant joints/members.

Earlier joint designs are generally inadequate, often not even meeting the current API requirement stating that the joints shall have the capacity to resist, as a minimum, 50% of attached tubular member capacity. Punching shear, tear and collapse failures can be prevented by increasing chord wall thickness and/or introduction of ring stiffeners and gusset plates during design. In-service, the simplest method to increase the joint capacity is to grout it. Figure 4.2-1 illustrates K-, T- and X-Joints and potential failure modes.

The tubular brace members generally have excess capacity to resist applied environmental loads greater than design loads. This is primarily due to:

- Effective slenderness factors (i.e., "k") of 0.8 and 0.9 are typically used during design. However, end restraints on tubular brace members spanning between platform legs result in actual "k" values varying from about 0.56 to 0.65. Thus, the tubular members will have built-in reserve capacity.
- Design load conditions are selected to maximize applied loading on each member on one frame. However, platform symmetry is maintained and excess capacity

is provided.

Thus, tubular brace failures are usually associated with additional bending stresses introduced by attachments or reduced member capacities due to dents resulting from impact damage.

The seafloor soil supporting platforms in the path of Hurricane Andrew consists mainly of normally consolidated silty clay. Undrained shear strength is the key soil parameter affecting pile response to applied loading. A platform may topple over due to foundation failure as observed in a number of instances. However, the piles would most likely overload, form plastic hinges and cause the platform to tilt in one direction or another.

Unequal pile capacities, either due to non-uniform soil resistance or due to pile damage during installation, can cause the structure to tear apart and collapse. Even when the collapse does not occur, the effects of load redistribution will result in an overload of some of the platform members and joints and potentially cause their failure.

4.3 CONCLUSIONS

A platform collapse or a failure of a component is due to :

- applied loads exceeding design loads
- predicted component capacities exceeding actual component capacities
- inadequate original predicted component capacities based on present methodology and information

The key parameters for failure, in order of importance, were observed to be foundations, joints and members. Member local and column bucklings were observed and post-buckling capacities allowed the structure to continue resisting applied loads. Joint failures, due to punching, crushing and tearing would result in a loss of joint stiffness and contribute to a progressive collapse mechanism.

Formation of plastic hinges at the platform/pile interfaces has the greatest impact on the survivability of the platform.

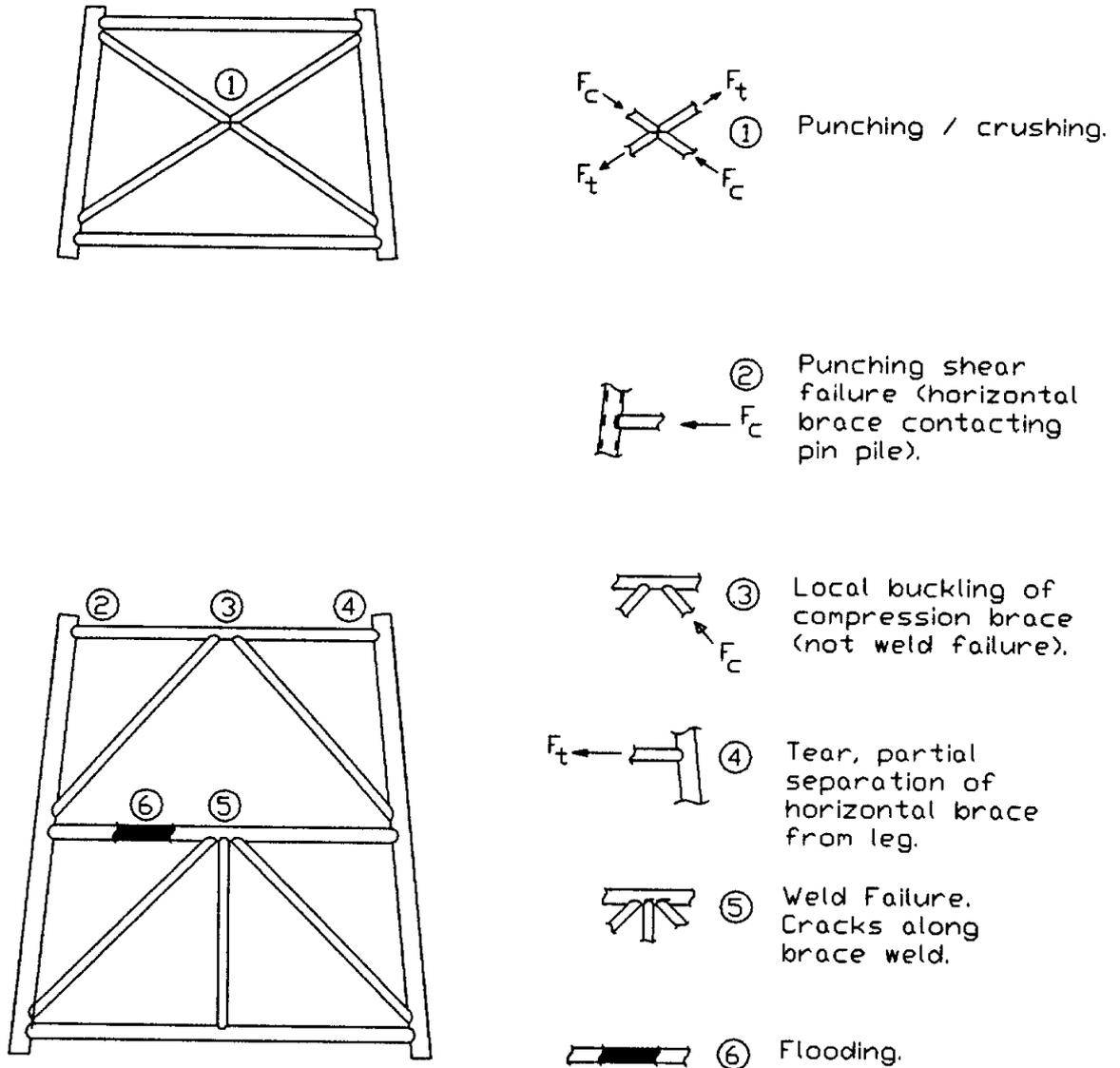


Figure 4.2-1
K-, T- and X-Joint Potential Failure Modes

5. DEVELOPMENT OF PARAMETRIC STUDY PROGRAM

5.1 DEVELOPMENT OF PARAMETRIC STUDY SCOPE

Post Mortem Platform Failure assessment work has demonstrated that platform response characteristics are dramatically affected by platform configuration, including number of legs, bracing system and foundation layout. However, even two seemingly similar structures may have different component failure sequences, collapse mechanism and reserve strength ratios due to:

- Differences in design criteria, including different environmental criteria and methodology, resulting in different applied loads.
- Differences in design methodology, resulting in different member, joint and foundation capacities/characteristics.
- Differences in as-designed and as-installed structure, including material and geometric imperfections and residual stresses introduced at the yard, as well as the damage to the platform during both pre-service (loadout, towing, launch and upending) and in-service (operating) conditions.
- Differences in in-service structure maintenance, repairs and modifications.
- Differences in in-service structure degradation due to corrosion and fatigue.

In addition, it is necessary to address the platform structural system effect on platform response. The number of legs, bracing system, and foundation layout directly influence platform redundancy, load path and reserve strength.

Thus, the work scope needs to include review of all pertinent parameters and the study of those parameters contributing to failure mechanisms.

5.2 STUDY EXECUTION PLAN

A simple screening method was implemented to determine those parameters not suitable for an in-depth analytical study. The following parameters were considered too difficult to document and reliably address:

- Imperfections and residual stresses introduced during fabrication.
- Dents introduced due to dropped objects.
- Degradation of members and joints due to in-service corrosion and fatigue.

Thus, for the purposes of this study, platforms were assumed to be in "pristine" condition and would exhibit the characteristics of "as-designed" structures. The primary parameters studied are grouped into platform component members, joints and foundations. Table 5.2-1 presents the specifics of the parameters addressed.

Table 5.2-1 Parameters Studied

COMPONENT	PARAMETER	COMMENTS
1. MEMBERS	Member Slenderness	End rigidities and "k" values (Theoretical vs. API recommended "k" values)
	Member Post-Buckling Capacity	(%) of initial capacity (0, 25, 50, 75 and 100%)
2. JOINTS	Joint Flexibility	Joint deformations (rigid vs. nonrigid joints)
	Joint Capacity	Punching vs. crushing considerations (Rigid joints, 100% and 50% of API punching capacities)
3. FOUNDATIONS	Soil Capacity	Soil skin friction ("t-z") and end bearing ("q-z") Soil lateral resistance ("p-y") (50, 100, 200% of soil capacities)
	Pile Flexibility	Pile cross-section variations

The joint, member, and foundation parameters relate to component capacity and/or stiffness. All of the above parameters relate directly to component capacity except for joint flexibility and pile flexibility. These excepted parameters may relate to component capacity for some types of structures. For example, the joint and pile flexibilities will definitely affect the displacement-load response of a platform but may only have a minor effect on ultimate capacity.

A proper requalification analysis will produce a correct platform response and ultimate capacity, whereas, an improper but adequate analysis will produce only the correct ultimate capacity. An improper and inadequate analysis will produce incorrect results on both counts. Thus, one objective of this parametric study is to determine the parameters for which the proper and/or adequate analyses are the most sensitive.

A careful literature survey was conducted prior to performing parametric studies. This included:

- All papers covering the work performed by the API Task Group responsible for development of API RP 2A Draft Section 17 (References 3 through 9).
- Other papers covering platform structure and component member capacities, damaged member capacities and analytical methods (References 10 through 12).

5.3 MODELING OF COMPONENTS AND SELECTED PLATFORMS

Initial parametric studies were performed to study the effect of modifying one component on itself, on adjacent components and the overall system. Sketches of various framing systems were drawn, free-body diagrams developed and the component studied to determine realistic upper- and lower-bound response.

Computer models, ranging from simple frames to entire platforms, were developed to study the effect of varying one parameter on another. Parametric study findings and conclusions were verified by performing complete "Push Over" analyses of structures having significantly different geometries and response characteristics.

The computer models of the following structures were developed and used to validate one or more of the parameters under investigation:

- API JIP "Benchmark" Platform (see Figure 5.3-1).
- Conoco's "Grand Isle" Platform (see Figure 5.3-2).
- Pacific Operators Offshore's "Hogan" Platform (see Figure 5.3-3).
- Trunkline Gas Company's "South Timbalier ST52" Platform (see Figure 5.3-4).

Work performed in this parametric study and assessment of failure mechanisms includes assessment of parameters affecting the behavior of component members, joints and the foundations. Extreme storm and earthquake loads were generated and the platform components evaluated. Then, ultimate strength level loads were generated and the loads applied on the platform increased incrementally until its collapse. Component members reaching capacity were tracked. Parametric studies included assessment of the primary variables controlling the ultimate capacity of component members, joints and foundations.

Tubular component failures were evaluated for different load combinations (i.e., axial tension and compression combined with bending and hydrostatic compression/tension). Variation of behavior for primary brace and leg components were evaluated separately from piles. Strain hardening effects and ductility of such components were assessed as to their effect on component capacity.

Critical joints not meeting the full capacity of the attached members can not be assumed to be rigid in a nonlinear analysis used to determine overall platform reserve strength. Deformation formulations were investigated to determine an effective approach to define interactions between component members and less-than-rigid joints. This effort covers typical Gulf of Mexico joint configurations as well as the more unusual larger diameter

joint chords.

One of the key components that can initiate the failure of a platform is the foundation system. Pile modeling alternatives as well as the foundation axial and lateral response characteristics were studied.

5.4 OTHER STUDIES

In 1994, a Joint Industry Project (JIP) was coordinated by PMB Engineering, Inc. (References 13 and 14) that:

- performed a trial application of the API Draft Section 17 guideline in its entirety for JIP participant selected platforms, and
- performed a trial application of the ultimate strength analysis procedure of the draft guideline to a common platform (i.e., "benchmark platform") by interested participants or other parties, in order to determine the variability in the ultimate strength analysis results.

Twelve organizations participated in the benchmark analysis. This work was performed on a voluntary basis and, thus, it is possible that the analyses may not have been performed to the same degree of precision or detail in all cases. However, the variability in results is indicative of the fact that not all pertinent factors in a nonlinear analysis are being considered by all participants.

The load-displacement results for a diagonal storm load direction are summarized in Figure 5.4-1. Two important points are immediately obvious:

- 1) The reported platform ultimate capacity varies from a high of about 3200 kips to a low of about 1500 kips, a variation of more than 100 percent.
- 2) The variation in initial stiffness, which is more or less linear, is on the order of 100 percent.

These are significant variations for a requalification assessment step that is intended to serve as final justification for platform requalification. The reasons for such variation need to be addressed and the large variability eliminated from future nonlinear analyses.

One reason for the variation in ultimate capacity may be incorrect modeling or accounting for the platform pile capacities. The assumed pile capacities were not reported by the participants in the JIP study. Assuming that the pile compression capacity is on the order of 3000 kips, the point of pile inflection is approximately 50 ft below mudline, the center of applied lateral load is approximately 130 ft above mudline, and a resisting lever arm of approximately 100 ft, the maximum platform lateral load based on overturning considerations would be about 1700 kips. The above numbers are only approximations but clearly show that the reported higher ultimate capacities are not feasible when the foundation capacities are included. Thus, it is extremely important that pile capacities be determined, documented and validated as a part of a requalification assessment.

Two reasons for the varying response stiffnesses would be differences in modeling techniques used for the foundation soil and pile characteristics and the accounting of joint flexibility in the analysis. Accounting for these effects will result in reduced overall platform stiffness; therefore, in the JIP study, the responses with less initial stiffness are probably (more correctly) accounting for these effects in their analyses. However, the stiffness variation is of secondary importance in a requalification assessment, since the load distribution in typical (relatively symmetric) platforms is not likely to change with assumed stiffness variation.

