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by

Rajiv Kumar Aggarwal

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ABSTRACT

This dissertation focuses on the development of a methodology for assessment of the safety of existing steel jacket platforms. The primary focus of this study has been on a methodology that can be used by regulatory bodies, owners, and operators to assess the safety of the offshore platforms located in the Gulf of Mexico.

The safety assessment of ageing offshore platforms has become of increasing importance in the recent years in the United States and worldwide in order to ensure their safe operation and continued production of crude oil and gas. This has created a dilemma for the government regulatory bodies, owners, and operators in the United States, where a major number of platforms exist. In recent years, their interest in maintaining the safety of offshore platforms against loss of life, environmental pollution, and loss of resources and property has increased due to the awareness of the public of the consequences of their failure.

Regulatory bodies, such as the Minerals Management Service (MMS) established in the United States, have a role to play to ensure that the structures operating in the offshore waters are safe to life, environment, property and loss of production. However, they do not have a methodology to make such routine assessments for the more than 4,500 platforms operating in the Gulf of Mexico.

In this study, a 4-cycle screening methodology has been developed for making routine assessments of the safety of platforms against storm wave loads. Simplified techniques have been developed for platform capacity evaluation which will be of special importance for identification of potentially critical platforms. Judgements on their safety can be made based on a probabilistic approach, which utilizes the nominal estimates and the uncertainties in the load, strength, and consequences of the failure of a platform.

Although the emphasis of this research is on safety assessment of the Gulf of Mexico platforms against the storm wave loads, the principles and theoretical background developed will be of use for assessment of the offshore structures in other areas and against other load sources.

The ideas developed in this study could also be utilized in the re-qualification and rehabilitation of other civil engineering structures. In the State of California, CALTRANS, a government body, needs a process to evaluate the safety of more than 9,000 existing bridges. There have also been increased concerns for safety of public buildings, and other important civil structures.

This research is believed to be timely, and of importance to regulatory bodies, owners, and operators of offshore platforms.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION AND OVERVIEW

1.1 Objectives and Need for this Project

The first steel platforms were installed in 1947 in the Gulf of Mexico, and since then they have been extensively used in the development of offshore fields around the world. The design criteria for these platforms has changed significantly over the past 45 years. Safety of these platforms depends upon the criteria and procedures used in their design, operation, and maintenance.

In the United States, the public responsibility for the structural and operational safety of the offshore platforms in the federal waters falls under the jurisdiction of the Minerals Management Service (MMS). Platform owners and operators have the responsibility for safe operation of the platforms and to maximize their utility. In order to facilitate maintenance of these platforms in a safe state, guidelines have been developed by the industry and the regulatory bodies.

The American Petroleum Institute (API), formed by the industry, first introduced recommended practices for the offshore platforms (API-RP-2A) in 1969, to provide a consistency in their design, fabrication, and installation. API has periodically updated these recommendations, and the 19th edition [*API, 1991*] now forms the general standard for design of the Gulf of Mexico platforms.

The present day Minerals Management Service (MMS), U.S. Department of Interior, introduced new requirements, 30 CFR part 250, which largely follow the API guidelines with some additions [*MMS, 1988*].

these requirements, the Outer Continental Shelf (OCS) Order No. 8 introduced in 1979 was followed, which provided mandatory requirements for maintaining the safety of platforms [*USGS, 1979*]. The current requirements define the need for periodic assessment of the existing platforms for their continued use (Clause 250.142a) [*MMS, 1988*]:

> All platforms installed in the OCS shall be inspected periodically in accordance with the provisions of API RP2A, section 7, Surveys. However, use of an inspection interval which exceeds 5 years shall require approval by the Regional Supervisor. Proper maintenance shall be performed to assure the structural integrity of a platform as a workbase for oil and gas operations.

Due to significant variations in the configuration of the platforms and the technology used to design, operate, and maintain them over the past 45 years, their structural and operational safety levels differ. The structural integrity of these platforms against the various sources of overload or deterioration of the structural components is most important for their continued operation in a safe state and for their optimum utilization. In addition, operational procedures and equipment must ensure safety against significant loss of life, and pollution.

Some of the primary factors which have influenced the structural and operational safety levels of offshore platforms are listed below:

1) <u>Significant difference from present acceptable load criteria:</u> A large number of platforms installed before 1965 and still in operation, were designed for the 25-year return period waves, whereas most of the platforms installed after 1965 are likely to have been designed for criteria based on the 100-year return period storm conditions. Hence, the load level based on the present

acceptable criteria on the earlier platforms is likely to be significantly higher than that of their original design criteria.

2) Change (reduction) in strength level: The main reasons for change in the strength level of the platforms are as follows:

a) Ageing of structure: The platforms which are operating beyond their original design life of 20-25 years are prone to reduction in their strength level due to the ageing processes such as corrosion, crack formation, and crack propagation (fatigue).

b) Damage of members: The strength of platforms may be reduced due to damage of their components from other load sources over their in-service period (dropped objects, collisions).

c) Modification of platforms: Modifications made to the structure and functions of the platforms have a direct effect on their structural and operational safety levels (e.g., added risers and operational loadings).

3) Salvaged and reused platform: In some cases, old platforms have been salvaged and reused at different locations, frequently with little or no refurbishment. The safety level of such platforms may not be comparable to the equivalent new platforms due to the reasons mentioned under 1 and 2.

4) Significant increase in number of operators: The number of operators in the Gulf of Mexico have increased from 64 in 1980 to more than 120 at present [*Arnold et al, 1989*]. The operational philosophy followed by an operator influences the Inspection, Maintenance, Repair (IMR) program followed; the personnel access and evacuation program; and the production safety measures incorporated. Hence, the IMR philosophy followed is likely to vary considerably, which may have a direct influence on the structural integrity of a platform and the consequences if it were to fail.

As discussed, a very large number of platforms exist with a significant variation in their functions, loading, structural, and foundation characteristics. In addition, the present condition of a platform may significantly differ from its "as-designed" state, due to its damage or degradation (age-effects); the operational and IMR philosophy followed; and the modifications made. Therefore, such platforms may not necessarily meet the structural and operational safety standards followed today.

Hence, a methodology is needed for periodic evaluation of the suitability for service of a large number of platforms and to ensure that they meet the minimum requirements promulgated by API, MMS, and the State regulatory bodies, to provide a safe workbase for oil & gas operations. Such a methodology should consider large variations in their load, strength, and operational parameters, and the limited resource in terms of time, cost, and manpower available with a regulatory body to undertake such a task.

In order to meet the above needs, a practical methodology for safety assessment of the existing platforms has been developed in this study.
The following three objectives were defined for this study:

a) To develop a screening process to group platforms according to the degree of their fitness for purpose.

b) To develop a practical and economic technique for determining the Reserve Strength Ratio (RSR), an index of the structural safety of the platforms, and

c) To demonstrate the feasibility of a computerized knowledge based expert system for organizing the knowledge components of this methodology and assisting with its implementation.

These three objectives were focussed on drilling and production oil and gas platforms, and operations in the Gulf of Mexico (GOM).

Further in this chapter, an overview of the development of offshore platforms in the United States is made, which aims at describing the importance and application of this study. The failure modes and mechanisms against which the safety of a platform should be assessed are then identified and confirmed with review of the actual cases of platform failures in the Gulf of Mexico. Then, an overview of the safety assessment methodology followed in this study is presented.

1.2 Overview of Offshore Platform Development

The majority of offshore steel platforms have been used to provide a fixed base for hydrocarbon production. Currently, there are more than 4,500 offshore platforms located in the United States. On the Outer Continental Shelf (OCS) of the Gulf of Mexico alone, there were over 4,400 structures, with approximately two-thirds in water depths up to 100 ft. [*Dyhrkopp, 1990*]. The Pacific OCS has 23 structures of various ages. In addition, there are many old structures in the state waters of Alaska, California, Louisiana, and Texas.

The Minerals Management Service (MMS) groups the platforms currently operating in the OCS of the Gulf of Mexico under two categories: major and minor. MMS considers the platforms provided with 6 conductor slots or more, or with at least 2 pieces of production equipment, as major platforms. The other platforms are classified as smaller structures, quarters platforms, or equipment platforms. The number of platforms, grouped according to their age (period since installation), under these categories are given in Table 1.1.

From Table 1.1, it is noted that 1,643 (37 %) of the total 4,400 (year-1,990 figure) platforms are more than 20 years old i.e., they were installed before 1970 and may be operating beyond their original design life.

Table-1.1: Platforms Operating in the Gulf of Mexico- Outer Continental Shelf [Based on statistics given in *Dyhrkopp, 1990*]

S. No.	Age Group	Year Installed	Major #1 Platforms	Minor #2 Platforms	Total Platforms
1.	> 25 years	Before 1965	370	646	1,016 (23.1%)
2.	20-25 years	1965 - 1970	298	329	627 (14.3%)
3.	15-19 years	1971 - 1975	290	178	468 (10.6%)
4.	10-14 years	1976 - 1980	418	301	719 (16.3%)
5.	1-10 years	After 1980			1,570 (35.7%)
	TOTAL				4,400

#1: Platforms with 6 conductor slots or more, or with at least 2 pieces of production equipment

#2: Smaller platforms, quarters platforms, or equipment platforms.

Note that in addition, approximately 148 platforms are installed each year in the GOM.

An approximately equal number of platforms, 1,570 (36 %), were installed after 1980 i.e., they have been in operation for less than 10 years. The remaining 27 % have been in operation for the intermediate period of 10 to 19 years.

The structural configurations of these platforms range from single-well caissons to multi-leg jacket or tower platforms. In 1987, there were 1,969 (64%) wellhead platforms, 355 (11%) tender-assisted platforms, and 756 (25%) self-contained drilling & production platforms in the Gulf of Mexico OCS [*Dodson, 1987*]. The wellhead platforms are single well caissons or multi-well 4-legged jacket platforms. The tender-assisted platforms are usually 4-legged jackets and the self-contained platforms are mostly 8-legged jackets or tower platforms.

Caissons consist of a large diameter cylindrical or tapered tube, installed in water depths up to about 125 ft., and driven into the seabed to penetrations of approximately 20-30 times their diameter. They support minimal equipment.

Jacket platforms are the most common type and they usually consist of 4- and 8- legs (or piles) constructed of thick tubulars. A significant difference in the structural configurations of the jackets is noted, because of the variations in their design criteria. Such variations are likely due to water depth; environmental and geotechnical parameters; selected construction and installation equipment; fabrication technology of the period; operational criteria; and design philosophy. The jacket sub-structure is transported on a barge, lift-installed by a derrick crane or launched by sliding along its one side at the site, upended, and placed on the seabed.

A tower platform is designed to self-float on one side from the fabrication yard to the installation site. It is provided with very large diameter legs on one side to provide floating capability to the structure. Such platforms have usually found application in harsh, remote environments such as the Cook Inlet, and the North Sea. A few of them were installed in the Santa Barbara Channel (Southern California) due to the availability of small size barges during that period, and also in the unstable soil zones of the Gulf of Mexico.

The emphasis in this project is on the steel jacket platforms located in the Gulf of Mexico. A steel jacket platform can be divided into three major parts: deck (air-above water medium); jacket (water medium); and foundation piles (soil medium) as shown in Fig. 1.1. The interaction of these three parts of a platform with the surrounding media varies for different sites and depends upon the structural properties and configuration of the platform, the length and properties of the medium, the forces imposed by the medium, and

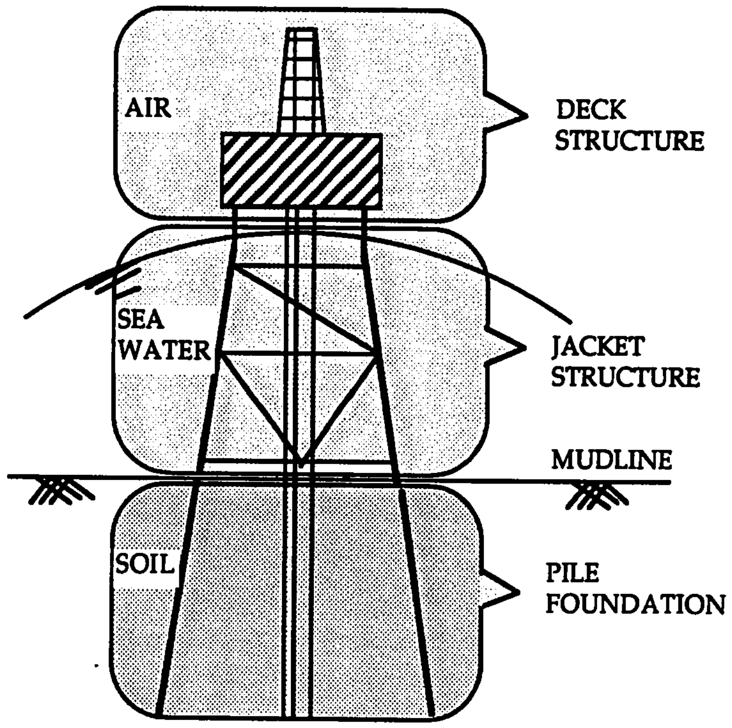


Figure 1.1: Major Parts of a Steel Jacket Platform

the forces absorbed by the medium. A number of complex phenomenon occur due to interaction of these media among themselves or with the structure.

Due to distinct differences in the structural configurations and behavior of these three parts, their failure modes and mechanisms differ. A summary of the failure modes and mechanisms in these parts, based upon a review of the past failures, is given in the next section.

1.3 Failure Modes and Mechanisms

A failure mode describes the mode in which a component member may fail. Some of the failure modes for a structural steel member are identified as: rupture; buckling, shear, bending, and tearing. Failure mode is initiated by overloading of a member above its current strength in that mode against the various load cases and their practical combinations.

A failure state (collapse mechanism) of a platform may form due to two reasons: successive failure of members; or propagation of partial failure due to the secondary moments induced by the vertical loads at large displacements of the platform (P-Δ effect). The number of members required to fail to form a collapse mechanism depends upon the degree of static indeterminancy of the structure or sub-structure under consideration. A failure state is reached when a part of the structure is unable to transfer the loads above it to the structure or foundation below it.

The possible failure modes and mechanisms in the three parts of a steel jacket platform, when induced by hurricane or storm waves, are given in Table 1.2. The directions of storm waves, which are more likely to develop these failure modes and mechanisms are also mentioned.

The likelihood of occurrence of these failure modes and mechanisms has been based upon review of the records of the past failures of platforms, and of the results reported in the literature for the failure analyses of a

Table 1.2: Possible Failure Modes and Mechanisms

Bay Type	Type of Mode or Mechanism	Storm Wave Direction
Deck Bay	Yielding of all deck legs	Diagonal
	Failure of deck leg-pile connection	Diagonal
Jacket Bay(s)	Buckling or yielding of all diagonal braces and jacket legs in the vertical frames between two adjacent horizontal levels	Orthogonal
	Joint failure in addition to buckling and/or yielding of one or more braces	Orthogonal
Foundation Bay	Yielding of all piles	Diagonal
	Pullout / plunging failure of piles	Diagonal

number of platforms. A summary of the failure modes, which occurred in the Gulf of Mexico platforms, is given in Table 1.3 and is presented according to the platform type and water depth [*Bea and Audibert, 1980; Cooper, 1967; Dyhrkopp, 1987; Lee, 1981; McClelland & Cox, 1976, Sterling and Strohbeck, 1975*]. The following observations are made from a detailed review of the past failures in the Gulf of Mexico due to hurricanes or hurricane-induced loads:

1) A total of 38 platforms collapsed or were severely damaged due to hurricane or hurricane-induced loads. These platforms were located in water depths from 30' to 327'. The actual failure modes are known for only 24 platforms out of the total 38 cases.

2) In most of the cases, the failure occurred due to overload from one or more of the following sources:

* Wave hit the deck: When the cellar (lower) deck elevation was too low, either due to use of 25-year wave design criteria or due to incorrect computations of the wave crest.

* Wave induced soil-movement occurred at the location: It resulted in additional loads on the platform from soil movement.

* Increase in wave load: When the platforms were designed for 25-year return period criteria instead of current 100-year wave criteria. In some cases, the thickness of marine growth on the members was higher than considered in the original design. All of the platforms except the four which were located in the mudslide zone were designed for the 25-year wave criteria.

* Increase in lateral loads: The platform was simultaneously impacted by the storm wave and supply ship.

Table 1.3: Failure of Platforms in the Gulf of Mexico due to Hurricanes (1947-1990)

PLATFORM TYPE	NO. OF PLATFORMS	WATER DEPTH (FT.)	FAILURE MODE
2-pile caisson brace	2	102	Brace-caisson connection due to: variable welding; and wave into deck.
Tripod well protector	6	30-125	Pile yielding
4-leg well protector	6	60-92	Pile pullout; Braces pullout(100-yr); Joint failure; Pile yielding; Mudslide (1-100yr-60' w.d.)
4-leg tender	4	50-192	Pile yielding
4-leg header	1	30	Corrosion and cracks in braces
6-leg tender	2	60	
6-leg self contained	1	87	
8-leg tender	7	50-215	Previously damaged; Braces failure; Deck leg shear; Joint failure; Vertical collapse
8-leg self-contained	5	172-327	Braces failure; Pile yielding; Mudslide(3no. 280'-327':100-yr)
8-Leg central facility	3	51-95	Storm and barge simultaneous; Corrosion; Holes in braces; Broken/missing members;
10-leg self-contained	1	87	
TOTAL	38	30'-327'	

3) In some cases, the soil strength adjacent to the pile foundation was insufficient to transfer the loads. Soil strength was reduced in one case due to jetting of the soil plug during installation.

4) In one case, the platform was damaged earlier due to ship collision and was unrepaired when the storm hit. The other damaged states noted for the platforms were as follows:

* Excessive Corrosion of the structural members
* Holes in the braces or legs
* Missing members in the jacket

5) The following six failure modes and mechanisms are reported for the 24 platform cases for which the type of failures are known:

* Yielding or bending of piles ---------- 9 platforms

  [Four of these platforms were designed for the 100-year wave design criteria, but were located in the mudslide zone.]
* Tension-pullout of piles ---------- 2 platforms
* Yielding of deck-jacket connection ---------- 2 platforms
* Yielding or buckling of braces ---------- 3 platforms
* Splitting of brace: Joint failure ---------- 3 platforms
* Settlement of jacket ---------- 1 platform

From this review, it is noted that overloading of a platform and its components could occur when: the load criteria recommended by the latest API edition is higher than the design load criteria; the wave hit the deck due to increase in the current wave design criteria or due to settlement of the jacket ; the wave-induced soil movement loads were not considered in design; the marine growth is excessive; the water depth changed due to the installation errors; and the supply vessels collided the platform simultaneously with the storm.

On the other hand, the capability of the components of a platform to transfer loads may reduce due to: damaged members not repaired ; defects exist in the welds; and insufficient strength of the major connections. In addition, the soil strength may be insufficient to transfer foundation loadings.

1.4 An Overview of the Safety Assessment Methodology

Safety assessment methodology should determine the structural strength of existing platforms against the load sources identified in Section 1.3, which may initiate failure of one or more of the structure components. The failure modes and mechanisms against which the strength of structure components and system should be checked were identified in the previous section.

In addition, it should determine the consequences upon failure of a platform to the environment, human life, property, and resource losses. These consequences should remain within acceptable limits, in case of insufficient strength of a platform against predicted loads over its remaining life.

In case these two criterion (adequate capacity and acceptable consequences) are met for a platform, then it can be classified as suitable for continued service in its existing state. An overview of the methodology, which is based on the evaluation of these two criterion, is presented in this section.

To evaluate the suitability for service of a platform, its current condition, the loads imposed and induced in the structure, the strength and capacity characteristics of the structure, and the potential consequences, if the platform fails to perform satisfactorily, are the four most important evaluation criteria. The characterization of the current condition of a platform could be improved with the implementation of an improved Inspection, Maintenance, Repair (IMR) program. The loading on and the strength of a platform are combined and expressed through Reserve Strength

Ratio (RSR = Ultimate Capacity/Minimum Reference Level Force). The RSR is an index that represents the overload capacity of the platform against the present requirements for design loading. The potential consequences upon failure of a platform would depend upon its physical characteristics, its functions, and the operational and safety philosophy followed.

Because of a large number of platforms, which are operating at an advanced age and which need a periodic re-assessment, rapid means are needed to enable evaluation of their safety and serviceability in continued service. In order to meet this criteria, a four cycle safety assessment methodology has been developed (Fig. 1.2).

At each of the four-cycles, essentially the capacity and consequence levels are evaluated and a decision on suitability of a platform in continued service is made by a comprehensive assessment of its capacity and consequence levels. Based on such an evaluation, the platforms are categorized as "Fit For Purpose (FFP)," "Marginal," and "Unfit For Purpose (UFP)." By such a 4-cycle process (Fig. 1.2), a detailed evaluation of the reserve strength and consequence levels is required for only a few platforms. Thus, this methodology is likely to be cost-effective and would form a more consistent basis for the safety evaluation of platforms.

Upon classification of a platform as marginal or unfit for purpose (UFP) at each of the 4-cycles, the next step in the process is to make a decision on the feasibility of the IMR program proposed for a candidate platform (Fig. 1.2). The IMR program plays a key role in maintaining the safety of a platform in continued operation. A platform classified as Marginal or UFP at any cycle will need revision of its IMR program and evaluation of its feasibility. If the revised IMR program for a platform is found to be adequate, then its safety in upgraded state would be evaluated at the same cycle or at the

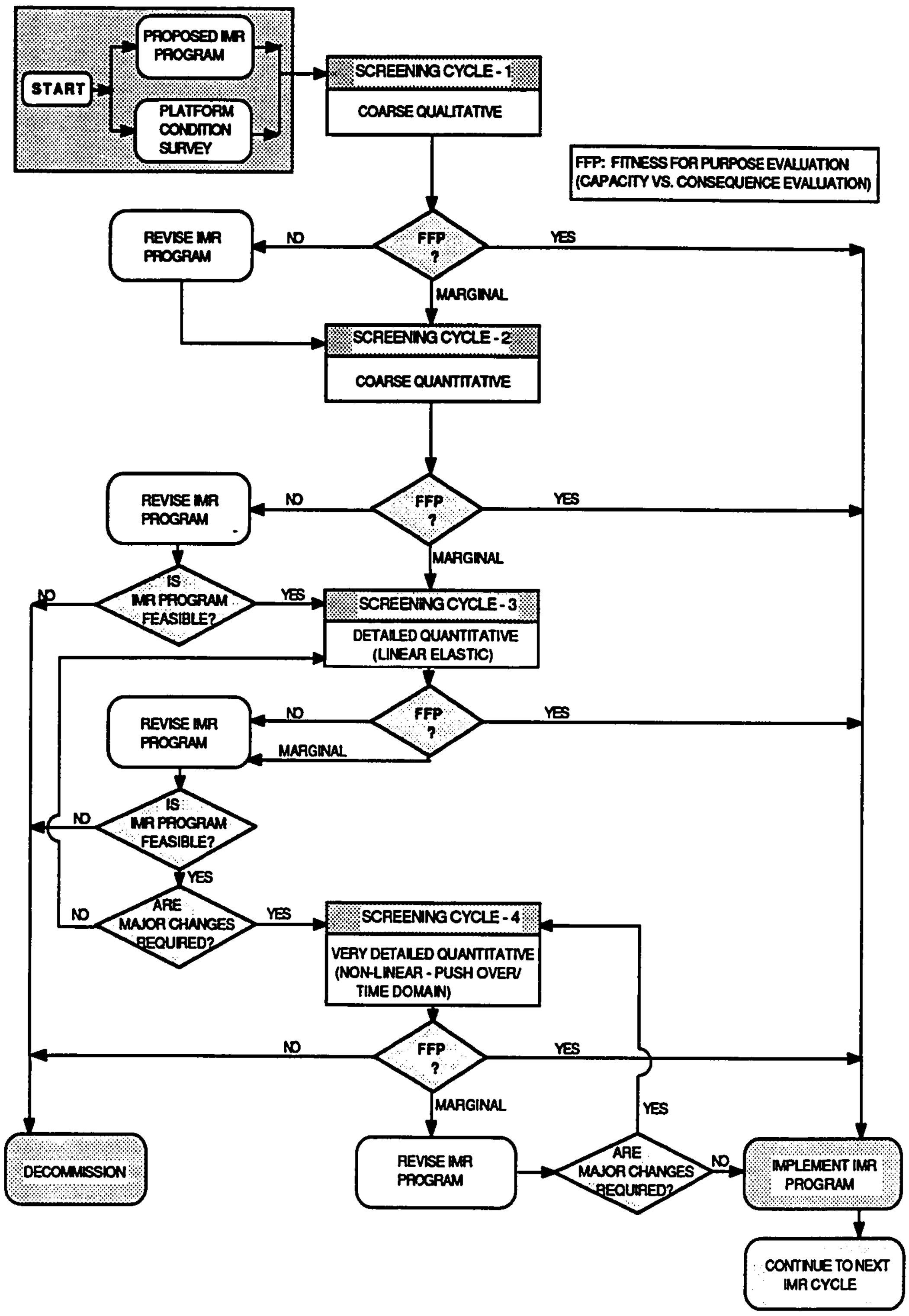


Figure 1.2: Algorithm for Safety Assessment of Offshore Platforms

next screening cycle. If the revised IMR program is found insufficient, then the platform may be considered for decommissioning. The platforms which are categorized as FFP will not need any further evaluation and will be thus approved for continued operations with a requirement to periodically implement the IMR program.

No similar comprehensive screening methodology for the safety assessment of offshore platforms is available in the public-domain. However, some operators follow different methodologies to make a decision on the need for survey and retrofitting of platforms. Their methods stress on optimization of the time interval for implementation of the IMR program in order to reduce its cost. Some of the important works in this direction have been reported by *Bea and Smith (1987), Bea et al (1988), Marshall (1979), UEG (1990)*. The methodology proposed by UEG considers the failure consequence of a component in making a decision on the IMR interval cycle, and is based on a subjective method to prescribe importance to the members. These methodologies are based on use of the conventional linear structural analysis and in some cases on the detailed non-linear analysis techniques. However, attempts have been made in the past to develop screening processes for the safety assessment of buildings against earthquake loads *[Okada and Bresler, 1976; Yao, 1985; Zhang and Yao, 1986]*, bridges *[Moses, 1987]*, and waterfront facilities *[Scola, 1989]*.

Note that a very large number of platforms exist in the Gulf of Mexico, which may need a periodic evaluation of their safety to determine whether the current MMS guidelines are met. These platforms may be operating with significant variations in their physical and operational characteristics, from their as-design stage. The variations in their physical condition may have occurred due to ageing effect, structural damages during their operation, inadequate design, and other sources which were described in Section 1.1. Such variations may have impact on their loads, structural strength, and failure consequences as were described in Sections 1.1 and 1.2. About 37 % of these platforms are reported to have been in-service for more than 20-years [Table 1.1]. Due to their age and the variations which occurred during their in-service state, it is likely that the structural strength level of many of these platforms will be lower than that at the design stage or that required as per the current design standards.

The maintenance of a platform in its safe and serviceable state during its continued operations will need a periodic assessment of its safety. The structural safety of a platform should be evaluated for possible failure modes and mechanisms listed in Section 1.3. In addition, at each cycle an evaluator will have to assess the consequence level upon failure of a platform and the feasibility of an operator's IMR program to maintain the platform in the safe and serviceable state. For such an assessment of a platform at screening cycles 1 to 4 , its details and data as presented in the "data-sheets" (Appendix-A) may be needed.

In the following sub-sections an overview of the process developed for each screening cycle of this methodology is presented.

1.4.1 Screening Cycle-1:

Screening cycle-1 is the first step in the 4-cycle safety assessment methodology described in Fig. 1.2. The primary goals at screening cycle-1 are to achieve the following:

1) To make it feasible for a regulatory body to undertake the task of safety assessment of a very large number of existing steel platforms.

2) To screen the platforms according to their need of an in-depth evaluation. It should be feasible to perform screening with the minimal platform data available with a regulator.

3) To make an optimal use of the resources of the regulators.

These would require an accurate characterization of a platform in its "as-is" state to make a meaningful assessment. Such a characterization would require the details and basic data for the platform, which could be obtained from: design and as-built drawings; basic design criteria and documents; installation records; operation records; and the periodic Inspection, Maintenance, and Repair (IMR) records. In some cases, in order to establish the current state of a platform, a condition survey may be needed.

The basic approach followed at screening cycle-1 is given in Fig. 1.3. At this screening cycle, the platforms are evaluated based on the need for an in-depth investigation of their safety in continued use. This screening process has been developed considering that its periodic application on a very large number of platforms would be needed, and that the regulators have limited resources. Therefore, screening cycle-1 is based on the identification and qualitative assessment of the factors that would be evaluated by an experienced offshore engineer. The process is organized so that it can be applied by a competent engineer in a methodical manner. It is expected that about 50 % of the existing GOM platforms would qualify as fit for purpose (FFP) at the end of screening cycle-1.

Screening cycle-1 comprises of two phases, as shown in Fig. 1.3. In Phase-A (Cycle-1A) of evaluation, the emphasis is to determine the major factors which individually may have a significant effect on the structural integrity and safety of a platform, so as to require further investigation.

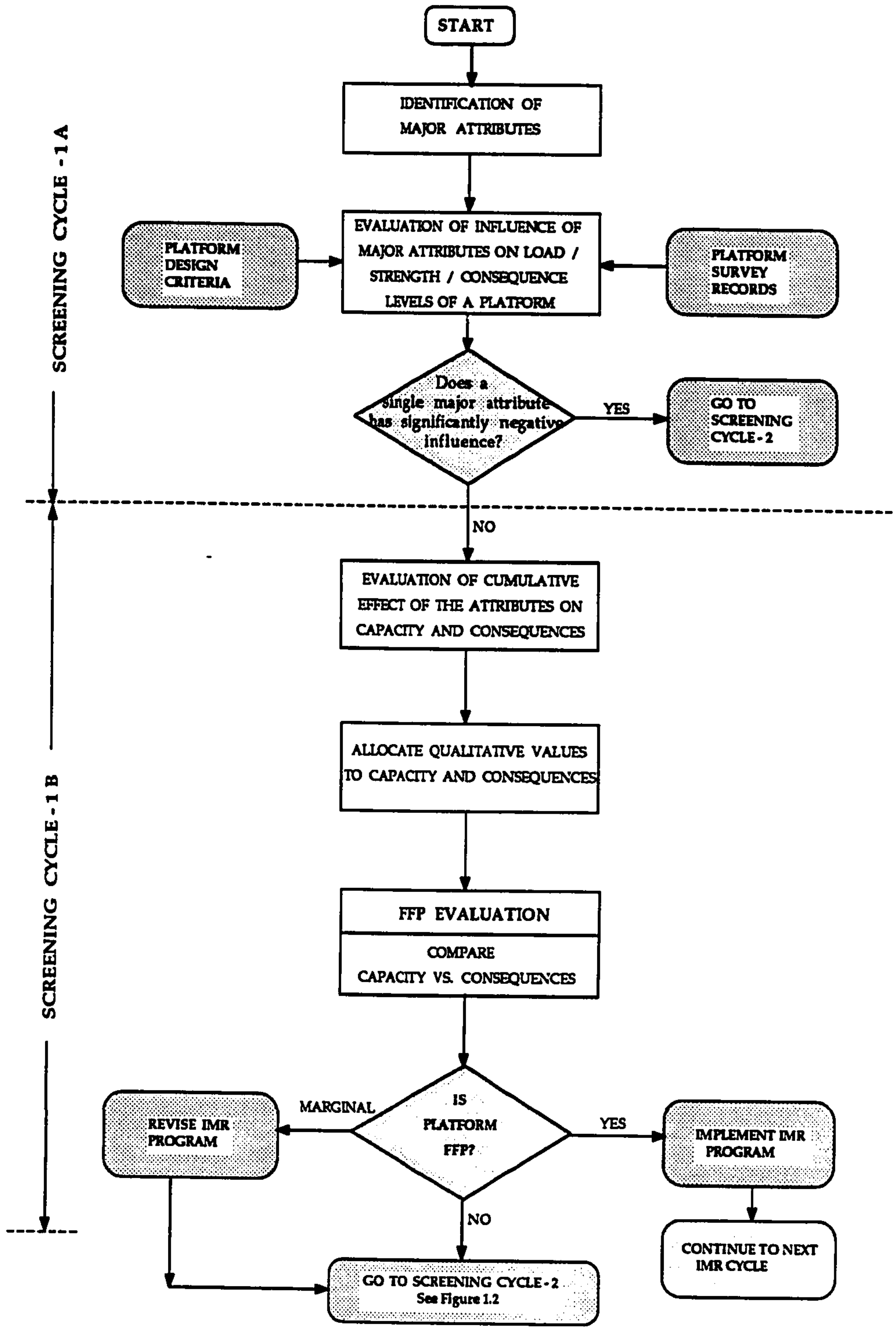


Figure 1.3: Algorithm for Safety Assessment at Screening Cycle-1

In this phase, the degree of influence of the major attributes on the capacity and consequence levels of a platform is evaluated. Then, an evaluation is done to select those platforms, which have incurred a significant negative influence on their capacity and consequence levels due to one single major attribute. Such platforms are screened out for a quantitative evaluation at screening cycle-2.

In Phase-B (Cycle-1B), an evaluation of the cumulative effect of variations in the major factors is made to identify the platforms whose capacity and consequence levels would have significant negative influence. Such platforms are screened out for further investigation at the next cycle.

The important steps in the process at screening cycle-1 are discussed below (refer Fig. 1.3):

1) **Gathering necessary data for a candidate platform.** The platform data is generally obtained from its records, inspection reports, and other documents supplied by an operator. In some cases site visits and surveys would be needed.

2) **Identification of the major attributes .** The major attributes, which influence load, strength, and consequences of a platform are identified.

3) **Evaluation of the effect of these attributes, on an individual basis, on the capacity and consequence levels of a candidate platform (screening cycle-1A).**

4) **Evaluation of the effect of these attributes, on a cumulative basis, on the capacity and consequence levels of a candidate platform (screening cycle-1B).**

Based on the relative values of RSR and consequences for a platform, a decision is made on its suitability for continued service. The platforms which fall in the "UFP" category are further evaluated at screening cycle-2. The platforms which fall in the "Marginal" category are evaluated for the feasibility of their IMR program to maintain or improve their safety level and the necessary measures are evaluated, and they are further evaluated at screening cycle-2.

1.4.2 Screening Cycle-2:

Screening cycle-2 is the second step in the 4-cycle screening process shown in Fig. 1.2. At this screening cycle, the objective is to make a coarse (simplified) quantitative estimate of the suitability for service of a platform which is intact, or has suffered only minor damages, or is an upgraded platform based on the screening cycle-1 assessment. The process is based on the application of a coarse (simplified) quantitative method to determine the capacity, consequence, and fitness-for-purpose of the platforms. It is expected that another 25 % of GOM platforms would qualify for service at the end of cycle-2 (total = 75 %).

The emphasis is to evaluate if the platform has sufficient reserve structural and foundation capacity, when compared against the reference level forces. At this screening cycle, the capacity is expressed by a quantitative index, RSR, which is determined by a simplified technique. Such a technique has been developed by utilizing the knowledge gained from review of the past failures of platforms and a review of the research results published for the system reliability and ULS capacity evaluations.

For RSR evaluation at this screening cycle, the possible failure modes of a platform are selected and the lateral loads required to initiate these modes are determined. The accuracy of the results depend upon the selection of realistic failure modes for a platform. The possible failure modes and mechanisms in the three portions (deck, jacket, pile foundation) of a platform were listed in Table 1.2.

The aim at this screening cycle is to get an approximate "lower bound" estimate of RSR, which will represent the "weak link" or "weak zone" in the structure. Then the RSR and consequence levels for the platforms are compared in a capacity-consequence diagram and a decision is made on their suitability for service. The platforms which fall in the "Marginal" category are further evaluated at screening cycle-3. The platforms in the "UFP" zone are evaluated for suitability of the various upgrading techniques and a decision is made on the feasibility of the revised IMR program (see Fig. 1.2).

1.4.3 Screening Cycle-3

Screening cycle-3 is implemented on the platforms which were screened out at screening cycle-2 due to their having marginal safety level against the reference level loads, and the platforms whose safety level was found to improve with the updated IMR program. At this cycle, a more accurate estimate of the strength of a platform is made by the conventional linear-elastic computer based analysis. The process followed at this cycle is based on an evaluation process that would normally be employed in the verification of a new platform. It is expected that another 10 -15 % of GOM platforms would qualify for service at the end of screening cycle-3.

RSR estimate is based on the extrapolation of results from linear structural analyses. Note that by following a more refined computer-based analysis, the modelling uncertainties would be reduced from those at screening cycle-2. At this level the non-linearities are not clearly considered, except that provided by the soil springs. A better estimate of RSR is made by extrapolation of the results and by accounting of the strength beyond the failure of the first member. However, bias in the results obtained by this analysis would remain.

Note that if other combinations of the force parameters are chosen, then the wave force and moment should be at least equal to the reference level.

The integrity of intact members and joints are checked as per API-RP-2A guidelines with the factors-of-safety or resistance factors [*API-RP-2A, LRFD*] set at unity. Interaction ratios of unity for a component would indicate that it was loaded with a load level corresponding to its best estimate ultimate strength. The yield strength of steel is normally upgraded to account for the difference between the mean and nominal yield strengths, and the loading strain rate effects.

Upon evaluation of RSR by the linear structural analysis and the consequence level, a decision is made on the overall safety (FFP) of a platform by a comprehensive assessment of the capacity and consequence levels. For the platforms categorized as "Marginal" or "UFP," their IMR program is revised and its effectiveness is evaluated. If major upgrading of the platform structure is required, then the platform will be evaluated further at screening cycle-4. If minor upgrading is needed, then its capacity will be re-evaluated at screening cycle-3.

1.4.4 Screening Cycle-4

At screening cycle-4, the platforms whose safety levels at screening cycle-3 were determined as "Marginal" or "UFP" and which were upgraded with major changes, are further evaluated on the basis of the ultimate capacity of the structural system. Such a non-linear analysis is obviously time-consuming and costly. The number of platforms which would require evaluation at screening cycle-4 would be a few critically important platforms.

At this screening cycle, the system strength of a platform is emphasized rather than its component strength. The system strength is useful to determine the possibility of progressive collapse in a platform against a load level. The ultimate capacity of a platform is evaluated based on non-linear analyses, which consider redundancy in the structural configuration and the post-ultimate capacities of the members.

At the simplest, the analysis can be done by performing member-replacement "static push-over analysis," by monotonically increasing the lateral load on the platform and determining the response of the members. Such analyses are difficult, time consuming, and expensive.

The platforms classified as FFP by such evaluation are recommended as safe, provided the proposed or revised IMR program is implemented, whereas those classified as UFP would be recommended as unsafe and may need decommissioning to avoid negative consequences. A revised IMR program would be needed for the marginal cases. If major modifications are needed in upgrading of a platform, then the modified platform would need re-evaluation at screening cycle-4. Otherwise the platform would be recommended safe upon implementation of the updated IMR program.

1.5 Overview of the Report

Chapter 2 presents details of the important parameters and the considerations made in establishment of the "as-is" state of a platform and in evaluation of the load and strength levels of its components, which are the basic input quantities in the safety assessment process. The parameters and the methods used in evaluation of the load and strength, and the uncertainties associated with their estimates are identified and discussed.

Chapter 3 includes details of the procedures developed for load and strength evaluations at the four screening cycles. The emphasis is to identify the processes which account for the differences in the results at the various screening cycles. A quantitative estimate of bias in the models is made.

Chapters 4 presents details of the various considerations made in the qualitative and quantitative evaluations of the consequence level.

Chapter 5 presents the basic details of the comprehensive assessment procedure for evaluation of the fitness-for-purpose of a platform.
The variation of the FFP criteria with the various parameters is presented through sensitivity studies. The most important considerations in upgrading of the platforms are identified through such study. The procedure for evaluation of the IMR program are discussed in Chapter 6.

This methodology is applied on three example platforms: one 4-legged, and two 8-legged, and the results are presented in Chapter 7.

The basic steps in the development of a computerized knowledge-based expert system are discussed in Chapter 8. The knowledge trees are developed as a first step in formation of the knowledge base at the screening cycle-1 . The methodology is summarized and future work is identified in Chapter 9.

The detailed computations for evaluation of RSR at the screening cycle-2 for Platform-A and Platform-B are given in Appendix-E and Appendix-F respectively.

CHAPTER 2

PRIMARY ELEMENTS FOR EVALUATION OF SAFETY

The safety assessment methodology outlined in Chapter 1 (Fig. 1.2) is based on the evaluation of four criteria for a candidate platform:

1) **Capacity (or Reserve Strength);**

2) **Consequences upon its failure to human, environment, property, and resources;**

3) **Fitness-for-purpose (risk evaluation vs. risk acceptability);** and

4) **Proposed inspection-maintenance-repair (IMR) program for maintaining it in a safe state.**

In order to evaluate the above four criteria, the best estimates of the current physical and operational states, the loads acting and load effects, and the strength of a platform are necessary. Estimates of these depend upon a number of individual parameters, the methods, and the numerical models used. The parameters vary based on actual condition of a component and its location. In normal practice the best estimate values of the parameters are selected and simplifications are made in the methods for evaluation of these quantities. Therefore, the estimates of capacity and consequences for a platform will have associated uncertainties, and the risk or safety estimate will be "notional" instead of "actuarial."

The structural safety of a platform is related to the safe state of its components in the three zones: deck, jacket, and foundation piles, which in turn depends upon the magnitudes of load (or load effects) and strength, and the physical condition of its components. Their magnitudes vary with time and have associated uncertainties and biases. The difference in the present condition of a platform from its as-designed and as-fabricated stages influences its load and strength levels. These could be characterized as random variables, with a best estimate (central tendency) and uncertainties (dispersion) in their estimates.

Estimates of the load and strength parameters, the uncertainties introduced, and propagation of the uncertainties through the process could be evaluated and combined in a meaningful way, through application of probability theory, to obtain the safety index or probability of failure for a component. The safety index provides a measure of the number of standard deviations the mean safety margin falls in the safe zone.

In general, very high uncertainties are associated with the loads acting on a platform, and lower uncertainties are associated with its strength. But, for a platform with damaged and deteriorated components, the strength uncertainty is likely to be high due to the errors in prediction of the damaged state and residual strength of the components.

In this chapter, first a formulation of the safety index is presented. Then the parameters and models used in condition assessment and in evaluation of the load and strength are identified and the associated uncertainties introduced in their evaluations are focused. This work utilizes the state-of-the-art information available in the literature.

2.1 Safety Parameters

In the deterministic sense, a component is identified as "safe" against a particular failure mode, when its resistance (R) is greater than the load effects (S) acting on it. The best estimates of R and S would have associated uncertainties, due to randomness of the parameters used to describe the load and strength and the errors introduced by the simplified models used in their estimation. Thus, probabilistic methods are used to make a decision on the safe state of a component, by considering the uncertainties associated with the mean (best) estimates of the load effect and strength. The safe state of a component could be described by a safety margin, M, and the associated uncertainties in its estimate.

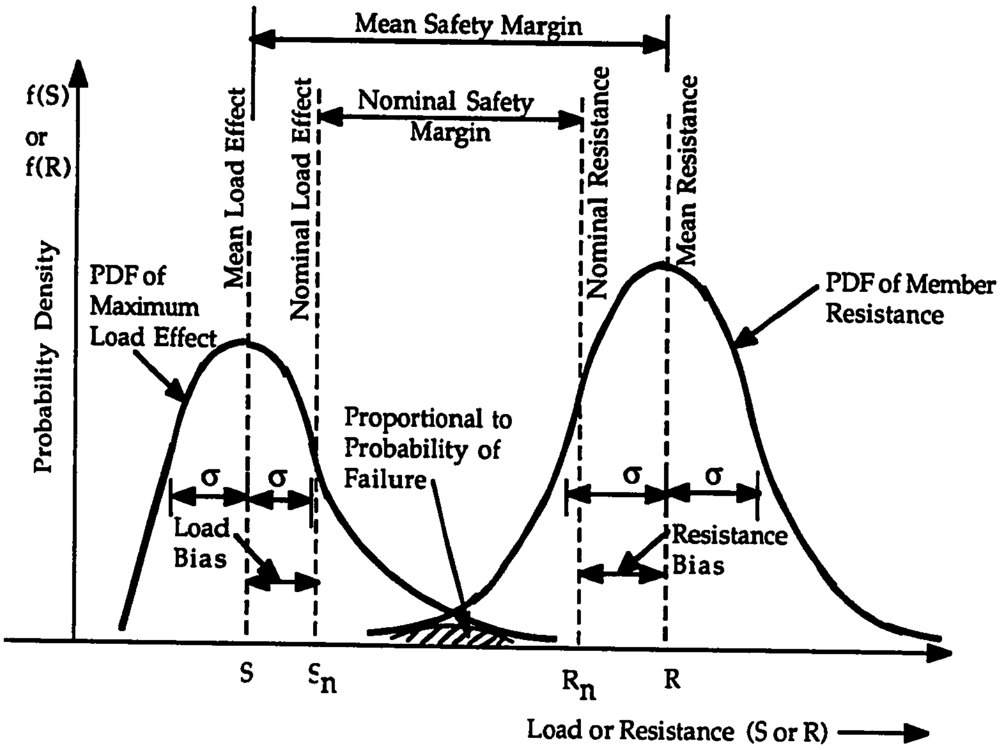
Mean safety margin, $$M = R - S \qquad \text{....(2.1)}$$

Uncertainty level, $$\sigma_M = [(\sigma_R)^2 + (\sigma_S)^2]^{1/2} \qquad \text{....(2.2)}$$

Fig. 2.1 illustrates the fundamental reliability formulation based on probability distributions of load and resistance and the safety margins. The risk depends on the degree of overlap of the load and strength probability density curves. The estimates of load effects (S) and strength (R) of the components are based on multiplication processes, and their estimates will be positive. Therefore, by virtue of the central limit theorem, their (R & S) distributions would be log-normal i.e., the logarithms of their median values would follow normal distribution (represented by mean values). The median (or 50th percentile) values, R_m and S_m, would correspond to the best estimates of their true values and would have associated uncertainties.

The uncertainties in the parameters may be characterized in three forms: randomness (aleatory, natural, or inherent); prediction or modelling uncertainties (epistemic, unnatural, or cognition); and human errors in the parameters. The uncertainties due to inherent randomness in a physical process are called as type-I uncertainties in this study. Type-I uncertainties are not reducible with time or additional data, e.g., randomness in the magnitudes of wave height and yield strength.

Prediction or modelling errors are called as type-II uncertainties in this study and these are introduced due to simulation of the actual behavior of or the loads on a platform by simplified empirical and numerical procedures.



$P_f = P[R \leq S]$

$B_S = S / S_n \leq 1.0$

$B_R = R / R_n \geq 1.0$

Figure 2.1: Illustration of Load and Resistance Probability Density and Safety Margins

Type-II uncertainties vary with time (due to change in platform condition), and are reducible by improved data on the platform condition through inspection, by load and strength evaluation by improved models, and with improvement of technology through research and development. This uncertainty is represented as cumulative variance for the parameters used in the models.

Human errors are introduced from personal judgement and preferences, errors in design, lack of knowledge, etc. The influence of human errors on the structural safety has been considered by *Melchers [1989], Pate'-Cornell and Bea [1989].* The human errors in prediction of the load effect and strength can be reduced with improved training of the personnel, improved documentation of the procedures, peer review, etc.

In addition, the control of human errors in operation, which may lead to adverse effects on the load and strength of a platform and its components, are likely to have a more significant effect on the safety of a platform. In this study, the human errors in re-evaluation of the capacity and consequence levels of a platform have not been considered. However, many of the human errors introduced during the design, construction, installation, and in-service operation phases can be accounted by the improved state of knowledge of the platform and by the improved estimates of the load effects and its strength.

The quantitative assessment of the major sources of uncertainties underlying the load and strength parameters and the prediction models constitutes a principal task in the application of the reliability framework. The uncertainties are represented by bias and variance in the mean or median values of the parameters. Bias is defined as the ratio of the actual (or measured) to the predicted (or nominal) values of a parameter or a resultant quantity.

$$\text{Bias} = \frac{\text{Actual or measured value}}{\text{Predicted or nominal value}} \quad \text{....(2.3)}$$

Bias represents an error factor in the process (methods) used for evaluation of a quantity when compared with its actual or measured value or that obtained from instrumentation monitoring. For the methods followed for estimation of S and R, the biases in S and R would thus represent the systematic or model errors in their central (mean or median) estimates.

The variance component in the uncertainty level, is obtained as the sum of the squares of the standard deviations of the logarithms of the strength and the load effect, and the bias component is incorporated in the median value by multiplying bias to the nominal estimates of the load effect and strength.

Mean safety margin, $M = R - S = \ln(R_m / S_m)$ ....(2.4)

Uncertainty level, $\sigma_M = [(\sigma_{\ln R})^2 + (\sigma_{\ln S})^2]^{1/2}$ ....(2.5)

The uncertainties in safety estimates for a component or the complete structure could be considered through a reliability measure, designated as the safety index (β). β is a probabilistic measure of the structural safety, and is approximately described by the mean-value first order second moment (MVFOSM) method first proposed by *Cornell [1969],* as the ratio of the mean safety margin to the uncertainty level in the assessment of the safety margin.

$$\text{Safety index, } \beta = \frac{\text{Mean Safety Margin}}{\text{Uncertainty Level}} = \frac{\ln\ [R_m / S_m]}{[(\sigma_{\ln R})^2 + (\sigma_{\ln S})^2]^{1/2}} \quad \text{....(2.6)}$$

where, the parameters of a log-normal distribution are given as follows *[Ang and Tang, 1984]:*

$\ln R_m = \ln R - 0.5\,(\sigma_{\ln R})^2$ $\qquad$ $\ln S_m = \ln S - 0.5\,(\sigma_{\ln S})^2$

$(\sigma_{\ln R})^2 = \ln\,[1 + (V_R)^2]$ $\qquad$ $(\sigma_{\ln S})^2 = \ln\,[1 + (V_S)^2]$

The above formulation of safety index gives approximate results due to non-linearity in load effects characterization. In common practice this formulation is used. A more accurate estimate of the safety index could be obtained by using the Hasofer-Lind transformation [*Hasofer and Lind, 1974*].

In general, $X_m = [X / \{1 + V_X)^2\}^{1/2}]$, where X represents the mean value, X_m the median value, and V_X the coefficient of variation (= standard deviation / mean value) of the mean value of parameter, X. The variance (square of standard deviation) of the log of the median value of X is given by, $\ln [1 + (V_X)^2]$. Hence, the safety index is obtained as follows:

$$\beta = \frac{\ln [R / S] + \ln [\{1 + (V_S)^2\} / \{1 + (V_R)^2\}]^{1/2}}{[\ln \{1 + (V_R)^2\} + \ln \{1 + (V_S)^2\}]^{1/2}} \quad \text{....(2.7)}$$

The mean values of the load effect (S) and strength (R) should represent the true values for a component or a structure. However, in engineering estimates of the load effects and strength, various approximations and simplifications are made, and their nominal values are obtained. The nominal values of the load effect, S_n and the strength, R_n for a component or a platform (sub-structure or complete) are determined by using the methods described in API-RP-2A Recommended Practices and MMS Guidelines, and by the other methods discussed in this study.

Nominal estimates R_n and S_n would differ, depending on the parameters and methods used, and may differ from the true values of the load and strength for a specific component or a platform. Thus, the factors which were not considered and the simplifications made in estimations of R_n and S_n should be identified and their effects could be incorporated by the biases, B_R and B_S. The biases associated with the nominal estimates, R_n and S_n, could be used to determine their mean value estimates:

Mean strength, $R = (B_R)\ (R_n)$

Mean load effect, $S = (B_S)\ (S_n)$

In the safety index expression, the COV in R and S could be further characterized as the type-I and type-II uncertainties discussed earlier. By incorporating the biases, the nominal values, and the type-I and type-II uncertainties, the safety index is expressed as follows:

$$\beta = \frac{\ln [R_n / S_n] + \ln [B_R / B_S] + \ln [\{1 + (V_{SI})^2\}\ \{1 + (V_{SII})^2\} / \{1 + (V_{RI})^2\ \{1 + (V_{RI})\}]^{1/2}}{[\ln \{1 + (V_{RI})^2\} + \ln \{1 + (V_{RII})^2\} + \ln \{1 + (V_{SI})^2\} + \ln \{1 + (V_{SII})^2\}\ ]^{1/2}}$$

For the lower values of COV up to 0.3, $(V_{RI})^2 \approx [\ln \{1 + (V_{RI})^2\ ]$. Also, the third term in the numerator will be very small, and can usually be neglected for practical purposes. However, for the platforms with low RSR or the components with low factor of safety, the third term in the numerator should be considered in order to obtain more accurate results. By neglecting the third term for the cases with lower variances, the above expression can be simplified as follows:

$$\beta \approx \frac{\ln [R_n / S_n] + \ln [B_R / B_S]}{[(V_{RI})^2 + (V_{RII})^2 + (V_{SI})^2 + (V_{SII})^2\ ]^{1/2}} \qquad \text{....(2.8)}$$

In the above expression, the ratio $[R_n / S_n]$ corresponds to the nominal safety margin $(FS)_n$ of a component or the nominal reserve strength ratio $(RSR)_n$ of a structure, and $[B_R / B_S]$ represents the bias ratio (BR) in the estimate of $(FS)_n$ or $(RSR)_n$ for a component or structure respectively. In the API-LRFD format, the factor of safety can be characterized by the ratio of load to resistance factors. The type-I and type-II variances include the variances in the estimates of FS or RSR, and BR. The safety index described above is evaluated as an annual safety index or the lifetime safety index, depending upon the annual maximum or lifetime maximum load effect considered.

Note that the nominal estimates of the load effect and strength will vary as the screening process proceeds from cycle-1 to cycle-4, due to the differences in their parameters and methods. Consequently the bias and variance estimates will also vary for each cycle. However, ideally, in case all the differences in the methods used in estimations of the load effects and strength are identified and accounted in their nominal values and biases, the products of the nominal values and biases at each of the four cycles should be the same and represent their mean values. The bias estimates would also have associated uncertainties due to variations in the physical characteristics of platforms and the natural processes. The COV level would also vary for the screening cycles 1 to 4, because it is entirely dependent on the type-I and type-II uncertainties in the parameters and methods used.

In order to obtain the safety index for a platform at a particular cycle, the estimates of propagation of uncertainties (bias and variance) in load and strength from parameters to model level, and from component to system level are important. In this chapter, the uncertainties associated with the conventional models used for evaluations of load and strength are described.

The estimates of bias and COV in the models used for evaluations of load effect and strength can be made by the following approach:

1) Identify the models and parameters used for evaluations of the load and strength for the different failure modes and mechanisms.

2) Determine the variances and biases associated with the member properties, load, and strength for the different failure modes and mechanisms. The member properties vary with time due to ageing effects and corrosion. Identify type-I (B_{IX} and σ_{IX}) and type-II (B_{IIX} and σ_{IIX}) uncertainties.

3) Perform a sensitivity study to identify the most important parameters for each failure mode and mechanism. This would provide a basis to select the optimal measures to reduce the uncertainties.

Note that type-II uncertainties can be reduced by better estimation (analytical model) of load effects and strength at the higher screening cycles, and with the more precise information on the present state of the platform. With the improved state of knowledge of a platform, its nominal load (load effects) and nominal strength estimates would approach their true (mean) values.

<u>Sensitivity coefficients</u> help in determination of the degree of importance of the different parameters on the estimations of load (load effects) and strength. By sensitivity analyses, the parameter sensitivity factors and the importance factors could be determined. The parameter sensitivity coefficients help in identification of the basic variables whose uncertainty contributes significantly to the overall uncertainty of the load effects or strength. Therefore, the measures can be undertaken to reduce the uncertainties of such parameters to improve estimates of load effects and strength. The importance factors determine the relative importance of the uncertainties of each variable used in the limit state of function (failure function). The sensitivity coefficients could be utilized to evaluate the COV of the load effects or strength.

The COV of the load effects and strength could be evaluated in the following way:

1) Identify the models and parameters used for the load and strength evaluations for the different failure modes and mechanisms.

2) Evaluate the sensitivity of RSR to change in the magnitude of each major attribute (ΔA_x), while considering the mean value estimates for all the other attributes (μ_x). The sensitivity coefficients thus depend upon the mean values of the parameters in the formula.

Sensitivity, $S_x = (\Delta RSR / \Delta A_x)$ for $x = 1..\text{to}..N$ attributes.

3) Evaluate the contributions of each attribute (x) on the overall COV.

$$= S_x [\sigma_x / RSR_n]$$
$$= S_x [\mu_x / RSR_n] COV_x$$

4) Evaluate the COV of load effects or strength by considering the contributions of their n attributes.

$$(COV_{RSR})^2 = \Sigma [S_x \{\mu_x / RSR_n\} COV_x]^2 \quad \text{....(2.9)}$$

In general, it can be written in the following way:

$$(COV_{Formula})^2 = \Sigma [\{\Delta Formula/\Delta A_x\}^2 \{\mu_x / \mu_{Formula}\}^2 COV_x] \quad \text{....(2.10)}$$

2.2 Condition of a Platform

The current state of a platform is established by review of: design documents and installation reports; periodic inspection, maintenance, and repair (IMR) reports; and site visits, interviews, and inspections. The emphasis is placed on establishing the differences between 'as-design' and 'current' states of a platform. The physical and operational characteristics, which influence the estimation of load, load effects, strength, and consequences are important to compare.

Physical State: During the operating life of a platform, modifications in the topside facilities are normal and in some cases structural modifications are done to the deck and jacket. For some platforms, it may be difficult to predict their "as-is" physical characteristics due to the lack of design, installation, and inspection records.

The load level on a platform may have a significant influence due to the differences between the "as-design" and "as-is" states of a platform, particularly for its orientation, deck elevation, water depth, number of well conductors and risers, and the type and extent of marine growth. The strength of a component (or a platform) is influenced by the installation errors, corrosion (overall and local) of the members, extent of damage of the members, and by the missing members. Upon failure of a platform, the consequences to human life, environment, property, and resources will vary due to the modifications made in the functions of a platform from its as-design stage. The consequence level of the platform would vary with the modifications in the oil storage facilities, production facilities, and personnel access and evacuation facilities.

Operation Philosophy: The philosophy followed for operations of a platform is important, because it would affect its structural strength and consequences upon failure. The provisions of the facilities for personnel access and evacuation, fire protection, personnel and material handling, spillage and contamination handling, and auxiliary systems would describe the operational philosophy. Some of these facilities are designed for operations during the normal (operating) environmental conditions. Note that in the Gulf of Mexico, normally the practice is: to evacuate the personnel from the platforms in the event of a hurricane prediction, and to shut off the production wells and risers through emergency shut-down valves.

In this project, the emphasis is on the safety of a platform during an extreme loading event.

Present Condition Assessment: The current state of a platform can be established by: review of documents, inspection of topside facilities, diver or ROV survey of the jacket structure, and instrumentation monitoring of its behavior. An accurate assessment of the present condition of a platform is a difficult and timely process, due to the limitations of resources and technical feasibility. Besides the technical limitations, the capabilities of human (diver) or ROV to assess the correct condition of a platform in the offshore environment is important. This introduces uncertainties in the state of knowledge of the platform and its components, which would result in an inaccurate assessment of load and strength.

The uncertainties associated with establishing the as-is state of a platform can be reduced by implementation of one or more of the following:

a. Platform topside inspection

b. Diver or ROV Survey

c. Increase in the frequency of inspection

d. Instrumentation & Monitoring of behavior of a platform

API-RP-2A Section 14 provides the guidelines for in-place platform survey for monitoring the adequacy of the corrosion protection system and to establish the condition of a platform. The recommended frequency of survey for a platform is related to a number of factors: location, water depth, structural condition and configuration, operational philosophy, importance, consequence level, and its in-service history of the platform. Four levels of surveys are recommended, which requires an increasing degree of sophistication and frequency in their application. A summary is provided in Table 2.1 [*API, 1991*].

By following such a program, the state of knowledge of a platform and its components is improved and the nominal estimates of load and strength will improve. Note that the survey level required for a screening cycle would depend upon the availability of the records for a platform, review of its past performance, and the details required for the safety assessment at that cycle.

Table 2.1: Guideline Survey Intervals and Methods [API, 1991]

	LEVEL OF SURVEY	INTERVALS	METHOD	EVALUATE/ CHECK
1	LEVEL-I	1 years or after hurricane or earthquake.	Above water visual survey General examination of all structural members in splash zone and above water.	Corrosion protection effectiveness. Bent, missing, or damaged members. Indications of overloading. Inconsistent use of platform.
2	LEVEL-II	3 thru 5 yrs (Manned) 5 thru 10 yrs (Others) After completion of initial drilling plan. After severe accidental loading 1 yr after repair of damaged area.	General underwater inspection by diver or ROV. Measurement of cathodic potential of critical areas.	Excessive corrosion. Accidental or environmental overloading. Scour, seafloor instability etc. Fatigue damage. Design or construction deficiencies. Presence of debris. Excessive marine growth.
3	LEVEL-III	6 thru 10 yrs (Manned) 11 thru 15 yrs (Unmanned) Based on results of Level-II survey.	Underwater visual inspection of preselected areas or areas of suspected damage (after cleaning of marine growth).	Existence and propagation of damage.
4	LEVEL-IV	Based on results of Level-III survey.	Underwater NDT of preselected areas or areas of known or suspected damage.	Detailed inspection and measurement of damaged area.

The influence of variations in the basic parameters of a platform upon the load effects, strength, and consequence levels are discussed in the following sections and are not covered here.

2.3 Load (Demand) on a Platform

A platform is subjected to a number of different load sources during its lifetime from the fabrication to the salvage phases. In this study, the structural safety of a platform during its in-service phase is evaluated. During this phase, a platform is subjected to a combination of individual loads with constant and varying magnitudes, from sources such as: structural, equipment, operational, environmental, and accidental loads. The structural, equipment, and operational loads on the platforms are considered as dead and live loads in their design.

The environmental loads on a platform are due to a combination of loads from the physical processes such as: wind, wave, current, seabed movement, earthquake, and ice movement. In most regions of the Gulf of Mexico, the structural size and configuration of a platform are governed by the lateral loads from wind, wave, and current. In some areas of the Gulf of Mexico, the platforms may be subjected to additional lateral loads due to wave-induced soil movement, which may even exceed the magnitude of the wave force. The platforms located in the offshore regions of California and Alaska are subjected to earthquake loads in addition to storm waves, and in many cases the earthquake loads govern their design. The platforms in the Arctic zone are also subjected to loads due to ice or iceberg.

The effect of accidental loads, which may have occurred before the extreme environmental loads, can be considered by characterization of the residual strength of the affected structural components. The models considered for their reduced strength due to selected sources of damage and deterioration are discussed in the next section.

In this study, the in-service safety of the Gulf of Mexico platforms is evaluated against the environmental loads, in combination with the structural and operational loads. The joint probability of occurrence of the accidental loads with the environmental loads has not been considered in this study. Note that on the contrary, some structural components of the platforms may possess additional load carrying capability due to their design being governed by the loads during earlier phases such as: fabrication, load out, transportation, launch, upending, and pile installation. Such extra load carrying capability may be of advantage in the case of damaged state of some components or in the case of overload.

Of the above cited load sources, in most cases in the Gulf of Mexico, the wave loads form a major part of the lateral loads. The waves are cyclic and occur randomly with varying wave heights and frequency. The random waves influence the structural safety of a platform in two ways, namely extreme load effects due to reference level waves, and fatigue load effects due to normal operating waves. The extreme load effects influence safety of the members and joints of the deck, jacket, and pile foundation, whereas the repeated loading effects due to the normal waves influence safety of the welds and joints. During a hurricane in the Gulf of Mexico, a few extreme waves of nearly-equal magnitude occur, which besides inducing the extreme load effects may also induce the low-cycle fatigue effects on the structure. In this study, the fatigue load effects due to cycles of the normal and extreme waves have not been considered.

In normal design of the platforms in the shallow and medium water depths, the random waves are idealized by a single wave frequency and the maximum load effects are evaluated. The platforms in the deep-water locations (i.e., water depth ≥ wave length/2) have vibration modes with a reduced frequency (or increased period), which are closer to the wave frequency. The static wave loads on such platforms would increase due to their dynamic amplification from the resonance effect of the structural and wave frequencies. In this study, the wave loads on the platforms in the medium water depths are considered, for which the dynamic component of the wave forces would be negligible.

The dead load associated with the design wave load case is from the self weight of the platform structure, permanent equipment, and appurtenant structures. Dead load also includes the hydrostatic forces, which act on the structure below the waterline, the external pressure and buoyancy, and the weight of water enclosed in the structure. In general, a more accurate estimate of dead load is available. However, its estimates can vary due to the estimation errors and imperfect knowledge of the equipment and appurtenant structures. The uncertainties in the dead load are normally included by a bias (B_{DL}) of 1.0 and COV (V_{DL}) of 0.08 [*RP2A-LRFD, 1989*]. Note that the uncertainties may be lower for the more recent platforms, on which improved weight control techniques were applied.

Live load includes the weight of consumable supplies and fluids in the pipes and tanks, and the short duration operational forces exerted on the structure. As a normal design philosophy, the platforms are designed for reduced operational loads with the design environment parameters. The uncertainties in live loads are accounted by a bias (B_{LL}) of 1.0 and COV (V_{LL}) of 0.14 [*RP2A-LRFD, 1989*].

The COV in the gravity loads (dead and live loads) include variances in the load intensity (material density and volume) and load effect (analysis).

The vertical loads on a platform virtually remain constant, while the lateral loads vary.

The magnitude of extreme wave loads (100-year return period) is several times of the wind loads. Wind loads have been estimated to be of the order of 15 % of the wave forces in medium water depth cases [*Bea, 1989*]. In evaluation of the reference level wave loads, a combination of the parameters given in the latest API recommended practice are used to obtain a minimum acceptable reference level force.

In this section, the parameters and methods used in evaluation of the wind loads, wave loads, and their load effects on a component and a platform are described. The focus is on the assessment of nominal estimates of wave loads and load effects, and the associated uncertainties (biases and variances) in the mean value estimates of the loads and load effects for medium water depth cases.

2.3.1 Wind Loads

The wind force on the deck structure and topside facilities is calculated by the following formula. [*API, 1989*]:

$$F = 0.00256\ (V_y)^2\ C_s\ A \qquad \text{....(2.11)}$$

where: F = wind force in lb.; V_y = wind velocity in mph at elevation "y"; C_s = shape coefficient; and A = projected area of the deck in ft^2. C_s could be assumed as 1.0 for the overall projected area of the platform (or deck), as 0.5 for the cylindrical sections, and as 1.5 for sides of the buildings and for the beams.

The wind velocity (V_y) at height 'y' is given by:

$$V_y = V_H\ [y / H]^{1/n} \qquad \text{....(2.12)}$$

where: V_H = wind velocity at reference height, H (= 33 ft.), which is given as 98 mph for the Gulf of Mexico, and it is considered to act along with the extreme waves; y = elevation in ft.; 1/n would be 1/13 for gusts.

The reference one-hour average wind speed at 33 ft elevation, which is associated with the extreme waves, is given as 98 mph, with a variation of ± 10 % for the Gulf of Mexico. Hence, mean wind velocity is 98 mph with a COV of 0.05 [*API, 1989*], and it is assumed to act simultaneously and co-directionally with the guideline of 100-year extreme waves.

The bias and COV for the maximum lifetime (25 years) wind effects are reported as 0.78 and 37% respectively [*API-LRFD, 1989*].

2.3.2 Wave Loads

The wave loads and load effects on a platform are determined by the following four-step approach (refer Fig. 2.2):

1) Selection of the wave parameters.

2) Evaluation of wave kinematics based on a suitable wave theory.

3) Establishment of the structural characteristics and evaluation of wave loads: external member loads, and lateral shear and overturning moment at the base (bottom of jacket or seabed).

4) Establishment of the structural stiffness properties and evaluation of wave load effects: internal member forces.

Step-1: Selection of the wave parameters

In lieu of the site-specific wave parameters established by an operator, API-RP-2A provides generalized guideline parameters for the design wave in Section 2.3.4 of the Recommended Practice [*API, 1989*]. The design wave is described by the parameters of wave height (H), wave period (T), wave

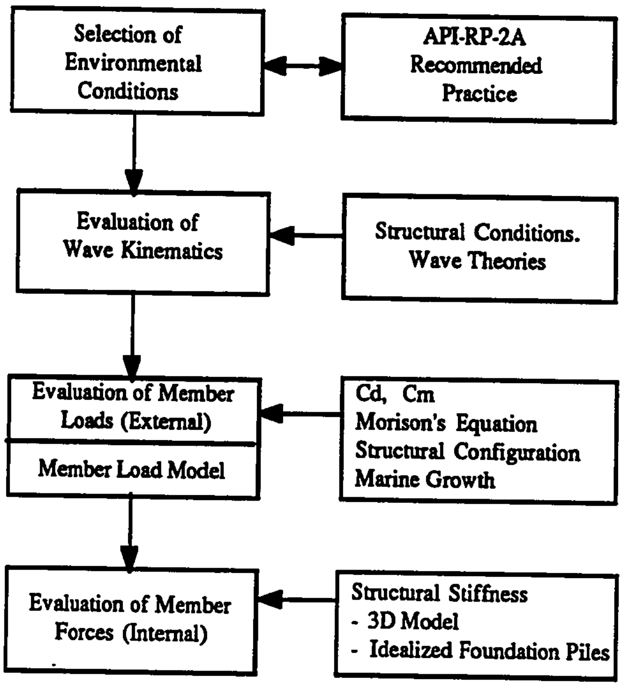


Figure 2.2: Procedure to Evaluate the Hydrodynamic Forces

steepness (S), approach direction (θ), and the total water depth including storm tide (d). The parameters selected as per the generalized API Recommended Practice may differ from the actual parameters for the specific site. Therefore, the biased wave parameters will subsequently influence the estimates of loads and load effects. These guideline parameters when applied on a platform under a specific set of conditions and a specific combination of other parameters should result in an acceptable reference level design force and overturning moment.

API recommends the guideline wave heights and storm tides for the Gulf of Mexico region, as given in API- Fig. 2.3.1, [*API, 1989*]. It specifies a "reference level" wave height and storm tide for a water depth, which would provide the reference level loads when used with the other specific provisions in the API-RP-2A. The guideline omni-directional extreme wave heights (H_m) with a nominal return period of 100-years for the Gulf of Mexico locations with water depth greater than 300 ft. are given by API, varying between 65 to 80 ft. (reference wave height of 72 ft.), with the wave steepness (S) varying between 1/11 to 1/15, and the associated storm tide of 2 to 4 ft. [*Table 2.3.4-1, API, 1989*]. The wave period can be evaluated by $T_m = [(2\pi / g)(H_{ref} / S)]^{0.5}$, which is equivalent to $T_m = 1.53 \sqrt{H_{ref}}$ for S = 1/12. Note that the wave steepness (wave height / wave length) of 1/12 represents its value during the hurricane.

This gives the appropriate "reference level static wave force" when *Morison's et al equation (1950)* is used with the full projected area of all structural members and appurtenances in the wave zone, a constant drag coefficient of 0.6, an inertia coefficient of 1.5 for members six feet in diameter or less, increasing linearly to 2.0 for members ten feet in diameter and greater, an appropriate wave theory, a wave period based on wave steepness of 1/12, and an appropriate allowance for marine growth.

The other guideline wave heights in the gray band shown in API- Fig. 2.3.1, [*API, 1989*] represent reasonable values which might be used to develop an equivalent level of force, with different values of other force parameters. For example, a lower wave height or consideration of directionality might be used in combination with the larger drag coefficient or storm currents. The magnitude of the force level by such combinations should not be lower than that obtained by using the reference level wave height and associated wave force parameters.

Note that in shallow water depths ($d/L < 0.05$), the wave height would change due to wave transformation and bottom dissipation of the wave energy. The wave transformation includes shoaling and refraction effects. The wave energy may dissipate due to a combination of mechanisms, such as:

bottom friction, percolation or soft bottom interaction, and other factors [*Bea et al, 1983*].

The COV for the predicted annual maximum wave height has been recommended by various researchers to vary between 19 to 36%. Typically, a type-I COV of 25% is considered for the Gulf of Mexico [*Moses, 1989*].

Step-2: **Evaluation of the wave kinematics based on suitable wave theory**

The wave kinematics, i.e., velocity (u) and acceleration (u') of the water particles, wave crest elevation, and decay of wave velocity and acceleration with depth are required to determine the loads and forces in members.

The wave kinematics is significantly affected by the change in water depth, because the water particles motion change from circular in the deep water to nearly-horizontal in the shallow water. In the deep water locations, where the water depth, $d > 0.5\ L$ (L = wave length), the wave kinematics become independent of the water depth. Waves in the shallow water locations ($d < 0.5\ L$) are affected by the seabed characteristics and thus their profiles change. In this study, such details are not given and reference is made to the literature *[SPM, 1981; Chakrabarti, 1987; Bea et al, 1983]*.

A number of wave theories are available for evaluation of the wave kinematics, and their suitability depends upon the water depth at the location. The results obtained by the different theories differ, due to the various assumptions made. The linear Airy's wave theory is the simplest to apply due to the simplifications made and the availability of closed form solutions. Airy's theory is based on the assumptions of infinitesimal wave height ($H \approx 0$), symmetric wave profile about the mean sea level, and that the water particles move in a closed orbit. By these assumptions, the strict application of Airy's theory would mean that the water particles above the mean sea level would have zero velocity and acceleration. The other theories are non-linear and consider the unsymmetry of the wave profile and the open water particle motion, which introduce the mass transfer effects. The results obtained by the non-linear theories are more accurate, but they are more complex to apply compared to Airy's theory. The errors introduced by Airy's theory could be reduced by stretching of the wave velocity and acceleration profiles above the mean sea level.