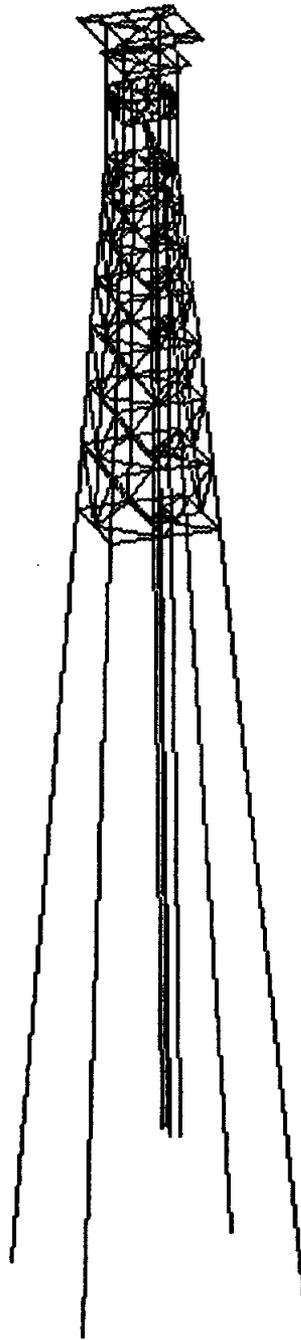


Figure 5.3-1
API JIP "Benchmark" Platform

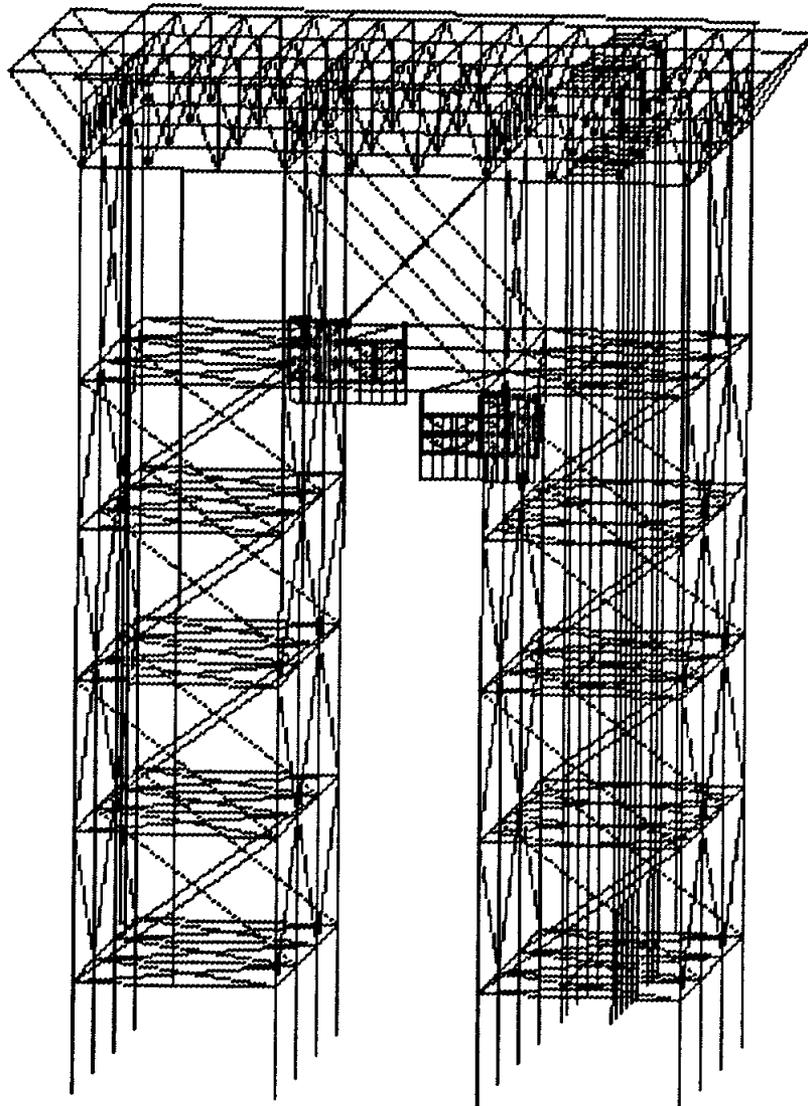


Figure 5.3-2
Conoco's Grand Isle Platform

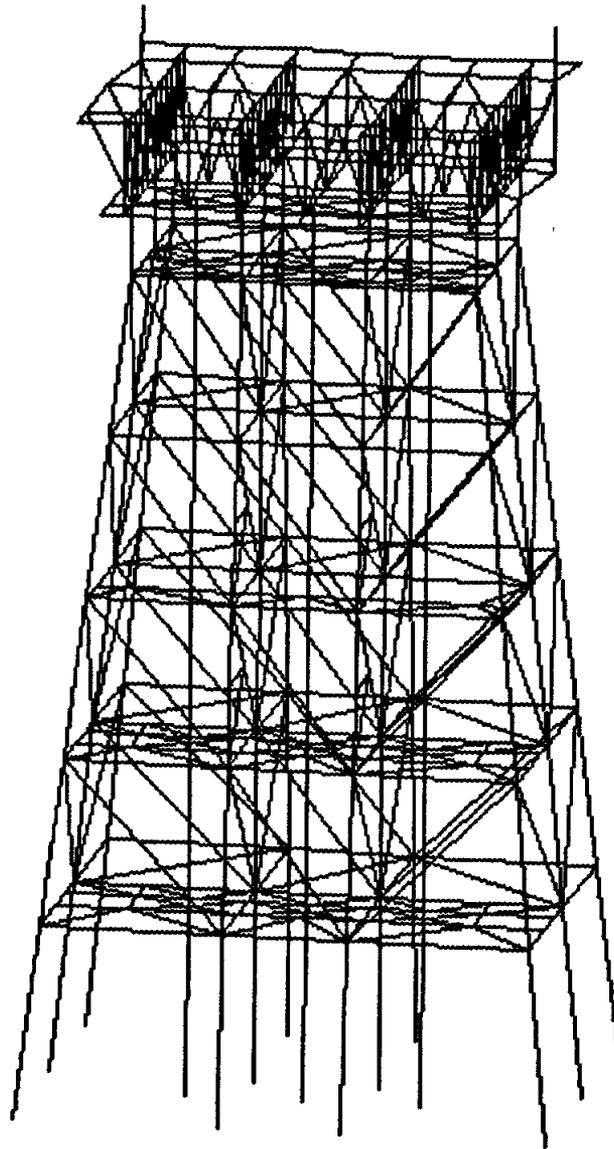


Figure 5.3-3
Pacific Operators Offshore's Hogan Platform

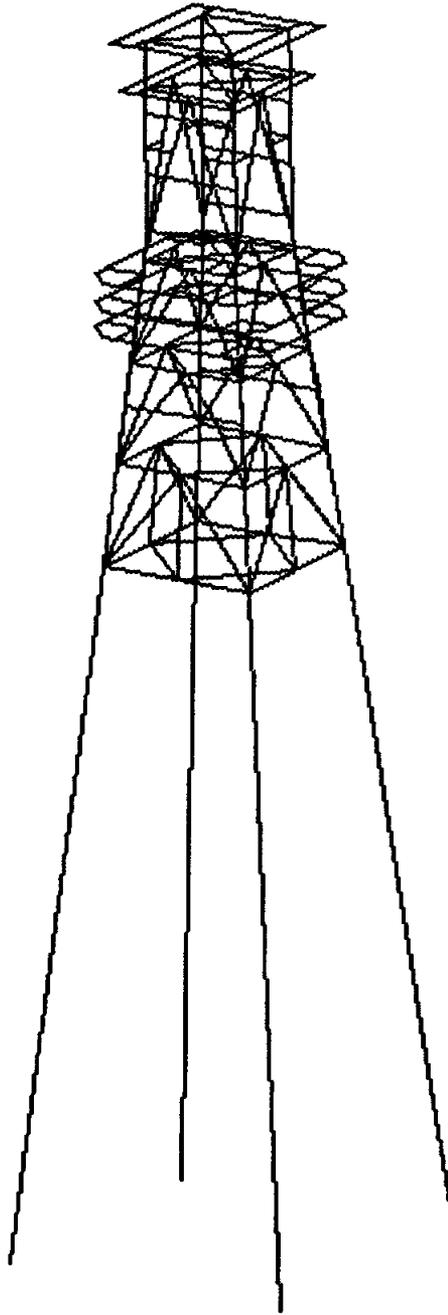


Figure 5.3-4
Trunkline Gas Company's South Timbalier ST52 Platform

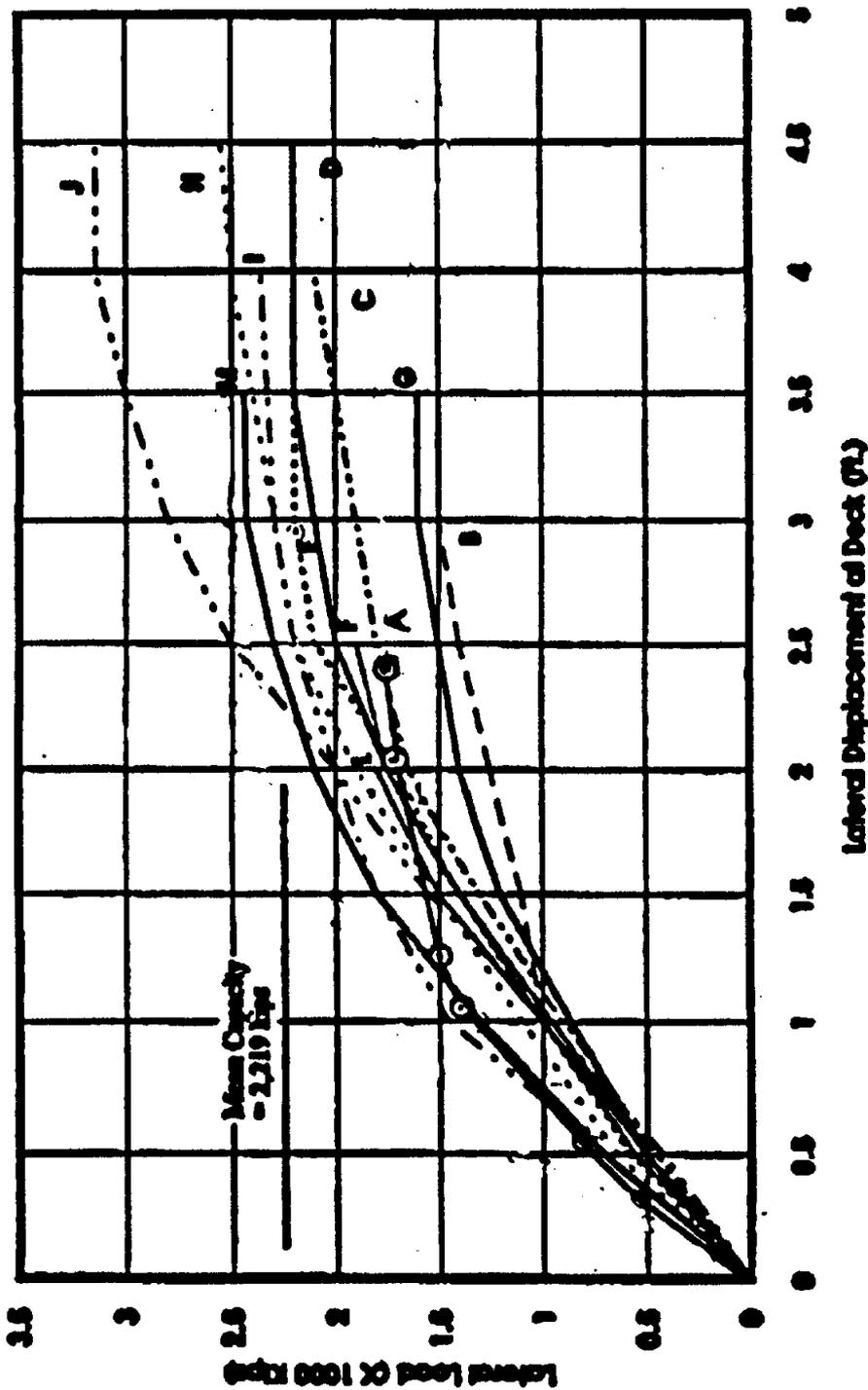


Figure 3-11 Load-Displacement Behavior - Direction 2

Figure 5.4-1
PMB JIP Benchmark Analysis Load-Displacement Results

6. DESIGN CONDITION CONSIDERATIONS

Requalification analyses, whether linear or non-linear, are highly sensitive to the magnitude and distribution of the applied loading condition. Present-day computer software provide the means for a precise computation of loads and distribution. However, the accuracy of such computation is only as good as the model and data input.

6.1 METOCEAN CRITERIA AND LOADS

6.1.1 Criteria

A requalification assessment of marginal platforms will be sensitive to the selected metocean criteria. API Draft Section 17 stipulates such criteria for domestic platforms. However, as the PMB JIP Benchmark Analysis demonstrated, there is still a variation in the interpretation and use of metocean criteria. To some extent this can be attributed to the interpretation of the API requirements; hopefully, any misinterpretation will be rectified in the near future.

The selection of applicable wave theory for use in load development can also vary the resultant applied load and its distribution on the platform. The appropriate wave theory and its combination with current loading for a given platform is not universally agreed upon. Couple this with the complexity of the API Draft Section 17 recommendations and a variation in resultant metocean loading is not surprising.

To further complicate this issue, API recommends specific wave height and period criteria. A maximum wave height criterion could be selected and taken as a conservative assessment. Wave period, on the other hand, may or may not be conservative, depending on the platform geometry and distribution of major load collecting elements. Shorter wave periods tend to have decreased applied lateral loading but increased applied overturning moment because the increased water particle kinematics tend to be located higher in the wave profile. The selection of the most critical wave criterion is not obvious without a simple parameter study. Perhaps the API should give a range of wave height, wave period combinations for use in requalification assessment.

6.1.2 Loads

The hydrodynamic model should accurately model all load collecting elements on the platform. Of particular importance are relatively large non-structural attachments such as conductors, risers, caissons and boat landings. A relatively large boat landing on a shallow-water platform could conceivably collect as much as 50 percent of the applied wave and current loading. This highly localized storm loading may be highly significant in limiting the ultimate capacity of the platform. Thus, particular attention needs to be given to the loads applied to the platform structure and appurtenances.

6.2 SEISMIC CRITERIA AND LOADS

6.2.1 Criteria

In a requalification assessment, the appropriateness of the assumed seismic response spectra should be confirmed by a qualified geotechnical or seismic consultant. A site specific seismic spectra with minimum built-in conservatism is preferred.

Seismic loads are usually developed using probabilistic methods. Resultant seismic loads in each platform orthogonal direction (two horizontal and one vertical) are typically summed to arrive at a total "most-likely" result. The resulting loads and their distribution are sensitive to the assumed percentage contribution in each direction. A typical present-day criteria is to assume 100 percent of the seismic load in each horizontal direction and 50 percent in the vertical direction. This is generally conservative, but not necessarily so. For example, relatively shallow major earthquakes have produced vertical accelerations approaching or exceeding the lateral accelerations. If seismic response with direction varies significantly and can be accounted for in the spectra, the additional conservatism, if any, can be removed from the assessment.

6.2.2 Loads

The mass model should accurately model all platform and hydrodynamic mass and its distribution. Appurtenance mass, especially that of relatively large appurtenances must be accounted for. The mass of the deck and associated equipment should be located at its actual center-of-gravity. The lateral position of mass will affect torsional loading on the platform; the vertical position of mass will affect the overall overturning moment on the platform.

Seismic load is often determined by response spectra techniques. A probabilistic value (e.g., SRSS or CQC methods) of load is determined for each platform joint. The directionality of seismic loading should be selected so that a minimum platform capacity is obtained. This may or may not be easily determined from the framing pattern of the platform (for example in a direction that loads the most compression bracing). When the critical direction is not readily determined, multiple load directions must be analyzed.

7. PLATFORM STRUCTURAL SYSTEM CONSIDERATIONS

7.1 NUMBER OF LEGS

Fixed offshore platforms are typically three-, four- or eight-legged platforms, although other variations exist. The benchmark and Timbalier platforms evaluated in this study are typical of four-legged platforms. Pacific Operators Offshore's Hogan platform has eight legs and Conoco's Grand Isle platform consists of two eight-legged platforms connected by cables and supporting the deck structure.

Three-legged platforms are more susceptible to collapse because they are less redundant, often with only one path available to resist load. They are particularly susceptible to foundation failure since one pile may be required to resist the entire foundation tension or compression.

Four-legged platforms are an obvious improvement over three-legged platforms but often still suffer from a lack of redundancy. This is especially true for first-generation design platforms which are typically "K"-braced. The failure of one brace is typically followed by a brace failure on the opposite frameline, drastically reducing the effective platform stiffness, and the only remaining path for resisting load is by severe distortion of the remaining portal frame.

Present day eight-legged platforms have added redundancy by cross-bracing at least one bay on each frame line between each plan level. Under this bracing scheme, the failure or softening of one (or more) braces does not significantly reduce the effective platform stiffness, providing additional load carrying capacity beyond the first member failure load.

Platform redundancy increases with the number of legs since the number of load resisting braced bents and bays also increases. Such redundancy should also increase the platform load capacity and ductility (or energy absorbing capability) by providing a structural system that has its inelastic behavior distributed more uniformly throughout the platform. A failure or softening of a particular structural component in a redundant system will also result in load redistribution to associated redundant elements without collapse of the entire system.