A suitable wave theory is selected on the basis of the magnitudes of d/T^2 and H/T^2, where d is water depth, H is wave height, and T is wave period. The curves developed for suitability of wave theory are given in Shore Protection Manual and other literature.

The uncertainties (bias and variance) would vary with the wave theory used in estimation of wave kinematics. It has been established that the global loads obtained from Dean's Stream Function and Stoke's 5th order theories are nearly the same *[OTH, 1988]*. The Stream Function wave theory is applicable over the entire range of d/T^2 values, because the kinematic condition error is zero. *Dean* has prepared the graphs for the estimation of wave loads based on the Stream Function theory, from which the results could be obtained for different water depths. For the 100-year design wave in the water depth range of 50 to 400 ft., d/T^2 and H/T^2 vary between 0.01-0.10 and 0.008-0.02 respectively.

The bias in wave kinematics obtained by these non-linear wave theories can be considered as approximately 1.0. The bias obtained by Airy's theory could be determined with reference to these theories.

Airy's linear wave theory gives biased results due to the various assumptions made to simplify its application The bias in the wave loads obtained by the linear theory ($H \approx 0.0$) would reduce, when the wave velocity and acceleration profiles are stretched to the instantaneous water level. Various different techniques are commonly used to stretch the wave kinematics profiles, and the results vary significantly.

When the velocity profile obtained by Airy's theory is stretched to the wave crest (surface) elevation, the bias reduces significantly, and in deep water depth the results are close to those obtained by Stoke's 5th order theory. Given that the results based on Dean's Stream Function theory provides an unbiased estimate of the nominal loads, the mean values and COV for different depth ranges can be established.

Additional bias would exist in the wave loads due to the neglect of wave surface effects. It has been determined that the wave velocity is not zero at the crest elevation but at $\{(u_{cr})^2 / (2\ g)\}$ above the crest elevation and the maximum force intensity occurs at an elevation $\{(u_{cr})^2 / (2\ g)\}$ to $\{2(u_{cr})^2 / (2\ g)\}$ below the crest elevation *[Torum, 1989]*. Such considerations become more important in case the wave impacts the deck or when the air gap is lower than that recommended in the API-guidelines.

The wave crest elevation evaluated by different wave theories would vary. The non-linear wave theories develop peaked profiles (more realistic wave profiles), whereas Airy's wave theory considers a linear wave profile with equal crest height and trough depth. An accurate estimate of the wave crest height is necessary to evaluate the air gap, and to determine the probability of the wave hitting the deck. The wave load increases sharply whenever the wave hits the deck.

Step-3: Establishing the structural characteristics and evaluation of wave loads

The characterization of wave loads on the individual members, and the computations of the global wave loads (base shear and base overturning moment) depends upon the wave kinematics, wave crest elevation, decay pattern of the wave kinematics with depth, and the member properties.

Wave Load on a Member:

The wave load (F) on a cylindrical member is computed as the sum of a drag force (F_D), which is related to the kinetic energy of water, and an inertial force (F_I) which is related to the acceleration of water. This wave load per unit length on a member is described by the empirical model given by *Morison et al [1950]*:

$$F = F_D + F_I$$

$$= [0.5\,\rho\, C_D D]\, u\, |u| + [\pi (D^2/4) C_M \rho]\, [du/dt] \qquad ...(2.13)$$

where: ρ = w/g = density of fluid

C_D = drag coefficient

C_M = inertia coefficient

D = diameter of cylindrical member including marine growth

u = component of velocity vector of water normal to the axis of the member

du/dt = component of acceleration vector of water normal to the axis of member.

In the above expression, the inertial term is very low for the conventional jacket platforms provided with small diameter members, the maximum inertia term being 90° out of phase with the maximum of drag term. The drag force term constitutes of drag coefficient C_D, diameter D, and wave velocity **u**, which are random variables. Upon substituting 'u' by the velocity obtained by Airy's theory, the following expression is obtained:

$$F \approx F_D = K_D\, u\, |u| \qquad ....(2.14)$$

where, K_D = $[0.5\,\rho\, C_D D]$

u = $[\pi H / T]\; f'(y)$

$$f'(y) = \frac{\text{Cosh}\,[k\,y\,d\,/\,(d + 0.6\,H)]}{\text{Sinh}\,[kd]}$$ for intermediate water depth, when

$f'(y)$ at mean sea level is stretched to the free surface (crest elevation)

$f'(y) = e^{ky}$ for deep water depth

$k = 2\pi/L$

L = wave length

d = water depth including storm tide

y = distance from seabed

$f'(y)$ represents the decay of fluid velocity with depth and it varies with the wave theory used. The difference in the wave profile due to the different theories is largely in the wave surface area. For the deep water locations ($d > 0.5\,L$), $f'(y) \approx 1.0$ at the crest elevation, . In case of the intermediate water depth locations ($d = 0.25\,L$ to $0.5\,L$), $f'(y)$ varies between 1.0 to 1.1 at the crest elevation. Drag force, F_D is highly sensitive to the variation in the wave height, and is less sensitive to the variations in, C_D, and D, The drag force equation (2.14) can be written as follows:

$$F_D = K_D\,K_U\,H^2 \qquad \text{.....(2.15)}$$

where, $K_D = [0.5\,\rho\,C_D\,D]$

$K_U = [(\pi / T)\; f'(y)]^2$

In these expressions, the uncertainties are introduced by the C_D, D, T, H, d parameters. The mean value and variance of the wave load, F_D could be determined as follows:

$$F_D = K_D\,K_U\,H^2$$

$$V_F^2 \approx V_{K_U}^{\,2} + V_D^2 + V_{Cd}^2 + (2\,V_H)^2 \qquad \text{....(2.16)}$$

In the medium to deep water depths, the base shear and overturning moment obtained by Airy's theory are under-estimated by about 15% (bias =

1.18). In the shallow water depth, the base shear and overturning moments obtained by Airy's theory are underestimated by 16% to 27% (bias = 1.19 to 1.37) respectively *[OTH, 1988]*.

Drag Coefficient, C_d: The drag coefficient depends upon the member characteristics such as: diameter of member, marine growth thickness, properties of marine growth, roughness of member, and the flow properties such as Reynold's number. The roughness of a member depends upon the type of marine growth accumulated on it. The value of C_d varies between 0.6 to 1.2 for smooth to the marine roughened cylinders [*Heideman et al, 1979*]. In general, for the steel jacket platforms, C_d is assumed between 0.6 or 0.7. The Gulf of Mexico platforms are subjected to soft marine growth, which accumulates at a slower rate, and the importance of marine growth is reduced on the estimates of C_d. A correct measure of marine growth would depend upon the survey level and location of the members.

From *Rodenbusch [1986]* measurements, COV in C_d is obtained as 0.10, which is considered here as type-I uncertainty. From the Ocean Test Structure (OTS) project of Exxon, for the rough (marine fouled) cylinders and high Keulegan Carpenter (KC = uT/D) flow condition, a mean C_d of 1.0 and a V_{C_d} of 0.25 have been obtained, whereas for a smooth cylinder and high KC flow condition, a C_d of 0.68 and a V_{C_d} of 0.26 have been obtained [*Heideman et al, 1979*].

Normally, in design, in the evaluation of the global loads, the nominal C_d of 0.6 is used for the marine-roughened cylinders. By considering the results obtained in the OTS test structure measurements, a bias of 1.67 (= 1.0/0.6) would remain in the C_d value. This bias of 1.67, the type-II modelling error, and the COV of 0.25, which were obtained in the OTS project have been considered in this study.

Uncertainty in the member diameter, D used in the base shear equation arises due to errors in prediction of the marine growth. The bias is considered here as 1.0 and a COV of 0.05 to account for the variations in marine growth estimates. Therefore, the uncertainties in the nominal estimate of K_d would include a bias of 1.67 and a COV of 0.26.

The closed form solution of the Base shear, S_D for a unit-diameter vertical pile, which extends from the seabed to an elevation above the mean sea level, as obtained by the Airy's theory is given as below:

$$(S_D)_m = [0.5\,\rho\, C_D D]\, K_w\, H^2 \qquad \text{....(2.17)}$$
$$= K_D\, K_w\, H^2$$

where, $K_w = 0.125\,g\,[1 + 2kd/\{\text{Sinh}\,(2kd)\}]$

For the deep water locations, K_w would be 0.125g by the linear wave theory, and $K_w \approx$ 0.222g by the Dean's Stream Function theory [*Dean, 1974*] for a wave with a of steepness (H/L = 1/12). For the shallow water locations, $K_w \approx$ 0.25g by the Airy's theory.

K_W as given in equation 2.17 depends on the variations in the wave parameters and water depth. The linear theory gives lower values for K_W, when stretching is not considered. Due to the use of Airy's theory, a higher bias would exist in K_W, which increases with the decreasing water depth. For a water depth in range of 250-400 ft., K_W is 0.22 by Dean's theory and 0.13 by Airy's theory, whereas for a water depth in the range of 25-50 ft., K_W ranges between 0.66-0.74 by Dean's theory and 0.225 by Airy's theory. The bias in the 400 ft. water depth would be around 1.7 to about 3.2 in 25 ft. water depth.

K_W would vary with a number of other parameters such as wave steepness, wave directionality, and current. The variation in wave steepness is due to the inherent characteristics of waves and it can not be precisely predicted. Its influence can be introduced as a type-I uncertainty, B_{IK_u} = 1.0

and $VIK_u = 0.10$. The effect of wave directionality could be considered as a type-II uncertainty. The wave load in any specific direction reduces when directionality is considered. The effect of current on the wave kinematics is not fully established by the available measurement data. In common practice, it is considered that its effect is accounted by neglecting the reduction of forces due to shielding, blocking, and near-surface effects [*Bea et al, 1988*].

Base moment, M_D is available in a closed form solution by Airy's theory. For a vertical pile of diameter D, M_D is given by the following expression:

$$(M_D)_m = (S_D)_m \, d \, Y_D \qquad \text{....(2. 18)}$$

where, $Y_D = 0.5 + (0.25 / n) + (0.5 / n) [\{1 - \text{Cosh}(2kd)\}]/[2kd \, \text{Sinh}(2kd)]$

$n = Cg / C$ = group velocity / wave celerity

By Airy's linear wave theory, Y_D range from 0.82 to 1.0 for the deep water depth locations and from 0.5 to 0.82 for the intermediate water depth locations. These values are obtained when no Wheeler's stretching is considered. By Stream Function theory, Y_D varies between 0.9 to 1.1 for the intermediate depth range near the breaking point, and between 0.9 to 1.0 for the deep water depth.

Step-4: <u>Establishing structural stiffness properties and evaluation of wave load effects: internal member forces</u>

The next step is to determine the environmental load effects on the members and to estimate the uncertainties in such estimates. The load effects may be the member forces, member displacements, joint displacement, etc. The forces in a member are due to two major components: local loads acting on the member and the load component transferred to it from the global structural behavior of the complete platform. The load component due to the global behavior of the structure depends upon the relative stiffness of the member in the plane of load application in comparison to the other adjacent members. It depends upon the structural configuration of the framing and the idealizations made of the stiffness including that of the foundation. Note that in the lower bays (below the wave zone) of a jacket, the global load component would dominate and the local load component would be insignificant due to a sharp reduction in the wave velocity profile.

The accuracy in the evaluation of load effects is directly related to the structural model considered. In this step, a number of improvements in the model can be done to obtain the load effects more accurately. One such modification is to consider the stiffness properties of the components more accurately by considering the brace lengths face-to-face instead of center-to-center.

2.3.3 Uncertainties in Reference Level Load

In this study, the global loads on a platform expressed as base shear and overturning moment have been used in the safety assessment process. The focus of this study is on the platforms located in medium to shallow water depths. In such water depths, base shear governs and gives the maximum load effects than compared to the base moment. Therefore, the uncertainties associated with base shear are needed in a probabilistic assessment.

Base shear expression as given in equation 2.17 could be expressed in probabilistic formulation as follows:

$$(S_D)_m = K_D \, K_w \, H^2 \qquad \text{..... (2.19)}$$

where, $K_D = [0.5 \, \rho \, g \, C_D D]$

$$V_{K_D} = [(V_{C_d})^2 + (V_D)^2]^{0.5}$$

The uncertainties (bias, and COV) associated with the above formulation of base shear have been studied in detail in the recent past by a number of researchers. Some of the more important results published by *Moses (1990), Olufsen and Bea (1990), Nikoladis and Kaplan (1991)* have been reviewed to select appropriate values.

The COV of predicted annual maximum wave height (H) varies between 19% to 36%, and typically V_H of 25% have been recommended for the Gulf of Mexico environment and structural conditions.

Moses (1990) considered K_D and K_w together as analysis variable and determined bias and COV of 0.93 and 25% when the life of a platform is 20 years. Based on these the bias and COV associated with the global loads are determined as 0.70 and 37% respectively *[Moses, 1990]*. These values have been recommended in API-LRFD recommended practice for the offshore platforms *[API-LRFD, 1990]*.

Olufsen and Bea (1990) explicitly considered the type-I (random) and type-II (modelling) uncertainties and considered that the modelling uncertainties do not vary with the return period. In this way, they obtained significantly different uncertainties for the Gulf of Mexico. They reported that COV would vary between 0.73 to 0.98 for the Gulf of Mexico conditions for 100-year wave return period.

Bea (1989), and *Nikoladis and Kaplan (1991)* obtained COV of 0.66 for forces due to 20-year return period wave and COV of 0.63 for 100-year return period wave when the modelling uncertainties were considered perfectly correlated from one year to another. They obtained COV of 0.38 and 0.31 for 20-year and 100-year return period waves, when the modelling uncertainties were treated as independent from one year to another.

In this study, the values reported from latest research have been considered for the example cases. The formulation of fitness for purpose presented in later chapters is based on COV in load effects equal to 0.75.

2.4 Strength of a Platform:

A steel jacket platform is composed of three distinct structural systems: deck, jacket, and pile foundation (Fig. 1.1). Each of these system constitutes of a number of components with similar or varying behavior and failure modes. The piles, jacket frame, and the deck legs are constructed of tubulars while the deck structure is built of WF beams and/or plate girders. The members may exist in intact, deteriorated, damaged, or failed state.

These systems are subjected to a combination of multiple load sources, with load effects transferred in the form of axial load and bending moments in the three directions of each member. The behavior, failure modes, and mechanisms for these three parts differ due to surrounding medium, structural configurations, and the loading characteristics, as were described in Section 1.3.

In this section, the factors which influence component strength of tubular members and ultimate strength of joints are discussed. The emphasis is on identification of the methods used and on evaluation of the uncertainties associated with the parameters and methods.

The typical damages which may occur during the operating life of a platform are identified and the models used to evaluate their reduced strength are reviewed.

The factors and processes which are important to determine the strength of each system (deck, jacket, and pile foundation) or a complete platform are described.

Wave loading is cyclic and random in nature, and thus the component strength and behavior would differ from the static load case. The difference in strength and behavior of components due to cyclic and random loads are described.

2.4.1 Component Strength

The methods used for the evaluation of the ultimate strength of tubular members, interconnecting joints, and welded connections are presented in Appendix-C. These methods are used for the evaluation of component and system strengths, and the uncertainties in the strength estimates. The component strength of tubulars against the following failure modes is required, in order to evaluate the strength of a bay or of the complete platform:

1. Axial compression strength of a tubular.
2. Axial tensile strength of a tubular.
3. Ultimate moment capacity of a tubular.
4. Strength of damaged members.
5. Joint strength: leg-brace and vertical brace-horizontal brace joints.
6. Tensile strength of welded connection: deck-jacket connection.

The models given in Appendix-C considers the effect of axial, bending, and hydrostatic loads acting on members. The influence of out-of-roundness and out-of-roundness of members on their axial strength has been identified for these models.

Tubular braces that are compact (i.e., diameter to thickness ratio = 8 to 25) develop full plastic bending capacity. Such compact sections possess sufficient rotation capacity to redistribute moments and to form a plastic mechanism. The braces with ratio of diameter to thickness greater than 60 are non-compact and fail in plastic buckling range and possess negligible rotation capacity.

The strength of a component is influenced by a number of other phenomenon. The strength of a component is influenced by system effect, i.e., due to other members in the structure. The nature of loading (static or cyclic) influence the component strength. The various damages and deteriorations of member which occur to the member from construction to the current state of a platform have a direct effect in reduction of the mean strength of the components. The primary aspects of some of these phenomenon are reviewed and discussed in the following sub-sections.

2.4.2 Effect of Damages and Deteriorations

The damage and deterioration of a component (member or joint) have a significant effect on behavior of components and they reduce ultimate strength of components. The influence of some of the typical damages to steel offshore platforms are discussed here. The influence of a damage would differ for axial and flexural strengths of a member. Therefore, the location of a member and the load effects on it are important in determination of effect of damage on its strength. In general, a damaged member would be more flexible compared to its undamaged state, which would reduce the post-yield resistance of the member.

In this section, the influence of dents, corrosion, and cracks on component strength are reviewed. These damages could occur during fabrication to operation stages due to a number of reasons which were identified in Section 1.3. The damages could have occurred due to other load sources such as collision, fatigue, dropped objects, etc. The important work in this area has been reported by *Moan (1987), Moan et al (1991), Taby and Moan (1987), UEG (1989), CIRIA-UEG (1985).*

Dents: The strength of tubular members, with damages in the form of permanent lateral deflection (buckling) and local dents have been extensively studied. The location of dent. its size, load type on component (in compression or in tension) are some of the important factors.

The effect of dents and lateral deflection on strength of tubular braces could be determined by the DENTA model developed by *Taby and Moan (1987), Moan and Taby (1987)* and validated by laboratory tests performed for various combinations of forces, end fixity, and sizes. The DENTA model includes the effects of premature buckling, and growth of buckle in post-ultimate regime. A very small bias of 0.9857 and C.O.V. of 0.0722 is reported by this model when compared with the tests.

The post-ultimate behavior of tubulars with large diameter to thickness ratio is significantly influenced due to presence of dents or increase in lateral deflection.

Corrosion: Corrosion occur due to lack of adequate cathodic protection, which occurs due to low potential, missing, or loose anodes. The corrosion may occur throughout the jacket with general loss of thickness of structural members or it may be localized with resulting holes in members. The effect on strength of the components due to general corrosion effect could be determined in similar way as for the non-corroded sections.

The influence of presence of holes on strength of tubular members have been recently investigated by *Hsu and Krieger (1990)* through model tests and analysis. They have found that the strength of a tubular member is significantly influenced by the hole size and slenderness ratio of member, and moderately influenced by the hole aspect ratio. The effect of hole location, number of holes, spacing of two holes on ultimate strength is minimal. Very small holes do not reduce the compressive-load capacity of members.

Cracks: Cracks may occur during fabrication and progress due to in-service cyclic wave loads. The reduction in strength of a member and structural system becomes more important when the crack grows to abnormal size. The time and number of cycles required to propagate a crack from abnormal size or full penetration state to fracture state could be more accurately determined by fracture mechanics approach.

The formation of cracks results in loss of cross section area of a member. The direction of formation of crack and the loads on a member are important to determine the loss in cross section area of the member. Moan (1987) discussed that at the stage of fracture of a member, the dynamic forces would be induced on the structure due to sudden re-distribution of force from the fractured members.

Sollie (1982) reported that the cracks identified in some of the joints in the offshore platforms in the North Sea had an average crack size of 70 mm, with a C.O.V. of 0.6.

2.4.3 Uncertainties and Biases in Strength Evaluation

The bias is a function of load type (axial/bending); reduced slenderness ratio; geometrical imperfections (shape imperfections, out-of-straightness deviations); boundary conditions; residual stresses; material properties, etc.

The inherent uncertainties varies for different components and failure modes. Thus the uncertainties associated with different failure mechanism would differ, in accordance with the cumulative magnitude of uncertainties and biases associated with the physical characteristics and strength evaluation of the component members.

Moses (1989), Frieze (1990) have reported the biases and COV for the Gulf of Mexico conditions, which have been considered in the development of API-LRFD code. From their results, the following values of biases and COV's have been extracted.

Strength Variable	Bias	C.O.V.
Yield	1.10	13%
Tubular Bending	1.26	11%
Tubular Compression	1-1.1	10-19%
Local Buckling	1.05-1.2	12%
Connections (Yura)	1.15-1.55	10-40%
Hydrostatic	1.05-1.10	10-14%
Foundation Piles	1.0	20%

The above values respresent the combined uncertainties in each case and take account of testing uncertainties, differences between test results and strengths as predicted using the design equation, materail variability dimensional variations [*Frieze, 1990*].

The uncertainties associated with a damaged member would be higher than the uncertainties associated with undamaged members. The bias and COV obtained for a dented and cracked members were reported in the previous section. In this study, in evaluation of the example platforms, a constant COV of 0.25 has been assumed for demonstration purpose.

2.5 Summary

The primary elements for evaluation of safety have been discussed. The importance of the best estimates of loads, load effects, and strength has been emphasized. The uncertainties inherent in the parameters describing the estimates of loads, load effects, and strength have been discussed. The current state of the platform and its components should be established for making a reasonable assessment of safety of a platform and its components.

The process involved in evaluation of wave loads on a platform has been discussed in detail. The models used for evaluation of the strength of components of a platform in their intact and damaged states have been discussed. The formulas involved have been presented in Appendix-C.

CHAPTER 3

CAPACITY OF A PLATFORM

The capacity of a platform is one of the major components in safety assessment. It depends upon the mean (nominal x bias) estimates of the load and strength of its components, and the uncertainties (variances and biases) in their estimates, which were discussed in detail in Chapter 2.

The conventional API-RP-2A based working stress design, considers that the strength of a platform would reduce upon failure of any of its component. Therefore, the design aims at a strength level of the platform at which under the specified reference level loads and nominal maximum allowable stresses in the components, none of its component is overstressed.

In recent research on the system reliability and ultimate strength, the contributions of the system effect (complete structure) to maintain the platform in a safe state, beyond failure of a single component has been studied. In this study, the results from these research works have been utilized to develop the simplified techniques for capacity evaluation, which are aimed at their utilization by the regulatory bodies and large operators, who may need to periodically assess the safety of a large number of platforms.

In this chapter, first a quantitative characterization of the capacity of a platform is discussed in detail. Then, the load-deflection behavior of a platform with increasing lateral load is discussed. The qualitative and coarse quantitative approaches developed in this study for the evaluation of capacity are discussed. An approach is presented, which utilizes the results obtained from the conventional linear-elastic analysis, to make an assessment of the ultimate capacity, as characterized in this study. Finally, the non-linear analysis techniques commonly used for the evaluation of ultimate capacity of a platform are reviewed and discussed.

3.1 Reserve Strength Ratio (RSR)

3.1.1 Definition of Reserve Strength Ratio

The capacity of a platform is expressed with the Reserve Strength Ratio (RSR). The RSR is defined as the ratio of ultimate load capacity of the platform, R_u, divided by a reference force, S_r (Fig. 3.1):

$$RSR = \frac{\text{(Ultimate capacity of a platform)}}{\text{(Reference level lateral load)}} = \frac{R_u}{S_r} \qquad \text{....(3.1)}$$

In this expression, R_u represents the best estimate of the capacity of a platform at the Ultimate Limit State (ULS). The ULS capacity, R_u thus represents the limit state of a platform beyond which any increase in load would result in its collapse. In Fig. 3.1, R_{UI} represents the ultimate capacity of a platform in its intact state and R_{UD} in its damaged or deteriorated state.

S_r represents the reference level global lateral load (base shear) on the platform, which should be based on the minimum acceptable 100-year return period wave force implied or suggested by the currently accepted guidelines or requirements for design of an equivalent new platform, such as given in Section 2.3.4g of API-RP-2A [*API, 1989*]. The details of evaluation of the API-guideline reference level wave force are given in Section 2.3 of this study.

Figure 3.1 shows a non-linear load-displacement (P-Δ) behavior of a steel jacket platform with increase in lateral load. The non-linear behavior becomes dominant upon failure of the structural members, due to a sharp reduction in the lateral stiffness of the platform. Thereafter, its lateral displacement increases rapidly with further increase in lateral load.

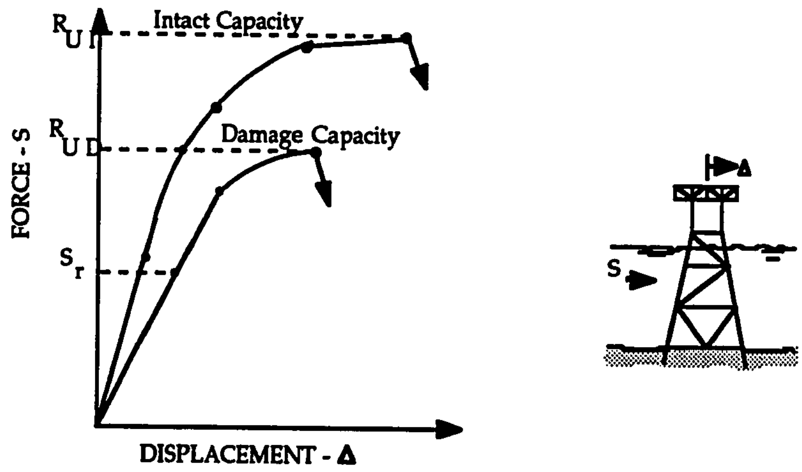


Figure 3.1: Definition of Reserve Strength Ratio (RSR)

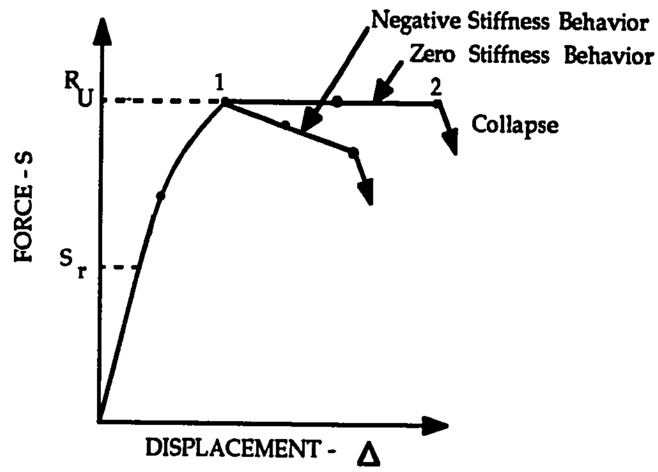


Figure 3.2: Load- Deflection Diagram

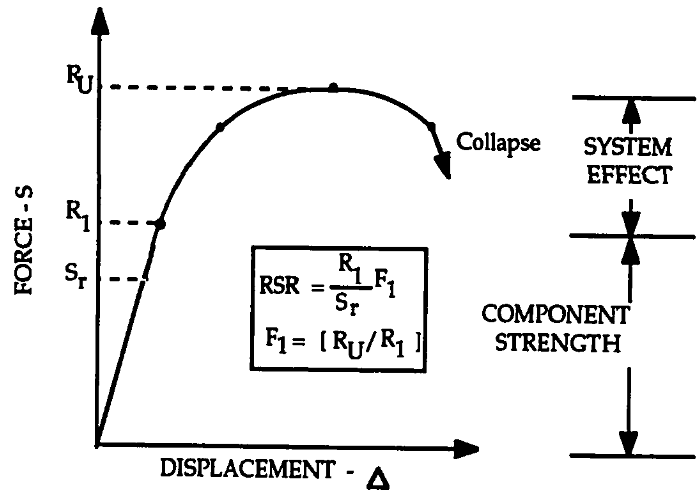


Figure 3.3: Load- Deflection Diagram

Beyond a limiting lateral load level, with the formation of a partial failure mechanism and a significant global displacement of the platform, the vertical load carrying capacity of the platform decreases. This decrease is associated with either failure of other members or due to the additional load component from the secondary P-Δ effect (instability effect of vertical load).

In order to support the vertical loads (dead & live loads), the lateral load level on the platform must remain the same (zero stiffness behavior) or decrease (negative stiffness behavior) as shown in Fig. 3.2. Ultimately with the formation of a full plastic mechanism, the platform will collapse. The magnitude of displacement (Δ) from point 1 to point 2 in Fig. 3.2 depends upon the material properties and configuration of the platform structure.

Alternate Representation of RSR

RSR can also be represented on the basis of the load level at failure of the first member and the structural redundancy of a platform (refer Fig. 3.3).

$$R_U\ (R_{UI} \text{ or } R_{UD}) = (R_1) \times (RF_1) \qquad \text{....(3.2)}$$

$$RSR = R_U / S_r = [R_1 / S_r]\ RF_1$$

$$= (CS_1)\ RF_1 \qquad \text{....(3.3)}$$

R_1 represents the lateral load (base shear) on the platform upon failure of its first member. R_1 is due to the difference in the magnitude of the ultimate strength of a component relative to the load acting on it i.e., it corresponds to the global load level (base shear) at the first member failure. CS_1, component strength, corresponds to the ratio of global lateral load at the failure of the first member to the reference level load. Note that this component strength is different than a conventional factor of safety, because here the reference level global lateral load is used. However, the magnitude of component strength would be proportional to the factor of safety (FOS) implied in the working stress design (WSD) or in the load and resistance factors included in the LRFD code.

RF_1, the redundancy or system factor, represents the overload capacity of a platform beyond the load level (R_1) at the first member failure. Redundancy factor is thus the ratio of lateral load (R_U) at formation of a failure mechanism to the lateral load (R_1) at failure of the first member. It depends upon the post-failure load carrying capacity of the failed members and the properties and behavior of the other components of a platform. The equation (3.3) can be rewritten as follows:

$$\textbf{System RSR} = \textbf{Component Strength} \times \textbf{Redundancy Factor} \qquad \text{....(3.4)}$$

Where:

$$\text{Component Strength } (CS_1) = \frac{\text{Lateral load at First Member Failure}}{\text{Reference Level Lateral Load}} \qquad \text{....(3.5)}$$

$$\text{Redundancy Factor } (RF_1) = \frac{\text{Ultimate Structural Resistance}}{\text{Lateral load at First Member Failure}} \qquad \text{....(3.6)}$$

Redundancy exists in a statically indeterminate structure i.e., a structure in which the alternate load paths allow it to support the same or higher load than the load at which the first member failure occurs. Offshore platforms are typically statically indeterminate structures and thus they are likely to show redundant behavior by withstanding the same or higher loads even after failure of one or more members.

Alternatively, a redundant member can be considered as a member with very low utilization ratio (resultant stress interaction ratio) and whose removal does not affect the load path in the structure or for which the loads at first member failure remains the same. Note that some redundant members may become active, or their utilization ratio may increase, due to the load transfer upon failure of the first member. *Lloyd and Clawson [1983],* and *Nelson [1987]* classified the members of a jacket platform in different categories according to their importance. The existence of such redundant members may contribute to the redundancy factor (RF_1) by the formation of alternate load paths. RF_1, corresponds to the minimum combination of members (or a cut-set in the system reliability terminology) leading to the formation of a failure mechanism.

Thus, the redundancy factor for a platform or the overload capacity of a platform beyond its first member failure depends upon:

1) **Degree of static indeterminacy of the platform structure.**
2) **Degree of unequal loading (member stress ratio) among members.**
3) **Post failure behavior (ductility) of the individual members.**
4) **Feasibility of structural configuration to provide alternate load paths.**

The ultimate collapse load for a platform with a low degree of static indeterminacy may not differ much from the load at its component strength level. If all the primary members (vertical braces, deck or jacket legs or piles), which are likely to form a collapse mechanism are of the same size and length, equally loaded, and all are in compression or in tension, then a failure mechanism is likely to form upon failure of the first member.

An assessment of unequal loads in the major members is made based on their utilization ratios (or resultant member stress ratios). The post failure behavior of a component depends upon its slenderness and D/t ratios. The alternate load paths in a platform become important for transfer of load from the failed member(s) to other members in the platform.

RSR should be taken as an index, due to the various idealizations made in the evaluations of load and strength for a platform, and due to the uncertainties associated with the different parameters and methods used.

Due to the simplifications made in the analytical methods, the computed values may differ from the true RSR. RSR should not be viewed as a single number, but a quantity that can vary within a range of possible values. The mean and variance of RSR would depend on the mean values and variance of the parameters used in the load and strength evaluations.

Note that this RSR would represent a best estimate value (or nominal value, RSR_n) with associated uncertainties, i.e, a bias and variance around its estimate. The bias and COV are determined based upon the parameters identified in Chapter 2 for the evaluations of load and strength, and the methods used in estimation of RSR.

$$(RSR_{mean}) = (B_{RSR})\,(RSR_n) \qquad \text{....(3.7)}$$

with $$V_{RSR} = [\,(V_{R_u})^2 + (V_{S_r})^2 + (V_{B_{RSR}})^2\,]^{1/2} \qquad \text{....(3.8)}$$

where, RSR_{mean} corresponds to the value obtainable by tests or by sophisticated analyses, RSR_n (or nominal) depends on the method used in its evaluation and B_{RSR} represents the associated bias in RSR_n. The COV (V_{RSR}) in mean RSR includes the variances in the estimates of R_u, S_{ref}, and B_{RSR}. Variance in bias, B_{RSR} would exist due to its change with the variations in the parameters and in the estimates of R_u, and S_{ref}. Fig. 3.4 presents a schematic representation of the bias (model error) in strength evaluation by simple methods, when compared with its true estimate.

For the intermediate to shallow water depth GOM platforms considered in this study, it is assumed that the results obtained by the non-linear static pushover analysis represent the best estimate of ultimate strength. The estimates of the biases and variances in the nominal RSR by the simplified methods are based on the results obtained by the non-linear analysis.

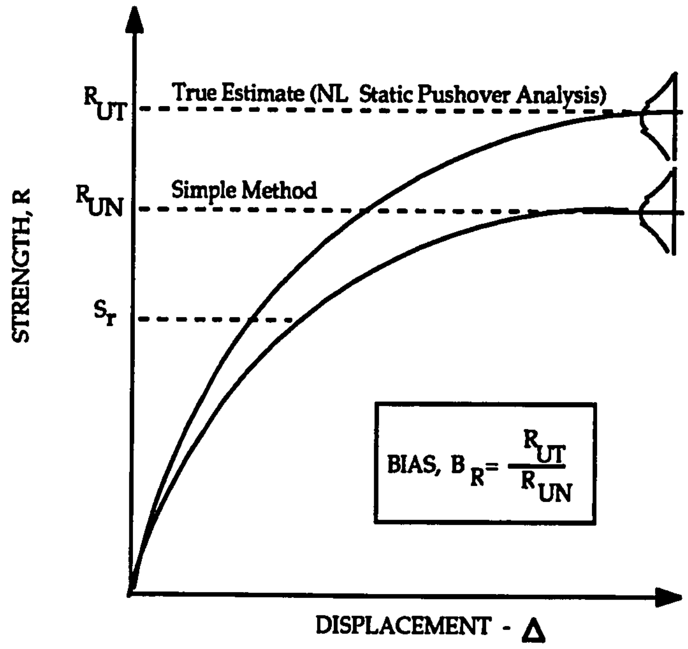


Fig. 3.4: Definition of Bias in Strength

Note that for the platforms with the variations in their parameters and configurations, different values of B_{RSR} would be evaluated. B_{RSR} for a 4-leg platform will differ from that for an 8-leg platform.

$$\text{Thus, } (RSR_{mean}) = (B_{RSR})\,(RSR_n) \quad \text{or}$$

$$= (B_{RSR})\,(CS_n)\,(RF_n) \qquad ....(3.9)$$

$$\text{with} \quad V_{RSR} = [(V_{B_{RSR}})^2 + (V_{R_1})^2 + (V_{S_r})^2 + (V_{RF})^2]^{1/2} \qquad ....(3.10)$$

The above description of RSR is based on the strength of a platform against formation of the failure mechanisms, when increasing lateral loads are imposed. In a similar way, RSR could be described for strength of a platform against formation of the axial failure mechanisms (such as deck leg-jacket connection failure, pile-soil failure, pile-grout failure).

RSR is required in a platform to maintain it in serviceable state in its intact, deteriorated, damaged physical state, or upon failure of some of its structural members (partial mechanism formation stage). For the 4-legged platforms in the Gulf of Mexico designed by the current API guidelines, RSR is generally above 1.5. For an 8-legged platform in the Gulf of Mexico, RSR can be more than 2.5. On the contrary, for some of the older platforms, RSR can be less than 1.0.

In addition to RSR, **residual strength** has also been used in the past to represent the overload capacity of a damaged platform. It has also been used to represent the ultimate load level at collapse, which a platform can resist beyond the load level upon failure of one or more of its members. Residual strength would exist in a platform essentially due to its structural redundancy, which could keep it safe and serviceable in its damaged condition. Residual strength could be defined as below:

$$\text{Residual Strength (RS)} = \frac{\text{Ultimate Capacity of a Damaged Platform}}{\text{Ultimate Capacity of an Intact Platform}} = \frac{R_{UD}}{R_{UI}} \quad \text{....(3.11)}$$

RSR has been considered by researchers in deterministic and probabilistic ways to represent the ultimate capacity of a platform. Their definitions are briefly discussed further in this section.