7.2 BRACING SYSTEMS

Redundant bracing systems, such as "X" bracing, provide more load capacity than non-redundant systems, such as "K" bracing. When one "X" bracing component reaches its yield or buckling capacity, the remaining components still form a triangulated, or trussed, system and the remaining components will continue to resist the imposed load by truss action in the braces. When one (of two) "K" bracing components reaches capacity, the resisting system is no longer triangulated, the system becomes more flexible, and the remaining components resist load through verendiehl frame action. The result is usually higher moments and a quicker ultimate failure of the "K" bracing.

7.3 FOUNDATION LAYOUT

Foundation considerations consist of pile orientation and arrangement, and soil properties.

Piles can be either battered or vertical. Vertical piles are more typical of first-generation platforms in the Gulf of Mexico because they were easier to install at the time. Today, battered pile installation presents no difficulty and is the preferred choice. Battered piles provide a relatively stiffer foundation laterally and markedly increase the lateral capacity of the foundation. The reason for this is that for most platforms and associated loading, even with a minor batter (say 5 deg), a significant portion of the horizontal shear will be resisted by axial load in the pile. The degradation in vertical load capacity is minor for battered piles.

Platform piling is generally arranged in one of three ways; skirt piles, pile groups, or a hybrid system.

Skirt Piles

Skirt piling consists of piles more or less evenly distributed around the platform perimeter, often at or near each leg for eight-legged platforms. Skirt pile systems are generally used in relatively soft soils on larger (eight legs or more) platforms and are especially efficient where vertical loads are relatively high and lateral loads are relatively low. Skirt piling systems are relatively inefficient in resisting lateral loads because the interior piles are not as effective in resisting platform overturning.

Corner Leg Piles of Pile Groups

Pile groups consist of clusters of piles located at the corners of the platform. A four-legged platform with one pile in each leg would also be in this classification. Pile groups are the most efficient in resisting lateral load since they provide the most leverage in resisting overturning moment. On large platforms (more than four legs), pile groups are relatively inefficient in resisting vertical load since load must be transferred through the platform framing, but this is only a minor consideration compared to the improvement in lateral load resistance. Pile groups are the layout of choice when lateral loads are high and/or the soils are relatively weak.

Hybrid Systems

Hybrid systems combine pile groups and skirt piles. Such systems combine the beneficial effects of pile groups and skirt piles. They are more typically of newer platform designs in unique locations and are more typical in foreign waters.

7.4 CONCLUSIONS

The relative platform redundancy should be assessed. Platforms with three or four legs and an associated number of piles, non-triangulated and/or non-redundant bracing will be less redundant and will have a relatively lower ultimate load capacity. Such platforms will be more sensitive to any or all of the specific component reduction parameters

addressed in this study.

The platform foundation layout should be assessed with respect to its efficiency in resisting loads. For example, skirt pile foundations are less efficient in resisting lateral loads (which are the basis for requalification), and the overload of a specific pile prior to reaching the full foundation capacity is a distinct possibility.

Platforms meeting the above conditions should be assessed with these component capacity reductions in mind.

8. COMPONENT MEMBER CONSIDERATIONS AND PLATFORM RESPONSE

8.1 MEMBER DESIGN AND MATERIALS

Platform jacket members are usually unstiffened tubular members, fully welded at the jacket joints. The steel material may be higher strength steel, but is often mild steel especially for first generation designed platforms. Platform legs will often be grouted especially if a pile is located inside the leg. The grout, if intact, will contribute substantially to the platform stiffness and load resistance capacity.

Platform legs are normally flooded and bracing is normally buoyant. For typical brace D/t ratios, the buoyancy effect counteracts the member weight and reduces the total vertical load on the member. However, hydrostatic induced circumferential stress interaction with brace longitudinal stress can be critical for members with high D/t ratios.

8.2 MEMBER BUCKLING CAPACITY

The member buckling capacity is a function of member yield strength, D/t ratio, effective length factor ("k"), and L/D. Actual member yield strength is often greater than that specified in design and can be determined from mill test results for specific members. Member slenderness ratios, especially for braces, are often less than assumed in design. The conservative combination of these two parameters can significantly under-predict the brace capacity of individual braces. However, the effect on ultimate capacity for a specific platform may be minimal, especially if the platform bracing is highly redundant, or more importantly, if the platform ultimate capacity is a function of some other platform component, such as the foundation.

Member capacity may also be significantly less than computed values due to in-service degradation, such as corrosion, dents and/or fatigue. These effects should be considered for each platform on a case by case basis.

8.3 MEMBER POST-BUCKLING CAPACITY

Member post-buckling capacity is usually reduced relative to the member post-buckling capacity as a function of member geometry (effective length factor "k," L/D and D/t ratios) and P-delta effects.

The member shape has a significant effect on post-buckling capacity. A wide-flange shape will have a higher post-buckling capacity relative to its initial buckling capacity than will a tubular section. Local buckling of thin webbed or flanged wide-flange sections or tubulars with high D/t ratios will also result in reduced post-buckling capacities.

P-delta effects on member post-buckling capacity are more difficult to ascertain since the effect is a function of applied load, in addition to member geometry. Determination of an exact value requires additional calculation for each member and iteration during an analysis.

An overall post-buckling reduction factor of 0.15 to 0.50 has been commonly applied to analyses to account for the total reduction in member post-buckling capacity. This approach may suffice for an initial look at platform response but will over- or under-predict the individual member post-buckling capacities and the resultant platform capacity.

Member post-buckling capacity has a significant effect on platform ultimate capacity since once buckling occurs, the load carrying capacity of the particular brace and eventually the entire lateral load resisting system, is reduced. To ignore the effect of member post-buckling capacity will result in over-prediction of the platform ultimate capacity.

8.4 MEMBER HYDROSTATIC CAPACITY

Buoyant platform tubular members are subject to circumferential stresses due to hydrostatic pressure. The interaction of such stress with member longitudinal stress is well known and has been incorporated into the API RP 2A design methodology for some time.

For tension members, the member tension capacity is always reduced due to the presence of hydrostatic pressure, the reduction increasing with increasing ratio of hydrostatic induced stress, f_h , to member hydrostatic critical stress, F_{hc} . Typical designs incorporate a safety factor of 2.0 for hydrostatic pressure; thus, f_h/F_{hc} is not likely to exceed 0.50. For a ratio of 0.5, the member tension capacity is reduced by at most 37 percent. Considering that typical members are designed for other load components in addition to hydrostatic pressure, the actual reduction will be less. As another example, a tension member with a D/t of 40 under 300 ft of head will have a f_h/F_{hc} of about 0.17, resulting in a tension capacity reduction of about 6 percent. Other tension capacity reduction factors assuming unstiffened tubulars and 36 ksi steel are shown in Table 8.4-1. Even a small reduction effect can be significant if it applies to a sufficient number of members.

Table 8.4-1 Unstiffened Member Tension Capacity Reduction Factors Due to Hydrostatic Pressure

Head (ft)	Member Tension Capacity Reduction Factor ¹			
	D/t = 20	D/t = 40	D/t = 60	D/t = 80
200	0.99	0.96	0.82	0.26
300	0.98	0.94	0.67	0.00 ²
400	0.98	0.91	0.47	0.00 ²

- Notes: 1. $F_y = 36$ ksi assumed, higher yield strengths may result in higher factors.
 2. Unstiffened members in this category can carry no tensile load.

For compression members, the member compression capacity may increase or decrease due to the presence of hydrostatic pressure. However, due to the design safety factor used in hydrostatic design, decreases in member compression capacity will not typically occur. Increases in member capacity are slight and are best ignored in any requalification analysis.

8.5 MEMBER DAMAGE

In-service platforms are susceptible to various types of damage including buckled members, dented members, corrosion, and weld fatigue or failure. Member capacity reduction due to any or all of these effects should be incorporated into a requalification analysis, especially where the platform ultimate capacity is limited by member capacity. Member buckling is typically assessed as a part of a requalification analysis and buckling parameters have been parametrically evaluated as a part of this study.

Member corrosion may be accounted for by reducing the wall thickness of corroded members to account for the reduction in load capacity. This is a simple procedure in any analysis. The member corrosion should be quantified as a part of the platform inspection and subsequent requalification analysis.

Weld failure can be similarly accounted for by eliminating, if applicable, the affected member from the requalification analysis. Again, this is a simple analysis procedure.

The assessment of the effect of member dents requires consideration on a member by member basis. There are a variety of formulations for predicting member capacity as a function of dent size and location for "typical" dent shapes. Although it is a simple matter to incorporate these formulations into the analysis for dented members, the difficulty problem occurs when dents are not typical and/or the impact causing the local dent has also introduced a global deformation (bending). In this case, engineering judgment or more refined engineering analysis is needed to quantify the reduction in member capacity. The location and size of member dents should be quantified as a part of the platform inspection and subsequent requalification analysis. Damage-induced member global displacement should be so specifically accounted for in the analysis. Kallaby and O'Connor (Reference 10) recommends an integrated damage assessment, including dent profile measurement and capacity reduction methods.

8.6 STUDY OF MEMBER BEHAVIOR AND PLATFORM RESPONSE

The effect of various member parameters considered important to member and platform capacity were assessed by performing nonlinear pushover analyses. The foundation elements were deleted from these parametric analyses in order to eliminate the foundation effects from the analysis results.

8.6.1 Brace Member Slenderness Ratio

The benchmark platform was analyzed for the orthogonal storm load pattern for both the base case structure with API RP 2A recommended member slenderness ratios and assuming theoretical member slenderness ratios. API recommends member slenderness

ratios assuming an effective length factor, k , of 0.8 for braces. Theoretical k values of about 0.65 were determined from the relative rigidities of supporting structure and the brace under consideration. Foundation effects were eliminated from the analyses by pinning the structure at the mudline.

8.6.2 Brace Member Post-Buckling Capacity

The effect of member post-buckling capacity was studied by analyzing the benchmark platform for the orthogonal storm load pattern with post-buckling capacities of 1, 25, 50, 75, 90 and 100 percent of the member initial buckling capacity based on theoretical slenderness ratios. A zero (0) percent reduction factor represents a "member elimination" analysis where a buckled member can carry no load. A 100 percent reduction factor means that a buckled member continues to hold its entire initial buckling load. Foundation effects were eliminated from the analyses by pinning the structure at the mudline.

8.7 FINDINGS AND CONCLUSIONS

8.7.1 Brace Member Slenderness Ratio

The resulting responses and ultimate capacities for each analysis are shown on Figure 8.7-1. The results show that platform response and ultimate capacity are somewhat sensitive to assumed member slenderness ratios when member post-buckling capacities are not reduced. This sensitivity would be more significant for platforms with relatively slender braces or higher strength steels. For example, assuming 36 ksi steel, slenderness ratios of 65 (based on "theoretical") and 80 (based on API RP 2A recommendations) produce critical stresses of 87 and 80 percent of yield, respectively, a sensitivity of about 9 percent. Table 8.7-1 shows other slenderness ratio comparisons and their associated sensitivities for 36 and 50 ksi steels

Table 8.7-1 Brace Member Capacity Sensitivity to Slenderness Ratio

Slenderness Ratio (kL/r)		36 ksi Steel			50 ksi Steel		
		Critical Stress, F_{cr} (ksi)		Variation (%)	Critical Stress, F_{cr} (ksi)		Variation (%)
API RP 2A	Theoretical	API RP 2A	Theoretical		API RP 2A	Theoretical	
16	13	35.7	35.8	0.3	49.4	49.6	0.4
32	26	34.8	35.2	1.1	47.8	48.5	1.6
48	39	33.4	34.3	2.7	45.0	46.7	3.8
64	52	31.4	32.9	5.0	41.1	44.1	7.4
80	65	28.8	31.2	8.5	36.0	40.8	13.2
96	78	25.6	29.1	13.9	29.9	36.7	22.9
112	101	21.8	26.6	22.1	22.8	31.9	39.9

Note: Theoretical kL/r values are assumed for demonstration purpose only.

$$F_{cr} = (1 - 0.5 * (kl/r)^2 / C_c^2) * F_y, \text{ where } C_c = (2\pi^2 E / F_y)^{1/2}$$

For the benchmark platform analyses, the ultimate capacity was increased by about 6 percent when theoretical "k" values are used instead of those recommended by API RP 2A. Thus, it is slightly conservative to use the API RP 2A recommended design values. The amount of conservatism depends on whether the platform capacity is a function of brace buckling, the L/r of the braces, and the brace yield strength. If brace compression instability is the primary mode of platform collapse, then additional load capacity may be justifiable by using theoretical (or actual) "k" values in the requalification analysis.

The level of conservatism is appropriate as the typical brace member is not in pristine condition and degradation due to in-service effects (corrosion, dents, fatigue) will more than offset any computed gains in member capacity due to the use of conservative slenderness ratios.

8.7.2 Brace Member Post-Buckling Capacity

The platform response and ultimate capacities are shown on Figure 8.7-2. All responses are linear up to the buckling of the first brace member. The subsequent loading peaks become flatter with reduced post-buckling capacity. In general, the "zigzags" shown on the figure occur when braces on opposite frames buckle. The platform response is sensitive to variations in post-buckling capacity.

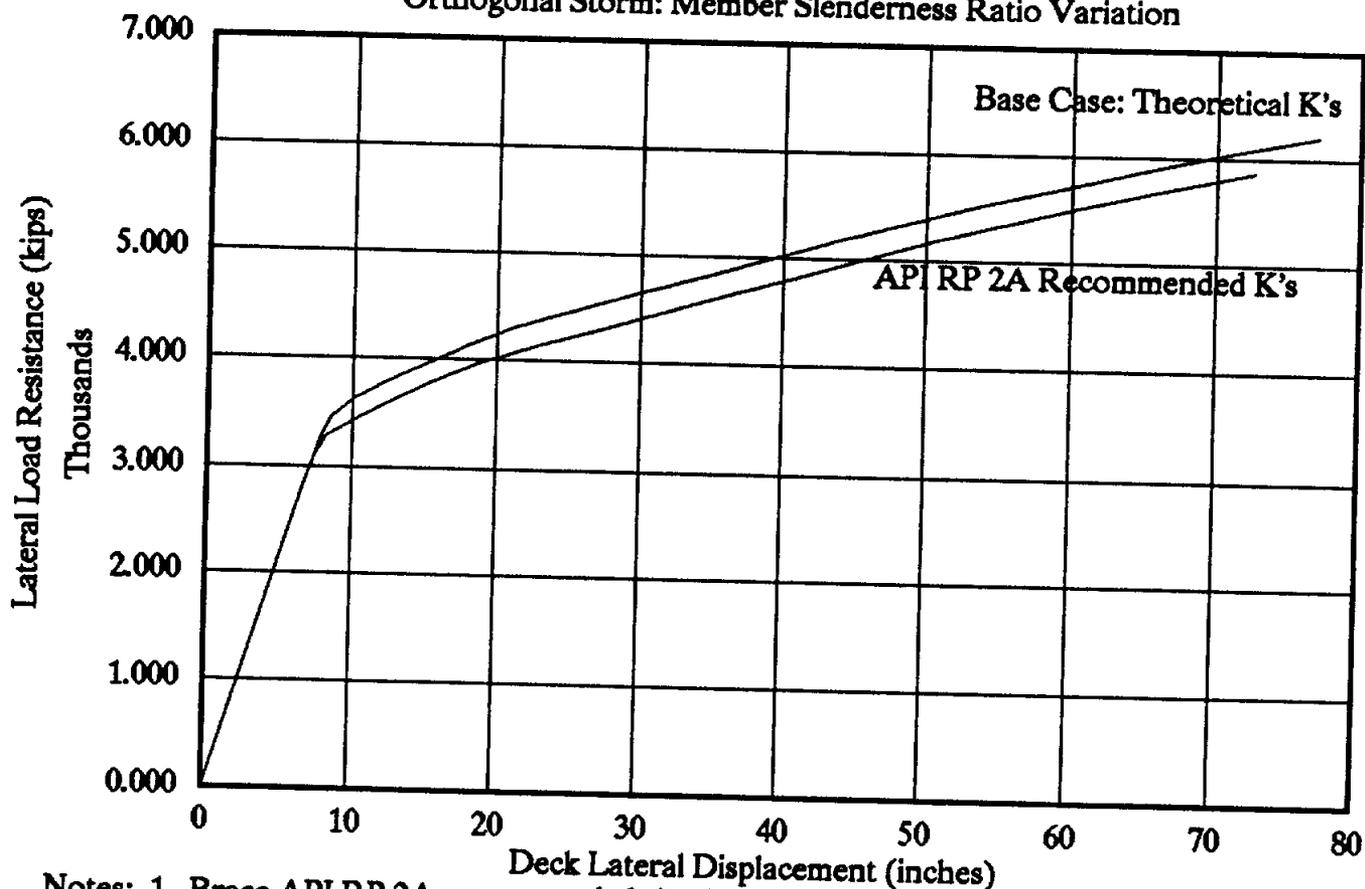
This study implies that member post-buckling capacity must be considered in any requalification analysis and that the maximum post-buckling reduction factor for each member should be quantified. Values ranging from 0.15 to 0.50 are often used in such analyses but even this range gives a large spread in the results. Further work would be

desirable to more accurately quantify appropriate member post-buckling reduction factors and their effects on platform ultimate capacity as a function of slenderness ratio and P-delta effects.

8.7.3 Hydrostatic Pressure

Based on the discussion presented in Section 8.4, the effect of hydrostatic pressure on member capacity should be accounted for in any requalification analysis where the hydrostatic pressure was not explicitly accounted for in the design. Where included in the design, tension members with high D/t ratios in deep water should be investigated for possible tension capacity reductions.

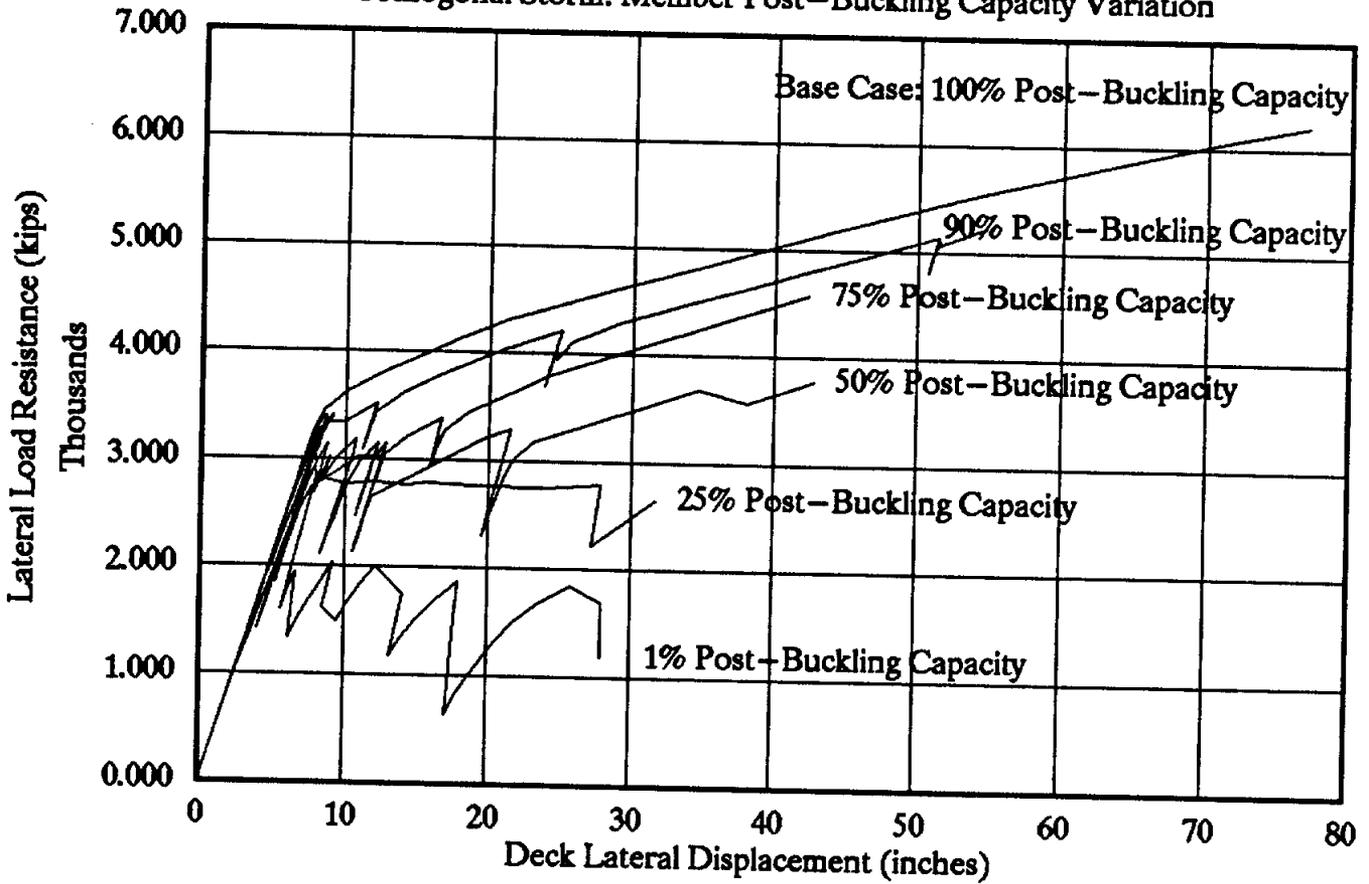
API JIP Benchmark Platform Pushover Analysis Results
 Orthogonal Storm: Member Slenderness Ratio Variation



- Notes: 1. Brace API RP 2A recommended slenderness ratios range from 46 to 61.
 2. Platform pinned at mudline for these parametric analyses.

Figure 8.7-1
Benchmark Platform - Brace Member Slenderness Ratio Study

API JIP Benchmark Platform Pushover Analysis Results
 Orthogonal Storm: Member Post-Buckling Capacity Variation



Note: Platform pinned at mudline for these parametric analyses.

Figure 8.7-2
 Benchmark Platform - Brace Member Post-Buckling Capacity Study

9. COMPONENT JOINT CONSIDERATIONS AND PLATFORM RESPONSE

9.1 JOINT DESIGN AND MATERIALS

Platform jacket joints generally consist of unstiffened or stiffened tubular joints. The joint material may be either mild steel (A36 or similar) or higher strength steel. Leg joints may or may not be grouted; the condition of any grout may be questionable with time as platform damage, platform loading and seawater corrosion work to degrade the grout performance.

First generation designed platforms may not have higher strength and increased thickness joint cans. The absence of such cans may result in joints with inadequate capacity relative to the jacket bracing. Furthermore, for first generation designed platforms, the actual joint design may be deficient by today's standard since punching shear design formulations have become more conservative with time and crushing collapse considerations may not have even been considered during the platform design.

However, even the present day designs may be inadequate. If the joints are designed to meet the minimum API requirements (namely, designed to resist 50 percent of the brace member capacity), they need to be scrutinized more carefully.

9.2 UNSTIFFENED JOINT PUNCHING SHEAR CAPACITY

Joint punching shear is defined as a joint chord shear failure due to brace axial and moment loading. Joint punching shear capacity may be a critical parameter for platform ultimate strength, particularly for first-generation platforms where joint design was done to less conservative punching shear formulations. The punching shear capacity may also be important for grouted joints where the participation of the grout over time becomes questionable. In these cases, the joint punching shear capacity will limit the force which can be transferred through the joint to or from the connecting brace or braces, thereby reducing the platform ultimate capacity.

9.3 UNSTIFFENED JOINT CRUSHING CAPACITY

Joint crushing collapse is defined as excessive chord wall circumferential bending and axial stress in the chord wall resulting in material yielding and subsequent joint collapse. Joint crushing capacity may be a critical parameter for platform ultimate strength, especially for first-generation platforms where joint designs are less adequate than present-day standards. The present-day API RP 2A recommendations purport to include the crushing capacity into the punching shear formulations for simple tubular joints; this was shown to be true only for simple tee joints and cross joints with "beta" ratios close to 1. More complex unstiffened joints and stiffened joints, especially those stiffened for in-service load conditions, may require checking for crushing collapse. In particular, all stiffened joints, and unstiffened cross and tee joints with differing brace and chord diameters should be evaluated for crushing collapse.

9.4 JOINT FLEXIBILITY

A properly designed stiffened joint will be substantially stiffer than an unstiffened joint with the same D/T ratio. Stiffened joints generally have stiffnesses which are considered "rigid" for analysis purposes. This is a reasonable assumption considering that unstiffened joint stiffness has a discernible, but not overly significant effect on platform response and associated capacity.

The reduced stiffness of unstiffened, simple tubular joints can result in effective brace stiffnesses significantly less than the brace alone. This effective brace stiffness may be written as:

$$K_{\text{eff}} = 1 / (1 / K_{\text{js}} + 1 / K_{\text{br}} + 1 / K_{\text{je}})$$

where:

K_{eff} = the effective brace stiffness

K_{br} = the brace stiffness alone (e.g., AE/L)

K_{ji} = the joint stiffness at the brace start or end

The actual joint stiffness is a function not only of joint geometry but the brace loads applied to the joint can. The reason for this is that load applied at one location on the chord location causes varying radial deformation around the entire chord circumference which further varies along the chord length. The computation of stiffness must therefore be calculated for every load condition, and the analysis iterated until acceptable stiffness-load compatibility is achieved. For even relatively small analysis models, this calculation and subsequent iteration becomes tedious and time-consuming, and therefore it is rarely done in practice. Ignoring the joint stiffness effect in its entirety, however, can result in over-predicting the platform stiffness and possibly the platform ultimate capacity.

9.5 JOINT DAMAGE

In-service platforms are susceptible to various types of joint damage including punching, crushing, grout and weld failures. Joint capacity reductions due to these effects should be incorporated into a requalification analysis, especially where the platform ultimate capacity is limited by joint and/or member capacity. Joint punching and crushing capacity effects have been parametrically evaluated as a part of this study. Joint weld failure can be accounted for by eliminating, if applicable, the affected connecting member from the requalification analysis. This is a simple analysis procedure.

Grouted joints would normally be assumed as rigid for analysis purposes. Degraded grout should be reflected as reduced leg properties in the analysis. Furthermore, the joint capacity of unstiffened joints with degraded grout should reflect the associated brace member capacity reduction, if any.

9.6 STUDY OF JOINT BEHAVIOR AND PLATFORM RESPONSE

The effect of various joint parameters considered important to joint, connecting member and platform capacity were assessed by performing nonlinear pushover analyses. The foundation elements were deleted from these parametric analyses in order to eliminate the foundation effects from the analysis results.

9.6.1 Joint Punching Shear Capacity

The benchmark platform was analyzed for the orthogonal storm load pattern by setting the joint punching shear capacities at 25, 50, and 100 percent of the API RP 2A 20th Edition capacities. A decreasing number of brace member ends were controlled by the joint capacity limits as shown in Table 9.6-1. If a structure has been properly designed in accordance with API RP 2A, no brace member ends should be limited by joint punching shear. The benchmark platform does not meet this criterion when joint punching capacity is set to its actual capacity (100 percent times API RP 2A computed values).

Table 9.6-1: Benchmark Platform - Brace Capacities Limited by Joints

Actual Joint Capacity / API RP 2A Design Capacity	Member Ends with Reduced Capacity
100%	45 %
75	50 %
50	50 %
25	50 %

For this analysis, the platform was pinned at mudline and the soil-pile foundation eliminated in order to remove the foundation capacity effects from the study. The analysis lateral load-displacement response is presented on Figure 9.7-1.

9.6.2 Joint Crushing Capacity

The crushing capacity of simple tubular cross and tee joints was parametrically evaluated relative to the connecting brace capacity. A check against the API RP 2A recommended punching shear formulations was also done to verify the assumption that the punching formulae accounted for the crushing collapse effects. "K" joints were not evaluated since collapse forces are not transferred around the chord circumference for this class of joint.

The ultimate brace tensile load was determined for each joint geometry and the associated chord circumferential axial and bending stresses were determined from a rigid ring analysis. Three chord diameters were assumed effective in resisting the imposed forces. The brace load was assumed applied at the extreme edge of the brace diameter; this is an unconservative assumption on the brace load distribution which will minimize chord wall stress and produce an upper-bound joint crushing capacity. An upper-bound

estimate was made since any other assumption of brace load distribution is questionable.

9.6.3 Joint Flexibility

The effect of joint stiffness was evaluated. For this work, it was assumed that the joint stiffness was unaffected by loading from adjacent braces. This results in an under-prediction (i.e. more flexible) joint stiffness which maximizes the effect of joint stiffness on platform response and ultimate capacity. The purpose of the study then becomes to determine the applicable joint and brace parameters which significantly affect joint stiffness and platform response and ultimate capacity.

The brace stiffness was determined based on EA/L for each brace. Joint stiffness was determined in accordance with Reference 15, Figure 15. The brace load normal to the chord was assumed divided into two equal parts located at the centroid of each half of the intersecting brace. The "d" parameter was calculated and the deflection coefficient, C_m , was determined from Figure 15. The effective joint stiffness was determined from the resulting joint radial deformation at the centerline of the brace consistent with the assumed applied brace load.

The joint stiffness computation is straightforward but is dependent upon an assumed effective width of chord acting under the intersecting brace. The API RP 2A assumption of three chord diameters may be too liberal whereas the API RP 2A recommendation of $1.1 * (D*T)^{0.5}$ is applicable only to ring-stiffened joints. A practical minimum effective width equal to the projected diameter of the brace at the face of the chord was assumed. Again, this results in a lower-bound joint stiffness which maximizes the effect on the analysis.

The benchmark platform was analyzed for the orthogonal load pattern assuming the platform pinned at mudline in order to eliminate the foundation capacity effects. Unstiffened joint stiffnesses were varied in a number of ways as follows:

- 1) All joint stiffnesses based on API RP 2A recommended effective width of three chord diameters; this represents a relatively stiff joint assumption.
- 2) All joint stiffnesses based on effective widths equal to projected brace diameter; this represents a relatively flexible joint assumption
- 3) Same as 2), but with no joint can increased thickness. The purpose of this variation was to represent first generation platforms designed without can thickness increases and/or with degraded grouting.
- 4) Same as 2), but only for the leg cans on one transversely loaded frame. The purpose of this was to create a torsional load distribution on the platform structure.

9.7 FINDINGS AND CONCLUSIONS

9.7.1 Joint Punching Shear Capacity

The resulting responses and ultimate capacities for each analysis are shown on Figure 9.7-1. The results show that the platform response varies significantly with joint punching shear capacity with an associated decrease in platform ultimate capacity.

This study implies that joint capacity reductions due to punching shear capacity must be incorporated into the requalification analysis if an accurate platform capacity is to be obtained. Particular attention should be paid to first generation platform designs since these designs were done to less stringent joint design criteria.