Lloyd & Clawson [1983] defined RSR by a Reserve Resistance Factor (REF), the ratio of ultimate system strength to the design environmental load. They also defined residual strength by Residual Resistance Factor (RIF), the ratio of residual strength of a structure upon failure of a member or of a severely damaged platform to the load level at ULS of an intact platform. It could be viewed as a value corresponding to the inverse of redundancy factor defined in expression 3.6. When (REF x RIF > 1.0), the damaged platform would be safe against the design environmental loads. *Lalani & Shuttleworth (1990),* reformulated the definition of RSR considered by *Lloyd & Clawson (1983),* as the ratio of lateral load at structural collapse to the design load. In such descriptions of the residual strength of a damaged platform, its ultimate strength in the intact state will also have to be evaluated, whereas in the platform, its ultimate strength in the intact state will also have to be evaluated, whereas in the characterizations of RSR given in the expressions 3.1 to 3.6, the ultimate strength of a platform is evaluated only in its as-is state (intact or damaged), thus avoiding a separate estimate of the residual strength.

3.1.2 Sources of Reserve Strength

Reserve strength exists in a platform when either the magnitude of the lateral load (base shear) at failure of the first member is more than a reference level load (S_r). or when the redundancy factor (RF) is greater than 1.0 and the product of the lateral load at first member failure and RF is greater than S_r. Conventional design aims to achieve a strength for each component above the load effects on it from the reference level load on the platform during its in-service state. It is normally achieved when the interaction equations for the various combinations of the axial and bending stresses from the vertical and lateral loads, the stress due to hydrostatic pressure, and the allowable stresses for the material are satisfied at all of the cross-sections along the length of members [Section 3, *API 1989*]. In conventional design, there is no direct consideration of the system effect, RF, whereas in the re-evaluation phase of the aged platforms, the contributions of both the component strength and system redundancy factor are important in order to obtain an accurate estimate of RSR and prevent excessive over-conservatism. In this section, the various sources which contribute to the reserve strength of a platform are presented.

The reserve strength associated with the component strength has been well documented and supported by the detailed analyses and laboratory model tests, whereas estimation of the redundancy factor, which is a system effect, has been only recently attempted by the detailed analytical procedures and in some cases by simple plane frame model tests [*Grenda et al, 1988; Lalani and Shuttleworth, 1990; Zayas et al, 1982*]. However, due to practical limitations, no attempt has been made to establish the redundancy factors associated with the large number of different configurations which are used for GOM jacket platforms. A reasonable estimate of the redundancy factor will normally require detailed structural analyses.

Component strength depends on the strength of members and joints, and the loads acting on them. The strength of a member is evaluated by use of standard formulas prescribed in API-RP-2A [*API, 1989*], and described in Chapter 2 and Appendix-C. These have been well established over time and are supported by a substantial data base from the strength tests. On the other hand, the empirical formulas for the ultimate capacity of a joint have been only recently established and then only for a few simple configurations. Some of these formulas are based on interpretation of the test data, while for those joint types with insufficient data, it has been necessary to incorporate a very conservative approach. Therefore, the estimates of component strength based on these formulas may contain large uncertainties.

The reference level load will have a major influence on the component strength, CS_1 (the load at first member failure) and a moderate influence on the redundancy factor, RF_1. The uncertainties in CS_1 would correspond to the cumulative uncertainties in evaluations of the wave loads and member capacity. The uncertainties would vary with the failure modes of a member. Thus, by establishing the failure modes which may occur in a particular member, a more accurate estimate of the uncertainty level in the ULS capacity of the platform (or its sub-structure) can be made.

The redundancy factor depends upon the structural characteristics of a platform, i.e., structural configuration (bracing pattern), ductility of components, and design philosophy. The uncertainties associated with the material properties and component behavior produce variances in the redundancy factor. Some of the sources which are likely to introduce reserve in capacity at the component level are listed below:

* **Material properties;**
* **Code specified stress check and strength formulas;**
* **Design to first yield instead of the capacity at the formation of full plastic hinge;**
* **Overdesign and over-sizing of the components;**
* **Excess capacity due to extra steel provided for safety against other load sources;**
* **Simplifications made in the computations.**

The material properties considered in the conventional design of a platform would contribute to the reserve in capacity of a platform due to: the expected (mean or test) yield strength of steel is higher than its code specified nominal yield strength; the increase in yield strength due to the strain-rate effect is not considered in the normal design. For the mild steel, generally an elastic-perfectly plastic behavior is assumed, which introduces conservativeness in the design by neglecting the contribution of strain-hardening at large deflections.

The structural members may have been overdesigned for several reasons, such as: to reduce the number of design iterations; to provide a symmetry to the structural framing; to size the members for safety during the temporary design stages such as construction, transportation, and installation. In some cases, the structural members may have been ordered in advance before the detailed design stage to meet the tight project schedule. Also, an oversized member may be provided due to the availability of limited number of sizes. All these tend to increase the component strength and RSR.

The component strength computed by the current acceptable guidelines may differ for the aged platforms, due to the differences in the state-of-practice (design criteria) over time. Note that the early generation platforms were usually designed by hand computations of the wave loads and by the simplified structural analyses which considered the members pinned at their ends. It is likely that the component strengths for the similar old platforms may differ from one to the next structure. In addition, two platforms designed by the same design criteria and for similar conditions may have similar component strength, but their redundancy factor (and RSR) may substantially differ based on their structural configuration.

3.2 Evaluation of Reserve Strength Ratio

3.2.1 Load - Displacement Behavior

The typical load-displacement behavior of a steel jacket platform is as shown in Fig. 3.1. Such a behavior is obtained upon complete or partial failure of one or more of its structural components, when it is subjected to the increasing lateral loads. The key elements which must be addressed in the development of the load-displacement diagram are: the loads acting on a platform and its components, the strength and material characteristics of components, the load and strength levels at component failures and at formation of the failure mechanism, and the deflection (Δ) at which instability (P-Δ) would occur.

The global load (usually base shear or overturning moment) upon failure of the first component could be considered to represent a "lower bound" estimate of the ultimate load capacity of a platform. The global load level at formation of a failure mechanism, due to failure of several members, would represent an "upper bound" estimate of the ultimate capacity. The difference between "upper bound" and "lower bound" estimates depends largely on the structural characteristics, which introduce structural redundancy in a platform. The structural characteristics, which enable total or partial re-distribution of loads from a failed member to others, provide alternate load paths, and introduce redundancy to the system, were discussed in Section 3.1.2.

Note that for an intact, well designed platform according to the latest API-working stress design (WSD) practice, the "lower bound" of RSR would be higher than 1.0, due to the factor of safety introduced in the design of components. Up to failure of the first component, the platform is likely to behave in a near-linear elastic mode and the non-linearities would not have a significant influence. Therefore, several simplifications can be made in the determination of "lower bound" of ultimate capacity, which can be used as its first estimate for the screening of a large number of platforms.

A true estimate of "upper bound" of ultimate capacity would require explicit consideration of the structural characteristics which provide redundancy, and of the random and cyclic nature of the wave loads. A realistic description of the behavior of a steel platform should consider instability, second order effects, and displacement in addition to the strength criteria. With such considerations, the behavior becomes highly non-linear and could be developed by use of the non-linear analysis techniques.

A simplified technique called as "static push-over analysis" is widely used. More accurate results could be obtained by time history analysis which could consider cyclicity of the lateral loads, but are computationaly very complex, time consuming, and costly.

Less accurate (or biased) estimates of the ultimate strength (upper bound) can be based on simplified techniques, which utilize expert judgement in *a priori* selection of the possible failure modes and mechanisms. Such methods would be useful in screening a large number of platforms, which are not expected to have abnormal low strength. Such techniques aim at determination of a particular pattern or combination of members, whose combined failure would correspond to a "failure state" with the formation of a mechanism. In this way, we are dealing only with the failure or collapse state of the platform, while the intermediate stages in propagation of the structure to its collapse state are neglected. This is similar to the plastic analysis concepts and it aims at a significant reduction in the computations.

The three methods for evaluation of the **upper bound of ULS capacity** of a structure are as follows (Fig. 3.5):

1) **Rigid-plastic failure load analysis;**
2) **Rigid-plastic large deformation analysis;**
3) **Linear-elastic failure load analysis.**

Rigid-plastic analysis is based on the assumption that the global displacement of a platform will be null, even with the failure of successive members in the formation of a mechanism. This assumption seems unrealistic due to the finite displacement of the complete platform or its sub-structures, upon the failure (buckling or yielding) of its successive components. With an increase in the lateral deflection and the topside loads acting on the legs, additional moment would be induced on the platform, which is known as secondary moment (or P-Δ effect). A better estimate of the ULS capacity is obtained by consideration of the secondary moment effect.

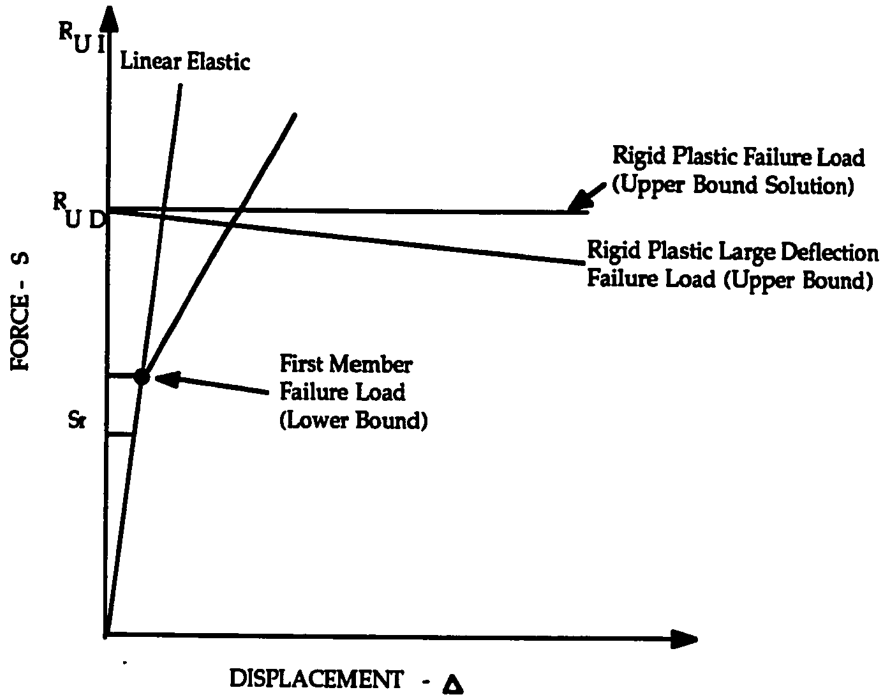


Figure 3.5: Coarse Quantitative Evaluation of Reserve Strength Ratio (RSR)

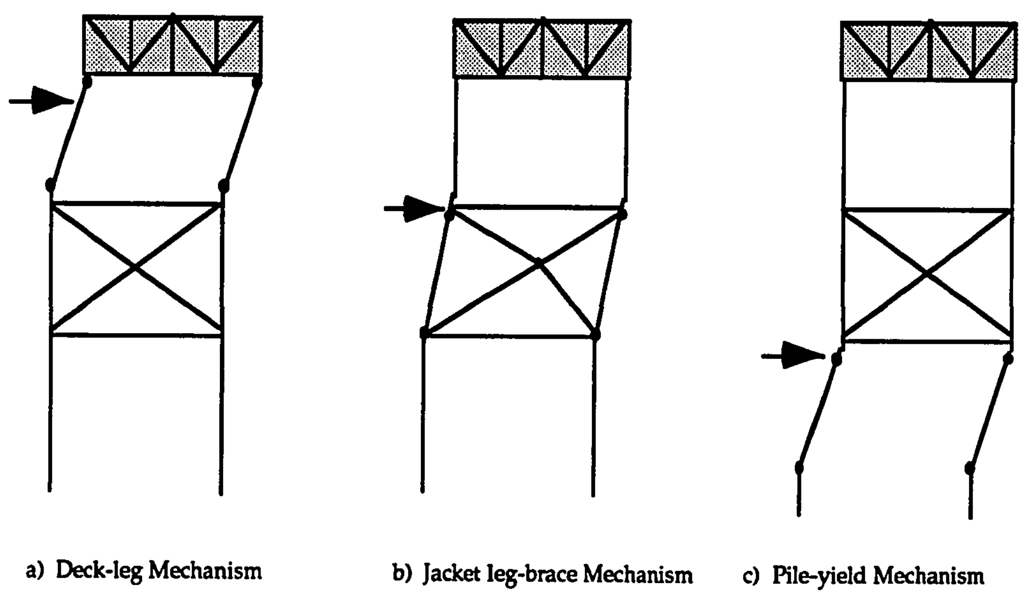


Fig. 3.6: Typical Bay Failure Mechanisms Against Lateral Loads

Under the third alternative the lateral stiffness matrix for a platform is determined at two or more different stages: the intact state, and upon failure of one or more members. In this way a stepwise load-displacement behavior is generated.

Rigid-plastic failure load analysis techniques would give a good estimate of the ultimate capacity of the failure mechanisms formed in the deck and foundation bays as shown in Figs. 3.6(a) and 3.6(c). A mechanism will form upon yielding of the deck legs or piles at their upper and lower ends or at intermediate elevations. The ultimate lateral load carrying capacity for the deck or foundation bays is evaluated by equating the internal work done in the formation of hinges (mechanism) with the external work done by the loads acting on the bay. In this way, it is assumed that upon formation of a hinge at a section, the loads beyond the hinge capacity are transferred to the other sections. The details in evaluation of the rigid-plastic collapse load are given below.

The internal work due to formation of hinges is given by:

$$W_i = 2\,[\Sigma M_{pj}]\,\theta$$

where, j represents the number of legs, M_{pj} represents the plastic moment capacity of leg j, and θ the horizontal deformation of the leg.

The external work due to the lateral load acting on the bay is given by:

$$W_e = P_1\,\Delta + P_2\,(X/L)\,\Delta$$

$$= [P_1 + P_2\,(X/L)\,]\,L\,\theta \qquad = [P_1\,L + P_2\,X]\,\theta$$

By the virtual work theorem, the external work should be equal to the internal work for equilibrium of structure.

Thus, $We = Wi$; $\qquad [P_1\,L + P_2\,X] = 2\,[\Sigma M_{pj}]$

For an 8-legged platform with uniform leg sections, the equation becomes:

$$[P_1 L + P_2 X] = 2\,[8\,M_p] = 16\,M_p$$

$$\text{If } X = L, \text{ then } [P_1 + P_2] = 16\,M_p / L \qquad \text{....(3.12)}$$

This value represents the ULS capacity against the formation of rigid-plastic mechanism in the deck bay of an 8-legged platform. In case of formation of the mechanism in the foundation bay, an additional term is introduced on the strength side to account for the load carrying capacity of the surrounding soil, thus the ultimate capacity would increase.

Rigid-Plastic with Large Deflection: The ULS capacity obtained by the rigid-plastic behavior will reduce with increasing deflection and consequent additional overturning moment (P-Δ effect) from the vertical loads acting on the legs. This effect of the vertical loads can be simulated by an increase in the lateral loads by an amount equal to ($P\,\Delta/L$), which would result in reduction of the rigid-plastic ultimate capacity by the same magnitude. Such a reduction in the ULS capacity will be important for the platforms with heavy topside facilities. In this case, the equation 3.12 reduces to:

$$[P_1 + P_2] = 16\,M_p / L - P\,\Delta / L \qquad \text{....(3.13)}$$

A diagonal wave is likely to govern the yielding of first yield section or the formation of hinge, which may occur in a corner leg along the wave direction. Upon yielding of the first section in a leg, the load re-distribution will occur through the deck girders or pile top framing. Further sections will yield with an increase in the lateral loads, leading to ultimate formation of a mechanism. The ULS capacity for the deck and foundation bays is likely to remain nearly the same for all of the wave approach directions, in case their framings provide sufficient load transfer capability for the formation of a ductile failure mechanism.

<u>Linear elastic failure load analysis (upper bound)</u>: An improved estimate of upper bound of the ULS capacity could be made by following the sequence of failure of members and subsequently updating (reducing) the linear-elastic stiffness of the platform. In this approach, a new physical state is considered upon failure of a component and a load-displacement diagram is developed as shown in Fig. 3.7. It aims at identification of the "most likely to fail" member at each physical state, and determination of the global load level (base shear or overturning moment) which would cause failure of that member in the current state.

The ultimate load capacity of the jacket part of a platform can be determined based on this approach. As stated earlier, the load carrying capacity of a jacket beyond failure of its first component will require an accurate characterization of the post-failure strength (residual strength) of the

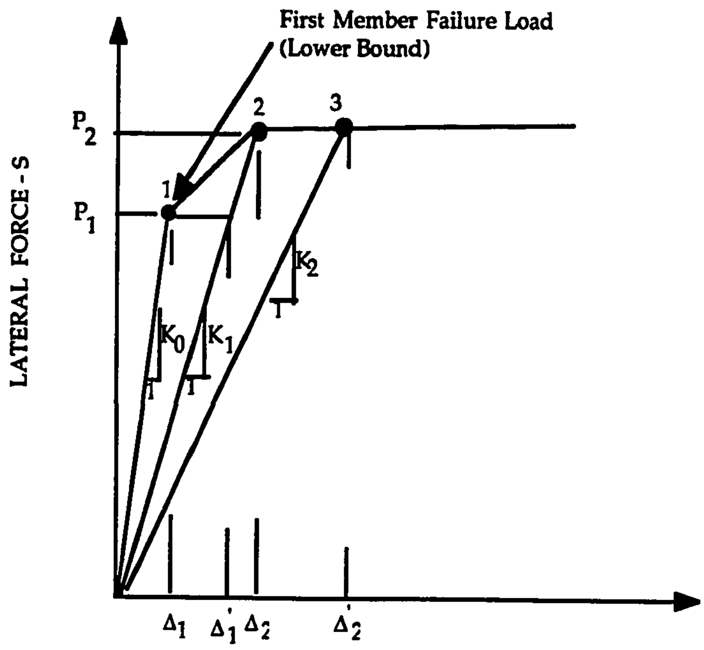


Figure 3.7: Stepwise Development of Load-Displacement Diagram

component. The stiffness of the jacket structure will change upon failure of a component. A piece-wise linear load-displacement behavior as shown in Fig. 3.7 can be traced by modifying the stiffness of the jacket upon failure of a component and reanalyzing the jacket to determine the load level at the failure of next component.

Note that this method considers development of load, stiffness, and displacement matrices for the complete platform. Hence, in this way the sequence of failure of the components in any part of the platform can be determined. The load-displacement behavior in Fig. 3.7 is essentially linear-elastic before failure (yielding or buckling) of the first component. The non-linearity in the behavior before failure of the first component will be primarily introduced by the soil-structure interaction (modelled by the soil springs). Such a behavior can be developed in the following way:

a) <u>Before failure of the first component:</u> In an undamaged and undeteriorated jacket structure, a compression brace is likely to fail first, if it is provided. From Fig. 3.7, we obtain the following:

$$P_1 = K_0 \Delta_1$$

where, P_1 is the global load (base shear) at failure of the first component, K_0 is the initial stiffness of platform, and Δ_1 is the global displacement of the platform usually considered at center of the upper deck level.

b) <u>Upon failure of the first component:</u> The stiffness of a component upon failure becomes zero (tension-brace) or negative (compression-brace) with further increase in displacement. At the moment of failure of the brace, stiffness of the platform reduces from K_0 to K_1 but the lateral load is assumed to remain the same. To maintain this load level (P_1) the displacement will increase to (Δ_1'), and the load shed by the failed component will be carried by the other members in the bay.

$$P_1 = (K_1)(\Delta_1')$$

$$(\Delta_1') = P_1 / (K_1)$$

c) <u>Before failure of the second component</u>: In case, the other components have the capability to carry more load, the lateral load will increase to P_2, till the next component fails.

$$P_2 = (K_1)(\Delta_2)$$

d) <u>Upon failure of the second component</u>: The stiffness of the second component likely to fail will reduce to zero or becomes negative, and the overall stiffness (K_1) of the platform will change to K_2. To maintain the same load level (P_2) at the reduced stiffness, the global deflection (Δ_2) will increase to Δ_2'.

$$P_2 = (K_2)(\Delta_2')$$

The instability effect due to secondary moment from the vertical loads must be included, as it will reduce the load carrying capacity at this deformation. In this way, the piece-wise linear P-Δ diagram 0-1-2-3 , as shown in Fig. 3.7 is developed and the ultimate load capacity of a platform is obtained.

The accuracy in the results obtained from this approach would depend upon the characterization of the geometric and material non-linearities for the components. Various degrees of approximations can be made in the characterization of post-failure capacity of the components. The post-failure capacity of a compression-brace depends upon a number of factors, which were discussed in Section 2.4. The global load at failure of successive components would depend upon the post-failure capacity of components.

3.2.2 Major Attributes for Reserve Strength Ratio

The major attributes, which have a significant influence on the capacity (RSR) level are identified and discussed in this section. These attributes have been selected upon a review of the properties, functions, design basis, past performance, and behavior of the platforms and their components.

The major causes for failure of the platforms in the Gulf of Mexico due to hurricane overload were summarized in Table 1.3. The primary reasons for failure were: wave hit the deck; wave-induced soil movement occurred at the location; inadequate design criteria was used; and prior damaged members were not repaired. During the operational life of a platform, the variations in its physical condition can influence its load and capacity levels.

The major attributes which have a significant influence on the RSR of a platform are: Age of platform; Variation in design criteria; Deck elevation; Platform modifications; Redundancy of platform; Bracing pattern; Platform condition. The influence of these attributes on the capacity of the three parts of a steel jacket platform are discussed further in this section and a summary of the qualitative assessment is presented in Table 3.1.

Age of platform: Ageing of a platform has a significant influence on the structural strength of the jacket part due to its deterioration in the corrosive medium, and the difficulties in the underwater maintenance. The ageing effect will be from corrosion of the structural components, formation and growth of the weld cracks. The influence of ageing can be accounted by reduction in the section properties of the affected components.

Design criteria variation: The maximum influence on capacity (RSR) will occur by increase in the load level of a platform, due to a significant variation in the reference level wave from that considered in design. In case

Table 3.1 : PLATFORM CAPACITY EVALUATION- ATTRIBUTES

S. No.	Attributes	Deck Bay	Jacket bay(s)	Foundation Bay
1.	Age of platform	Very low effect Medium effect, if unpainted	Corrosion Cracks propagation Fatigue	Very low effect
2.	Design criteria variation- H	If wave hits deck- High - Very High effect	Load increases	Base shear increases
3.	Deck elevation- wave ht. changes	High influence	Lateral load increases	Base shear increases
4.	Platform modifications	Knee braces Conductors number Appurtenances Vertical load	Strength improved by grouting Removal of appurtenances	Usually no modifications Foundation strength- ening case possible
5.	Structural redundancy	Compact sections Ductility effect	Very High influence	Compact sections Ductility effect
6.	Bracing pattern	Usually not applicable Some decks braced	High influence	Not applicable
7.	Platform condition- damages	Usually not damaged	Brace damaged Corrosion/age Boat collision Dropped objects	Installation overstresses Underdriven piles

the wave does not hit the deck, the safety of jacket and pile foundation would be affected due to an increase in the lateral load. The influence of change in wave height will be significant on the platform loads as given by equations 2.13 and 2.17 *[Section 2.3.2]*. A change in design criteria arising from wave induced soil movement will have a significant influence on safety of the jacket and foundation bays.

Deck elevation: The deck elevation in relation to the wave crest elevation is important. In case the reference wave changes and the wave hits the deck, the increase in load would be significant and the capacity of all of the three parts of a platform would be affected. When the wave hits the underside of deck, local slamming loads are imposed. In case a minimum of 5 ft. air-gap, the gap between the design wave crest elevation and the bottom of steel elevation of the lower deck, was provided to meet the API-RP-2A guidelines and the reference level wave is just below the deck, an increase in wave load is likely to remain lower than 25 %.

Platform modifications: The modifications to a platform from loading, structural strength, operational, and control aspects would have a direct influence on its safety level. The variations in the vertical loads, due to modifications in the topside facilities, will have a minor influence. The effect of addition or removal of the conductors, risers will be significant on the platform loads. Removal of appurtenances (boat landings, caissons) and part topside facilities (cellar deck equipment, mezzanine deck) will help in reduction of the wave loads. The variation in marine growth will have a linear effect on the wave loads. The strength may increase due to grouting of leg-pile annulus, grouting of braces, grouting of piles, and providing additional braces.

Structural Redundancy & Bracing Pattern: Structural redundancy in a platform affects the capacity of jacket and foundation. The structural redundancy of a jacket bay depends upon: the degree of static indeterminancy; the degree of inequality of loading in the members or the difference in their utilization ratios; the bracing pattern; the ability of framing to re-distribute the loads upon failure of a member; and the post-collapse behavior of a compression brace. The deck and pile bays would show a robust behavior when their sections are compact. Due to their ductile behavior, premature rupture is not likely to occur and the members would sustain their peak loads.

Platform Conditions: The damage to a platform in its in-service state is likely to be more in the jacket part from sources other than wave overload and fire. A decrease in strength may occur due to corrosion and damage of the members, growth of weld cracks, installation errors. The jacket components would have damage in the form of dents, holes, rupture of braces or legs, and weld cracks. The foundation bay capacity may reduce due to overstressing of piles during driving and due to underdriven piles.

3.2.3 Overview of RSR Evaluation

An overview of the overall methodology developed for fitness for purpose (FFP) evaluation, at the four screening cycles, was given in Section 1.4. An evaluation of RSR is needed at each screening cycle, which would differ due to the variations in the state of knowledge of a platform and the methods used for evaluation of RSR at a screening cycle.

At the initial screening cycles, a very large number of platforms would need evaluation of RSR. Therefore, the first cycles should be based on simplified techniques, which must be easier to apply and provide results in lesser time with minimal associated cost. A well designed platform in accordance with the latest API guidelines, would have an RSR greater than 1.0, whereas for a damaged platform or a platform with an increase in the reference level load, RSR could be less than 1.0. Hence, the likelihood of reduction in the capacity of a platform at the initial cycles could be determined by comparison of the design criteria of the platform with the current API- guidelines.

The conventional design of the platforms is based on the linear-elastic analysis. Such analysis techniques are well known and they take moderate time. Therefore, the capacity of the reduced number of platforms, which showed likelihood of reduction in their capacity levels should be checked by this technique.

The sophisticated non-linear analysis techniques would require expert knowledge and would be complex to apply to a large number of platforms. Thus, their application should be restricted to very less number of platforms.

The following approaches have been used to evaluate RSR at the four screening cycles. These are described in detail in the following sections of this chapter:

Screening Cycle-1	**:**	**Qualitative evaluation**
Screening Cycle-2	**:**	**Coarse Quantitative evaluation**
Screening Cycle-3	**:**	**Detailed Quantitative evaluation**
Screening Cycle-4	**:**	**Detailed Quantitative evaluation**

At screening cycle-1, a two-stage qualitative evaluation of RSR is made. The first stage is based on evaluation of the significance of the individual effect of the major factors on the structural integrity of a platform. At the second stage, the significance of cumulative effect of the major factors on the structural integrity of a platform is evaluated.

At the screening cycle-2, RSR is evaluated by a simplified quantitative analysis, which is based on the evaluations of strength of the sub-structures. A platform is appropriately cut into sub-structures or bays and the ultimate strengths are evaluated for each of them. A decision on the RSR of a platform is then made based on comparison of the bay strength pattern with the the global load pattern over the length of the platform.

At the screening cycle-3, RSR is evaluated by extrapolation of the results obtained by a conventional linear-elastic analysis. The conventional linear elastic analysis is based on the provision of factor of safety in the components. Strictly based on the working stress design (WSD), a platform is considered to have failed when the stress ratio in a member has exceeded 1.0. The stress interaction ratios obtained by linear-elastic analysis for the critical members, which are likely to form a mechanism, are considered and extrapolations are made to obtain the maximum load which the platform could carry before formation of a mechanism.

At the screening cycle-4, a detailed non-linear structural analysis is done to make a better estimate of RSR. A simplified non-linear analysis, known as static pushover analysis, is usually performed. This method is based on an incremental loading technique and determines the load levels and deflections at failure of the members in the formation of a mechanism. The post-collapse resistance and behavior of the members is explicitly considered in this method.

A more accurate approach at this cycle would be based on developing a time-history of the platform loads and displacements corresponding to the cyclic and random wave forces. This becomes very complex and time consuming, and thus would be limited to only a few platforms, where its use may be economically justified.

Details of these four approaches for the evaluations of RSR at the screening cycles 1 to 4 are presented in the Sections 3.3 to 3.6. The methods for the evaluations of RSR at the screening cycles 1 and 2 are covered in more detail. For the screening cycle-3, a technique for extrapolation of the linear-elastic analysis results is discussed. The various techniques developed by others for the evaluation of RSR at the screening cycle-4 are reviewed and discussed in Section 3.6.

3.3 Qualitative Evaluation of RSR

3.3.1 Introduction:

The qualitative evaluation of RSR is aimed at its use at screening cycle-1, where a very large number of platforms will need a periodic evaluation of their safety level. A qualitative (subjective) method should provide a pragmatic approach and optimally utilize the time and resources available with a regulator. An overview of the process followed at screening cycle-1 to classify the platforms as "FFP" (Fit for Purpose), "Marginal", and "UFP" (Unfit for Purpose), was presented in Section 1.4.1 and in Fig. 1.3 of this study.

The following basic assumptions have been made in the development of a method for the capacity assessment of a platform at this screening cycle:

1) The platform was well-designed in accordance with the design criteria of the period in which it was installed.

2) The platform was designed with the utilization ratio of near about 1.0 for one or more of its components, against the acceptable load level during its design period.

3) The difference in the expected and nominal yield strengths of steel was not considered at the design stage. Hence, in case of A36 steel, a reserve of approximately 1.25 would exist in material yield strength.

It means that a platform is assumed to be safe when significant changes have not occurred in the current physical, operational, and loading characteristics of the platform from its as-design and installed states. In general, in this approach the effect of improvements that may have occurred over time in fabrication procedures, load and strength prediction techniques, material technology, improved structural design, control measures for material and welding have been neglected at this qualitative assessment stage.

This screening cycle starts with the best possible characterization of the physical and operational parameters of a platform in its intact, damaged, or deteriorated state from design and installation documents and other means available. This step is considered to be the most important for making a reasonable qualitative assessment of the RSR at this cycle.

The details of a platform are established through the design documents, drawings, operation records, and the IMR (inspection, maintenance, and repair) records. A field visit and discussion with operating personnel about their opinion and any unusual response of platform may be required. It will be necessary to obtain the basic condition survey data (API Level-I survey) for the platform to confirm its present state. In addition, the details of the IMR program which an operator plans to implement on the platform should be available. The example data-sheets, which list the data needed to make safety assessments at screening cycles 1 to 4 are presented in Appendix-A.

For some platforms, sufficient documents may not be available and would warrant more effort at this cycle to establish the site parameters and platform details. In such a case, the work at screening cycle-1 will provide an effective pre-processor and will point at the deficiencies in the available data for the platform. For such platforms with their improved state of knowledge, a better estimate of RSR could be made at screening cycle-2.

The capacity evaluation at screening cycle-1 is done in two phases.

In Phase-A, the significance of the changes in the load and strength of a platform on its safety, due to the variation in a single major attribute is evaluated. The details of this phase are given in Section 3.3.2. In Phase-B, the significance of the cumulative changes in the load and strength on the safety of a platform, due to changes in more than one of the major attributes is evaluated, and is described in Section 3.3.3. The major attributes, which influence capacity (RSR) by varying the load and strength levels for the three parts of a platform, were identified and discussed in Section 3.2.2.

During the initial application of the method developed in this screening cycle, it is likely that more platforms may be screened out for the need of evaluation of their capacity at higher screening cycles. However, during subsequent applications (at the next periodic assessment cycle or in between), the assessment at screening cycle-1 may be sufficient for safety assessment of more number of platforms. The periodic assessment will normally follow the survey of a platform. In case the hurricane track passes near a platform, the platforms may need re-assessment of their safety following the post-hurricane condition survey.

The processes in the two phases of this screening cycle have been developed to provide a feasible basis for eventual preparation of a knowledge-based expert system. An expert system in this way would make it simpler and quicker to apply this method periodically on a very large number of platforms. The basic considerations and the steps needed in development of an expert system and its components are presented in Chapter 8 of this study.

3.3.2 Phase- A: Cut-Off Criteria

An overview of the process developed in phase-A of the screening cycle-1, which provides a cut-off criteria based on a qualitative assessment of the structural capacity of a platform was presented in Fig. 1.3 (Section 1.4.1). The process starts with an accurate characterization of the structural components and the loads acting on the three parts of the platform: deck, jacket, and pile foundation.

In phase-A, the emphasis is to establish the important conditions, which when met for a candidate platform would keep the increase in loads or load effects, or the decrease in strength of components and the platform within allowable limits. A platform could be classified to possess adequate structural capacity (RSR), when the following conditions are met.

Effect of loads and load effects on the platform: The lateral loads are not likely to increase significantly from those used in the design of a platform, when the following four conditions are met:

i) The current API-RP-2A guideline reference wave height and forces were used in original design of the platform;

ii) The bottom of steel elevation of the lower production equipment deck is 5 ft. (recommended airgap in API-RP-2A as an additional safety measure) above the crest elevation of the guideline wave;

iii) No significant modifications made to the platform;

iv) No adverse features in the environmental and geotechnical parameters are reported, since the installation of the platform.

Effect on strength of the components and platform: The strength of a platform is not likely to decrease significantly, when the following four conditions are met:

v) Good practices were followed in construction of the platform;

vi) The platform has been well operated, inspected, maintained, and repaired, as required;

vii) There have been no significant damage or deterioration of the primary components of the platform;

viii) The jacket was installed as planned and the foundation piles were driven to the design penetration.

These conditions provide a basis to determine the significance of variations in the parameters from the current API-guidelines, which influence the loads, load effects, and strength levels of a platform. Due to the distinct configurations of the three parts (deck, jacket, and pile foundation) of a platform, the above conditions should be verified for each of them.

Based on evaluation of these conditions in the platforms, logic diagrams have been developed as shown in Figs. 3.8 and 3.9. The platforms are screened in Fig. 3.8, on the basis of potential for a significant increase in their loads and load effects. In Fig. 3.9, the platforms are screened on the basis of the potential for a significant decrease in the strength level of its components.

In case there is a likelihood of a significant increase in the load level (load effect) or a significant decrease in the strength level of a platform by any single condition, the platform exits the first screening cycle. In this way, the platforms which are obviously critical are screened out for a further quantitative evaluation. Otherwise screening of the platforms at phase-A moves ahead with evaluation of the other major factors. The platforms, which are not likely to have a significant increase in the load or a significant decrease in the strength due to a single major condition, are evaluated in detail by the qualitative method in phase-B of the screening cycle-1.

The Figs. 3.8 and 3.9 have been developed to evaluate a steel jacket platform primarily for the seven conditions listed above. **In this study, it has been assumed that the significant variations in the capacity (RSR) of a platform would occur, when the loads acting on the platform are expected to increase by more than 25 % and the strength level of its component(s) are expected to decrease by more than 20 %.** The eight conditions listed earlier are further discussed below.

i) <u>Recommended RP-2A wave height $\leq$ the "as-design" wave height</u>

The on-bottom load (base shear) is likely to be lower than the "as-design" loads, when the reference level wave height is lower. The global

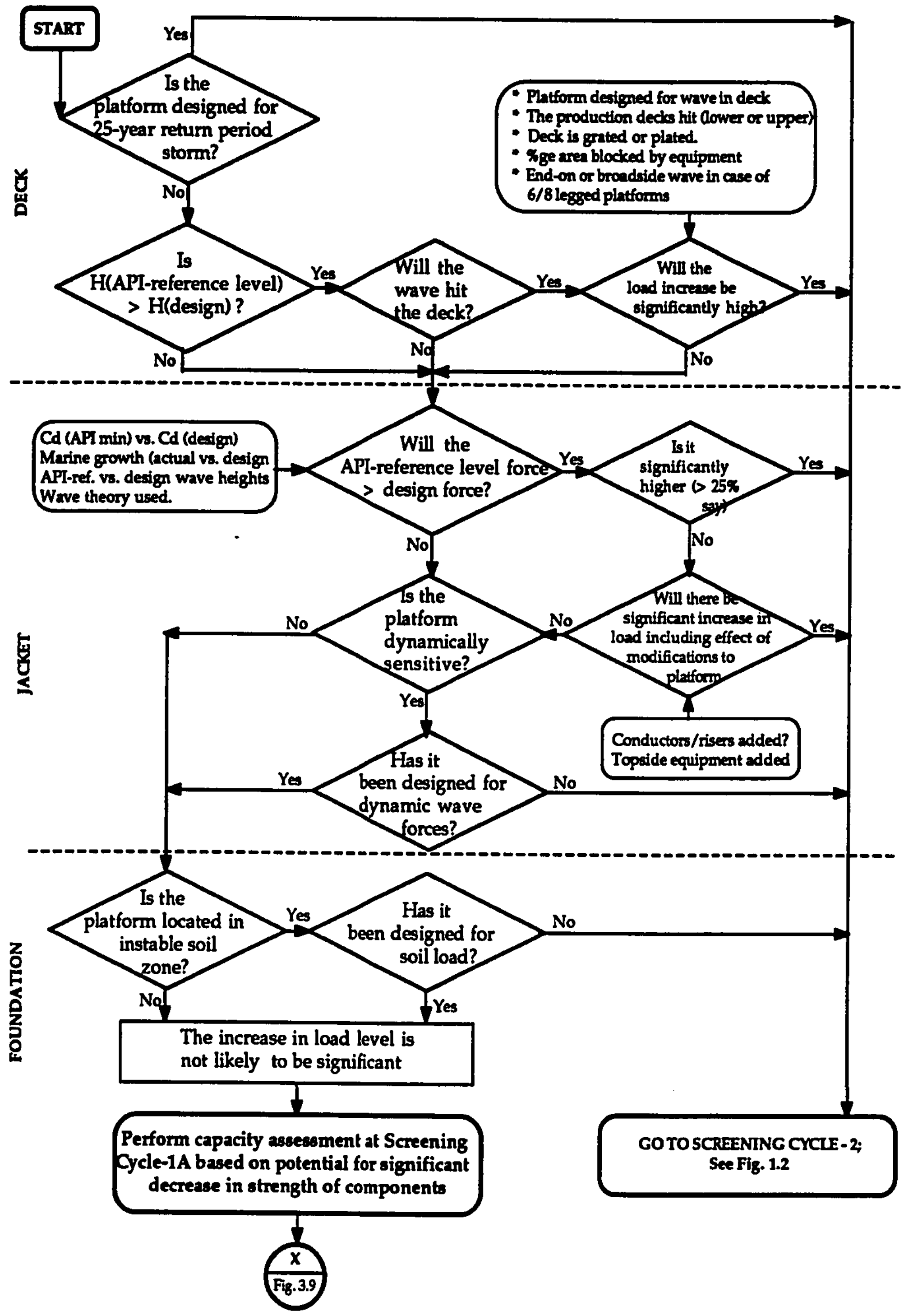


Figure 3.8: Platform Capacity Assessment at Screening Cycle - 1A Based on Potential for Significant Increase in Design Load Level

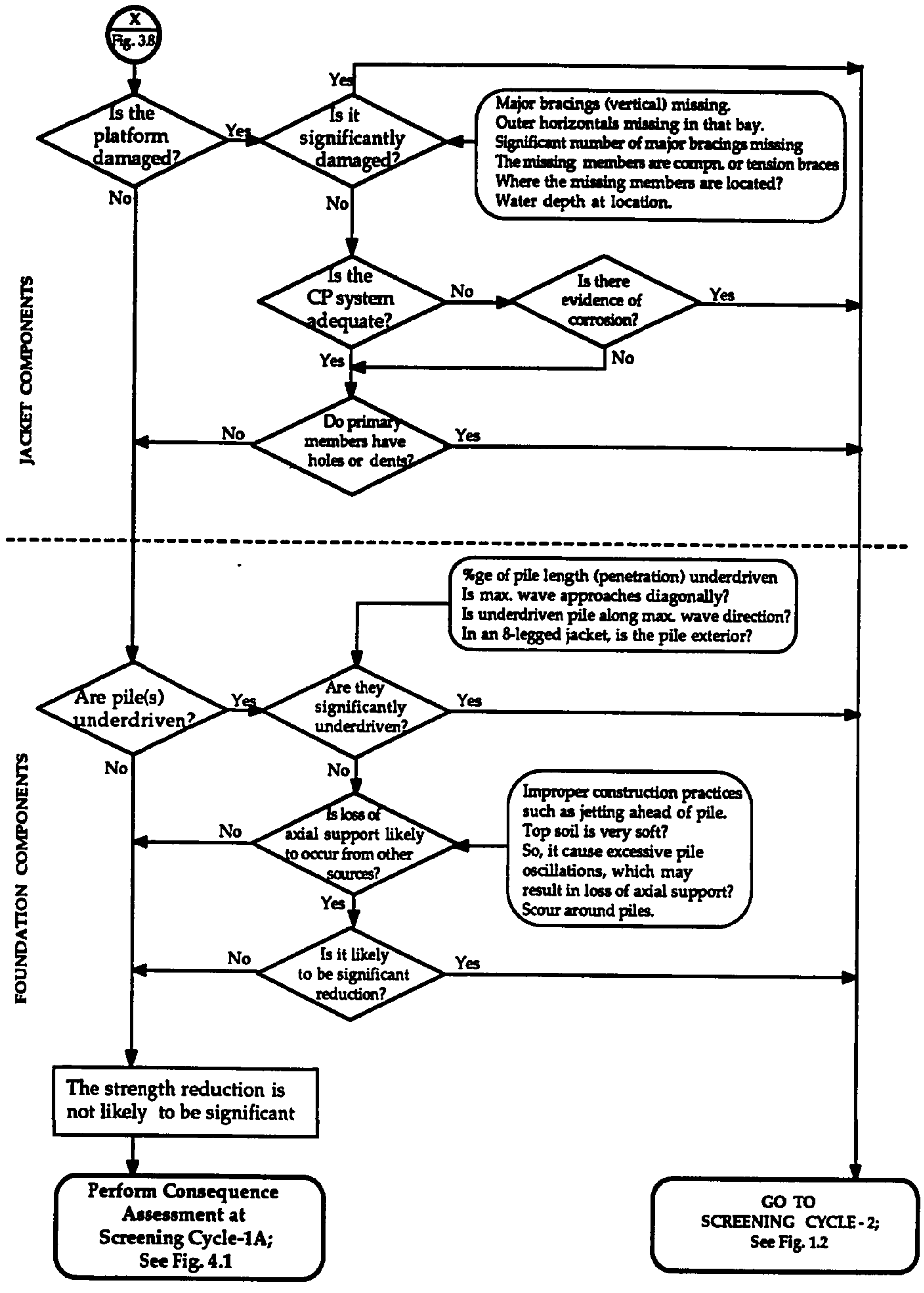


Figure 3.9: Platform Capacity Assessment at Screening Cycle - 1A Based on Potential for Significant Decrease in Strength Level of Components

loads on a platform are significantly influenced by the variations in the number of conductors, flowlines, pipeline risers, casings, caissons, and boat landings; or in the thickness of marine growth from that considered at the design stage. Their verification may need field visits and an underwater survey of the platform.

ii) Lower production equipment deck is above the wave crest elevation

In case the wave hits the deck of a platform, which was not designed for wave-in-deck condition, it is a major concern for its and equipment safety. The consequences will be more severe, when the production deck is hit by the waves. The magnitude of wave load on the deck would depend upon the percentage of the deck area blocked, and the type of deck plating. The type of deck plating also determines the magnitude of slam load. In case of grated deck, the vertical slam load would be minimal.

iii) No significant modifications made to the jacket: Normally the modifications to the platform are made on the deck to add new equipment and facilities when required due to change in production characteristics of the reservoir. The jacket modifications includes addition of conductors, risers, casings, and caissons. In some cases, additional boat landing may be added. The other modifications to the platform may be unintentional and during its installation. In some cases, the piles may need modification, e.g., in case of significant underdrive an insert pile may be used or grouting of pile with bell footing at the tip may be considered.

iv) No adverse features in the environmental and geotechnical parameters were reported at the platform location: One adverse feature, wave induced seabed movement, has been noted in some areas of the Gulf of Mexico. From a review and evaluation of the soil bore-logs, a decision on the succeptibility of the site to soil failure can be made. If the platform had not been designed for such loads, then the increase in lateral load is likely to be significant.

v) Good construction practices were followed: A review of the records for a platform will assist in determination of any unusual conditions during construction, installation, piling, hook-up and commissioning, and drilling phases of the platform. Good construction practice measures include selection of through-section property steel (Z-property steel) for the primary nodes such as leg-brace, providing cans or stubs at selected joint, and reducing stress concentration factor at nodes by providing simple or grouted joints, or profile grinding of welds.

vi) Platform has been well operated, inspected, maintained, and repaired: Periodic inspection, maintenance, and repair in accordance with the current API are required for a platform over its service life. A qualitative assessment of the importance of the damaged, deteriorated, or missing members is needed. If the primary members are damaged, then the capacity level of the platform will be directly affected.

vii) There are no significant damages and deterioration of the components of a platform: The damage and deterioration of the major components of a platform may reduce their strength significantly, and are likely to reduce the ultimate strength and RSR of the platform. The capacity of a platform with little or no redundancy, is likely to have a more significant influence of the damages of the components.

viii) The jacket was installed as-planned and the foundation piles were driven to the design penetration: If the jacket was installed at a different location, or in different water depth, then loads acting and its capacity would be influenced. If the piles were underdriven, then their axial ultimate strength may reduce. The lateral capacity of the underdriven piles may significantly reduce when the pile thickness at the mudline and below (where the bending moment is maximum) is lower than that provided based on design . If improper construction practices, such as jetting ahead of pile were used during installation, the axial support from the soil would reduce.

3.3.3 Phase-B: Qualitative Evaluation

In phase-B of screening cycle-1, the platforms whose capacity level is not likely to significantly reduce due to the variations in the eight conditions listed in phase-A on an individual basis, are evaluated. The effects of the individual conditions may be "moderate" or "low," but the combined effect of the variations in all of these conditions may have a significantly negative influence on the capacity level. In phase-B, the cumulative effect of the variations in the parameters associated with the eight conditions for assessment of the variation in the capacity level is evaluated.

A qualitative method is presented, which aims at assessment of the variation in the capacity (RSR) for each bay of the platform. In this approach, the changes in the capacity, which are likely to occur due to the differences in various physical, operational, and environmental parameters for the platform, are evaluated. This method basically follows the characterization of RSR, which is the ratio of the "ultimate strength of a platform" to the "API reference level lateral load." The effect of the variations in the major attributes, which affect the ultimate strength of a bay are designated by the factors, "R_i." These factors represent the likely change in the ultimate strength due to the variation in the magnitudes of the major attributes for the candidate platform, when compared with their values at the design stage.
In the same way, the effects of the attributes which influence the reference level lateral load are designated by the factors, "S_j." The likely variation in

the capacity (RSR) of a platform, when evaluated for the current reference level load parameters, is then obtained by multiplication and division of these factors, as expressed below:

$$RSR = [R_1 \times R_2 \times R_3 \times R_4 \times R_5] \;/\; [S_1 \times S_2 \times S_3] \qquad \text{....(3.14)}$$

where: R_1 = Material factor;

R_2 = Platform Condition factor;

R_3 = Platform Modifications factor;

R_4 = Structural Configuration factor;

R_5 = As-installed Stage factor;

S_1 = Design Criteria Variation factor;

S_2 = Deck Elevation factor;

S_3 = Platform Modifications factor.

These factors are further discussed in detail in this section.

Ultimate Strength: In this phase of evaluation an estimate of the likelihood of reduction in the lower bound of the ultimate strength of a platform is made. The ultimate strength of a platform depends upon the material properties, as-is condition and properties of its components (members, joints), and configuration of the structure. The variation in lower bound strength of a platform is directly proportional to the variations in the component members. Hence, the current state of its components is compared with their state considered in the original design, and the influence of variations on the strength level is determined. The five factors considered in equation 3.14 to represent such changes are described further in this section.

Material Factor, R_1: Due to the difference in the nominal and expected values of yield stress a reserve factor of 1.25 exists. For A36 steel the nominal value used in design is 36 ksi, whereas the expected ultimate value of this steel is approximately 45 ksi [*Moses, 1989*]. Hence, in all of the platforms based on conventional design, a material reserve factor of 1.25 would exist (R_1 = 1.25 for A 36 steel).

Platform Condition Factor, R_2: The strength of a platform is likely to reduce with age, when necessary measures have not been taken to maintain its as-designed strength level. The damage of the platform bay(s), which can occur from multiple sources, will have a direct influence on reduction in strength of the bay. If one out of 4 bracings damaged or missing, then R2 = 0.75. This factor will always be less than 1.0, due to age related effects on the platform.

Platform Modifications Factor, R_3: The structural modifications to a platform, which would vary its ultimate strength are as follows: addition of knee braces to the deck legs, grouting of the leg-pile annulus, grouting of the braces, improvement (profiling) of welds, providing insert piles, or grouting of the piles, etc. These measures are costly and therefore are applied in case a platform was damaged during its installation, or a primary member was damaged during the operational phase, or when upgrading of a platform is needed for its extended service.

The ultimate strength of a platform will usually increase due to its structural upgrading ($R_3 \geq 1.0$). In many cases, the reports indicate that desired improvement in strength did not occur due to lower reliability associated with the underwater repairs.

Structural Configuration Factor, R_4: The platforms are designed so that the appropriate combinations of the maximum stresses in the members and the nominal allowable stresses remain within safe limits. In general, the stress interaction formulas presented in the API guidelines are used. The stress interaction ratio obtained in this way is sometimes called as utilization ratio.

In an optimal design, a designer would aim to size the components such that the utilization ratio for a large number of components is close to 1.0 for optimal use of steel. Hence, a rough approximation may be made at the qualitative stage of evaluation that all the primary structural members in a bay have an interaction ratio (or utilization ratio) nearly 1.0, and upon failure of the first member the other primary load carrying members in the bay are likely to fail leading to formation of a collapse mechanism.

In addition to this a more accurate evaluation of R_4 would depend upon the vertical bracing pattern, adequate horizontal bracings, an appropriate joint design, and the compactness of leg sections. For example, a platform (bay) with X-brace pattern for the vertical framings would have higher strength than a platform (bay) with K- or diagonal-brace patterns.

If the deck legs are of non-compact size ($D/t > 60$), then R_4 is likely to be lower than 1.0. If the vertical bracings for a jacket bay is of X-pattern, then R_4 would be more than 1.0 for that bay.

As-installed Stage Factor, R_5: A number of differences may exist in the as-installed platform from its as-design state. These differences may occur during its fabrication, load out, transportation, and installation phases due to difficulties in construction, human errors, and other limitations. Such differences, in general will tend to reduce the strength of a platform.

For example, the piles may not be driven to the design penetration (tip elevation), which will have a direct influence on the ultimate compression and tensile capacity of the piles against the orthogonal and diagonal wave load conditions. In addition, the pile section at the mudline may be of lower thickness than that required by the design, and it will have a direct influence on reduction of the ultimate capacity of the foundation bay. In such cases, R_5 would be lower than 1.0.

API Reference Level Load: In this phase, the likelihood of increase in the lateral loads on the platform is assessed. Such an assessment is based on comparison of the loads used in original design of the platform,with the lateral loads based on the reference level load criteria given in the latest API-RP-2A guidelines. The various parameters, which determine the load level on a platform and were considered in the original design of the platform, are compared with the latest API-guideline parameters. The three factors, which forms the basis to evaluate such changes in equation 3.14 are discussed further.

Design Criteria Variation, S_1: The various parameters and methods, which are important in evaluation of the loads were discussed in detail in Section 2.3. In that section, it was demonstrated that variation in the wave height has the maximum effect on the wave loads, because the base shear and overturning moment would change in proportion to square (or higher) of the wave height. Therefore, an approximate estimate of S1 could be made by following expression:

$$S_1 = [(H_{\text{API-Ref}}) / (H_{\text{original design}})]^{\alpha}$$

where, α = 2.0 for drag dominated platforms

= 2.2 for platform with boat landings

= 2.5 for wave in deck condition

The elevation of the wave crest is also important in making a decision on the likelihood of an increase in the wave loads. For example, a platform was designed with 5 ft airgap, and the current wave criteria changed for the platform with change in wave height by 8 ft. i.e., approximately 5 ft. increase in the crest elevation. In this situation the crest elevation is just below the bottom of steel of the cellar deck girders. In case the original design wave height was 60 ft., the new wave height would be 68 ft. The value of S_1 would be 1.28 for α of 2.0.

In case the wave hits the deck, the load would significantly increase, and the magnitude will depend upon the wave crest elevation, the type of deck plating and the deck equipment.

In case of increase in the reference level wave height over that considered in the design, S_1 would normally be higher than 1.0. However, in case the reference level wave height was used at the design stage, with a combination of other parameters which produced higher wave loads, then S_1 would be lower than 1.0.

Deck Elevation, S_2: API-RP-2A recommends that the elevation of bottom of steel of the lower deck should be at least 5 ft. higher than the wave crest elevation. As demonstrated under S1, in the event the wave crest approach an elevation within or above this 5 ft. zone, the wave loads would significantly increase.

In case the original design provided 5 ft. airgap and the current reference level wave hits the deck, which was not designed for this condition, the wave load would increase by more than 28 % for original design wave height of 60 ft.,

Platform Modifications, S_3: The modifications, which may have occurred to a platform since its installation, are important to evaluate for their influence on variation in the loads. The lateral load on a platform would increase due to addition of risers, conductors, caissons, casings, boat landings, and due to formation of marine growth thicker than that considered in the design stage. The other sources of increase could be due to installation of platform in higher water depth or increased water depth due to settlement of jacket. The lateral load may reduce due to removal of marine growth, appurtenances, conductors, and risers.

The change in the vertical loads would be directly proportional to the changes in the topside facilities. Due to modifications to the jacket and flooding of damaged members, if any the vertical load would increase.

An attempt has been made to evaluate the differences in the loads and strength of a platform based on the above conditions through schematic representation given in Figs. 3.8 and 3.9. In case these parameters have not changed from those considered in the design stage, the load and strength of the platform would remain same as were evaluated at the design stage. In such case, the RSR is likely to be approximately 1.25 due to the difference between mean and nominal values for A36 steel.

In cases, significant differences exist from the as-installed stage of a platform, a more detailed evaluation will be needed to assess the effect on the RSR of the platform or a particular bay. At the screening cycle-1, the effect of redundancy of platform bays can be neglected, by assuming that the load level at the first member failure represents the lower bound of ultimate strength of the bay.

Normally, a platform whose parameters meet the current API-guidelines and whose strength does not reduce due to the other load effects such as fatigue or collision, its overload capacity would be approximately proportional to the reserve in the mean and nominal values of material yield strength which would be approximately equal to 1.25.

Note that the API-guidelines were primarily developed for use for the platforms in the medium water depth and with the medium consequence. Such platforms are reported to have RSR in the range of 1.5 to 2.5 [*Titus and Banon, 1988*]. A platform with a RSR of 1.25, based on assessment of the major attributes, can be classified as a platform with medium to high capacity level. In this study, RSR of 1.25 has been assumed to represent the boundary of medium and high capacity levels.

At this screening cycle, a qualitative classification of capacity (RSR) has been made. RSR based on this method would be classified as: very low, low, medium, high, and very high. In order to make a consistent decision on the capacity and safety levels of the platforms, the qualitative classifications have been considered to relate to the variations in the RSR based on phase-B qualitative evaluation based on evaluation of the the differences in the parameters of the original design criteria and as-is API criteria.

The following ranges of variations in RSR obtained as per equation 3.14 have been assumed for this study for the classification of capacity:

Qualitative Capacity Level	Assumed RSR range
Very Low	< 0.80
Low	0.80 - 1.00
Medium	1.00 - 1.25
High	1.25 - 1.50
Very High	> 1.50

These values have been assumed as a criteria for this study only and are also based on expert opinion, and the heuristic knowledge of the safety of platforms. These values may change with a different interpretation by a regulator, an operator, or a user. Upon implementation of this methodology on a large number of platforms and verification of RSR at the higher screening cycle, one may arrive at improved classification of the ranges for the qualitative classification of capacity (RSR).

3.4 Coarse Quantitative Evaluation

A coarse quantitative (simplified) method has been developed to determine the structural capacity index, Reserve Strength Ratio (RSR), for a platform at screening cycle-2. At this screening cycle, the objective is to make a quantitative estimate of the suitability for service of the platforms which are intact, or have suffered only minor damages, or are in upgraded state, or were classified as "Marginal" or "UFP" in the first screening cycle. Such a technique has been developed by utilizing the knowledge gained from a detailed review of the failure modes and failure mechanisms observed [*refer Section 1.3*] for the steel jacket platforms, and the results published for their ultimate capacity and system reliability evaluations.

The goal is to make an estimate of RSR based on simple hand computations without explicit need for the use of computers. Therefore, in such a method, the more complex aspects discussed in Section 3.2.1 for the load-displacement behavior of the platforms should not be incorporated. This method is limited to the determination of the "lower bound" capacity, which corresponds to the component strength as described in equation 3.4 (Section 3.1). In the evaluation of lower bound RSR, the "weak link(s)" in the structure is (are) identified and an approximation of the nominal RSR is made by extrapolation or interpolation of the results.

Based on the continued interest in application of this technique on a large number of platforms, effort may be initiated by others to develop user-interactive software to further reduce time in its implementation. This method is very useful, because it aims at a cost-effective technique for periodic screening of a very large number of platforms.

The capacity of the reduced number of platforms, which have been identified with questionable safety level at the second screening cycle, is then evaluated by more accurate computer based evaluations at the higher screening cycles. In addition, upon identification of the "weak links" or the "weak zones" in a platform by this simplified method, the inspection efforts can be focussed on accurate characterization of the structure in such zones to improve the accuracy in the capacity evaluation.

Beyond the lower bound strength level the system effects becomes important and the load-displacement behavior becomes non-linear, which were discussed in detail in Sections 3.1 and 3.2. The characteristics of a platform, which are likely to introduce errors in estimation of strength are: unequal loading in the different parallel frames; difference in loading with elevation of the members; effect of the relative stiffness of members in the load distribution among the members. Such effects can not be considered very accurately by the methods based on simplified hand computations.
The lower bound RSR results can be improved by introduction of error (or bias) factors to account for the variations due to such effects for a kind of platform in a water depth range. In this way, an improved estimate of the lower bound RSR can be obtained upon identification of the errors introduced in the simple method and consideration of the factors which modify the strength estimate.

A logic diagram for the capacity evaluation at screening cycle-2 is shown in Fig. 3.10. The following steps are followed to develop the lower bound of capacity (RSR):

1) Platform characterization: The process starts with an accurate characterization of the platform. The size, condition, and configuration of all the structural member of the platform are obtained and their differences from those at the design (as-installed) stage are noted. The non-structural members and marine growth, which are important for the determination of loads are

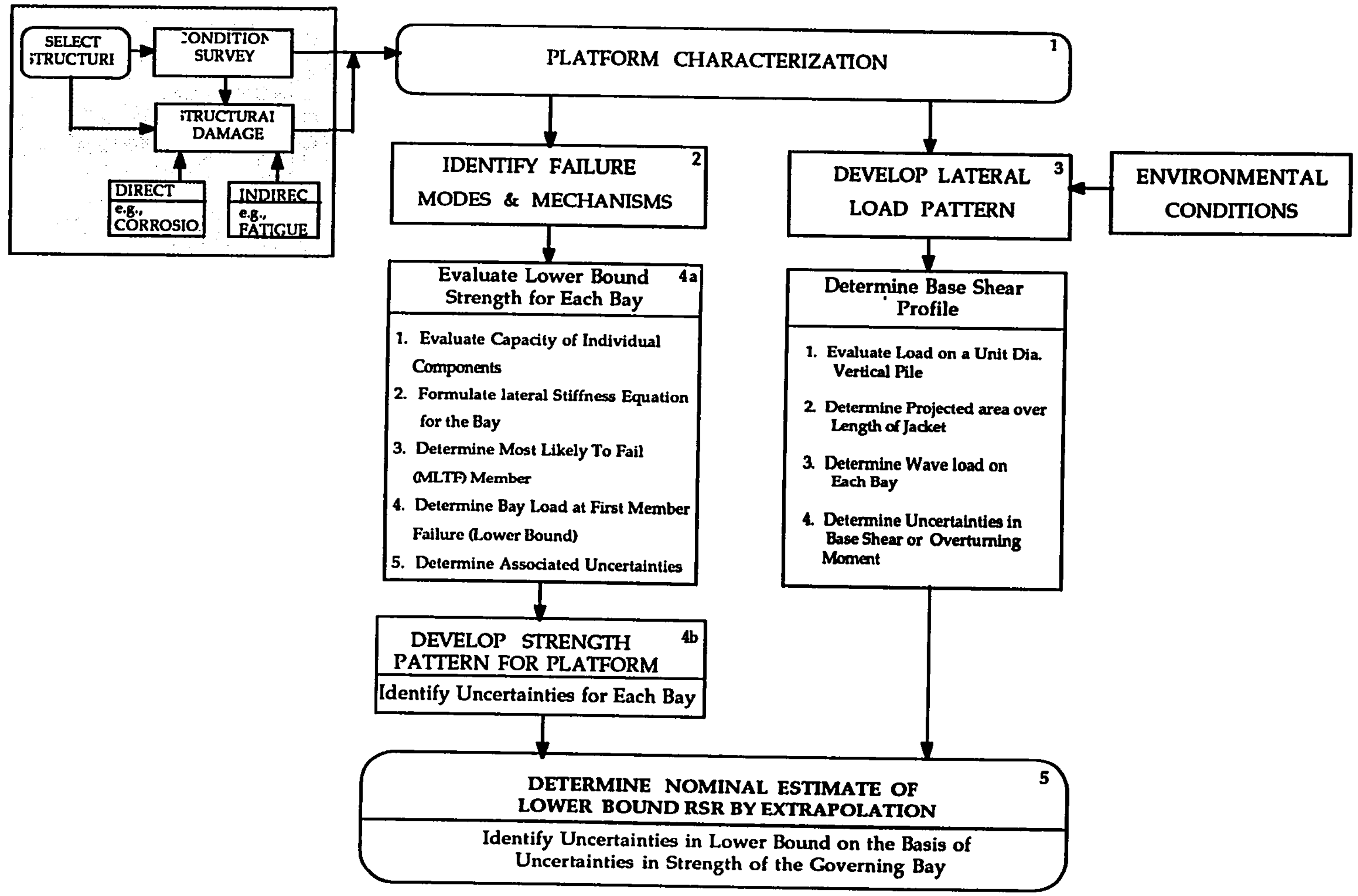


Figure 3.10: Development of Strength Pattern at Screening Cycle-2

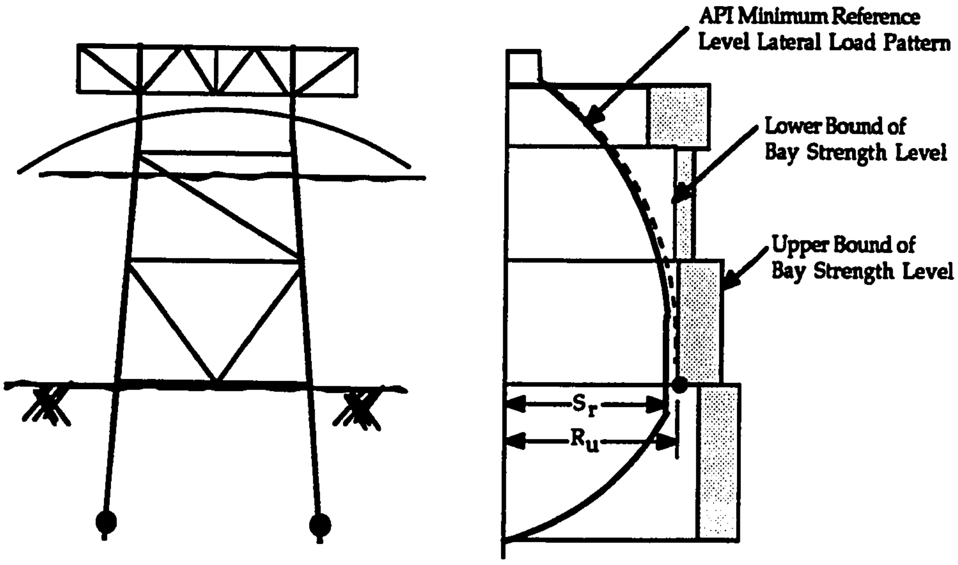
also accounted. The soil parameters, which are important to determine its strength to transfer the pile loads and the possibility of soil liquefaction are established from the records.

A condition survey of the platform or other means identified in Section 2.2 may be required to accurately characterize the above-mentioned parameters. Such an effort may be more effective by surveying in detail the weak links or weak zones identified in the first screening cycle.

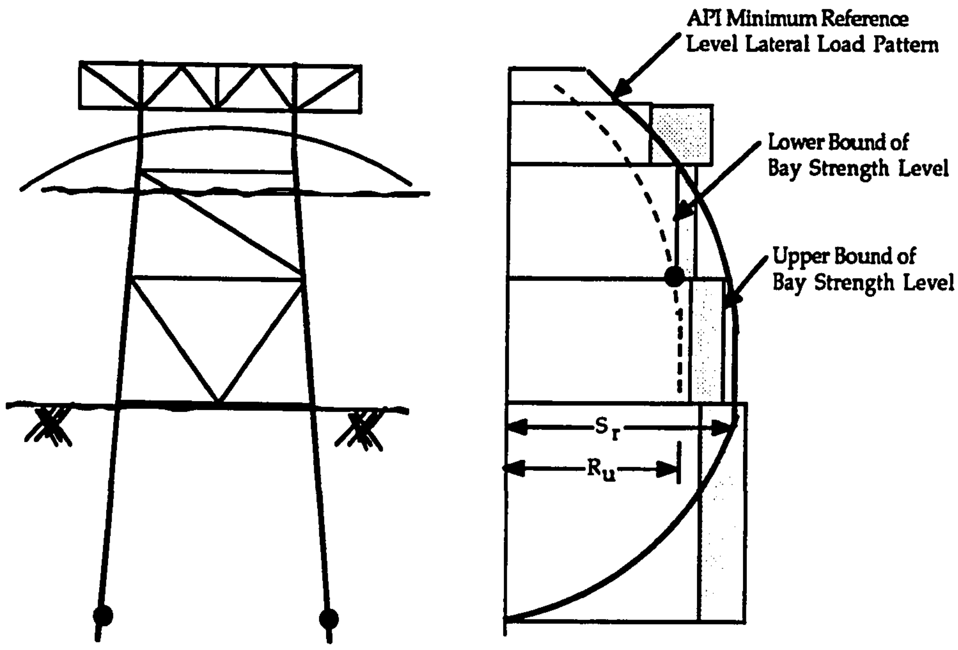
2) Identify failure modes and mechanisms: The failure modes and mechanisms for the three parts of the platform are identified, utilizing the description given in Section 1.3. In their identification, the evaluations made at the first screening cycle would be very useful.

3) Development of load pattern: In Section 2.3, it was mentioned that the API-reference level lateral load for the platform would be obtained, when they are evaluated by the API-reference level wave height and the other wave force parameters, wind loads, and storm tide specified in API. Note that, if current is included with these conditions, the resulting wave load will exceed the API-reference level force. The reference level lateral load is represented by S_r in Fig. 3.11. The various techniques which could be used for the development of load pattern are described in Section 3.4.1.

4) Development of strength pattern: The lower bound strength pattern, which corresponds to the global load level (base shear or overturning moment) at failure of the first component is desired. In case of the deck and foundation bays, the upper bound strength can be easily determined, whereas for the jacket bays it becomes complex. The details of the method developed for development of the strength pattern are given in Section 3.4.2.



Case - A: Reference Level Base Shear Pattern < Ultimate Strength Pattern



Case - B: Reference Level Base Shear Pattern > Ultimate Strength Pattern

Figure 3.11: Coarse Quantitative Evaluation of RSR at Screening Cycle-2

5) Determination of the lower bound capacity (RSR): The capacity index (RSR) is obtained by comparison of the load and strength patterns, and by manipulation of the results to obtain the load (strength) level at failure of the most-likely-to-fail member. Fig. 3.11 demonstrate the two cases, which may occur in the determination of RSR by using the load and strength patterns over the jacket length.

RSR as defined in Section 3.1 is based on the capacity (RSR) level of the entire platform and its estimate will require development of the load-displacement diagram for the complete platform. The complexities in the establishment of the load-displacement diagram would reduce by dividing the platform into a number of sub-structures or bays. Then the ultimate strength (lower bound) could be obtained for each of the bay and a stepped strength pattern for the complete platform is developed. More details are given in the further sub-sections.

The strength (component or ultimate) of a platform when represented by a global lateral load can be considered as the base shear at which the "reference level lateral load" profile, when scaled up or down, matches the "bay strength profile (lower or upper bound)" at any elevation. The corresponding base shear value will thus represent R_u, which is a measure of the component strength or the ultimate strength. The component strength thus obtained represents the lower bound strength of the platform and the ultimate strength at the formation of a failure mechanism would represent the upper bound strength. The upper bound strength of a bay will include contributions of both the component strength and redundancy factor for the bay. The upper bound RSR as described by the equations 3.2 to 3.6 (Section 3.1.1) includes the redundancy effect (RF_1) of the platform beyond (CS_1).

In this way, an estimate of the upper or lower bound of RSR is made for the overall platform. The difference between upper and lower bound depends upon the material properties, geometry, and loading characteristics for the platform. Beyond the failure of the first component in the jacket bay(s) complexities are introduced, which can not be correctly incorporated by the simplified methods. The failure of the members required to form a failure mechanism (called as cut-set in system reliability terminology) may not necessarily happen due to failure of all the members in a single bay, and may involve the components in more than one bay. Such a sequence of failure of members in several bays cannot be easily accounted by this simplified approach. The upper bound estimates for deck and pile bays, which correspond to the rigid-plastic large deformation analyses presented in Section 3.2.1, can be correctly determined by this approach.

The lower bound estimate of RSR, based on the component strength of each bay, can be determined more precisely by this method. However, caution is needed in its evaluation for a platform with unsymmetrical configuration and/or unsymmetrical loads. In such cases, biases (correction factors) can be introduced to improve the results.

The following assumptions have been made in the development of this simplified approach:

1. **In the formation of a failure mechanism, it is assumed that all of the components which form the mechanism in a bay would fail together.**
2. **The soil-structure interaction is not considered.**
3. **The non-linearities due to the material and the geometric properties are idealized on the conservative side.**

3.4.1 Development of Load Pattern:

In this section, simplified approaches are presented for development of the lateral load pattern needed to estimate RSR at the second screening cycle. The lateral load patterns in the two orthogonal directions and a diagonal direction are required to evaluate RSR against the different failure modes in the deck, jacket, and foundation bays.

The four-step process for determination of the wave loads on the members and a platform, and the uncertainties in the results were described in detail in Section 2.3. The following approaches utilize the details given in Section 2.3. The three options are described below:

1) Option A : In case, the global lateral loads (base shear and overturning moment) for the platform are available from the design documents or are supplied by an operator, an assessment is required to establish that they are similar to the reference level loads obtained by the latest API-load parameters.

It could be done by comparison of the load parameters which were considered at the design stage with the latest API-reference level parameters. In case the parameters differ between the two criteria, the values of the base shear and overturning moment for the design parameters will have to be updated.

The differences are likely to arise primarily due to the parameters: wave height (H), water depth, current, deck elevation, wave crest elevation, marine growth, and drag coefficient (C_d). Among these parameters the variations in the drag coefficient and member diameter (due to marine growth) have a linear influence on the global wave loads. The wave height has the maximum influence on the global wave loads. The base shear could be updated by the following formulation, based on equation 2.15, for the differences in the wave height criteria:

$$(\text{Base Shear})_{ref} = (\text{Base Shear})_{design} \; (H_{ref}/ H_{design})^{\alpha}$$

where for drag dominated steel jacket platforms, the value of α is 2.0 for the platforms without the boat landings, 2.2 for the platforms with boat landings, or 2.5 for the platforms with wave into the deck *[Lloyd, 1983]*.

In the same way, the overturning moment at base could be updated by the following formulation:

$$(\text{Base Moment})_{ref} = (\text{Base Moment})_{design} \; (H_{ref}/ H_{design})^{\alpha} \; (d_{ref}/ d_{design})$$

2) **Option-B:** By using the wave kinematics and wave force profiles developed based on the Airy's linear wave theory, for a 1-ft diameter pile positioned at the center of structure. The load level on the complete jacket is approximately evaluated by use of the profile for the unit wave force. Note that in this case, all of the members in a jacket are lumped together and are assumed to be located at the center of the jacket for load evaluation purpose, and the spatial variation of wave kinematics is neglected.

It can be developed by hand computations or by use of simple wave load computer software. More details of this approach are given in the following sub-section.

3) **Option-C:** By using the available software for evaluation of the wave loads and load effects on the individual members. The software packages which are commonly used in the design of offshore platforms can be utilized for development of wave load profile. Some of the software commonly used are identified here as: SACS, Strucad, SESAM80, and ASAS.

The conventional procedure used at the design stage for evaluation of the loads would be followed in this option. Therefore, the details are not reproduced in this study. Note that by following this option, the bias and variance in the wave loads and their parameter would remain the same as for the conventional design, because it would give more precise estimates of the loads and load effects.