9.7.2 Joint Crushing Capacity

Numerous joint chord and brace diameters and thicknesses were evaluated and the results tabulated and plotted. It was determined that the ratio of chord collapse load to brace ultimate load was effectively a linear function of brace D/t and an inverse square function of the chord D/t when the brace-to-chord ratio ("beta") is constant. Plotting the results in this manner provides a convenient figure for determining when the joint crushing capacity will limit the maximum possible brace load. Figures 9.7-2 and 9.7-3 present these curves for unstiffened tee and cross joints, respectively. A resulting capacity reduction factor of less than 1.0 means that the brace capacity is controlled by joint crushing rather than the brace ultimate stress. The plotted factors need to be modified by the ratio of chord to brace ultimate stress and further revised to account for the brace-to-chord angle. The factors may be read directly for brace-to-chord angles of 90 deg and equal chord and brace ultimate stresses.

The check against the API RP 2A punching shear formulae was found to be within 1 percent for simple tee joints and cross joints with diameter ratios approaching 1. However, the discrepancy in results increased with decreasing "beta" ratio for cross joints, with the crushing capacity significantly less than the API predicted punching capacity (see Figure 9.7-4). Thus, it can not be concluded that the API punching formulae adequately include the crushing effect for cross joints. All joints should be checked for collapse as a part of good design practice since the calculated crushing capacities in this study were always less than or equal to the punching shear capacities and the crushing capacities are upper-bound.

The curves in Figures 9.7-2 and 9.7-3 may be used as a guideline to determine the reduction in brace capacity, if applicable, to be used to account for joint collapse (and punching shear) for simple tee and cross joints. Reductions for simple "K" joints should be determined from the API RP 2A formulae. More complex or stiffened joints should be individually evaluated. Joints with reduction factors less than 1.0 should have the reduced brace member capacities included in any requalification analysis. The figures allow MMS and other interested parties a quick method for determining when such an approach is definitely required.

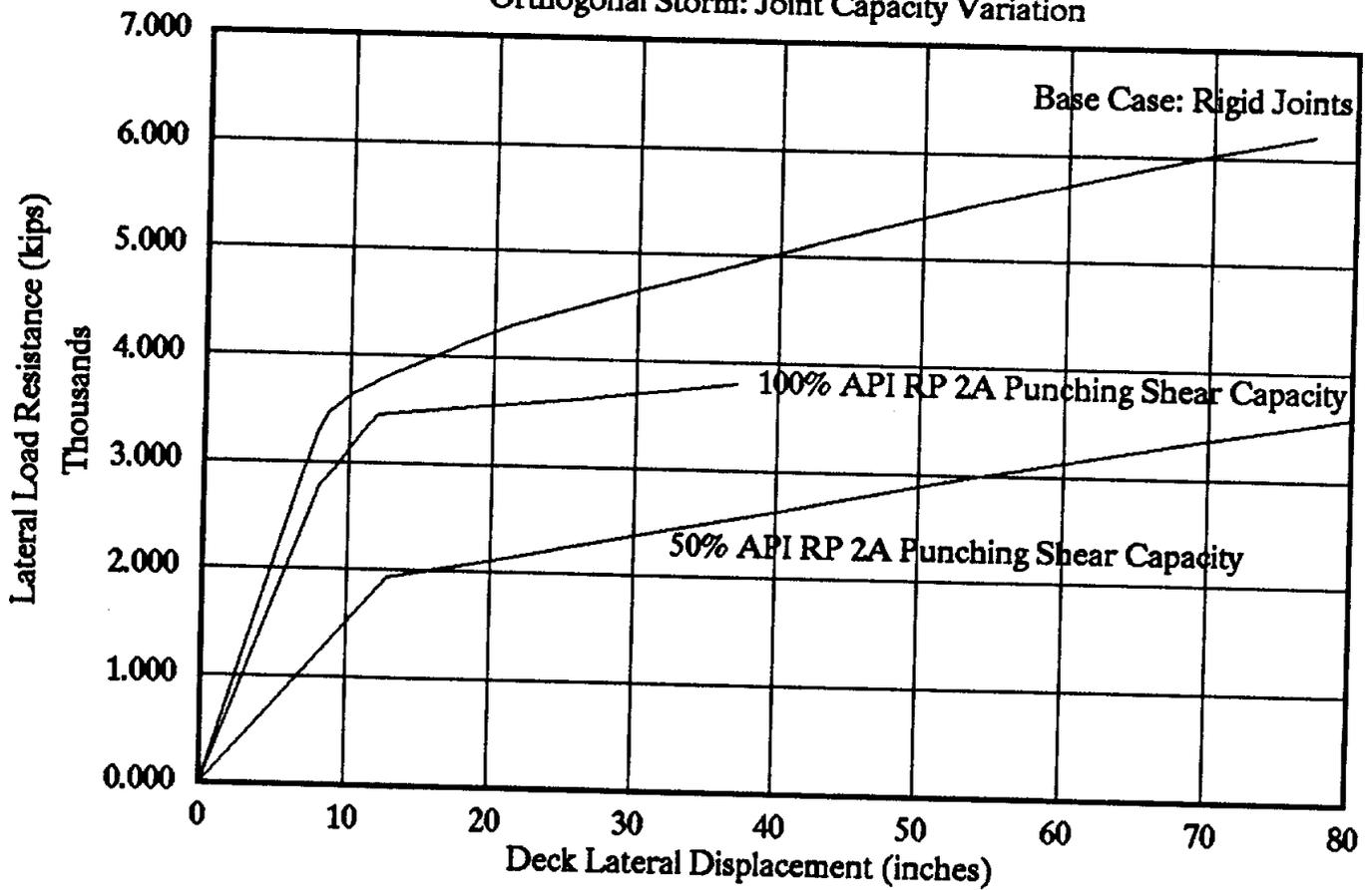
The factors in the figures are upper-bound estimates of the joint capacity and actual capacities will be less. Further study is needed to establish the actual brace distribution in each case in order to better quantify the joint capacities.

9.7.3 Joint Stiffness Effects

The platform response and associated ultimate capacities are shown on Figure 9.7-5. The platform response does vary with assumed joint stiffness and stiffness distribution, initially in the linear range and again as ultimate capacity is reached. However, the ultimate capacities are all approximately the same. This is because a homogeneously (i.e. symmetric) more flexible structure will deflect more but still distribute the resisting forces in about the same manner. Since the member and joint capacities are not a direct function of joint stiffness, the platform capacity remains about the same whether or not joint stiffness is incorporated into the analysis.

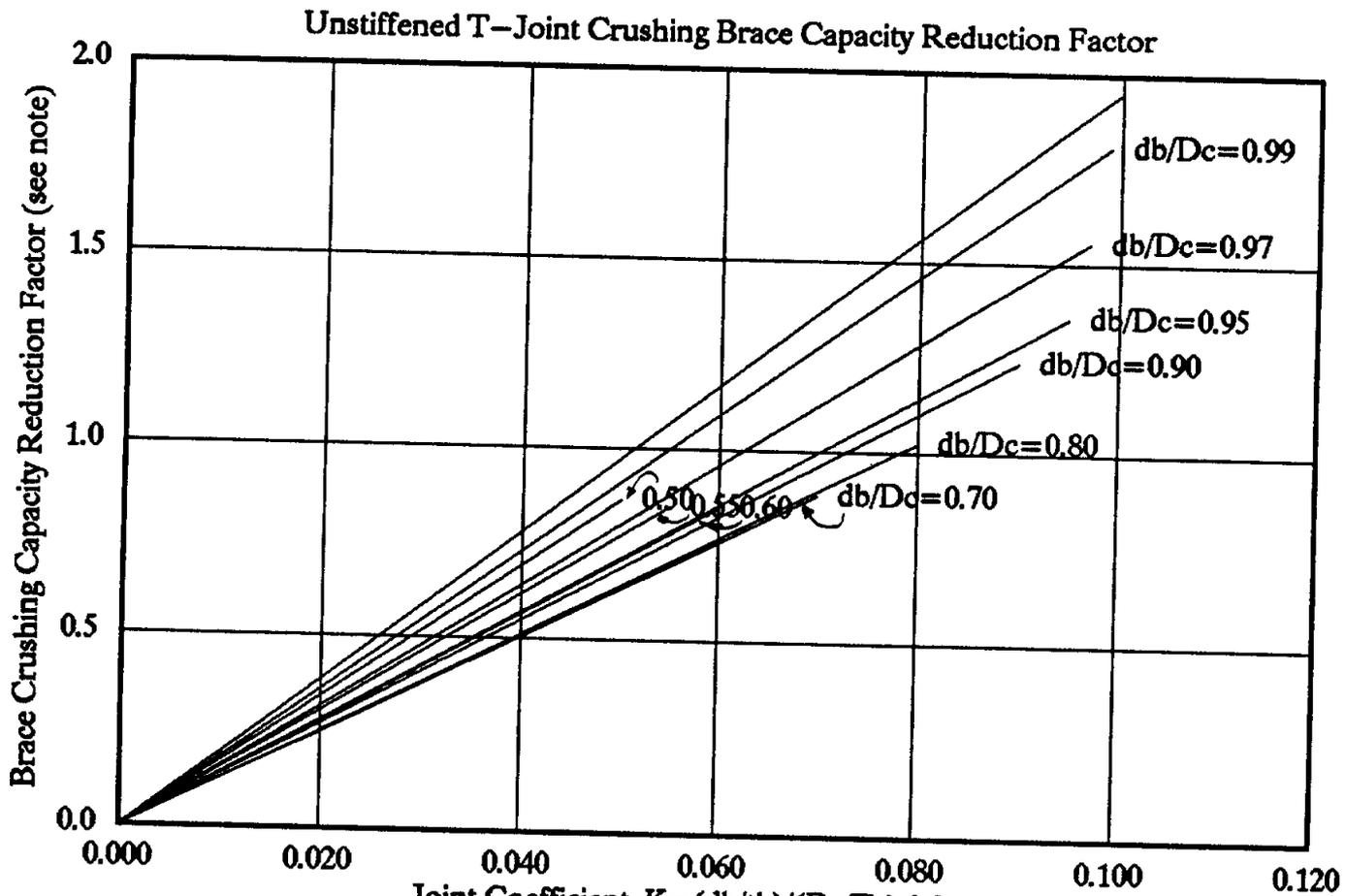
An exception to the above would occur when the joint stiffness variations are not symmetrically distributed throughout the platform. This might occur if the grouting in one or more legs (but not all) became deficient. In this case, the load redistribution would be non-symmetric and platform ultimate capacity would be affected. Figure 9.7-6 shows a case where two legs incorporate the effects of joint stiffness and two do not. The center-of-resistance shifts towards the stiffer legs and a torsional loading distribution results. The decrease in platform ultimate capacity for the benchmark structure is about 10 percent.

API JIP Benchmark Platform Pushover Analysis Results
 Orthogonal Storm: Joint Capacity Variation



Note: Platform pinned at mudline for these parametric analyses.

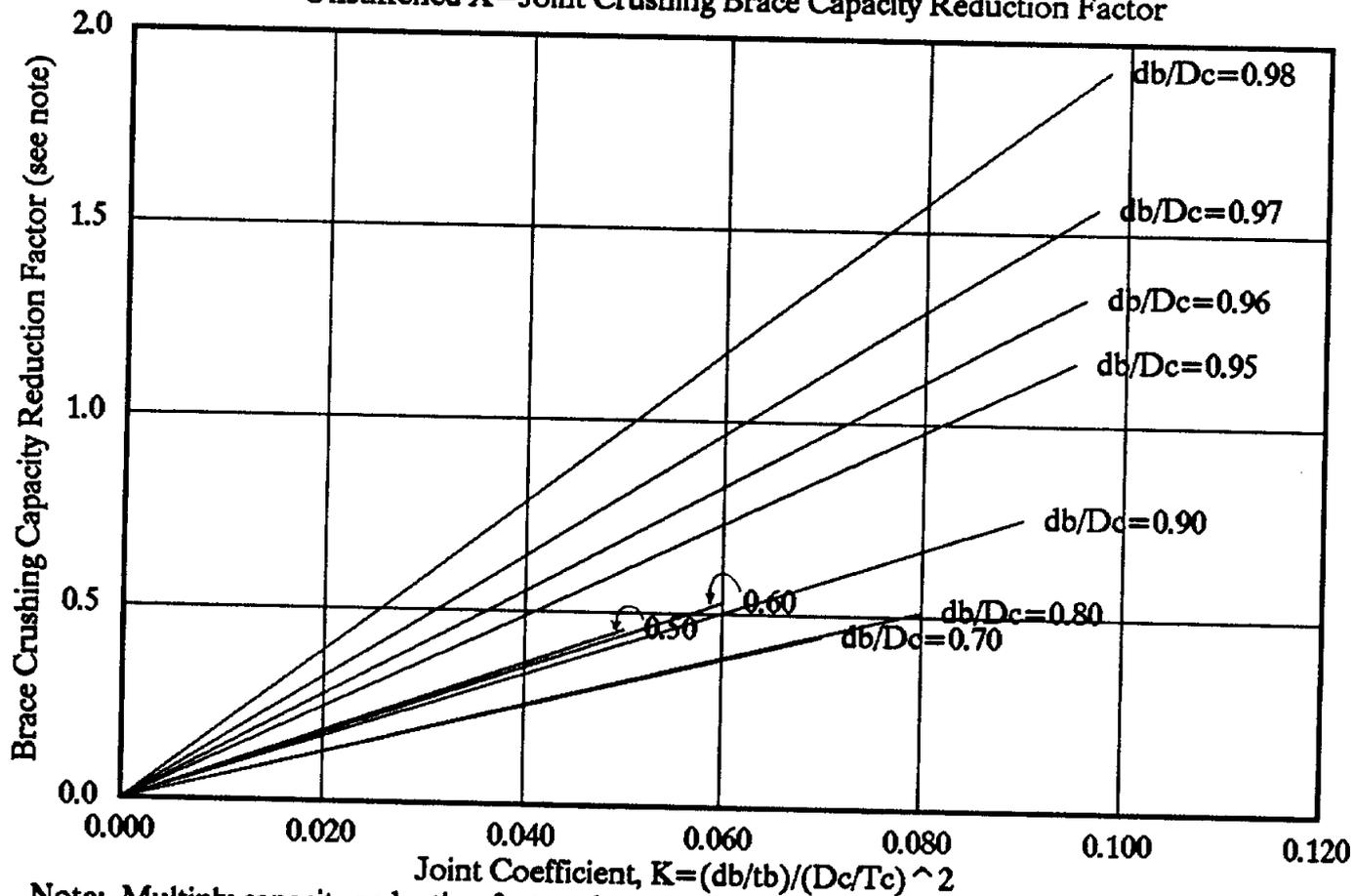
Figure 9.7-1
 Benchmark Platform - Joint Punching Shear Capacity Study



Note: Multiply capacity reduction factor above by $F_{yc}/F_{yb} \cdot \sin(\theta)$, where θ is angle between brace and chord, and F_{yb} is the brace ultimate stress.