3.4.1.1 Option-B: Wave Loads on the Platform

In this section, a simplified approach for determination of the wave loads on a steel jacket platform is given. This approach utilizes, the wave kinematics and wave force profiles developed based on the Airy's linear wave theory on a unit-diameter vertical pile positioned at the center of the platform. The unit wave load profiles are developed when the wave crest hits the pile. The global wave loads of interest are shear force and overturning moment at the base of jacket. In addition, lateral shear values at each horizontal framing elevation, and the lateral shear and overturning moment at top of the jacket are desired.

The wave loads on the complete jacket are approximately determined by using the "unit wave force" profile. In the simplest case, all the jacket members (legs, braces, conductors, risers, and appurtenances) can be considered lumped together at the center of the jacket and the appropriate "equivalent diameters" could be determined between the different elevations. Then, the total wave load on the jacket would be the summation of the multiples of the unit wave loads and equivalent diameters for the different elevations. Alternately, the loads in the different members in a bay are evaluated and added. By this approach, the wave loads would be biased due to neglecting of: the spatial variation of the wave kinematics; contribution of the inertia loads; wave load effect on some of the inclined members.

It can be developed by hand computations or by use of simple wave load computer software. In this study, the wave loads evaluation by hand computations are given.

The diameters of the jacket components up to the mean sea level (or below the splash zone) are increased to account for the marine growth.

The contribution of the large diameter jacket legs and well conductors is the maximum on the total lateral loads. The vertical braces in the orthogonal frames, risers, casings, caissons, boat landings, and other appurtenances have lesser contribution to the the wave loads. The horizontal braces which are perpendicular to the wave direction and located in the wave zone will also contribute to the loads.

The simple procedure would be to cut the jacket at different elevations, where the shear and overturning moment values are desired. The ideal way to cut the jacket into substructures is to consider different bays, as has been done for the capacity evaluation. The wave loads acting on the members in a bay are evaluated by using the unit wave force profile on a vertical pile.

Such a wave load pattern is generated for the two orthogonal wave approach directions. An approximate estimate of the wave loads for the diagonal wave approach is made by adding the components of the orthogonal wave loads along the diagonal direction.

The design or sizing of most of the components of the jacket bays are governed by the waves along the orthogonal directions of a platform. In the jacket bays, the braces and joints are normally the most-likely-to-fail first and they are most stressed by the orthogonal waves. The waves approaching diagonally will govern the stresses in the most-likely-to-yield first sections in the deck and foundation bays, because the lateral loads from diagonal directions will impose maximum compressive or tensile stresses in one leg or a pile or a joint. Besides first yielding of a leg or a pile section (the ductile failure modes), the diagonal wave approach direction will be critical for the deck leg-pile connections, the jacket leg-pile connection failure, and failure due to axial pullout of piles (the brittle failure modes).

Simplified Evaluation of Wave Loads on a Jacket Platform:

The four-step approach used in the evaluation of loads on a jacket platform was described in detail in Section 2.3.2. In this section a simplified approach is described, which utilizes the closed form solution given in equation 2.10 for the wave loads on a unit diameter pile by the Linear wave theory. The step-by-step approach is given below:

1. Selection of the Parameters: Select the reference level wave height (H), storm tide (S), and the wind speed from the latest API guidelines. A wave steepness of 1/12 is selected, as it is representative of the hurricane conditions. Determine the associated time period of waves. Consider the wave crest elevation at (0.6 H + S) above the mean sea level.

Check, whether the wave hits the deck, or the air-gap is lesser than 5 ft. as recommended by API guidelines.

2. Wave Kinematics: Evaluate the horizontal water particle velocity, u(t), at the wave crest and the other elevations, by using the Airy's Linear wave theory.

3. Wave Force Pattern: Evaluate the wave force pattern for a unit diameter vertical pile by considering the drag coefficient, C_D as 0.6, and neglecting the inertia load contribution. Closed form solutions are obtained for the base shear and overturning moment by the Airy's theory, as given in equations 2.13 and 2.14 in Section 2.3.

The wave force magnitudes for each bay are required to develop the overall wave load pattern. The wave force profiles for the complete jacket are evaluated by using the unit wave force profile developed for a vertical pile.

It can be developed in several ways. One way can be to move from top to bottom and compute the wave loads at the different elevations and at the base by addition of the loads on the different members and the appurtenances in the bay. The other way will be to evaluate the base shear and then approximating the wave force profile as triangular in the wave zone. The first approach will provide more accurate results.

4. Uncertainties: The uncertainties (bias and variance) in the estimates of base shear and overturning moment by this simplified approach are identified and a quantitative estimate is made. The estimates of uncertainties are obtained by comparison of the results for an 8-legged platform by this simplified approach with the results obtained by the computerized analysis for the Dean's Stream Function theory. Note that for other platforms, the bias values will differ due to variations in their physical characteristics and water depth from those for the 8-legged platform considered in these estimates. Therefore, these bias estimates should be used with caution. The variance depends upon the parameters used in the process, and will remain constant irrespective of the configuration of platforms.

The major contributions to the bias are from the basic assumptions made in evaluation of the loads. Some of these are listed below:

a) The wave load model considered here assumes that all of the members are lumped together and located at the center of the platform (jacket). The spatial variations in the forces on the individual legs and other members are neglected. Therefore, this model gives conservative estimate of the drag load.

b) The contribution of the inertia load, though insignificant, is completely neglected. The inertia load becomes important for the larger diameter legs.

c) The unit wave forces in the horizontal direction are considered in evaluation of base shear. The other parameters used in evaluations of wave loads are for the condition with the wave acting perpendicular to the member. Due to this, bias will exist and conservative results will be obtained.

d) Strictly speaking, Airy's wave theory is based on infinitesimal wave height ($H \approx 0$), which results in significant differences. When stretched to the crest level, which for deepwater could be taken approximately equal to 0.6H and for shallow water approximately equal to H, the bias in wave loads would reduce.

e.g.,

1. Wave load profile on the deck-bay will be as follows:

$$F_1 [(N_{leg} D_{leg}) + (N_{cond.} D_{cond.}) + (N_{riser} D_{riser})]$$

where, F_1 is the total wave force on a unit diameter vertical member from under the bottom deck to the elevation at top of first horizontal framing, i.e., within the deck bay elevations; D is the diameter of a member in feet; and N denotes the number of components of a type.

2. Wave load profile on a jacket-bay will be as follows:

$$F_1 [(N_{leg} D_{leg}) + (N_{cond.} D_{cond.}) + (N_{riser} D_{riser})] +$$

$$F_2 [(N_{vertical\ braces\ broadside} D_{brace} f_{inclination}) + (N_{vertical\ braces\text{-}end\ on} . D_{brace.})] + F_3 [\Sigma (L_{equiv.\ horz.\ members} D_{brace} / 12)]$$

where, F_1 is the wave force on a unit diameter vertical member between upper two horizontal framings; F_2 is the wave force on a unit diameter vertical member from top horizontal framing level to the seabed; F_3 is the wave force on a unit diameter horizontal member at elevation of the bay; D is the diameter of a member in feet.

3.4.2 Development of Strength Pattern:

A steel jacket platform consists of three distinct parts: deck, jacket, and pile foundation, which have different behavior modes due to variations in their configurations, their media, and the loads imposed (Fig. 1.1). The self weight of the platform, and the operational and environmental loads imposed on it are transferred from top of the deck, through the jacket and supporting pile foundation, to the soil medium. The platform can be considered as a cantilever beam fixed at some distance below the sea bed in the soil medium. Therefore, the strength pattern (variations in the strength at different elevations) for the platform can be developed with an independent evaluation of the strength of its different parts from the top (deck level) to the bottom (foundation). The platform is divided in a judicial manner into a number of sub-structures (called hereafter bays), such that its strength determination against the various failure modes and mechanisms is considered. Such a division of a platform into sub-structures is based on their behavior reported in the literature for the actual failure cases and their responses as determined from the advanced analyses.

The failure modes and mechanisms, which are possible in a steel jacket platform were identified and discussed in detail in Section 1.3. From this review, it is noted that the deck and foundation bays are likely to develop a ductile failure mechanism against the lateral loads, with the yielding of all legs or piles, and a brittle failure mode due to the failure of one leg (leg-pile connection) or pile (pile-soil interface, pile plunging) against the axial loads. The failure mechanisms of the jacket are likely to develop due to lateral loads, with the buckling or yielding of the primary braces and legs in a bay. The jacket mechanism could be ductile or brittle depending on the size, type, configuration, and loading on the braces. In addition, a brittle failure mode can develop in a jacket bay due to the failure of a primary (leg-brace) joint.

Fig. 3.12 shows the division of a steel jacket platform into sub-structures or bays. A bay is considered in this study as a 3-dimensional sub-structure of the complete platform cut at different elevations. The uppermost bay, the deck bay, comprise of the deck structure, deck legs, appurtenances, and any bracings up to the first jacket framing level. The intermediate bays, the jacket bay(s), comprise of jacket horizontal framing, jacket legs, vertical braces, and other intermediate members and appurtenances. The number of jacket bays will be equal to one less than the number of horizontal framings in the jacket. The bottom-most bay, the foundation bay, comprise of the lowest horizontal framing of the jacket (with or without mudmats), the piles (main and skirt), and other elements such as conductors which extend below the seabed and provide a secondary support.

Then the component strength or the strength of a bay at failure of its first component is determined. This would represent a lower bound of the bay strength. In addition, the bias and variance associated with the lower bound of strength are determined. Note that bias and variance in the strength would vary for different bays, due to variations in their configurations and types of component. In case of the deck and the foundation bays, an estimate of the upper bound of the bay strength is also made due to their ductile portal behavior.

In the following sub-sections, the detailed procedures for estimation of the strengths for deck, jacket, and foundation bays are presented. Note that this procedure is a simulation of the actual behavior of the bays and it is based solely on simple characterization of the strength of the components and hand

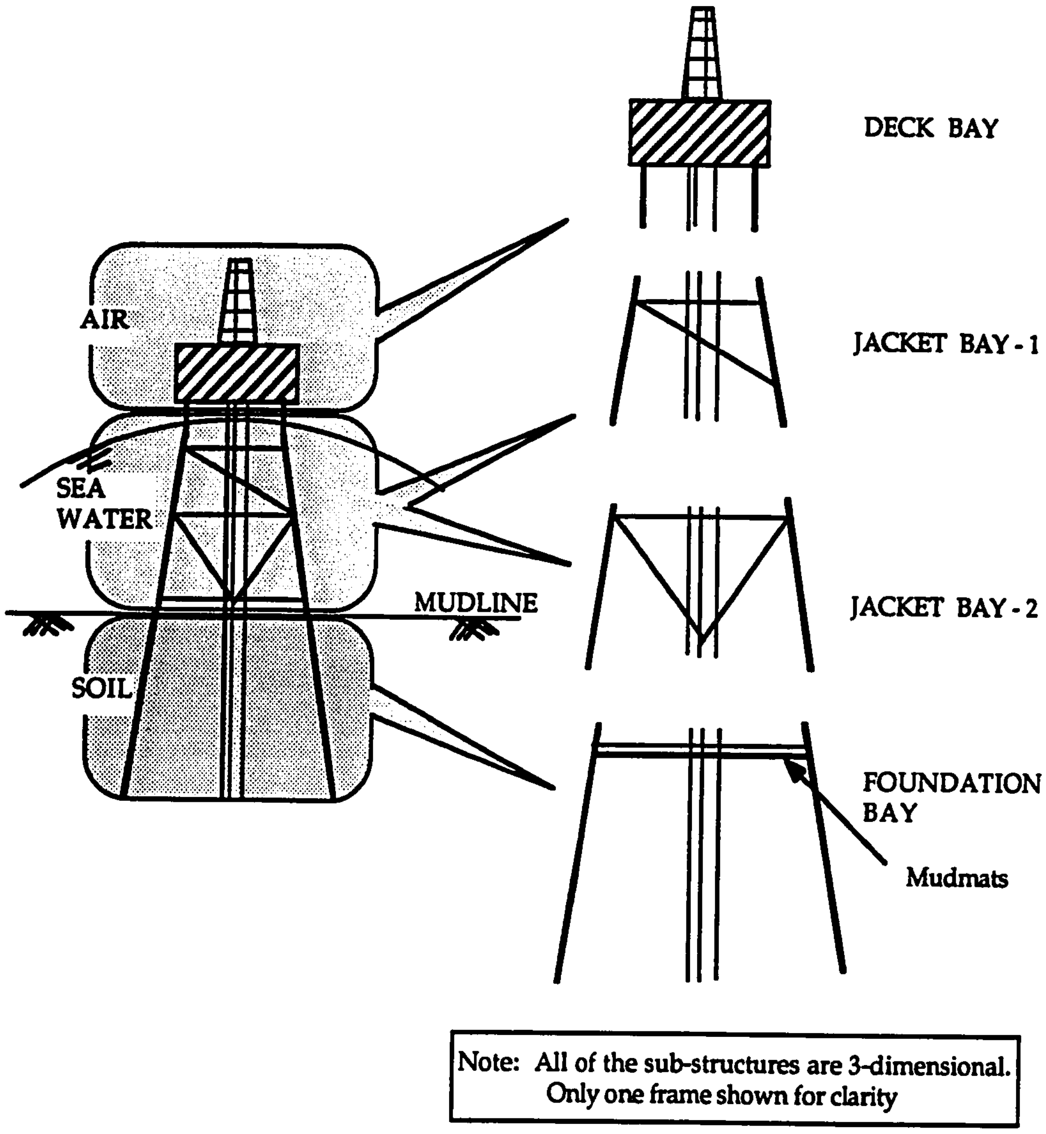


Figure 3.12: Sub-structures for a Steel Jacket Platform

computations. It does not require conventional finite element modelling for the bay structures and does not necessarily require computer based analysis.

3.4.2.1 Strength of a Deck Bay

The deck legs are connected at their upper end by large size deck girders. The primary girders are laterally braced by the secondary girders, which in turn are restrained against horizontal deflection by secondary beams and floor plating or grating. The deck bay comprising of deck legs with the upper and lower deck structural assembly behaves like a portal frame. The deck structure will deform like a rigid body and the girders with their high axial load-carrying capacity will provide the capability to re-distribute loads from one leg to another.

The failure modes and mechanisms can develop in the following way in a deck bay:

1) By formation of plastic hinges at the top and bottom ends of each leg of the deck under high lateral loads;

2) By failure of one deck leg connection with top of a pile or a jacket leg under high axial loads.

The hinges will form in the legs at the locations which are subjected to high bending moments and have higher stress ratios (= bending stress / yield strength of section). The yield strength of a leg section depends upon the material used and its section modulus. In case the deck girders are knee-braced, the upper hinges are likely to form in the legs at or below the elevation of knee brace-leg joints. Such a mechanism developed by yielding of the leg sections is of ductile type, and it shows ability of the deck bay to sustain the ultimate load level with further increase in the bay-deflection. Ductile behavior is observed when the leg sections are compact ($D/t < 25$) or semi-compact ($25 < D/t < 60$).

The connections of deck legs with piles or jacket legs are heavily stressed in axial mode in case the topside loads are high or the lateral loads are very high (wave hits the deck). In many cases, it becomes difficult to provide adequate weld strength to the field welded joints. In such cases, special measures are taken, such as increasing weld lengths by cutting grooves in the connecting plates. The maximum axial stress in a leg would occur when the wave approaches from a diagonal direction. In the past, at least two platforms have collapsed under hurricane loading, due to failure of deck leg - pile top connections (refer Table 1.3). The maximum and minimum loads in opposite legs depend upon contributions from lateral environmental loads (over-turning moment, OTM) and vertical deck loads. Due to cyclic action of waves, the failure of one such joint will bring the platform to a "fail-state." Upon failure of a leg-pile or leg-jacket connection, the load carrying capacity of the deck bay would substantially reduce, thus exhibiting a brittle behavior.

The ultimate moment capacity of a tubular member (leg) reduces with an increase in the magnitude of axial load on the member (leg), as was discussed in Section 2.4.2. The moment values in the legs will differ due to the variations in their size and configuration, axial loads (i.e., topside load distribution) and local wave loads (due to attached appurtenances, if any, or locations of conductors and risers) acting on them, and contributions from the overturning moment. In case the topside arrangement is symmetric, the difference in axial loads will not be significant, unless the wave hits the deck.

The methods used in evaluation of lower and upper bounds of the ultimate strength of the deck bay are presented further in this sub-section.

<u>Lower Bound Strength of Deck Bay</u>: The lower bound solution corresponds to the load level at failure of the first member (yielding of the first leg section). This load level is obtained by assuming that all the legs are subjected to equal magnitude lateral load and variable magnitude axial load. The leg with the maximum axial load will be the most likely to yield first.

The moment at first yield of a section is about 0.785 of its plastic moment capacity [*refer Section 2.4*]. Upon yielding of a section, the additional loads on it are re-distributed to the sections with lower stresses. An accurate determination of the sequence of yielding of the members will need use of sophisticated structural analysis.

An approximate estimate of the lower bound strength for a deck bay can be made based on the following steps:

1) Identify the leg with the maximum axial load from the topside facilities. Normally a wave approaching diagonally will introduce the maximum axial load in a corner leg. In case the API reference wave criteria varies with the direction, a different wave approach direction may be chosen. Determine axial and lateral load components on the leg for the reference level lateral wind and wave loads on the bay. A range of loads may be considered.

2) Determine ultimate axial (P_u) and ultimate moment (M_p, M_y) capacities for the leg section(s).

3) Evaluate the maximum allowable moment capacity of leg section considering reduction in capacity due to the range of axial load on the leg. The formulation discussed in Section 2.4 and given in Appendix-B is used. (M/M_p vs. P/P_y formulation).

4) For the (P/P_y) at the reference level load, determine (M/M_p) at the ultimate condition. Obtain allowable moment (M_x) in the leg with maximum axial load. The allowable moment for all the legs will vary between M_x and M_p.

5) Determine corresponding lateral load for the bay and it will give a lower bound estimate of deck bay strength.

Upper Bound Strength of Deck Bay:

The methods used for evaluation of the upper bound strength for a platform were described in detail in Section 3.2.1. An estimate of the upper bound strength of the deck bay can be made by the mechanism approach or by following the load-deflection diagram and considering the non-linearities introduced with the progressive failure of sections. The mechanism approach is followed due to simplicity in evaluation and ductile behavior of the deck bay.

The rigid-plastic large deformation analysis gives a more accurate upper bound estimate, by considering the instability (P-Δ) effect. It is evaluated by equating the internal work done due to the formation of a mechanism with the external work done by the loads acting on the bay, as given in Section 3.2.1 and is reproduced below. A mechanism will form upon yielding of all of the legs at their upper and lower ends or at intermediate elevations where a significant change in section properties occurs.

3.4.2.2 Strength of a Jacket Bay:

In most cases, the jackets are designed with battered legs, which are laterally supported by vertical and horizontal bracing systems at different elevations. The vertical loads are primarily carried by the jacket legs and the lateral loads are transferred from top to bottom of the legs, primarily by the vertical bracing system. The horizontal bracings are normally effective in re-distribution of loads from one brace to another upon yielding or buckling of the vertical braces. This is true in case the vertical brace is connected at top of a bay with the leg. Otherwise the horizontal braces transfer lateral loads to the vertical braces connected at middle or end of the horizontal braces. The battered legs with interior piles provide lateral resistance with their deformation and the moment-resisting action of the jacket legs. For a jacket with vertical legs, their contribution in carrying lateral load would be limited to their moment-resisting behavior.

Therefore, the strength of a jacket bay is obtained from structural properties and configuration of the three load transfer systems:

1) **Vertical brace system**
2) **Leg-horizontal framing system**
3) **Leg-brace joint**

The vertical braces act as struts whose ultimate axial load capacity is reduced by deflection due to the local lateral wave load acting on them.

The legs and horizontal framing together can be assumed to behave as a portal frame similar to the deck bay. However, due to lower rigidity provided by the horizontal braces, an ideal portal behavior (as considered for the deck bay) may not occur and a mixed mode failure may occur, with the deformation of horizontal framing in vertical direction.

In addition, the load carrying capacity of braces will be conditioned on the ultimate strength characteristics of the leg-brace joint. As a good design practice, the strength of a joint should be more than the ultimate axial strength of brace. Note that many of the earlier jackets were designed without leg cans, through-thickness steel properties at the joints, and grouted leg-pile annulus. Therefore, with increased load level, joint failure is more likely to occur.

<u>First Member Failure Load Analysis (Lower Bound Estimate):</u>

Usually, the compression brace is the first member to fail in a jacket bay due to its lower strength. In some cases, a joint may fail earlier. The lateral load on a bay at failure of the first member can be estimated by combining the load in other legs / braces at the moment of high load in the compression brace. The proportionate lateral load on the bay, P_1, which will initiate failure of its vertical compression brace is evaluated.

The steps in evaluation of the lateral load at failure of first member in a jacket-bay are given as follows:

1. Evaluate the capacity of individual members

   a) Evaluate the ultimate axial strength of major compression and tension braces in the horizontal and vertical planes and the axial and flexural strength of jacket legs. Consider reduction in axial strength due to flexural stresses (lateral load) in the wave zone, and due to hydrostatic pressure.

   b) Compute the horizontal component of the ultimate axial load for the vertical braces and jacket legs, P_{um}

2. Formulate the lateral stiffness for the bay

   Formulate the lateral stiffness equation for the bay along the load direction.

   $$K_o \approx [\Sigma\, EA/L\ Cos^2\Phi + \Sigma\, 3\, EI/L^3] = \Sigma\, K_{om}$$

   where, the first term is for the lateral stiffness provided by the vertical braces and the second term is for the lateral stiffness provided by the legs due to their deflection.

   The lateral stiffness of a leg, $3EI/L^3$, corresponds to the pinned end condition. For the fixed end condition, the lateral stiffness becomes $6EI/L^3$. The end conditions of a jacket leg fall between these two extremes states. However, as the contribution of the legs to the overall lateral stiffness is lower, the lower leg stiffness has been considered in this study.

3. Determine the Most Likely to Fail (MLTF) member

The most likely to fail (MLTF) first member in a bay is determined by comparison of the magnitudes of ULS strength of the horizontal components of the vertical braces with the ULS strength of the horizontal braces.

a) The load acting on a bay is described by the following formulation developed with the lateral stiffness components of its major load carrying elements:

$$P = K\,\Delta \quad = K_{01}\,\Delta + K_{02}\,\Delta + \ldots\ldots\ldots\ldots + K_{0N}\,\Delta$$

b) The horizontal component of the lateral load in a member (brace), P_m will be proportional to its contribution to the lateral stiffness of the bay.

$P_m \quad = (K_{0m} / K_0)\,P$

where, $K_0 \quad = K_{01} + K_{02} + \ldots\ldots\ldots + K_{0N}$

K_{0m} = Lateral stiffness of member along the load direction

N = Number of members

c) Determine P_{um} / P_m for all the major members. The member with the minimum ratio of $[P_{um} / P_m]$ will be the MLTF member in the bay.

4. Determine the bay load at failure of the first component

The lateral load acting on a bay, P_{bay}, to initiate collapse of its first member is determined by extrapolation. The horizontal component of the ultimate strength of the MLTF member (P_{um}) is taken along the load direction. This load corresponds to the stiffness contribution of the member to the whole bay. Based on the ratio of stiffness of the whole bay to the stiffness of this member, the load acting on the bay is evaluated by the following expression:

$$P_{bay} = (P_{um} / K_{0m})\,K_0$$

This load value gives a **lower bound** estimate of the ULS capacity for a jacket bay. The true value of the ULS capacity for a jacket bay can be obtained by tracing P-Δ diagram beyond first member failure. For such an evaluation, the second member likely to fail can be determined by considering the post-buckling strength of the first compression-brace or the yield strength of tension-brace at large deflection of frame. The second MLTF member is determined in the same as described above in steps 2 and 4.

The lateral load-deflection behavior for each bay of the jacket is required at screening cycle-2 for increase in the magnitude of lateral load, while the vertical load remains constant. The ultimate strength of each bay of the jacket is evaluated by neglecting interaction among the bays i.e., by assuming that the failure of members in the formation of a mechanism is restricted to one bay.

The capacity of a bay beyond first yield depends upon: magnitude of deflection and load shed by the brace (if buckled); load transfer capability of the horizontals; additional load carrying capacity of other vertical braces; the degree of static indeterminacy; and the lateral load capacity of legs. The above factors were together represented by "redundancy factor (F_1)" in Section 2.1.2.

If the horizontal braces connecting the jacket legs have lower strength and are not able to transfer the load shed by the buckled or yielded vertical braces, then the horizontal brace will collapse and it can initiate successive failure of other members in the bay, to form a plastic mechanism.

If the primary horizontal braces have sufficient capacity to transfer load, then the bay stiffness will reduce upon buckling of a compression brace, and thus the deflection of bay will increase. With further increase in lateral load, the bay stiffness can be assumed to remain same up to yielding or buckling of second section. In this way by following successive buckling or yielding of all the braces in a bay, the load-deflection diagram for the bay could be traced in a piecewise linear fashion. The reduction in magnitude of stiffness from its original value depends upon the number of compression braces. In addition, the load carrying capability of the jacket-bay also depends upon the moment resisting behavior of jacket legs. The contribution of the moment resisting behavior of jacket legs to the lateral load capacity is usually very less compared to that of the vertical bracings.

In addition to, the moment resisting behavior of legs, the effect of secondary moment (P-Δ effect) on legs will also be considered. The P-Δ Effect, can be represented as the geometric stiffness of the legs, and its effect is to reduce the lateral load carrying capacity of the bay.

Upper Bound Estimate of Jacket Bay Strength:

An upper bound estimate of the ULS strength of a bay can be made by selection of the primary members of a bay, the failure of which will lead to formation of a collapse mechanism in the bay. However, depending on the type of behavior of a member and the sequence of members assumed to fail, the residual load carrying capacity of primary members is affected. Thus, it is difficult to establish upper bound value for the jacket bays.

For example, a mechanism can be assumed to have formed in a bay, when all the vertical braces, which are important for transfer of lateral load from top of leg to its bottom, have failed (buckled or yielded). This would be an upper bound solution. In such an evaluation, the uncertainties exist due to the following:

The compression and tension members do not fail simultaneously due to difference in failure load of compression and tension members. The strength of a C-brace is lower than the strength of a same size tension member. Thus C-braces fail first and their post-failure (post-buckling) capacity reduces with increase in deflection. Thus when a T-brace fails upon further increase in lateral load, the load carried by the C-member reduces due to higher node deflection. The T-brace keeps on holding load level upon its failure (yielding). Thus, uncertainty due to the deflection and load in C-braces, depend upon the number of C-braces relative to the T-braces.

Thus, an upper bound estimate can be based upon the residual strength (upon considering cyclic wave loading) of C-brace(s) and the yield strength of T-braces. Therefore, it is important to make an accurate estimate of the residual capacity of the C-brace(s), which depends on many factors. In addition, the contribution of jacket legs in carrying lateral load should be considered.

Note, that when there are two or more compression members, they are all likely to fail together, when the jacket has been optimally designed, with the major members having similar utilization ratio during the in-place condition. Thus, they are most likely to fail in succession.

When there are T-braces, then a coarse evaluation of redundancy level can be made by neglecting C-braces altogether (at the screening cycle-2) and considering the stiffness of T-braces and legs. However, in such a case, the C-brace can be assumed to carry 15-25 % of buckling load. This gives a good estimate of the load carried by the bay upon failure of C-braces. Then this value is extrapolated to the level that the T-brace yields. All of the T-braces are likely to yield at nearly the same load level. Upon yielding, the T-braces would sustain the yield load, but its stiffness would become zero. Thus, the deflection would increase by substantial amount, and show **ductile behavior (robustness)** of the bay. At this time, the T-braces are carrying load corresponding to their yield value, the legs are carrying load due to their deflection (moment resisting behavior) and some load due to their axial stiffness component.

The ductile behavior (robustness) would depend upon the number of T-braces compared to C-braces in a bay. However, with a significant increase in deflection of bay, there will be reduction in load carrying capacity due to instability effect from the vertical load acting on the legs.

3.4.2.3 Strength of a Foundation Bay:

The foundation bay of a steel jacket platform comprises of three different types of structural components: **piles; conductors;** and **mudmats.** These components transfer the functional, environmental, and other loads imposed on the platform to the surrounding soil medium. The piles are the primary load carrying members in a foundation bay and are normally installed through the jacket legs. The load transfer from the jacket to pile is achieved in several ways: 1) grouting of the leg-pile annulus; 2) welding of piles to the jacket at its-top; or 3) combination of 1 and 2. The conductors are usually considered at the design stage for their capability to only transfer the vertical loads acting on them (self weight and christmas tree) to the soil medium. In addition, they can provide secondary load transfer capability essentially due to their deflection along with with the jacket, which can be accounted at the re-evaluation stage. The mudmats are designed to provide support to the jacket in its unpiled installation stage. In normal design, the capability of mudmats to transfer loads after installation stage to the soil medium are not considered. At the re-evaluation stage, their reserve capability can be accounted, which will depend upon the soil-type and the failure-mode.

The typical penetration (depth below seabed) of piles for the Gulf of Mexico platforms is about 250 ft. and they are usually lesser than 94 inch diameter. However, in some cases, pile penetrations up to 650 ft. have been provided. Note that very small diameter piles were driven for the early platforms. The penetration of the piles is achieved in several ways such as: driven piles; drilled & grouted piles; and installation by jetting. The different piles provided at the corners of a jacket template and connected to the its legs behave in a similar way. The jacket provides rigidity to the pile system and a ductile portal behavior is observed for the pile bay against lateral loads. The rigidity provided by the template will be higher for the case with grouted jacket-pile annulus.

In some platforms additional piles, known as skirt piles, are provided. The skirt piles are installed through the sleeves attached to the jacket legs in the bottom-most jacket bay. The annulus between the sleeve and skirt pile is also grouted. The load transfer capability from the short skirt sleeve to the skirt piles is enhanced by provision of mechanical aids such as weld beads, shear keys on the piles and the interior of sleeves. *Lloyd and Clawson, [1983]* determined that the RSR of a platform with the skirt piles may be 50% higher than that for a platform with only the main piles.

The conductors are laterally supported at different horizontal framing levels of the jacket by guides and thus they deflect with the jacket. Therefore, their deflection at the seabed will be approximately equal to that of the jacket-pile system.

The mudmats are provided at or below the seabed level to temporarily support the unpiled jacket, during the installation stage. The mudmats are designed to transfer load by shear and bearing and are conventionally located near the corner legs for their effectiveness. The effectiveness of the mudmats increase, when they are located near the jacket corner legs.

In the Gulf of Mexico, the soil medium consists primarily of clay with intermittent layers of silt and sand. The strength and stiffness properties of the soil layers vary with the inherent properties of the soil particles. The load carrying capability of the soil medium may be affected by a number of other phenomena. Some of these phenomena which influence the capacity of the soil media are: installation procedures adapted to achieve design penetration; effect of vibration of piles on soil. In case jetting of soil is done to achieve the design penetration, the strength of soil may reduce.

The foundation bay is subjected to the static (structural and topside) and dynamic (lateral environmental) loads transferred at the pileheads from the platform. The vertical loads on the piles are transferred to the surrounding soil by axial shear and end bearing. The lateral loads are transferred to the soil around the upper part of the pile. The cyclic lateral loads will deteriorate the load transfer capability of the soil and therefore the pile length to transfer the load and stresses in pile will increase.

The reference level wave (100 year return period storm) is likely to deflect the foundation system substantially near the seabed, and thus will disturb the surrounding soil due to wave cycles. The effect of cyclic load is to reduce the load carrying capacity of soil near the seabed in shear and bearing and thus the effectiveness of mudmats to transfer lateral loads diminishes. This will be true for the soft soil locations. If the mudmats are effective in transferring the loads, they will carry a part of the lateral load and some vertical load due to overturning moment at base.

For the diagonal wave approach case, the vertical load and the overturning moment result in very high compressive loads in one corner pile and low compressive stress (or tensile stress) in opposite corner pile. Thus, for determination of the first MLTF member, diagonal wave approach direction is considered, similar to the deck bay.

<u>Failure of the Foundation System:</u>

The foundation-system is subjected to a combined action of base shear, overturning moment, and vertical loads. In Chapter 1, eleven (11) platforms were reported to have failed in the Gulf of Mexico due to "insufficient reserve strength in the foundation elements" against the loads due to hurricanes. The specific reasons for their failures were as follows:

1) <u>Wave induced soil movement</u> occurred at some locations resulting in yielding and shearing of piles leading to collapse of platform.

2) <u>Pile pullout</u> occurred in a 4-legged well protector platform, due to deterioration of soil strength during installation. The deterioration occurred due to jetting of soil plug.

The <u>stability of pile foundation</u> depends upon the following:

1) The behavior of pile and soil under lateral movement due to ocean waves;

2) The bearing capacity of piles against compression plunging and tension pullout forces;

3) The possibility of scour and liquefaction of soil near the piles. The liquefaction which may occur during pile driving is recovered in time. Thus liquefaction potential due to earthquake shaking may be a consideration for platforms located in earthquake prone areas. In case cyclic storm waves are likely to cause liquefaction, it should be considered.

A correct estimate of the stress-strain curves (p-y and t-z) for the soil layers is the most important for evaluation of stability of foundation against lateral and axial movement of piles. A large number of uncertainties exist in the development of stress-strain curves for the soil. The lateral load acting on a pile will affect only its top portion, which shows oscillation due to cyclic wave load.

The bearing capacity of a pile depends on its diameter and its depth of insertion into the soil. The pullout force (maximum tension load capacity) of a pile is lower than its bearing capacity (maximum compression load capacity). The evaluation of pullout force is more imprecise than of its bearing capacity, as behavior of soils under tensile loads is not well known.

The scour around piles depend upon: type of sediments; amplitude of waves; and pile diameter. It results in reduction in effective depth of insertion of pile which reduces its bearing capacity and stability under lateral loads.

The p-y (lateral soil springs) and t-z (axial soil springs) curves are developed for different soil layers. These curves represent the capability of load transfer by the soil layers in lateral and axial directions. The soil strength in axial and lateral directions depend upon: the nature of soil and its consolidation level; the type and dimensions of piles, and the methods used for installation of piles. The magnitude of load transfer, besides the inherent properties (strength and stiffness) of surrounding soil, also depends upon the following:

* <u>Cyclic and dynamic loading effects:</u> The ocean wave is cyclic and has variable frequency. Thus, a hysteretic behavior of soil occurs due to pile motion, with the waves. Due to degrading behavior of soil (due to pore water pressure build-up), its strength and stiffness reduces in time.

The dynamic loading is induced due to impact and repeated loading, which occurs from storm waves (several hours) and earthquakes (few seconds).

* Remoulding effect: Clays experience a softening due to remoulding and built-up of excess pore water pressure. The remoulding effect is seen in clays due to relative pile-soil motion. The ocean wave load is cyclic, thus the pore-pressure changes in the soil near the seabed. In sand due to separation of soil contact with the pile near the seabed due to vibration, the excess pore pressure builds up in the soil due to pumping of water in between the sand grains. These two phenomenon are limited to the zone near the seabed. Due to these two phenomenon, the ultimate shear resistance and stiffness of soil can reduce with time.

In case of sand layers, the occurrence of storms is not likely to cause a significant buildup of excess pore water pressure. The build-up of pore water pressure is limited to the zone near the pilehead. Such a porewater pressure build-up is drained out immediately due to permeability of sand.

The p-y curves for sand indicates, that the soil upon yielding continue supporting the ultimate load level, with increase in deflection. However, for clay the soil strength reduces upon reaching the ultimate strength, and with further increase in deflection.

A large number of uncertainties exist in prediction of p-y curves for soil. The uncertainties exist due to scarcity of full scale test results. The development of p-y curve besides soil properties also depend upon: dynamic loading; reloading after extreme loading; scour; group effect, etc.

Pile Yielding Failure due to Lateral Load:

The load at pilehead is carried by the following two mechanisms:

1) Structural members: By yielding (hinge formation) of structural members, energy is absorbed.

2) Soil medium: By movement of piles, the soil particles are compressed and thus the resistance increases. Thus, the soil particles resist load by transferring it to the surrounding particles. At a certain load load, they also yield similarly to the different structural members. Upon yielding, their load carrying capacity reduces.

Under the action of base shear and vertical loads, the ultimate failure of the piles may occur by formation of a plastic mechanism. A plastic mechanism may form upon yielding of all the piles at two sections: at or near the pilehead, and at distance "L" below the pilehead. The distance "L" below pilehead corresponds to the point of zero shear. The ultimate moment carrying capacity of piles vary due to difference in axial load acting on them, in a similar way as for the deck legs.

The magnitude of ultimate lateral load capacity and the length "L" of piles for the formation of plastic mechanism is dependent upon the lateral resistance provided by upper layers of soil. The reaction of soil to movement of pile subjected to lateral loading is characterized by stress-strain curve of the soil. The soil modulus is dependent upon the disturbance of soil, rate of application of loading, and the experimental conditions. The soil layers near the seabed show inelastic behavior due to large deflections of the jacket-pile system, which reduces the load carrying capacity of soils. The effect of lateral load is usually limited to upper zone. It also depends upon the liquefaction potential of soil, its recovery later, and the scour near the piles. The soil layers at some distance below the seabed would show elastic behavior due to lower deflections.