**Figure 9.7-2
Brace Capacity Reduction Factors for T-Joints**

Unstiffened X-Joint Crushing Brace Capacity Reduction Factor

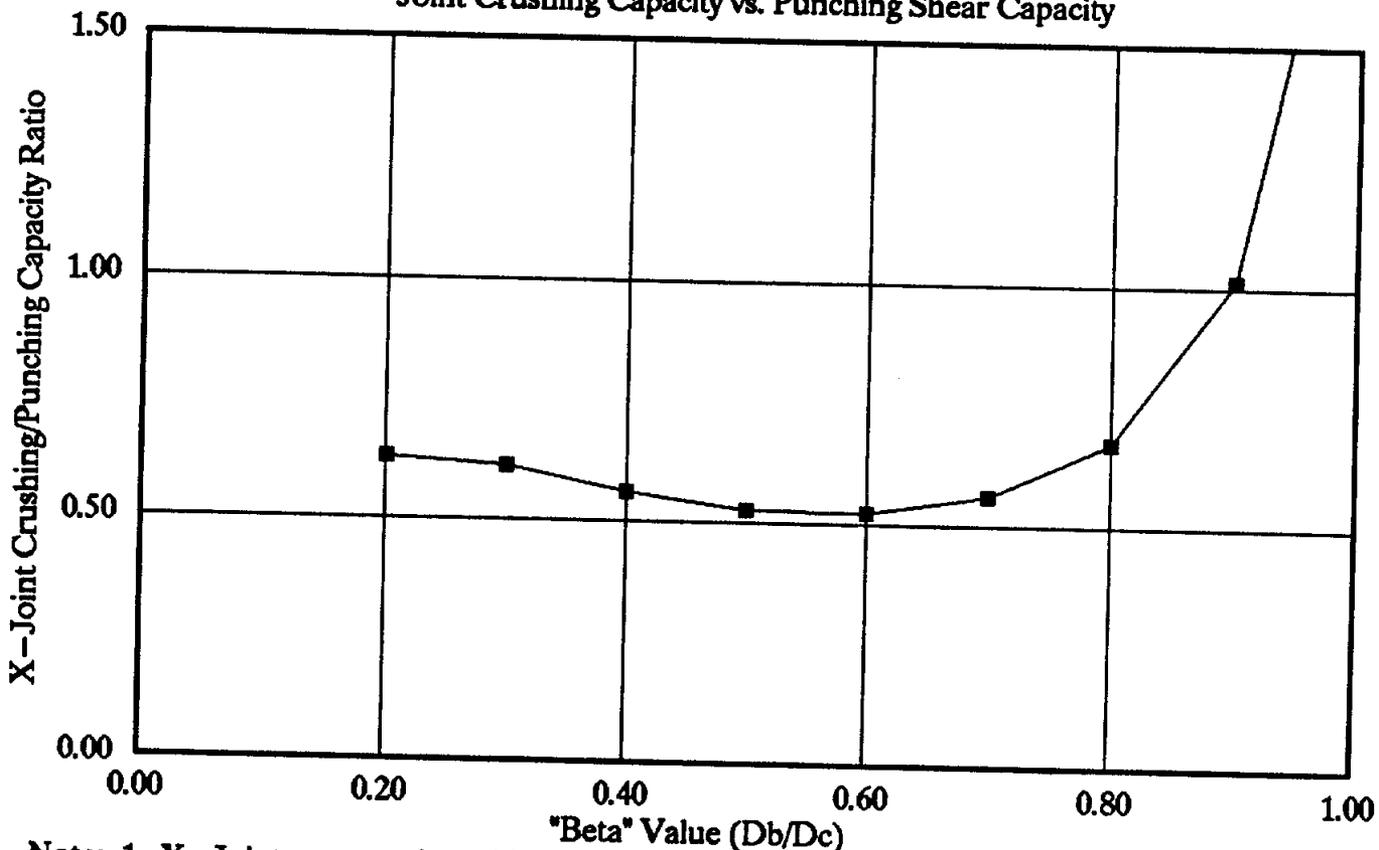


Note: Multiply capacity reduction factor above by $F_{yc}/F_{yb} \cdot \sin(\theta)$, where θ is angle between brace and chord, and F_{yb} is the brace ultimate stress.

Figure 9.7-3
Brace Capacity Reduction Factors for X-Joints

Unstiffened X-Joint Capacity

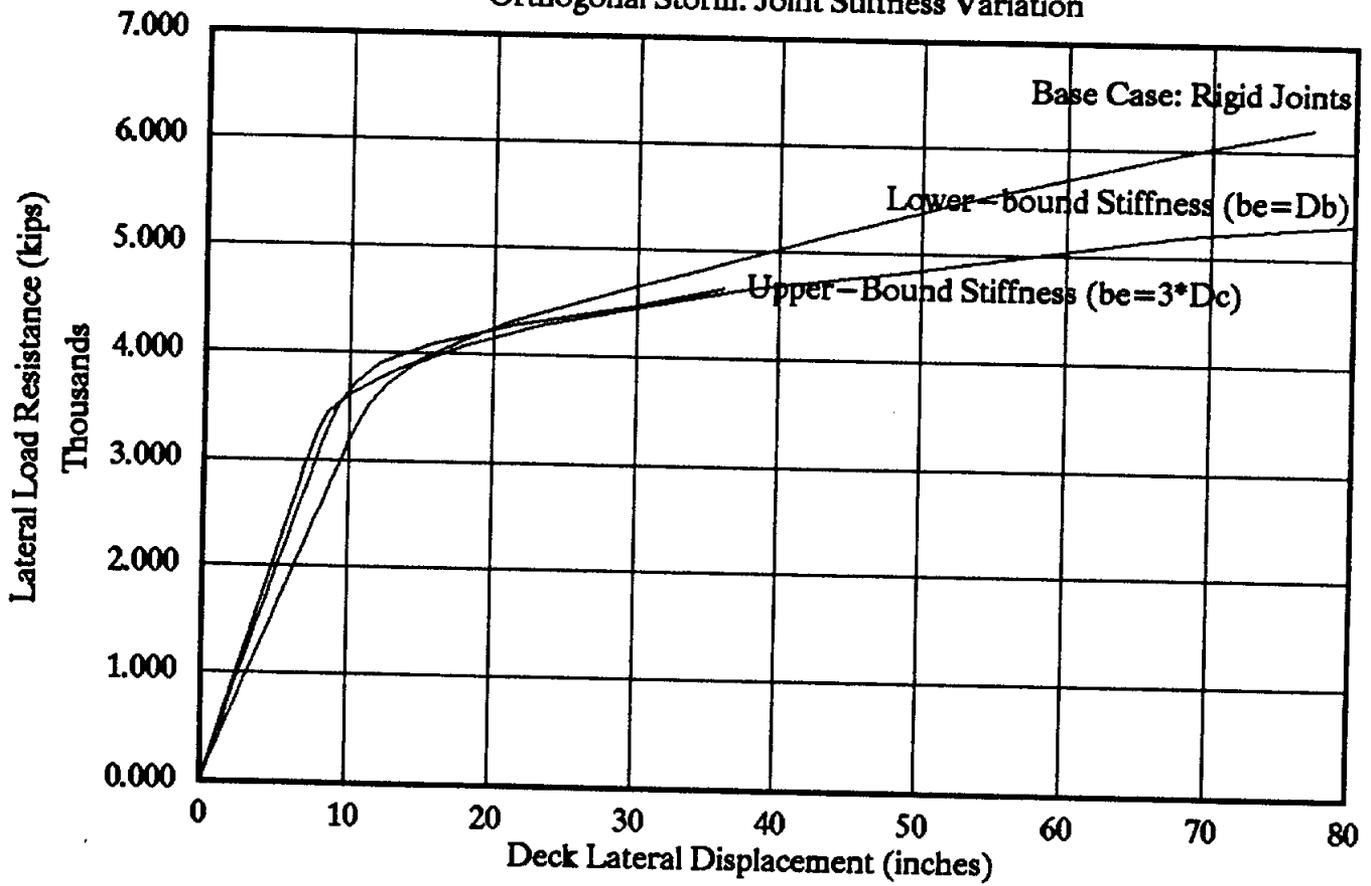
Joint Crushing Capacity vs. Punching Shear Capacity



- Note: 1. X-Joint computed crushing capacity is upper-bound.
2. Computed punching shear capacity per API RP 2A.

Figure 9.7-4
X-Joint Punching and Crushing Capacity Comparison

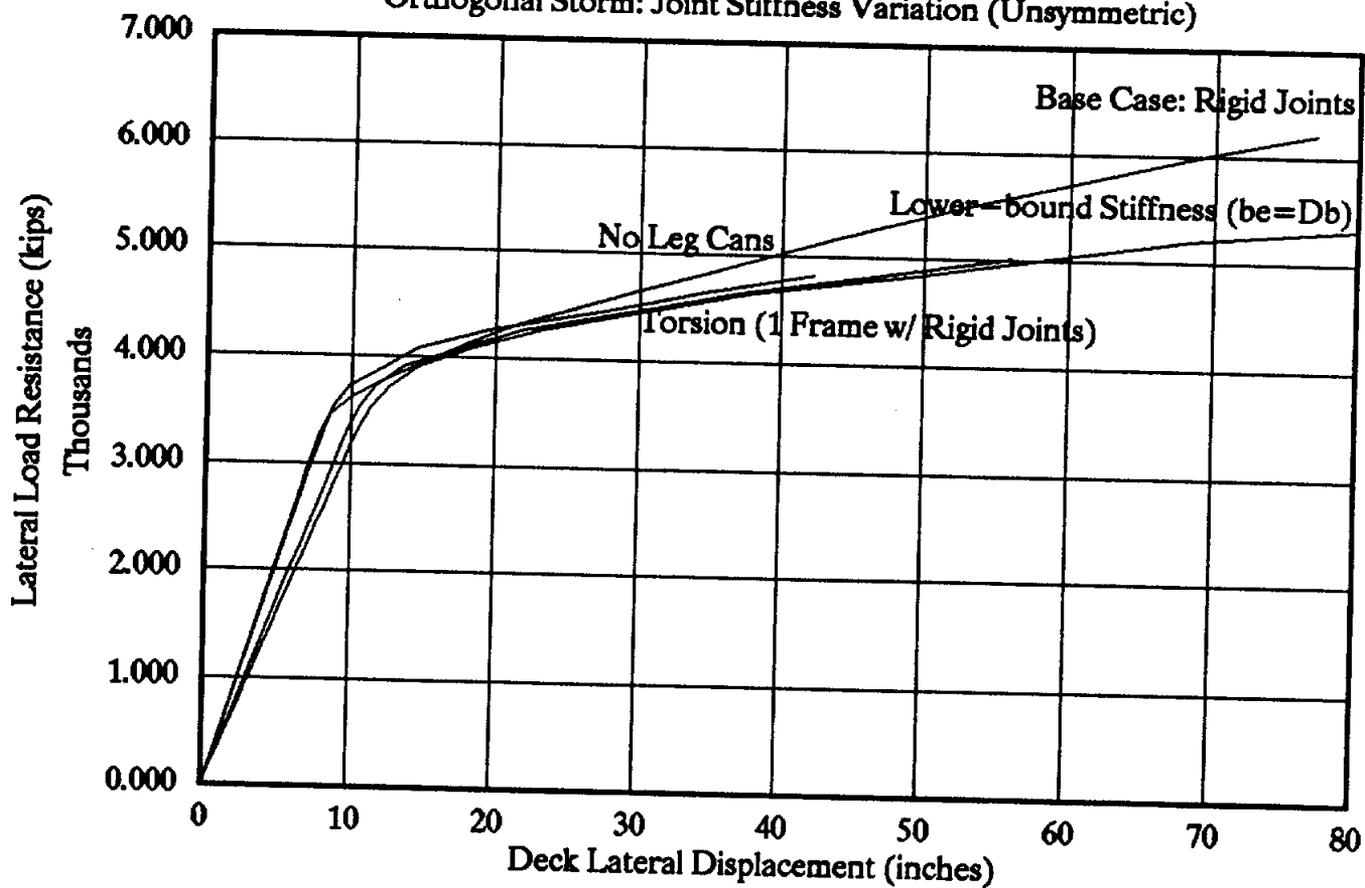
API JIP Benchmark Platform Pushover Analysis Results
 Orthogonal Storm: Joint Stiffness Variation



Note: Platform pinned at mudline for these parametric analyses.

Figure 9.7-5
 Benchmark Platform - Joint Flexibility Study

API JIP Benchmark Platform Pushover Analysis Results
 Orthogonal Storm: Joint Stiffness Variation (Unsymmetric)



Note: Platform pinned at mudline for these parametric analyses.

Figure 9.7-6
Benchmark Platform - Joint Flexibility Study (Unsymmetric Case)

10. FOUNDATION CONSIDERATIONS AND PLATFORM RESPONSE

10.1 FOUNDATION SYSTEM DESIGN AND MATERIALS (Groups/Skirts, Battered)

Typical Gulf of Mexico platform foundation systems consist of either battered or non-battered piles, usually distributed as individual pin or skirt piles, but sometimes distributed as group piles of two or more piles surrounding a corner leg.

At sites with low environmental load and/or soft soils, skirt piles are usually more efficient in resisting vertical load since the load transfer to the piles is more direct and group effects on soil resistance are eliminated. Group piles are sometimes used in firmer soils or where lateral loading is relatively high since group effect reductions are less and pile groups can take advantage of the increased lever arm for resisting overturning loads.

Battered piles are more efficient in resisting lateral loads since a large portion of the lateral load is resisted axially by the pile. This requires less pile steel, in general, than resisting the lateral load by lateral deformations of the pile and soil. Vertical piles are common among the first generation platforms and it is for these platforms that particular care must be taken in interpreting and using soil data.

10.2 PILE AXIAL CAPACITY

Pile axial capacity is a function of the pile-soil skin-friction ("t-z") and end-bearing ("q-z") resistance properties, and to a secondary extent, the pile cross-sectional area.

10.3 PILE LATERAL CAPACITY

Pile lateral capacity is a function of the pile-soil lateral ("p-y") resistance and the pile flexural stiffness.

10.4 FOUNDATION DAMAGE

Potential foundation damage includes soil failure, scour, pile plunging and pile pullout. A platform inspection should reveal specifically search for such damage. Evidence of such damage is of serious concern and would generally be grounds for rejection of requalification unless significant remedial measures are taken. Any such damage should be included in a requalification assessment.

Such damage should be included in a requalification analysis by assuming soil resistances compatible with the observed damage. For example, soil failure in one direction could be accounted for by reducing or eliminating soil resistance for the subject pile or piles in that direction. Scour could be accounted for by eliminating soil resistance over the scour depth. Pile plunging or pile pullout are more difficult damages to account for unless the magnitude of pile load causing the damage can be quantified.

10.5 CONDUCTOR LOAD RESISTANCE

Conductors may increase the platform lateral load capacity although any increase is usually neglected in design. The amount of the increase is a function of various parameters, including:

- Number, size and relative lateral stiffness of conductors compared to the piling
- Conductor support details and condition

Most conductor supports are designed to minimize conductor axial load transfer by providing only lateral support to the conductors. Hence, only the lateral load resistance is an important contributor to platform ultimate capacity. The relative importance will be directly proportional to the number and size of conductors relative to the number and size of piling. In relatively soft soils, such as those found in the Gulf of Mexico, the relative importance will be reduced significantly if the platform piling is battered in the direction of the applied load since the horizontal stiffness of battered piling will be significantly greater than the horizontal stiffness of the conductors. In calcareous soils, the conductor stiffness would be relatively more significant since the relative lateral soil resistance increases whereas the relative axial soil resistance decreases.

10.6 STUDY OF FOUNDATION BEHAVIOR AND PLATFORM RESPONSE

The benchmark platform was analyzed, using nonlinear static pushover analysis techniques, for the diagonal storm loading pattern. For a base case, the actual soil skin friction ("t-z"), end bearing ("q-z") and lateral resistance ("p-y") soil data were determined based on API RP 2A recommendations and input into the analysis. The pile size variations were also accounted for. The nonlinear effects of these pile and soil properties were analyzed at 50 equal locations along the pile length.

Additional parametric analyses were performed varying the input soil properties. Soil strengths of 50% and 200% of the base case t-z, q-z and p-y data were analyzed.