Note that the pile behavior is significantly influenced by the upper soil layers. In this simplified approach two distinct failure modes as extreme cases possible are considered. The first mode is the formation of two yield hinges in each pile and conductor and the jacket moving as a rigid body with zero rotation at top of the pile. The second mode is axial pullout of a single pile with bending (rotation) of piles. A case may arise, when a yield hinge has formed in a pile and axial pullout of an opposite pile occurs. Note that in addition to these combined failure modes may occur, which may need evaluation by detailed analyses.

In case of underdriven piles, a pullout failure will need a serious consideration. In case the jacket leg - pile annulus is grouted, a rigid body movement of a jacket against dominant lateral loads is more likely, whereas, in case of ungrouted annulus bending of piles is likely to occur and a mixed mode may be observed.

A limit equilibrium approach is used for evaluation of the ultimate capacity of the foundation bay. The contribution to the strength of foundation bay is provided by piles, conductors, and soil. The contribution of mudmats is neglected at the screening cycle-2 for simplicity sake.

However, an accurate evaluation of ultimate capacity of foundation bay will require development of a progressive load-deflection diagram, in the same way as for the jacket bays, considering the various non-linearities in load, strength, and soil behavior. The development of load-deflection diagram will require sophisticated analyses and thus it should be done at higher screening cycles.

At this screening cycle, a limit equilibrium approach is used, which is also called as dowel theory by some [*DnV, 1980; Janbu, 1982*]. A plastic analysis of piles is attempted with assumption that yield hinges develop in the piles and conductors along with a full mobilization of earth pressure between the two hinge locations. It is considered that the lateral deformation of piles are sufficiently large to plastify the soil completely.

The ultimate collapse state for the foundation bay is shown in Fig. 3.6. The scour at the site should be considered and accordingly the soil curves be modified.

The conductors will deflect relative to a point at some distance below the seabed, where the shear load on the conductor will be zero. In case, there is a hard soil strata, then that hard soil layer will generally provide the support point, by its ability to absorb the conductor shear load.

The uncertainties in the foundation bay capacity will be a combination of the uncertainties associated with a number of factors, such as: 1) soil properties and their natural variations; 2) methods used for evaluation of bearing capacities; 3) loads imposed and their transfer to subsoil;
4) influence of installation methods and process on soil properties.

The degree of uncertainties in soil properties will depend upon the availability of soil investigation data and its quality for the particular location.

A discrepancy may occur between the actual loads evaluated for the platform structure and loads acting on the piles. Some of the loads may be directly transferred from the jacket structure to subsoil through other components such as jacket legs, conductors, mudmats, and the mud-level horizontal framing. In case of deepwater jackets, with inertia and damping effects, the loads transferred will further reduce.

The overall resistance against the lateral load or base shear is provided by the piles and conductors installed and supported the jacket template.

The following steps will be required for simplified evaluation of the ultimate strength of the foundation bay:

1) Evaluate the ultimate resistance of soil, pu as per API regulations.

In soft clays, $P_u = 3c$ to $9c$, for $X = 0$ to Xr,

where, $X_r = 6D / [(\gamma D / c) + 0.5]$

For $X = 0$ to X_r

$P_u = 3c + \gamma X + [c / (2D)] X \quad = 3c + X [\gamma + c / (2D)]$

For the extreme wave load condition, the pilehead displacements would be very large. Thus the soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation. This would not need development of complete p-y curves for a pile.

Thus, $P = 0.72 [X / X_r] P_u$ and $P_u = 3c + X [\gamma + c / (2D)]$

2) Evaluate the ultimate strength of the foundation bay. The equilibrium equation at level of hinge formation in a pile as shown in Fig. 3.6 would be:

$$2 M_p = R_u L - 0.5 P_s (L-10)(L-10) / 3 \quad \text{......(3.17)}$$

$$R_u = 0.5 P_s (L - 100) \quad \text{.....(3.18)}$$

$$\text{or } L = 2 R_u / P_s + 10 \quad \text{or} \quad L - 10 = 2 R_u / P_s$$

Substituting in equation (1), we get:

$$2 M_p = Ru [2 R_u / P_s + 10] - (1/6) Ps [2 R_u / P_s]^2$$

$$= (2 R_u^2 / P_s)(1 - 2/6) + 10 R_u$$

$$M_p = (2/3)(R_u^2 / P_s) + 5 R_u$$

$$\text{or} \quad 2 R_u^2 + 15 P_s R_u - 3 P_s M_p = 0$$

$$\boxed{R_u = 0.25 [-15 P_s + \{225 P_s^2 + 24 P_s M_p\}^{0.5}]}$$

"for number sequence only"

3.5 Detailed Quantitative Evaluation

3.5.1 Introduction

At the screening cycle-3, the capacity level of a platform is evaluated by the conventional linear structural analysis. The API minimum reference level wave load, as per Section 2.3.4 g of API RP 2A [*API, 1989*], is to be used for evaluation of the reference level loads. In this way the relative stiffness of frames and stiffness of different components are explicitly considered. Therefore, an accurate estimate of the member loads can be made.

The reference level wave loads are computed using the Morisons equation, the full projected area of the structural members and appurtenances in the wave zone, a constant drag coefficient of 0.6, and an inertia coefficient of 1.5 for members six-feet in diameter or less, increasing linearly to 2.0 for members ten-feet in diameter and greater, the appropriate wave theory, a wave period based on a specified steepness, and appropriate allowances for marine growth. **It is important to note that these reference level forces do not explicitly include currents.** As noted in Section 2.3.4g, if other combinations of the force parameters are chosen, the wave force and moment level at the seabed should be at least equal to the reference level.

In addition to the special considerations regarding the computation of the hydrodynamic forces, care must be taken when developing S_r that is based on the site-specific storm conditions. Of particular importance are the wave crest elevations that could have dramatic effect in increasing S_r as the wave crests reach the large projected area of the lower decks in a platform, and consequently in decreasing the RSR. Also, in the case of existing platforms, it is important to note that S_r can be reduced by the removal of unnecessary elements in the platform (e.g., unused conductors, boat landings, etc.), and removal of marine growth.

Given the loading specification and a linear structural analysis computer program, the loads and load effects on each member of the platform are determined. The integrity of intact members and joints are checked as per API-RP-2A guidelines with the factors-of-safety or resistance factors [*API-RP-2A, LRFD*] set at unity. The yield strength of steel is normally upgraded to account for the difference between the mean and nominal yield strength and the loading strain rate effects. The integrity of damaged, defective, and repaired members and joints is checked with special formulations that reflect the effects of damage, defects, or repairs on the ultimate capacity of the members and joints. Interaction ratios of unity for a component would indicate that it was loaded with a load level corresponding to its best estimate ultimate strength.

To form a collapse mechanism, a number of structural elements would have to have interaction ratios of unity. The RSR of the structure could be estimated by taking a weighted average of the reciprocals of the interaction ratios of the structural elements having the highest interaction ratios and which would form a failure or collapse mechanism (e.g., all vertical diagonal braces at a given elevation).

3.5.2 Evaluation of RSR

In this section, a simplified procedure for estimation of RSR is presented for use at the screening cycle-3. The following steps are considered in evaluation of RSR for a platform utilizing the results (member loads and member strengths) obtained by a linear structural analysis:

1) Development of Analysis model: Develop a 3-dimensional model for the platform in its "as-is" state. Some of the following may be included in development of the structural model for performing a linear elastic analysis:

* Soil-structure interaction;

* Strength reduction for damaged and deteriorated members;

* Improved load evaluation and distribution;

* Refinements such as edge-to-edge brace length instead of center-to-center length, and joint flexibility.

The model may be simplified by representing the stiffness of deck structure by only a few large diameter structural members connected at top of the jacket, and by lumping the vertical loads at a single point. In addition, the conductor guide area may be idealized by two diagonal members.

2) Identification of critical failure modes and mechanisms: Identify the failure modes and mechanisms for the candidate platform. The results of RSR (safety) assessment at the screening cycle-2 will be useful in identification of the critical failure modes.

3) Perform linear structural analysis: Perform a linear elastic structural analysis with the load parameters selected to obtain the API-reference level environmental force. Determine the load effects and stress ratios in members and joints.

4) Identification of overstressed components: Identify the components which are overstressed or the most likely to fail (MLTF). Develop a post-failure behavior for these components, e.g., the compression braces will have reduced strength upon buckling, whereas the tension braces will support their load level at yielding.

5) Approximate estimate of RSR: By incorporating the post-failure capacity of the highly stressed members, an approximation of RSR can be made for the cases with the likelihood of formation of failure mechanisms in the different bays. In essence an approximate estimate of the ultimate strength for a bay is made by considering the strength of components which will form a

mechanism in that bay. In this case, the reference level load corresponds to the base shear on the platform. The minimum RSR of all the cases would represent RSR for the platform.

Note that this RSR should reflect the ultimate strength of a platform, against the API-reference level force.

$R_u = (RSR)_{min}\, S_r$

$[K_d\ K_u\ (H_u)^{\alpha}] = (RSR)\ [K_d\ K_u\ (H_r)^{\alpha}]$

$H_u = (RSR)^{1/\alpha}\, H_r$ .....(3.19)

where, H_u reflects the wave height at which the first failure mechanism would form in a bay in the platform. H_r is the reference level wave height. α is equal to 2.0 for the drag dominated platforms, 2.5 when boat landings are provided, and 3.0 when wave hits the deck.

In this way, H_u would be evaluated for the formation of a mechanism in each bay.

6) Re-run the linear elastic analysis for the platform, with the different load cases corresponding to wave heights within the range determined in step 5, to determine member and joint loads and stresses. In some software packages, it is possible to save the results of the first stiffness analysis and make a re-run for additional load cases.

7) Improved estimate of RSR: An improved estimate of RSR will correspond to a wave height, H, which will cause overstressing of several members in a bay or several members have stress ratios close to 1.0. Such a scenario will point that a failure mechanism is propagating in that bay, with increasing wave height (or increasing load level). From review of the results obtained under step 6, such a bay and corresponding wave height can be determined.

Note that in this method, the non-linearities due to pile-soil interaction are considered, through "p-y" and "t-z" curves specified for the piles. Whereas, the non-linearities, which are introduced upon failure (buckling or yielding) of members are not considered accurately. Hence, a conservative estimate of RSR is made by considering the residual strengths of such members at large deflections.

By such a linear elastic analysis at the screening cycle-3, we will get a good estimate of the lower bound of RSR, because structural non-linearities are normally not effective up to the failure of first member. However, we will obtain a coarse estimate of the upper bound of RSR due to the conservative approximation made in incorporating the post-failure capacity of members.

3.6 Very Detailed Quantitative Evaluation - A Review of Current Practice

At the screening cycle-4, the platforms whose safety level at cycle-3 was determined as 'Marginal' or UFP and were upgraded with major changes, are further evaluated on the basis of the ultimate capacity of the structural system. Such a non-linear analysis is obviously time-consuming and costly. The number of platforms requiring cycle-4 evaluation would be few.

At this level, the capacity of a platform is evaluated by performing a detailed non-linear structural analysis. Such an analysis is done by considering a non-linear load-displacement behavior of primary members of the platform, which reflect their plastic performance at the ultimate load limit. The non-linear behavior of legs, piles, and conductors is modelled by in-elastic beam-column elements. The horizontal and diagonal braces are modelled as 'strut' elements.

At the simplest, the analysis can be done by performing member-replacement "static push-over analysis," by monotonically increasing the lateral load on the platform and determining member response. Upon failure of a member, the member is replaced by its residual load-carrying capacity and the analysis is repeated. The analysis is repeated until the capacity of the platform drops with further increase in the lateral loads. This indicates the ultimate capacity of the platform.

Deficiencies in the static push-over RSR are due to the fallacies of the loading scenario considered in push-over analyses. Storm loadings are not a static pattern of lateral wind, wave, and current forces that are proportionately increased until the platform collapses. The loadings are transient, dynamic, and cyclic.

Additional deficiencies in the static push-over RSR are due to potential degradations in the capacity of the elements that comprise the structure during intense cyclic loading conditions. Many of the platform elements experience rapid degradations in capacity when the repeated levels of strains are in the plastic regime.

These deficiencies can be addressed through the use of time-history nonlinear analyses that take explicit account of the transient nature of the loadings, loading histories, and the potential degradations in the capacity of the platform elements. However, such analyses are extremely difficult and time consuming.

A number of researchers in academia and industry have developed various software and methods for evaluation of ultimate strength (RSR) of a platform. Some of the important analytical works and model testings reported are reviewed here.

Marshall and Bea (1976) first evaluated the ultimate strength of platforms by simple member strength models and studied the failure modes. Thereafter large scale research effort was made in the U.S. to study the behavior of offshore platforms against earthquake loading. This work was undertaken at the University of California, Berkeley and the results were published by *Zayas, Popov, and Mahin (1981, 1982).* In their work, one-sixth scale model tests on a frame of an offshore platform were conducted.

More recently *Exxon Production Research Company (EPRCO)* evaluated the ultimate strength behavior of offshore platforms. They performed model tests and analytical studies for offshore platforms. Much of their work was started due to its application in the project on upgrading of the bass Strait platforms. Results of this work have been published by *Lloyd and Clawson (1983), Titus and Banon (1988), Grenda et al (1988), Pike and Grenda (1987).*

Norwegian works are mostly based on utilization and extension of the simplified Idealized Structural Unit Method (ISUM), and comparing the results with more sophisticated FENRIS software package developed by DnV, NTH, and SINTEF team. ISUM method has been used in the research undertaken at the Norwegian Institute of Technology (NTH) and in Japan. The results of this research have been published by *Moan et al (1985, 1986), Søreide et al (1986), Aanhold (1983), Engseth (1984),* and *Ueda and Rashed (1987).*

ISUM method has been developed as a design oriented numerical approach. The basic idea is to model the structure by using only one beam element between each joint in the platform, i.e., the structure is divided into the biggest possible structural units. *Søreide* extended this method to include refinement of the formulation for large deflections.

In the U.K., a large number of projects were undertaken to study ultimate strength of tubular joints and members. Recently, Steel Construction Institute has undertaken joint industry project to develop a non-linear analysis code for determining ultimate strength of platforms and to prove it with results from large scale model testing of frames *(Lalani, 1987)*.

Various simplifications have been made in the past to determine the ultimate strength of the platforms. Three approaches are more common for evaluation of RSR based on non-linear analysis techniques, such as follows:

1) **Detailed non-linear analysis using static pushover approach.**
2) **Member replacement method.**
3) **Enhanced member replacement method.**

Static pushover analysis has been widely used by the industry to evaluate the reserve strength of the platform. In this approach, a series of static structural analysis are performed by monotonically increasing the pseudo-static lateral load on the platform. The analysis is stopped when a global instability is reached in the platform system.

Member replacement approach is an approximate method utilizing linear-elastic static structural analysis. In this approach, the static pushover analysis is performed, and at each successive analysis, the failed members are replaced by end forced and moments to approximate the post-failure behavior of the members. Therefore, in getting accurate results by this method, the post-failure behavior of platform members should be modelled accurately.

Enhanced member replacement method introduced by *Pike and Grenda (1987)*, where the post-buckling load-shedding behavior of relatively slender tubular braces is introduced. In this approach, the remaining stiffness provided by the surrounding members in the axial direction of the brace is considered.

3.7 Summary

Qualitative and quantitative methods for evaluation of the ultimate strength of platforms have been presented. The qualitative and coarse quantitative methods have been developed in this study. These methods utilize the experience with the steel jacket offshore platforms in the Gulf of Mexico. These methods are based on evaluation of a platform for the likely failure or collapse modes. These failure modes were based on extensive review of failure of platforms in the Gulf of Mexico.

The qualitative methods with its two phases would find extensive application in routine assessment of the capacity level of large number of platforms existing in the Gulf of Mexico and other regions. These methods would identify the likelihood of a platform to have equal or lesser strength since the last assessment.

The coarse quantitative method is relatively simple to apply due to evaluation of capacity of different sub-structures. Due to the inherent properties of a steel jacket platform, the results obtained have been good as initial estimate for screening purpose.

Simplified approach has been presented to evaluate RSR from reduced number of linear-elastic analysis.

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CHAPTER 4

CONSEQUENCES OF A PLATFORM FAILURE

Consequences form an important dimension in risk assessment and fitness for purpose evaluation of a platform. Risk is represented as the product of the capacity of a platform and the consequences upon its failure. It is very complex to make its precise estimate of consequence level for a particular platform due to a large number of socio-economic factors, which could vary with location and period. There is no direct method available for evaluation of the consequence level for a platform.

In this chapter, the consequences which are likely to occur upon failure of a platform and the major attributes which influence each of these consequences are identified and discussed. A subjective classification method to evaluate the consequence level is presented. The quantitative methods available for their evaluation are described.

4.1 Identification of Consequences:

The consequences of prime importance when a platform is deemed unserviceable due to severe damage or when it collapses are as follows:

a) Personnel: - **Injuries**

- **Loss of life**

b) Environmental: - **Pollution**

- **Cleanup**

- **Restoration**

c) Resources: - **Loss of production**

- **Inability to meet contractual agreements**

d) Property: **- Loss of production revenue**

- Cost associated with repair, salvage, platform replacement, plugging and abandonment of wells, and loss of contract.

A regulator is interested in the consequences related to personnel, environmental, and resources loss. An operator is interested in these consequences and in addition in the potential loss of property. In addition to the above consequences, the "public-political" reaction to the consequences may sometimes influence the operational philosophy followed by the "regulator-operator" for offshore production in an entire region (e.g., California, Gulf of Alaska).

The above consequence categories require evaluation on different scales and therefore can not be combined on a common scale. In effect, the process for evaluation of a comprehensive consequence level for a platform becomes very complex and variable dependent upon the affected parties. For example, how should the value of life and value of environmental pollution be assessed.

Various researchers have looked into these aspects in detail, but there seem to be no consensus on these issues. The above consequences are often termed as "tangible" i.e., which can be accounted in terms of money, and "intangible" i.e., which cannot be accounted in terms of money.

Some years ago, the environmental consequences were commonly considered as "tangible" but due to various accidents in the past decade in some areas, and the consequent increase in public awareness due to damage to flora and fauna, they cannot be considered in such a way in the environmentally sensitive areas such as California and Alaska. However, in the Gulf of Mexico they can still be accounted in terms of money.

The loss of resources may require an operator to obtain an equivalent quantity of the contractual agreement from other sources available to him. The property losses may be significant and would essentially require a cost-benefit evaluation to determine the feasibility of continuing development from the field. In addition, the hidden costs associated with such failures would be borne by the operator and the shareholders of the company.

The focus of this study is on safety assessment of the platforms located in the Gulf of Mexico. The structural and operational characteristics of these platforms vary from those located in Southern California and Gulf of Alaska. In this chapter, the consequence evaluation of the Gulf of Mexico platforms will be focussed.

Many of the platforms in the Gulf of Mexico are unmanned, and are remotely operated from a central platform or from shore stations. In some cases, the unmanned platforms are temporarily accessed during operations. As a general operational philosophy in the Gulf of Mexico, platforms are evacuated before the hurricane approaches. Hence, the possibility of loss of human lives during hurricane is significantly reduced. However, in case of manned platforms in the Gulf of Mexico, potential for loss of life would remain due to probability of maloperation of the emergency evacuation system provided on a platform.

Environmental pollution occurs primarily due to three sources: crude storage tanks, production wells, and risers. Most of the platforms in the Gulf of Mexico do not have crude storage on their decks. The crude is piped to other platforms, to shore stations, or transported to shore in barges. Down-hole sub-surface safety valves (SSSV) have been installed in the wells on all the platforms in the Gulf of Mexico to meet the MMS guidelines. Many of the major risers have been equipped with emergency shut-down and back-flow prevention valves to reduce environmental pollution. However, the potential for environmental pollution remains in such cases due to failure of valves to operate, as they are not 100 % reliable.

The consequence level of many platforms in the Gulf of Mexico would have a significant contribution from the cost effects associated with the loss of resources and property.

4.2 Major Attributes for Consequences Upon Failure:

The major attributes, which have a significant influence on the consequence level of a platform are identified and discussed in this section. These attributes have been selected based upon a review of the operational characteristics of various different types of platforms operating in the Gulf of Mexico.

The failure of various types of platforms which have occurred in the Gulf of Mexico was summarized in Table 1.3. They varied from individual well protector platforms to self contained production cum process facilities. In addition, there are platforms which are used solely to provide living facilities or support to the transportation system.

The major attributes which have a significant influence on the consequence level of a platform are listed below:

a) Platform is manned or unmanned

b) Emergency evacuation system

c) Crude storage on deck

d) SSSV installed in wells

e) ESD valves in risers

f) Production level

g) Functions of platform

h) Integration with other platforms

i) Lost production.

Of the above attributes, the consequence to loss of life or injuries upon failure of a platform is dependent upon the number of people on the platform and safe operability of the emergency evacuation system. The consequence to environmental pollution is related to the crude stored on the platform, the number of risers and production wells, the crude flow pressure in wells, and the operability of the emergency safety valves provided in production wells and risers. The loss of resource consequence is related to the crude quantity handled by the platform and the recoverable crude lost in the reservoir. The property losses relate to the functions of the affected platform, influence on operation of other platforms in its vicinity, and production level.

In the following sections, various methods to make qualitative and quantitative evaluations of the consequence level of a platform upon its failure to operate are described. These methods are based on the major attributes discussed in this section.

4.3 Evaluation of Consequences:

The consequence level of a platform can be evaluated based upon a comprehensive assessment of its functions, the operational philosophy followed, and its importance to the continued operation of the platforms connected to it. An estimate of the consequence level upon failure of a platform is required at the four-cycles of the screening process, which would differ due to the difference in the state of knowledge of the platform at these screening cycles.

At the initial screening cycles, a very large number of platforms would require assessment of their consequence level. Therefore, simplified techniques should be used at the first screening cycle. With improved knowledge of a platform, a quantitative assessment of consequence level could be made. However, due to the complexities involved in making a quantitative assessment of consequences related to loss of life and environmental pollution, it was considered that a broader classification of platforms be attempted according to its functions. Such a classification, which roughly correspond to the guidelines provided by MMS is given further in this section.

The consequence level associated with a platform has been characterized in this study into five categories, such as: Very Low Consequence (VLC), Low Consequence (LC), Moderate Consequence (MC), High Consequence (HC), and Very High Consequence (VHC). Such general qualitative categorization is important in making a decision on its "Fitness For Purpose (FFP)."

A VLC category platform would be the one with negligible consequences upon its failure to human life, environment, and property. For example, an unmanned, single gas-well caisson platform could fall in this category. A VHC category platform, on the contrary, would have severe consequences associated with its failure. For example, a multi-well, manned platform supporting production, drilling, and living facilities, and producing large quantities of oil & gas could fall in this category. If such a platform also provides processing and pumping support to the platforms in its vicinity, then its consequence level would further increase.

The characteristics of the platforms, based on which they could be classified in above five categories are summarized below:

Very High Consequence (VHC) platforms: A platform, which produces crude oil or not, which supports all the major operations (processing, storage, compression, or pumping) on it, which is manned (living quarters provided), and which supports operations of a number of other platforms is classified as a very high consequence (VHC) structure. **(Central processing platform)**

High Consequence (HC) platforms: A platform, which produces crude oil, supports all the major operations (processing, storage, compression, or pumping) on it, which is manned (living quarters provided), and whose operations do not affect other platforms is classified as a high consequence (HC) structure. **(Independent well/ process platform)**

Medium Consequence (MC) platforms: A platform, which produces crude oil and has **no other major operations** (processing, storage, compression, or pumping) on it, which is **manned** (living quarters provided for lesser number of people), and which transfers its crude to a central platform is classified as a medium consequence (MC) structure. **(Manned well platform)**

Low Consequence (LC) platforms: A platform, which produces lesser quantity of crude oil, has **no other operations** (processing, storage, compression, or pumping) on it, which is **unmanned**, and which feeds its production to a central processing or pumping platform is classified as a low consequence (LC) structure. **(Unmanned oil-well platform)**

Very Low Consequence (VLC) platforms: A platform, which produces gas and has no other operations (processing, storage, compression, or pumping) on it, which is unmanned, and which feeds its production to a central processing or pumping platform is classified as a very low consequence (VLC) structure. **(Unmanned gas-well platform)**

Note that the above classification of the consequence levels has been assumed for this study and may vary with a different interpretation by a regulatory body or an operator for the platforms located in a particular region, and with consideration of different functions and safety criteria.

The qualitative and quantitative methods should be based on the major attributes identified in the previous section and the consequence classification given above.

4.3.1 Qualitative Evaluation of Consequence Level

The qualitative evaluation of consequence level is aimed at its use at screening cycle -1, where a decision on safety level of a very large population of platforms is required. At this stage, it is likely that sufficient details of the platform facilities may not be available and may require prohibitive cost and time to obtain. Therefore, simple methods should be used to make a qualitative assessment of consequence level of a platform.

A two stage qualitative assessment method is proposed for evaluation at this screening cycle. The first stage is based on evaluation of the significance of individual attribute discussed in Section 4.2 on the consequence level. This provides a cut-off criteria, based on which the high consequence and very high consequence platforms are identified quickly for need of detailed quantitative evaluation at higher screening cycles upon gathering of necessary details of the platform. The platforms which are not considered to be of high consequence based on a single attribute are further evaluated at screening cycle-1, with an assessment of the cumulative effect of all the attributes on the consequence level.

At screening cycle -1, the major attributes discussed in Section 4.2 are evaluated for these three conditions. The various scenarios, which could meet these conditions are further discussed in this section.

i) Low risk or consequence to personal, property, and environment:

If a platform is unmanned, has necessary evacuation facilities intact and operational, has no crude storage on its deck, and has safety valves provided in its wells and risers, then it will have low risk to human and environment.

ii) No adverse effects on the safety of platform due to modifications in topside facilities: If the production facilities on a platform have remained the same as were originally planned, then its consequence level is likely to remain the same unless the new regulatory guidelines dictate modified safety standards and design of equipment.

As a normal practice, the topside facilities of a platform change over its operational life, with addition or removal of equipment or instruments. Additional risers and boat landings may be added to the structure. The equipment on the deck may differ from those at its "as-design" state. In such a case the impact of variations in major facilities on consequence level need to be considered. The major changes which may occur are: storage tanks for consumable or production, separators, compressors, generators, pumps, cranes, etc.

A thorough investigation of a platform is required to make an assessment of the above conditions. Such an assessment will be aided by completion of the data-sheets (gathering platform data) for a platform, as presented in Appendix-A of this study. These data-sheets have been prepared considering them to be completed by obtaining standard platform data provided by an operator, and they may require one or more site visits to update the operator supplied data. In the absence of operator supplied data, the effort required to gather the details of a platform will be significant. Alternately, 'databases' can be prepared and integrated with the eventual expert system based on this methodology to provide default data for the different platform locations.

4.3.1.1 Phase - A: Cut-off Criteria

In Phase - A, the emphasis is to establish the important conditions, which would provide a cut-off criteria and identify the high consequence platforms from low consequence platforms. The characteristics which would classify a platform in one of the five consequence levels were listed in Section 4.3. These have been assumed for use in this study. Therefore, the cut-off criteria should correspond to these characteristics.

The Figures 4.1 and 4.2 have been developed to demonstrate the procedure to differentiate between high and low consequence platforms on the basis of a single major attribute.

The cut-off criteria in Figure 4.1 is based on evaluation of the likelihood of loss of life and environmental pollution. The likelihood of loss of life is related to the operations of a platform, emergency evacuation system, and the safety philosophy followed for the platform.

The environmental pollution as discussed in previous sections is related to type of production wells, type and function of risers, facilities on deck and the safety equipment provided to remedy any negative influence due to failure of these components of a platform.

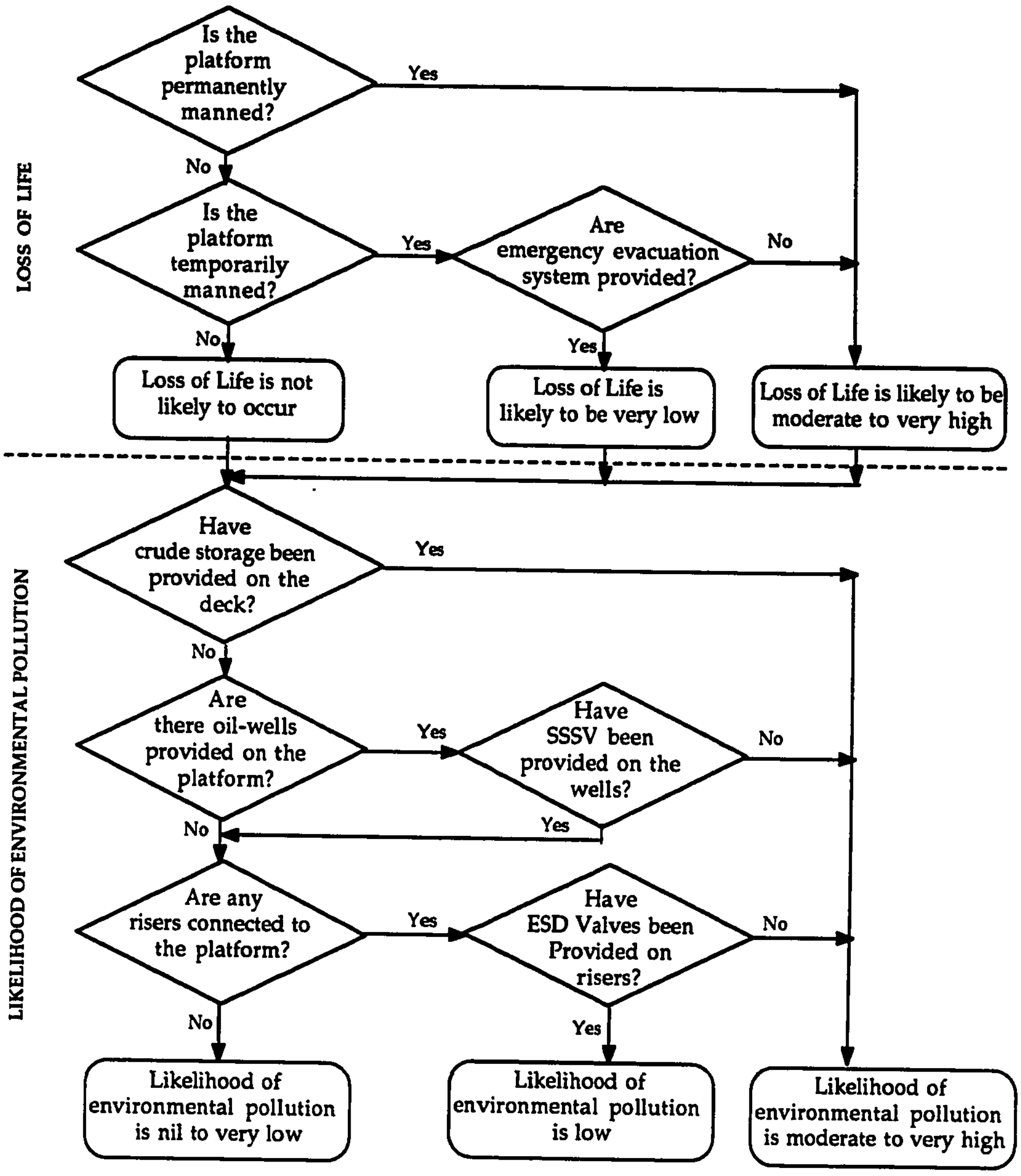


Figure 4.1: Cut-off Criteria Based on consequences related to likelihood of loss of life and likelihood of envirnmental pollution

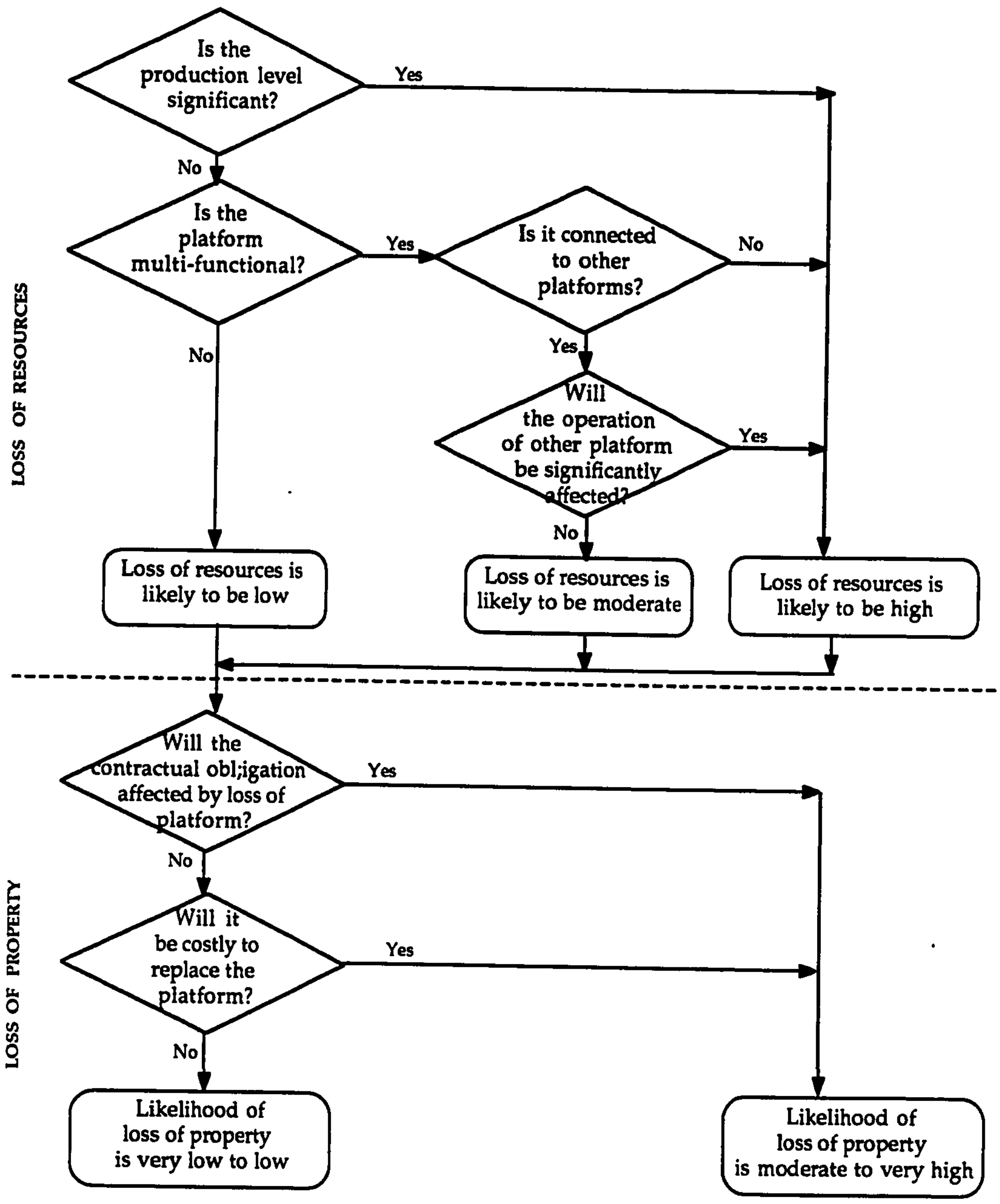


Figure 4.2: Cut-off Criteria Based on consequences related to loss of resources and loss of property

The platforms which are evaluated to have high likelihood of loss of life or environmental pollution in case of collapse of a platform due to storm condition, would require detailed evaluation of its consequence and capacity evaluation. The platforms with low likelihood of loss of life or environmental pollution could be evaluated by qualitative process.

The cut-off criteria presented in Fig. 4.2 is based on evaluation of likelihood of loss of resources and loss of property. The likelihood of loss of resources depends upon the functions of a platform, production level, importance of the platform to the overall production level from a reservoir, importance of a platform to other platforms.

The loss of property is related to the monetary loss due to failure of the platform. The losses will be in terms of the cost of the platform to replace, if required, contractual obligations of the operator.

The loss of resources and loss of property could be evaluated in monetary terms. The platforms which do not show high likelihood of any one of these losses would be evaluated for cumulative effect at Phase-B. The platforms which show high likelihood would be evaluated in detail at screening cycle-2.

4.3.1.2 Phase - B: Qualitative Evaluation

In Phase-B evaluation, the platforms whose consequence level is not dominated by one or two major attributes are evaluated. The consequence level of such platforms is determined based on a combined subjective evaluation of the major attributes or conditions identified in Figures 4.1 and 4.2. Such an evaluation is based on the "attribute weighting" approach and is done in the following way:

1) Identification of the major attributes, which significantly affect the five categories listed in Section 4.3.

2) Allocation of relative weights to the different attributes. These weights will represent the "Degree of Importance" for a category of platforms. These must be selected in consensus with the regulator and other parties involved.