10.7 FINDINGS AND CONCLUSIONS

Under a diagonal storm load, the first mode of failure of the benchmark platform is "pile plunging." The "pile-plunging" mode consists of failure of the pile and soil in skin-friction and end-bearing. For the benchmark platform, more than 90% of the pile axial resistance is in skin-friction. The results of this parameter study are shown on Figure 10.7-1. This study shows that a variation in soil strength by a factor of 2 will lead to an even greater variation in calculated platform capacity. Thus, the importance of proper foundation modeling and nonlinear analysis can not be overemphasized.

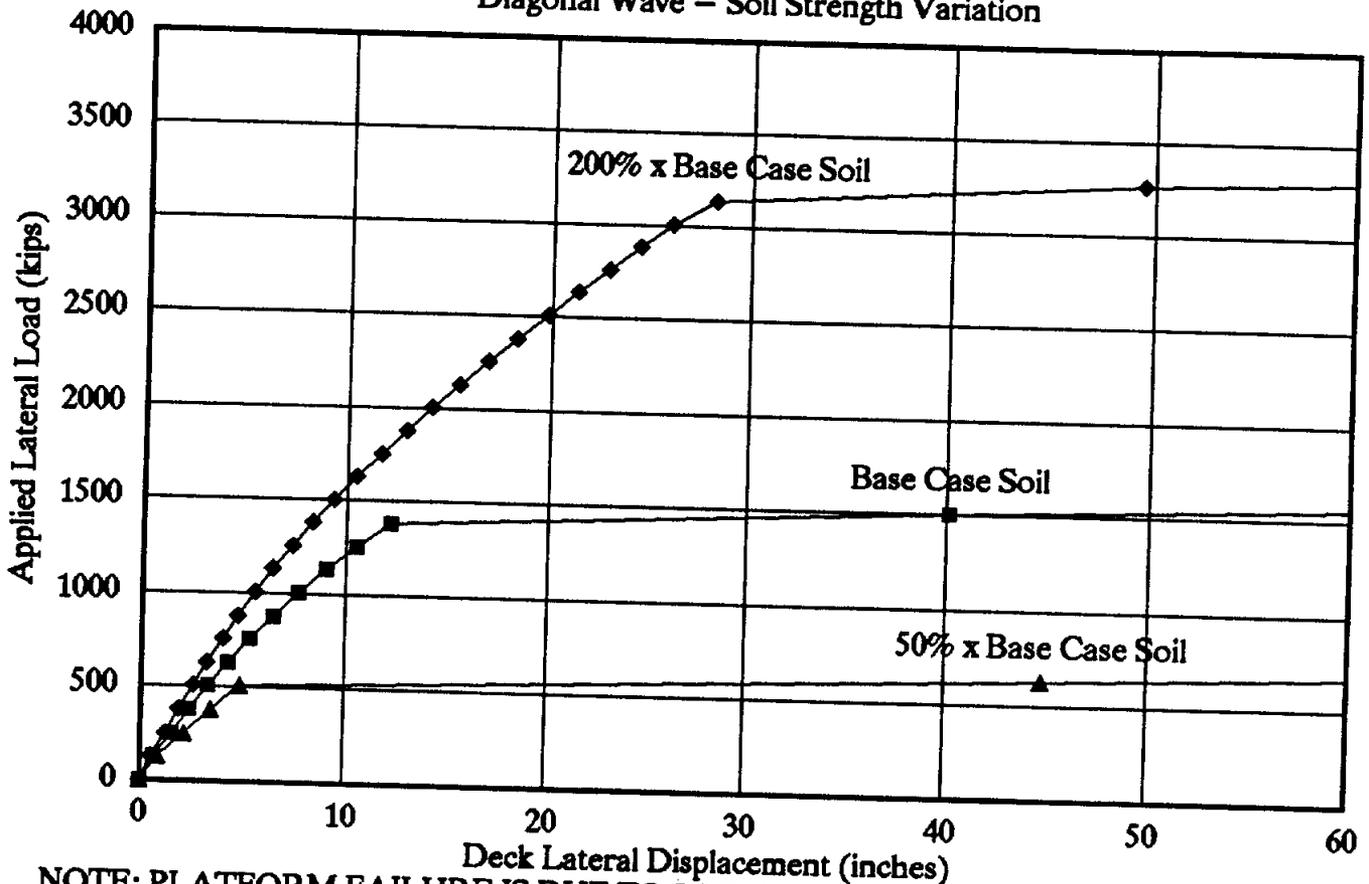
This first mode of failure is common among Gulf of Mexico platforms, especially those of first-generation design. Thus, for a requalification analysis where foundation failure is the first mode, the platform ultimate capacity is extremely sensitive to the input soil properties, pile modeling, and nonlinear techniques used to represent the foundation.

This foundation study has only quantitatively evaluated the effect of soil strength variation.

Other experience in nonlinear analysis of platforms leads to the following recommendations:

- The actual pile-soil properties should be properly accounted for in any requalification analysis. Linearized representations (e.g., linear springs) are inadequate since they are correct at best at only one load level. Linearized representations also produce incorrect load distributions into adjacent platform structure leading to possible incorrect evaluation of this structure.
- Variations in pile size with depth are common. This variation is important to the effective nonlinear stiffness and load distribution of the pile and soil. The size variation should be modeled. Averaging or smearing techniques are inadequate because the effective length of pile varies under load and varies from pile to pile.
- Pinning or fixing the foundation at mudline will underestimate the actual axial load in the pile. The actual pile inflection point is at some distance below mudline (this is the actual location for a pinned representation of the pile); however, the location varies from pile to pile and varies with pile load due to platform overturning moment. The corollary to this is that pile axial load cannot be accurately predicted based on platform overturning moment at the mudline and will be underestimated unless care is taken.

API JIP Benchmark Platform Pushover Analysis Results
 Diagonal Wave – Soil Strength Variation



NOTE: PLATFORM FAILURE IS DUE TO SOIL COMPRESSION FAILURE FOLLOWED BY SOIL TENSION FAILURE AT OPPOSITE PILE.

Figure 10.7-1
 Benchmark Platform - Soil Strength Variation Study

11. IMPLICATIONS OF FINDINGS AND CONCLUSIONS

11.1 DEFINITION OF KEY PARAMETERS AFFECTING PLATFORM RESPONSE

Although there are numerous parameters affecting platform response to applied loads and platform overall integrity, the primary parameters may be best addressed as to their actual effect on the platform and the implied effect on computer analyses of the platform.

The following two sections address these parameters and their role in the development of an assessment guideline.

11.2 BASIS FOR PLATFORM IN-SERVICE INSPECTION REPORT ASSESSMENT

The key parameters used in the development of a guideline for platform in-service inspection and assessment are:

- (1) Material imperfections introduced during fabrication that reduce elastic buckling stress of a component.
- (2) Residual stresses introduced during fabrication that reduce material plasticity factor and critical buckling stress.
- (3) Dents introduced due to impact during in-service condition that reduce critical buckling stress and member capacity.
- (4) Poor maintenance and corrosion that reduce member and component wall thickness will both increase applied stresses and reduce capacity.
- (5) Fatigue cracks initiated due to cyclic loading will reduce component stiffness and capacity.
- (6) Reduced load carrying capacity of a component will result in the alteration of a load path and increased loading on other components.
- (7) Installation of additional caissons and risers, subsistence or addition of component structures below the lower deck level will result in increased environmental loads.
- (8) Uncontrolled marine growth will result in increased environmental loads.
- (9) Installation of additional deck equipment will increase pile loading and reduce pile/foundation capacity to resist environmental loads.

Discussion of these parameters, an inspection checklist and the recommended actions are presented under separate cover in a document entitled: "Guidelines for Assessment of Platform Requalification Analysis Results."

11.3 BASIS FOR PLATFORM ANALYSIS MODELING, METHODOLOGY AND REPORT ASSESSMENT

The parameters used in the development of a guideline for the evaluation of platform assessments and/or requalification reports include a wide range of assumptions, modeling techniques and applied methodology. Some of these are:

- Accuracy of platform model and introduced simplifications that will affect the magnitude and distribution of applied environmental loads.
- Applicability of assumptions made on environmental criteria, including the wind and current profile, wave height and period range and the compatibility of combinations of wind, current and wave direction.
- Applicability of the proposed wave theory for the given wave height/period range and water depth and the methodology used to compute applied loads on each component member.
- Validity of computed member capacities and the effect of relative changes in joint stiffness and deformation on member slenderness ratio (i.e., "k" value) and load carrying capacity.
- Modeling of joints and the effect of joint deformation on load redistribution and strain hardening.
- Modeling of piles and foundation material to represent elasto-plastic behavior of foundation material and correctly represent variations in pile axial and lateral resistance on load redistribution.
- Appropriate definition of component member and joint post-buckling capacities.

Discussion of these parameters, an inspection checklist and the recommended actions are presented under separate cover in a document entitled: "Guidelines for Assessment of Platform Requalification Analysis Results."

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**GUIDELINES FOR ASSESSMENT OF
PLATFORM REQUALIFICATION ANALYSIS RESULTS**

**POST MORTEM PLATFORM FAILURE EVALUATION STUDY
CONTRACT NO. 14-35-0001-30747**

**MINERALS MANAGEMENT SERVICE
DEPARTMENT OF INTERIOR
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3.	GUIDELINES CHECKLIST								

1. INTRODUCTION

1.1 BACKGROUND

Platform requalification assessment can be a complex exercise which may involve complicated in-depth nonlinear analyses of platform structures. Such analyses should consider a variety of parameters including, but not limited to, the following:

- applicable loading criteria
- discrete modeling of platform members, joints and foundations
- initial failure and post-failure capacities of members, joints and foundations

Personnel who review, assess or approve submitted requalification analyses results may or may not be completely up-to-date on the techniques and specific details inherent in such analyses. This document provides a guideline to the more significant parameters and the conditions under which they may significantly affect the requalification assessment.

1.2 SCOPE

These guidelines are organized as follows:

- General guidelines covering general platform configurations, loading criteria and structure components.
- Specific guidelines pertaining to platform foundation considerations.
- Specific guidelines pertaining to platform member considerations.
- Specific guidelines pertaining to platform joint considerations.

2. GUIDELINES

This section presents important considerations in performing and evaluating a platform requalification assessment. Section 3 summarizes this presentation in a guidelines checklist.

2.1 GENERAL

The subject platform should be qualitatively reviewed to get a feel for the relative redundancy and/or ductility inherent in the platform. Basic structure considerations are:

- number of platform legs (e.g., three-, four- or eight-legged)
- type of vertical bracing (e.g., k-bracing, x-bracing)
- general platform condition (e.g., excessive corrosion, significant denting or missing members, weld failure, pile scour)

A platform requalification assessment should document, at a minimum, the following:

- Type (e.g., storm, seismic) and direction of loading conditions analyzed. Justification for the selected load type(s) and directions should be provided. The minimum number of load directions would be two for platforms that are completely symmetric about all axes in terms of structure and applied loading. For example, a three- or four-legged platform with identical bracing sizes and configuration on all vertical frames would require load application in at least one orthogonal and one diagonal direction. However, platforms usually are not loaded symmetrically due to the presence of various apertures (boat landings, conductors, caissons, risers) and their load resistance may not be symmetrical either. It does not take much effort to determine only the applied loads in eight directions (four orthogonal and four diagonal) as a justification for determining the critical loads and directions to be used in a complete analysis.

- API RP 2A metocean and/or seismic parameters and associated applied loads, including total applied base shear and overturning moment. Are the applied, overturning moment and base shear consistent with respect to magnitude, direction and effective center of load application? Generally, the center of load application is within the upper 20 percent of waterdepth. Is a maximum load condition applied and has this been demonstrated? The applied wave load will vary with wave crest position; the position for maximum loading in each direction should be determined.

- API Section 17 metocean and/or seismic parameters and associated applied loads, including total applied base shear and overturning moment. The API RP 2A considerations above also apply here.

- Applied load level at which the first structure component becomes nonlinear. When compared with the platform ultimate capacity, this load level is an indication of the relative reserve capacity of the platform which should be consistent with the

The ultimate capacity of older-design platforms located in relatively stiff soils may be

MEMBER CAPACITY CONTROLLED PLATFORMS

2.3

- The pile capacities need to be determined, documented and validated. Pile capacity information can be used to establish the validity of reported platform ultimate capacities.
- The variations in pile properties with pile depth. Single prismatic member modeling will be insufficient since most of the pile lateral load is resisted by the upper portion of soil while most of the pile axial load is resisted by the lower portion of the soil. The entire pile should be modeled with precision. Incorrect modeling can produce incorrect results with respect to pile axial load and overturning moments which may be either conservative or unconservative.
- The nonlinear characteristics of the soil lateral, skin friction and end bearing resistance. The nonlinear soil characteristics should be modeled with a fair degree of precision. Linear interpolations over great lengths of pile should be assessed for the level of conservatism or unconservatism. Nonlinear foundation elements that do not properly account for the local nonlinear soil characteristics should be evaluated with caution.
- Assuming that foundation pile and soil characteristics are significant factors in the determination of platform ultimate capacity, the following should be considered:

Most platforms located in soft soils will have ultimate capacities that are controlled by foundation considerations. Older-design platforms located in the Gulf of Mexico are typical of this condition. Platform requalification assessments that, without justification and/or proper analysis, do not report foundation limitations for such platforms should be treated with suspicion.

FOUNDATION CONTROLLED PLATFORMS

2.2

- Reserve strength ratio (RSR) for each analyzed load condition. Is the reported minimum RSR marginal or more than adequate? If marginal but acceptable (say less than 20 percent in excess of the minimum required), specific potential sensitivities of the analysis results will need to be addressed.
 - Failure mode and/or mechanism for each analyzed load condition. This information serves as a basis for specific component related questions and sensitivity assessments. The platform ultimate capacity is usually limited by foundation, member or joint component capacity considerations. If one class of component controls the ultimate capacity by a large margin, then any further evaluation can be concentrated on those elements and their corresponding significant considerations.
- overall platform redundancy (i.e., number of legs/piles, type and redundancy of framing, etc.)

limited by member capacities. This would also be expected for properly designed platforms since it is the intent of present-day designs to make the bracing members the weak link with respect to platform failure.

Initial-failure capacities are well understood for members in their original design condition. However, most platforms subject to requalification will have members with significant corrosion and possible dents or more significant damage due to impact. In addition, the post-buckling capacity of platform members has a significant effect on platform ultimate capacity for platforms with relatively robust foundation and joints.

Assuming that member capacities are a significant factor in the determination of the platform ultimate capacity, the following member characteristics should be considered:

- Member post-buckling capacity. This should be individually determined for each compression member. Global reduction factors should be discouraged unless they can be shown to be overly conservative while not affecting the result of the assessment.

- Member corrosion. The thickness of corrosion should be documented. Member thicknesses should be reduced when determining individual member capacities.

- Member denting or misalignment. Misalignments should be specifically modelled. Member capacities should be reduced to account for significant denting or damage. Neglecting a damaged or misaligned member in the analysis should be conservative.

- Member slenderness ratios. Actual brace member slenderness ratios are generally 10 to 20 percent less than values assumed in design, but the increase in individual brace capacity will usually be only a fraction of this percentage. The assumption of a reduced actual (or theoretical) slenderness ratio should be documented but such an assumption will only be relevant for requalifying marginal platforms.

2.4 JOINT CAPACITY CONTROLLED PLATFORMS

A major consideration for platform tubular joints is whether the joint capacity exceeds the connecting brace capacity. If it does not, the brace load must be limited to the joint capacity in the requalification analysis (linear or nonlinear). This will be the typical case for older-design platforms without thickened and/or grouted joint chords. Requalification assessments that, without justification, do not report limiting joint capacities should be treated with suspicion.

Assuming that the platform joint characteristics are a significant factor in determining the platform ultimate capacity, the following should be considered:

- Are simple unstiffened tubular joint capacities quantified and/or included in the assessment. If not, is this exclusion justified? Justifications may include joint grouting; if so, has the grout capacity been demonstrated and is the actual grout on the platform been inspected and known to be viable?

■ Are more complex and/or stiffened tubular joint capacities quantified and/or included in the assessment. If not, is this exclusion justified? If stiffening were not considered, would the equivalent unstiffened joint capacity suffice?

Simple unstiffened tubular joints can be classified, depending on framing and load distribution, as one of three types: x-joints, t-joints and k-joints. For most simple four-legged platforms, the classification of the platform joints is usually straightforward. Four the more complicated eight-legged platforms, the proper classification can be more difficult. API RP 2A provides guidance to joint classification; this guidance is reproduced on Figure 2.4-1.

Once the joint has been properly classified, API RP 2A punching shear recommendations can be used to determine the punching shear capacity of the joint. These recommendations also include the crushing capacity for t-joints and for x-joints with approximately equal chord and brace diameters (crushing should not control k-joints). The minimum capacity for these joints, including crushing effects where they govern, are presented on Figures 2.4-2 through 2.4-3.

These figures establish the maximum joint capacity as a ratio of brace capacity. This ratio is a function of geometry only; actual load distribution is not required. The reviewer can simply calculate the platform joint coefficients for each class of joint on the platform and determine whether each specific joint will limit the brace capacity. A ratio less than 1.0 means that the joint capacity governs and the brace capacity should be limited to the joint capacity.

Submitted qualification assessment results should document joint capacities at or below tabulated joint capacity values. The figures may also be used to validate the assessment procedures and assumptions as they pertain to platform joints. Stiffened joints require capacity assessment on a case by case basis, usually utilizing rigid ring analysis formulations to determine the joint capacity.

Figure 2.4-1
API RP 2A Tubular Joint Classification

FIG 4.3.1-1
EXAMPLES OF JOINT CLASSIFICATION

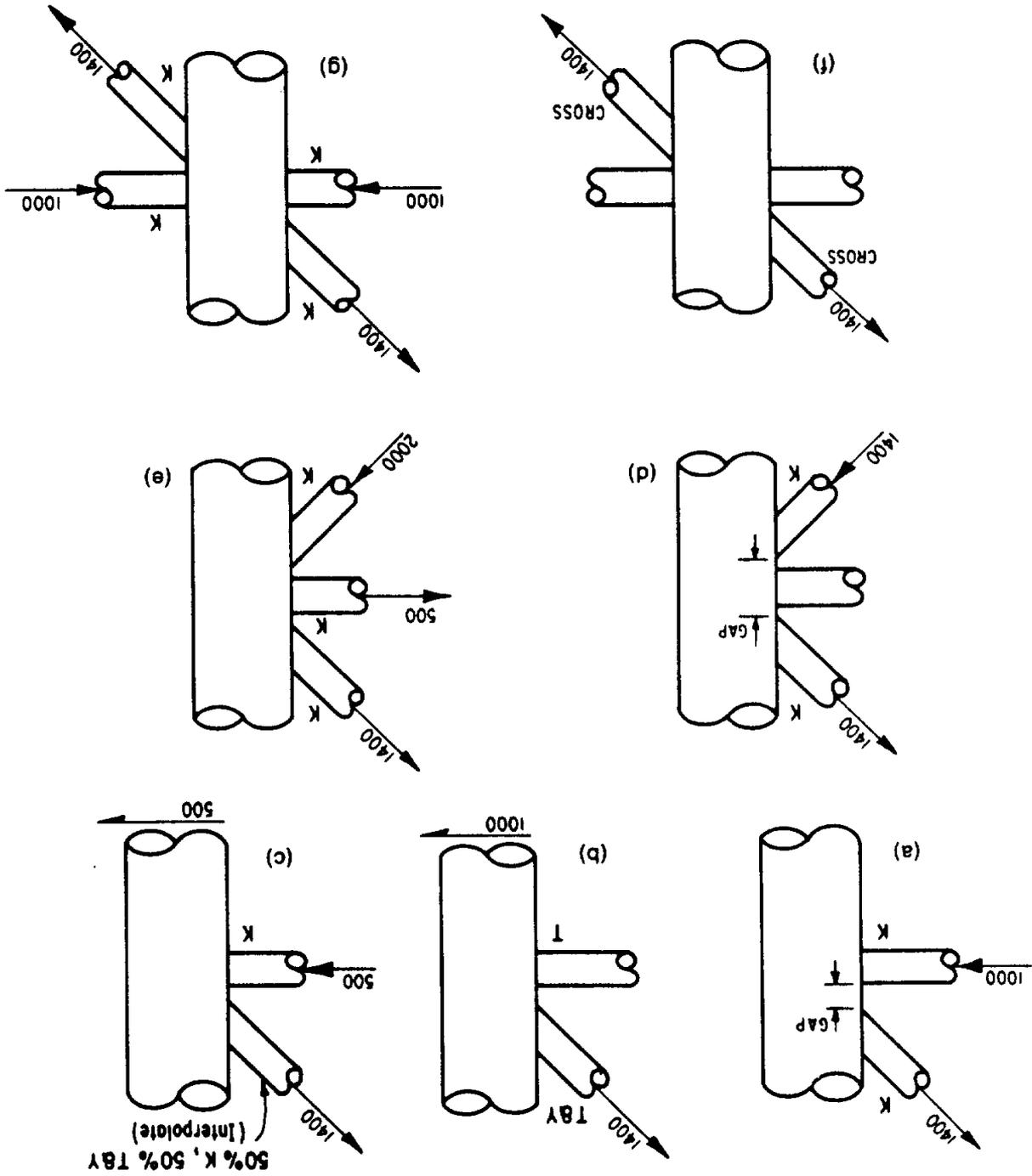


Figure 2.4-2
 Brace Capacity Reduction Factors for T-Joints

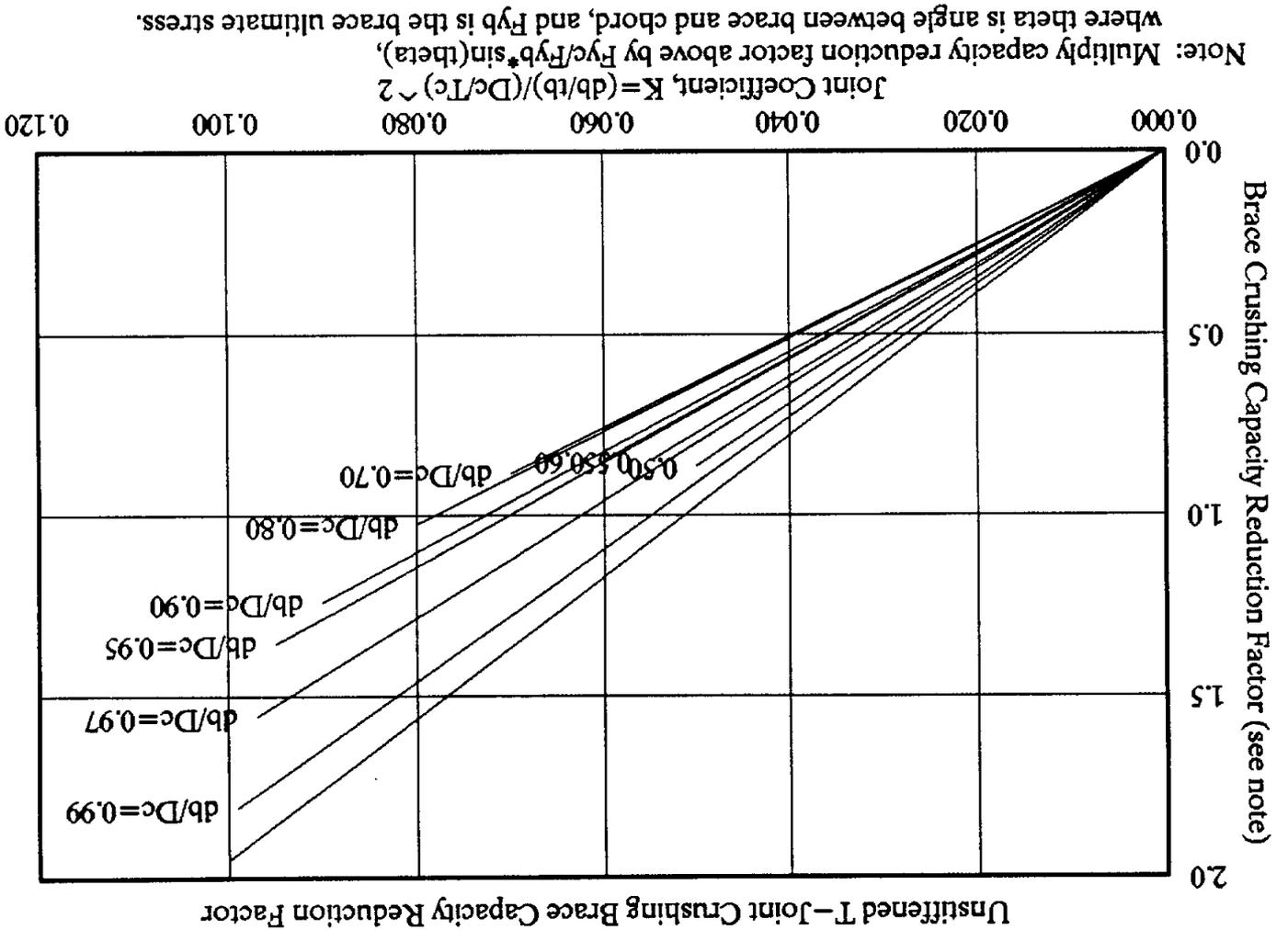
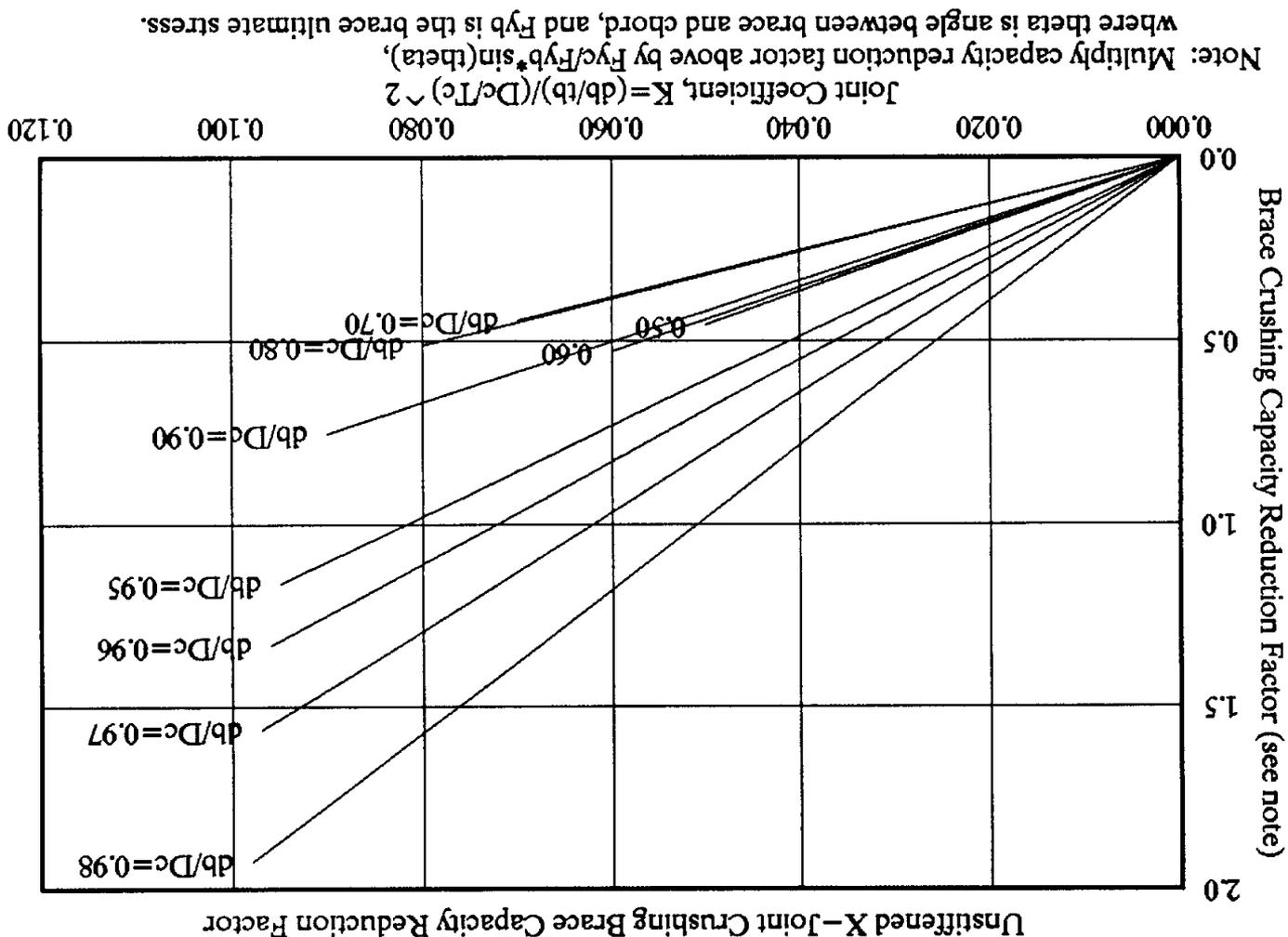


Figure 2.4-3
Brace Capacity Reduction Factors for X-Joints



3. GUIDELINES CHECKLIST

A checklist covering the previously discussed guidelines is presented. The general section of the checklist covers general platform parameters that provide a general feel for the robustness and/or ductility of the platform. These items by themselves do not directly validate a qualification assessment but would highlight potential sensitivities to be addressed during the assessment or evaluation.

The specific component topics should be considered significant when it has been shown that the particular component is a significant factor in the determination of the platform ultimate capacity. Negative implications of such factors should be investigated further. Additional analyses and/or documentation may be required to alleviate concerns about the assessment validity.

GUIDELINES CHECKLIST

CHECKLIST ITEM DESCRIPTION		RATING		
		Good	Fair	Poor
PLATFORM:		> 4 legs	4 legs	3 legs
■ No. of legs				
■ Vertical bracing type		x-braced		k-braced
■ Soil resistance		stiff soil		soft soil
■ General platform condition		good	fair	poor
LOADING and ANALYSIS:		6 or more	3 to 7	2 or less
■ Load directions				
■ Maximized API RP 2A loading achieved?		yes		no
■ First structure nonlinearity documented?		yes		no
■ Failure mode/mechanism documented?		yes		no
■ Failure mode?		members	joints	foundation
■ Reserve Strength Ratio (RSR)?		ADEQUATE	MARGINAL	INADEQUATE
FOUNDATION: (Rate if foundation controls ultimate capacity)		yes		no
■ Detailed soil properties modeled/analyzed?		yes		no
■ Detailed pile properties modeled/analyzed?		yes		no
■ Pile capacities determined, documented and validated?		yes		no
MEMBERS: (Rate if member characteristics control ultimate capacity)		yes	no	
■ Individual member post-buckling capacities considered?			yes	no
■ Global member post-buckling reduction capacities used?		no	unknown	yes
■ Member corrosion?		no	unknown	yes
■ Member denting or damage?		no	unknown	yes
■ Theoretical member slenderness ratios?			no	yes
JOINTS: (Rate if joint characteristics control ultimate capacity)		yes		no
■ Unstiffened joint capacity reductions used?		yes		no
■ Stiffened joint capacities documented?		yes		no
OVERALL:		good	fair	poor