3) Allocation of grades for different attributes for a candidate platform;

4) Evaluation of a comprehensive grade by summation of the products of grade and weight for each attribute.

5) Identification of a subjective level (VLC, LC, MC, HC, VHC) on the basis of comprehensive numerical grade fixed by the regulator.

The relative weights would change for a different geographic region and with different interpretation by the regulator. A platform is likely to have medium consequence level, in case of its failure (collapse) under the API-reference level load, when the following are met:

i) Low risk or consequence to personal, property, and environment;

ii) Adequate facilities provided for safety of personal, property, and environment.

iii) No adverse effects on the safety of the platform due to modifications in the topside facilities.

4.3.2 Quantitative Consequence Evaluation:

At screening cycles 2 to 4, a quantitative estimate of the consequence level of a platform is required. The objective is to make a quantitative consequence estimate of the platforms which are with high and very high consequence level, or which were classified as "Marginal" or "UFP" in the first screening cycle.

A quantitative estimate could be made in two distinct ways, as follows:

1. **Monetary evaluation;**
2. **Utility evaluation.**

Note that it is very difficult to incorporate loss of life etc. in monetary terms. In such cases, a utility method is beneficial. In this project, as the emphasis is on use of this approach by the regulatory bodies, "utility theory-decision analysis" approach has been applied for evaluation of the consequence levels at the higher screening cycles.

The potential consequences are assessed in monetary terms and are converted to a utility scale as shown in Fig. 4.3 [*Bea, 1990*]. The relationship between monetary and utility scales can be linear or non-linear. The linear scale represents risk neutral effect of the cost associated with the attribute on the utility (or consequence). A non-linear scale represents risk adverse or risk attractive effect of the cost associated with the attribute on the consequence level. The inconsistent effect of different magnitudes of an attribute on a consequence is reflected by a non-linear relationship.

The importance of a consequence (preference, priority) could be decided by the regulator. However, based on an expert's opinion, general values for the different consequences will be included in this method. It may be noted that the relative importance of the consequences may differ among the regulatory bodies, between a regulator and an operator, and for different geological areas. It may be possible that the DOI for consequences decided by a regulatory body today may vary in time, depending on the "public-political"awareness, needs, and concerns. Hence, the preference order could change with time.

A) COSTS (MILIONS $), X1

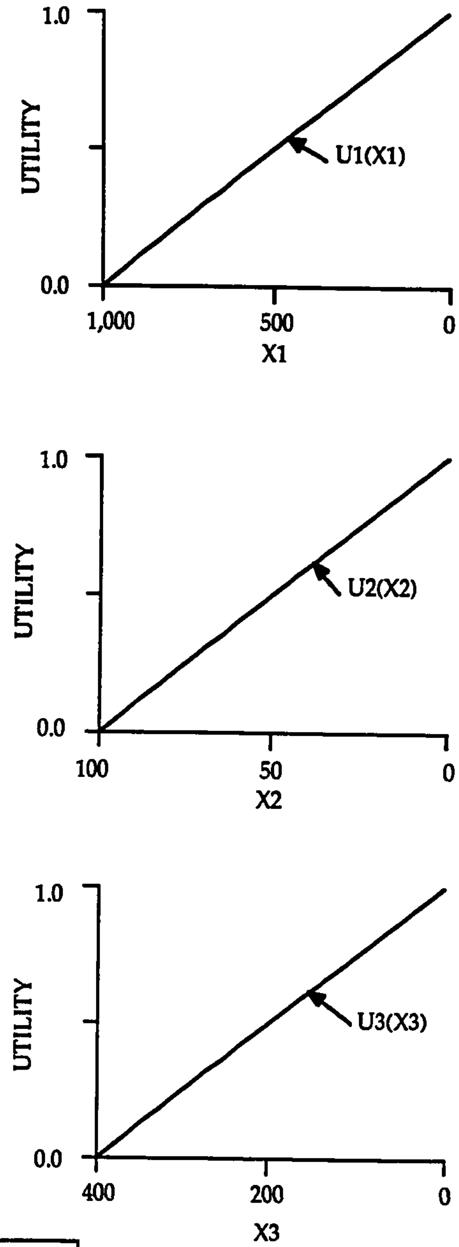
UI (XI) = 1.0 - X1/ 1,000

B) INJURIES (NUMBER), X2

U2 (X2) = 1.0 - X2/ 100

C) ENVIRONMENTAL IMPACT (100 BARRELS SPILLED), X3

U3 (X3) = 1.0 - X3/ 400



$$U = \frac{K1}{\Sigma K} U1 + \frac{K2}{\Sigma K} U2 + \frac{K3}{\Sigma K} U3$$

Figure 4.3: Utility Based Evaluation of Potential Negative Impacts Represented by Dollars, Injuries, and Barrels of Oil Spilled [Bea, 1990]

4.4 Summary:

Qualitative and quantitative methods for evaluation of the consequence level of a platform were discussed. The consequence level of a platform is based on a number of factors which affect personnel, environment, resources, and property. The consequence level varies with the geographical region.

In the Gulf of Mexico, there is very less likelihood of loss of life and environmental pollution, because the normal practice is to evacuate the platforms before storm warning and that the wells and risers are provided with safety valves.

The platforms with high consequence level could be distinguished from low consequence platforms by cut-off criteria presented. A cumulative effect due to various characteristics of a platform could be assessed by a matrix procedure, in which the user may choose the degree of importance of different consequences for the platform evaluated.

The quantitative method based on utility theory-decision analysis would provide satisfactory results at higher screening cycles, by including loss of life and environmental pollution in utility terms.

More work is required to develop the methods for evaluation of the consequence level. Such effort should be done in concurrence with MMS and other parties involved.

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CHAPTER 5

FITNESS FOR PURPOSE EVALUATION

A method for Fitness For Purpose (FFP) evaluation of a platform should provide a basis to make the following decisions:

- The platform is safe for continued production;

- The platform requires more refined estimations of its capacity and consequence levels;

- The platform requires structural strength upgradation or reduction in the loads acting on the platform;

- The platform is unsafe for current use under its existing state and parameters, and may be considered for an alternate application;

- The platform may be abandoned and decommissioned.

No such method is available in public-domain for use by the offshore regulatory bodies and the industry. Such decisions so far have been made based on re-assessment of deterministic structural strength of a platform. In such an approach, the consequence dimension is reflected in the code specified factor of safety.

The approach followed in this study is based on a comprehensive probabilistic evaluation of fitness for purpose of a platform using the estimates of capacity and consequence levels. The approach described here has some similarities to the approach followed in a joint-industry program Assessment, Inspection, and Maintenance (AIM) [*Bea and Smith, 1987, Bea et al, 1988*].

In this chapter first the important characteristics of FFP evaluation are discussed. Then the methods suggested for qualitative and quantitative evaluations are discussed in detail.

5.1 Introduction

The Fitness for Purpose (FFP) evaluation could also be called as the risk assessment phase in re-evaluation of the safety of a platform. It is based on two important premises: "Risk Identification" and "Risk Acceptability" for a type of platform. Fitness for purpose of a platform could be decided based on identification of its risk of failure against one or more initiating events and comparison of the platform's risk level with the risk level for that region acceptable to the public-regulatory bodies.

The **risk identification phase** requires development of a decision making method based on a combination of the capacity (RSR) and consequence levels for a platform, whereas the **risk acceptability phase** requires development of a method to differentiate between the acceptable and unacceptable risk levels from the public-regulatory perspective. In this study, it has been assumed that the "acceptable risk level," corresponds to the risk level acceptable to a public-regulatory body for a "new design" platform, and the "marginal risk level" corresponds to the risk level acceptable to the public-regulatory body for an "old platform."

Conventionally total risk is represented as the product of "likelihood of failure" and "consequence of failure."

$$\text{RISK} = \sum [\text{Likelihood of Failure}]_i \times [\text{Consequence of Failure}]_i \quad \text{...(5.1)}$$

where 'i' represents a failure initiating event. The likelihood (probability) of failure of a platform can be inversely related to its RSR, which could be evaluated as described in Chapter 3. Consequence of failure of the platform is evaluated as described in Chapter 4. A decision on the fitness for purpose of a platform is then based on a comprehensive assessment of its RSR and consequence levels.

This study focuses on the risk associated with overload from storm waves or hurricanes. The total risk associated with a platform will be a summation of the risk associated with the different load sources such as hurricane, earthquake, fatigue (normal waves), fire, dropped objects, ship collision, etc.

On the basis of total risk equation 5.1, a platform with a high likelihood of failure (or a low RSR) and a low consequence level could have a risk level similar to that associated with a platform with a low probability of failure (or a high RSR) and a high consequence level. It seems pragmatic, because a platform with a high RSR is likely to represent a more redundant structure, for which failure of one component is not likely to lead to the formation of a mechanism, and therefore it could be subjected to higher consequence level and meet the safety level acceptable to the public-regulatory bodies. This philosophy has formed the basis in development of the methods presented in this chapter for FFP evaluation.

The likelihood of failure is proportional to inverse of RSR of a platform, which was discussed in detail in Chapter 3. RSR for a platform is determined based on the estimates of loads acting on it and its strength. As discussed in Chapter 3, the nominal estimates of load and strength of a platform have associated uncertainties (biases and variances) due to various sources. Therefore, a failure probability (i.e., the probability of load or demand exceeding the strength level) would be associated with the nominal capacity (RSR) due to randomness of physical phenomenon and lack of precision in evaluation of parameters and quantities.

On the contrary, the factor of safety, specified by the codes and used in the conventional design, is a deterministic quantity, because in its evaluation the uncertainties in its parameters are not considered. Therefore, a failure probability would be associated with the deterministic factor of safety and the factor of safety could be considered as a "nominal" measure of structural safety.

The correctness of the assessment in these two phases will depend on the accuracy of the input data of the parameters considered in the capacity and consequence levels. The bias associated with FFP evaluation for a platform would depend upon the biases associated with the capacity and consequence estimates made at each cycle of the screening process.

5.2 Qualitative Evaluation:

The screening methodology presented in Section 1.4 is based on a comprehensive evaluation of capacity and consequence evaluation at different screening cycles. At screening cycle-1, a qualitative evaluation of capacity (RSR) and consequence (C) levels is made. The capacity and consequence levels evaluated in screening cycle-1, on the basis of the procedures discussed in chapters 3 and 4, are classified in five categories ranging from very low to very high. The following basis was assumed in this study for forming a qualitative classification of the capacity and consequence levels.

Category	Capacity (RSR)	Consequence (C)
VH	> 1.50	> 300
H	1.25 - 1.50	100 - 300
M	1.00 - 1.25	40 - 100
L	0.80 - 1.00	10 - 40
VL	< 0.80	< 10

The above classification has been assumed for this study to demonstrate the process. A user may elect to consider different ranges for very low to very high capacity and consequence levels, depending on particular characteristics of a platform and the desired safety level.

These estimates for a platform are then compared in a diagram similar to that shown in Fig. 5.1 and a qualitative estimate of fitness for purpose (FFP) of a platform is made. Based on this diagram, the platforms will be screened into three categories: Fit for Purpose (FFP), Unfit for Purpose (UFP), and Marginal.

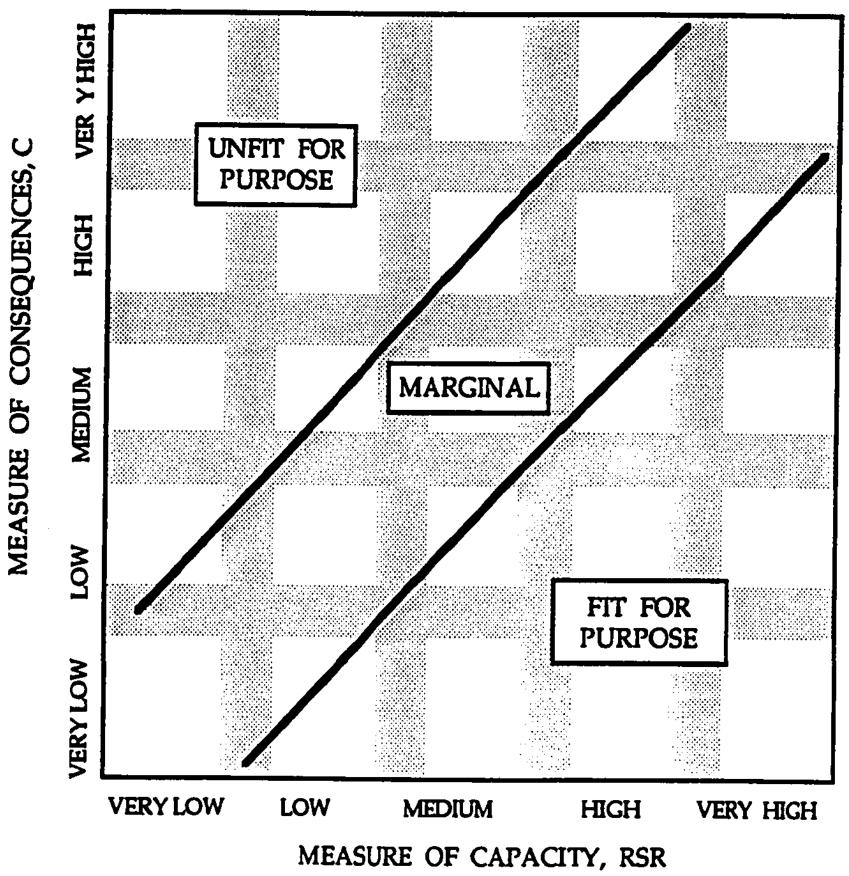


Figure 5.1: Capacity - Consequence Evaluation (Screening Cycle -1B)

In this case, the risk acceptability has been simply based on equation 5.1 and the values given in the above table.

If the combination of capacity and consequence levels for a platform falls in the FFP zone, the platform exits the re-assessment process (Fig. 1.2) with recommendation of implementation of the proposed IMR program. If a combination of capacity and consequence levels falls in the UFP or Marginal zones, the safety-assessment should be performed at the next screening cycle (coarse quantitative) upon obtaining necessary data and details of the platform, and with more refined estimates of capacity and consequence levels.

5.3 Quantitative Evaluation:

A quantitative evaluation of the FFP of a platform is made at screening cycles 2 to 4. Such characterizations of FFP is derived from the safety index formulation discussed in detail in Chapter 2 and given as below:

$$\text{Safety index, } \beta = \frac{\text{Mean Safety Margin}}{\text{Uncertainty Level}} = \frac{\ln\,[R_m / S_m]}{[(\sigma_{\ln R_m})^2 + (\sigma_{\ln S_m})^2]^{1/2}} \quad \text{...(5.2)}$$

where, R_m = Median ultimate strength of a platform

S_m = Median load effect on a platform

$(\sigma_{\ln R_m})^2$ = Variance of log of the median ultimate strength

$(\sigma_{\ln S_m})^2$ = Variance of log of the median load effect

The above expression can be rewritten in the following form:

$$R_m = S_m \exp(\beta\,\sigma)$$

where σ represents combined standard deviation for R_m & S_m. Upon dividing both sides of the equation by RSR, it becomes

$$(R_m / RSR) = (S_m / RSR) \exp(\beta\,\sigma)$$

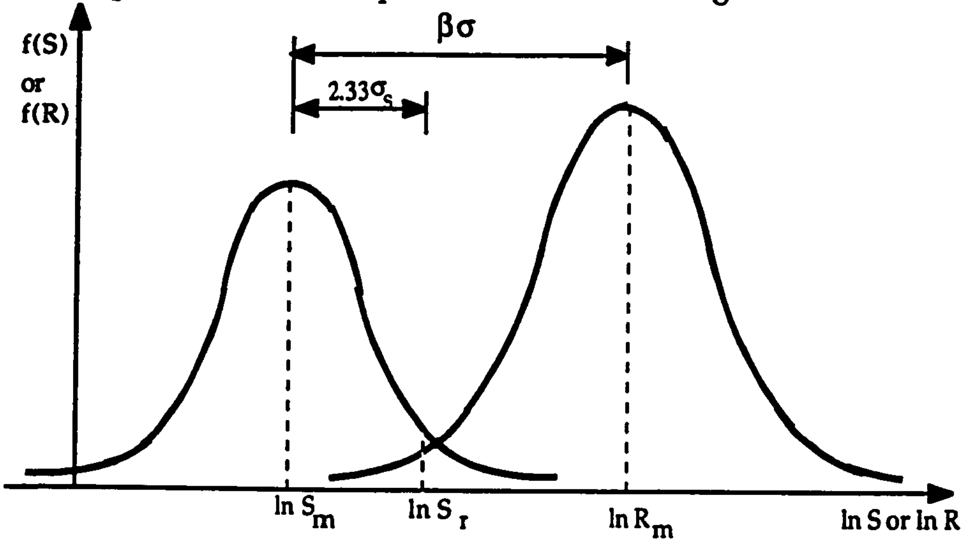
where RSR represents true RSR equal to the ratio of median ultimate strength divided by true estimate of reference level force (S_r). Therefore, the above expression reduces to:

$$S_r = (S_m / RSR) \exp(\beta \sigma)$$

or $$RSR = (S_m / S_r) \exp(\beta \sigma)$$

or $$RSR = FR \exp(\beta \sigma) \qquad ...(5.3)$$

where FR is a force ratio, β is a safety index (a measure of the platform reliability), σ expresses combined standard deviation in the loadings and capacities of the platform, and RSR is the true (mean) reserve strength ratio. As discussed in Chapter 2, R and S follow log-normal distributions, and thus the above expression could be represented as shown in Figure 5.2.



$$\ln S_r = \ln S_{100}$$
$$(RSR / FR) = R_m / S_m$$
$$= \exp(\beta\sigma)$$

Fig. 5.2: Probability Distribution of Expected Annual Maximum Wave Height and Expected Ultimate Strength of Platform

The uncertainty measure, σ is obtained by combining the standard deviation of the log of the loading, σ_s, and the standard deviation of the log of the capacity, σ_R. The standard deviation (or variance) in the consequence level of a platform could also be included in σ as follows:

$$\sigma = [(\sigma_{\ln R_m})^2 + (\sigma_{\ln S_m})^2 + (\sigma_{\ln C})^2]^{1/2}$$

The force ratio is the ratio of the median annual expected maximum force on the platform, S_m, to the reference level force, S_r:

$$FR = S_m / S_r \qquad \text{...(5.4)}$$

S_r would normally be represented as the load value with 0.01 or 1% probability of exceedence per year or which is exceeded once every 100 years. S_m represents the median load acting on the platform.

The expected annual maximum wave force is assumed to follow a log-normal distribution, due to the properties of the central limit theorem for a model with the multiplication of a number of parameters. In an assumed log-normal probability distribution of the expected annual maximum wave heights at the platform site as shown in Fig. 5.2, the probability of exceedence of 1% would correspond to the value at (2.3263 x standard deviation) above the median value, S_m.

$$\ln S_r = \ln S_m + 2.33\ \sigma_s$$

$$\ln (S_m / S_r) = -2.33\ \sigma_s$$

$$FR = \exp(-\ 2.33\ \sigma_s) \qquad \text{...(5.5)}$$

Therefore, the force ratio (FR) at 1% probability of exceedence depends only upon the variance in the loads acting on a platform. In case of 2% probability of exceedence (or 50 year return period), the 50-year return period force level will correspond to the value at ($2.05\ \sigma_s$) above the median value, and the force ratio, $FR = \exp(-\ 2.05\ \sigma_s)$. In case of 5% probability of exceedence (or 20 year return period), the 20-year return period force level will correspond to the value at ($1.65\ \sigma_s$) above the median value, and the force ratio, $FR = \exp(-\ 1.65\ \sigma_s)$.

The difference in values of force ratios, FR with the return period and variation in load effect standard deviation (σ_s) is demonstrated in the following table.

σ_s	Force Ratio, FR for S_{100}	S_{50}	S_{20}
0.25	0.560	0.600	0.660
0.50	0.310	0.360	0.440
0.75	0.174	0.215	0.290
1.00	0.097	0.129	0.190

From the above table, it is noted that change in force ratio (FR) is more significant due to change in the uncertainty level associated with the load effect than due to change in the return period associated with the annual maximum wave height distribution.

S_m corresponds to the load due to the annual mean wave height, H, at the platform site with a standard deviation (σ_H). The standard deviation of the expected annual maximum wave force (median load), σ_s, will include the uncertainties due to inherent randomness of parameters and modelling as described in section 2.3.3. The maximum total lateral force developed on a platform, S_m, is a function of the expected **annual** maximum wave height, H_m, raised to an exponent α, then:

$$S_m = K_D \, K_i \, (H_m)^{\alpha} \qquad \text{... (5.6)}$$

where K_D is a hydrodynamic force coefficient, and K_i is a wave-current kinematics integrating coefficient. The wave height exponent (α) is approximately equal to 2.0 for drag force dominated platforms and designed according to API-RP-2A guidelines. The value of α could be as high as 2.5 for the platforms with wave into the deck. More details in evaluation of S_m were given in section 2.3.

The median annual maximum wave force corresponds to the 50th percentile value in a cumulative wave height distribution, F(h). The average return period, ARP, associated with the median wave height can be determined from the following formulation:

$$\mathbf{ARP} = 1 / [1 - F(H)] \qquad \text{...(5.7)}$$
$$= 1 / [1 - 0.5] \qquad = 2 \text{ years}$$

where F(H) expresses that the annual expected maximum wave height, H, at a structure location is equal to or less than a given value, h. Hence, the median value of the annual maximum wave force distribution would represent the wave force level corresponding to the lateral force based on a 2-year return period wave height. Therefore, β in expression 5.3 would represent a safety level against the maximum expected annual force level.

The RSR expression in (5.3) can be rewritten upon substitution of FR from expression (5.5) as follows:

$$\mathbf{RSR} = \exp(\beta \sigma - 2.33\ \sigma_s) \qquad \text{...(5.8)}$$

This description of RSR is demonstrated pictorially in Fig. 5.2. Therefore, RSR essentially depends upon the uncertainties in load (load effects), strength, and consequences, and the acceptable annual risk level for a platform. Normally, the standard deviation or variance in load (load effects) on a platform is very high compared to the variance in strength. So, the variance in load (load effect) plays a major role in the evaluation of an acceptable RSR. The uncertainty level associated with loads (load effects) would vary for a type of platform, water depth, and the computation method. In addition, it also depends upon the uncertainties associated with the prediction of annual maximum wave height for the particular site.

Hence for the given characteristics of a platform, a safety level acceptable to a public-regulatory body or the RSR required for a platform could be obtained.

β reflects the desirable level of reliability for a platform. The safety index is fundamentally a function of the level of consequences associated with the failure of a platform. Utility, cost-benefit evaluations and historic, standard-of-practice evaluations can be used to define this quantity. A detailed description of these methods for evaluation of these two methods has been covered by a number of researchers. The specific works reported by *Bea (1990), Jordaan (1988), Melchers (1987), Reid (1990), Vrijling (1989)* are related to this study and have been used in the following descriptions of the acceptability criteria.

The safety index, β is related to the probability of failure (P_f) of a platform by the following standard normal distribution (zero mean and unit variance) [*Melchers, 1989*]:

$$P_f = P(R - S \leq 0) = P(Z \leq 0) = \Phi\,[(0 - \mu_Z) / \sigma_Z]$$

where Z = Safety margin

$$\mu_Z = \mu_R - \mu_S$$

$$[\sigma_Z]^2 = [\sigma_R]^2 + [\sigma_S]^2$$

The above equation could be written as follows:

$$P_f = \Phi\,[-(\mu_R - \mu_S) / \{(\sigma_R)^2 + (\sigma_S)^2\}^{1/2}\ ] = \Phi\,[-\beta]$$

or $\quad P_f = 1 - \Phi\,[\beta]$ ...(5.9)

The values of β and $\Phi\,[-\beta]$ for the standard normal distribution N(0,1) are given in a table in Appendix -D for ready use. Alternately, it could be obtained by the following formulation which gives good results for β between 1 to 3 [*Bea, 1990*].

$$P_f = 0.475 \exp\,[-\beta^{1.6}]$$

or $\beta = [-\ln(2.105\, P_f)]^{0.625}$ ...(5.10)

The probability of failure in the above expressions, which would be acceptable to the public-regulatory bodies could be obtained by comparison with the historic risk acceptable to the society for offshore platforms in a particular geographical region, or by utility (cost-benefit) evaluations.

Socially acceptable level of risk: *Bea (1990)* utilized the characterization presented by *Whitman (1984)*, as shown in Fig. 5.3, which summarizes annual probabilities of failure and consequences associated with a wide variety of engineered structures and facilities. The probabilities of failure have been based on historic rate of accidents (failures) and the consequences are based on the ranges of monetary costs, and/ or fatalities that have been associated with the accidents. The two lines shown in Fig. 5.3 could be termed as "acceptable" or "marginal" combinations of likelihood of consequences. These lines have been expressed analytically by *Bea (1990)* as follows:

$$P_{ft} = 10^{-(0.74 \log C + 1.12)} \quad \text{...(5.11)}$$

$$P_{fm} = 10^{-(0.60 \log C + 0.95)} \quad \text{...(5.12)}$$

where P_{ft} represents the tolerable annual probability of failure, P_{fm} is the marginal annual probability of failure, and C is the present value of the total cost or the number of fatalities or a combination thereof associated with the failure of a platform.

The positioning of the two lines in Fig. 5.3 would vary depending on the geographical location and with the trade-off between consequences and risk acceptable to the parties involved. Note that over a period of time, the risk level acceptable to the society could change due to socio-political reasons. In such a case, the formulations presented above would have to be modified.

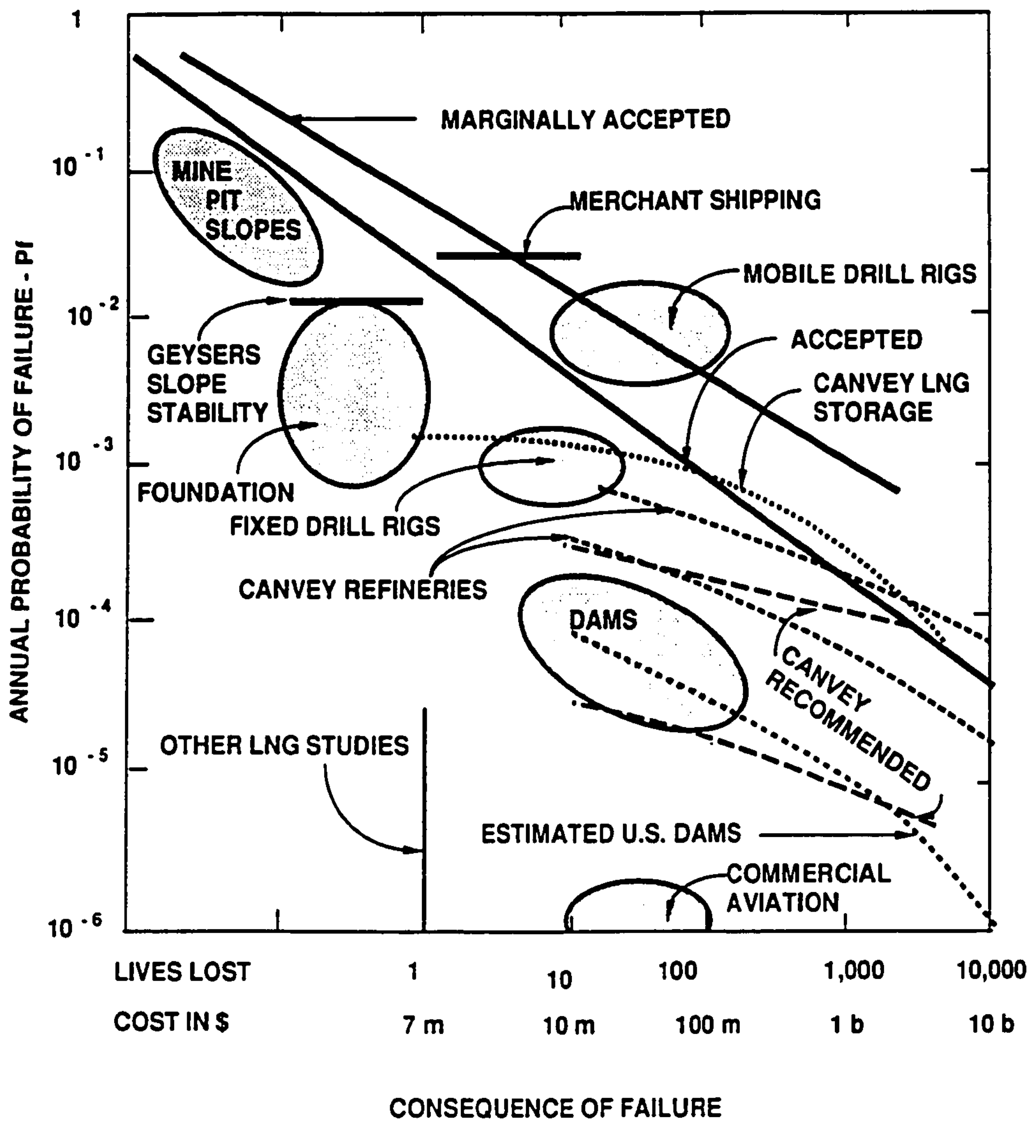


Figure 5.3: Historical Relationship of Risk and Consequences for Engineered Structures *[Bea, 1990]*

Utility (cost - benefit) evaluation: The cost-benefit evaluation provides an analytical approach based on which the acceptable risk level (or probability of failure) could be determined corresponding to the minimum cost or maximum utility of a platform. The details of this evaluation has been covered by *Stahl (1986), Bea (1990)* and others.

The approach is based on determination of expected total associated with failure of a platform, which is computed as summation of the products of present value estimates of individual costs and the likelihood of experiencing those costs.

$$E(C_t) = E(C_i) + E(C_f) \qquad \text{...(5.13)}$$

or $$E(C_t) = C_i + [C_f (PVF)]\, P_{fa} \qquad \text{...(5.14)}$$

where, C_i represents the initial cost associated with the platform and the risk cost (C_f, a consequence measure) would include the costs associated with injuries and fatalities, environmental damage, productivity loss, property loss, and resource development loss. PVF represents the present value function used to discount the future risk costs listed above to the present value. Pfa represents the annual likelihood of experiencing these costs.

The initial cost could be written as follows:

$$C_i = C_o + C \log_{10} P_f \qquad \text{...(5.15)}$$

where Co and C are the constants as shown in Fig. 5.4. The cost of a safer structure increases linearly with $(-\log P_f) = \log(1/P_f)$.

By substituting the above equations (5.14 and 5.15) into total cost equation (5.13), and optimizing for maximum utility the equation w.r.t. P_f, by differentiating and equating to zero, we would obtain the following expression:

$$P_{fa} = 0.435 \,/\, [(PVF)\,(CR)] \qquad \text{...(5.16)}$$

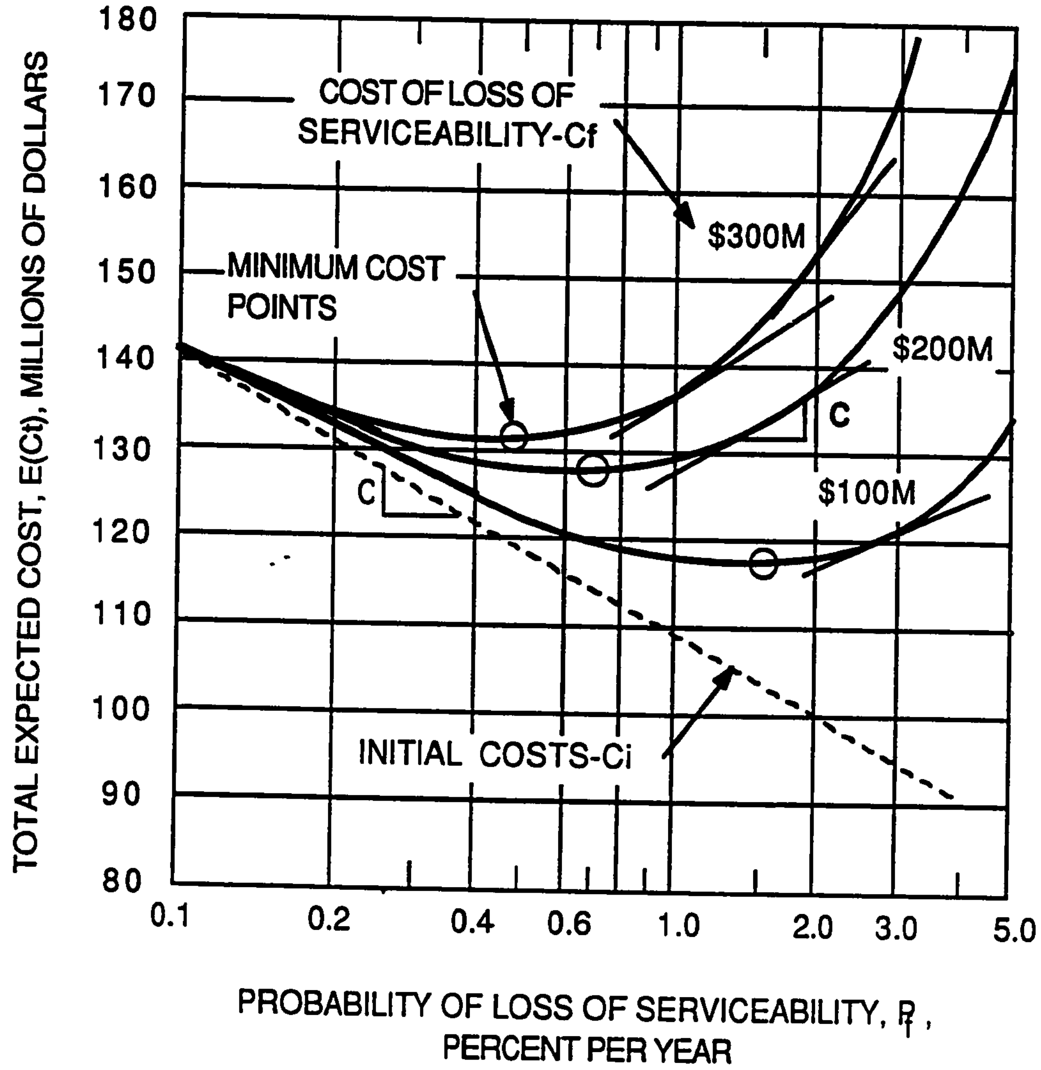


Figure 5.4. Optimized Utility - Cost - Benefit Evaluation [Bea, 1990]

where CR is the ratio of C_f and C. C represents the cost needed to decrease the annual probability of failure for the platform by a factor of 10. C equals the slope of C_i in Fig.5.4.

Upon eualuation of the acceptable probability of failure, P_{fa} of a platform by equation 5.16, the required safety index could be determined by equation 5.9 by using the standard normal distribution table provided in Appendix-G. Alternately, safety index, β could be determined by using simplified equation 5.10. In this way, we would obtain the acceptable safety index.

If the marginal probability of failure is considered as two times the acceptable probability of failure, then

$$P_{fm} = 0.870 \ / \ [(PVF)\ (CR)] \qquad ...(5.17)$$

From the history of failure of offshore platforms due to various reasons, it has been reported that approximately 20 % of all the failures are due to storm waves [*Bea, 1990*]. In this study, the failures of platforms due to storm waves have only been considered and 20% failure probability has been assumed to demonstrate the process, and the following expressions for acceptable and marginal probabilities of failure have been used.

$$P_{fa} = 0.087 \ / \ [(PVF)\ (CR)] \qquad ...(5.18)$$

$$P_{fm} = 0.174 \ / \ [(PVF)\ (CR)] \qquad ...(5.19)$$

Upon substituting these values of P_f in equation 5.10, the following expressions for acceptable and marginal safety index are obtained:

$$\beta_a = [-\ln \{0.183 \ / \ (PVF)\ (CR)\}]^{0.625} \qquad ...(5.20)$$

$$\beta_m = [-\ln \{0.366 \ / \ (PVF)\ (CR)\}]^{0.625} \qquad ...(5.21)$$

In order to perform a more realistic establishment of the acceptability criteria, it would be prudent to do more research on the sources of failures for different kinds of platforms, e.g., well platforms, production platforms, process platforms, living quarters platforms, riser platforms, etc. and establish more realistic likelihood of failure associated with the storm waves. Note that the consequences for each of these platforms would vary. Therefore, difference in acceptability criteria could result.

Given these characterizations of acceptable and marginal safety index, β (5.20, 5.21), and the values of uncertainties associated with load effects, strength, and consequence levels of a platform, the corresponding acceptable and marginally acceptable values of RSR could be determined by using the formulation derived in equation 5.3. From the equation 5.3, the definition of the acceptable, marginal, and unacceptable combinations of RSR and consequences (reflected in the safety index by product of PVF and CR), for the given force ratios and the loading-strength-consequence uncertainties could be obtained.

Force ratio (FR) can vary with the platform, its location, and water depth due to variation in uncertainty levels associated with load effects (σ_S). For the Gulf of Mexico, in medium to shallow water depth range, FR is likely to vary between 0.1 to 0.3. Loading-strength uncertainties (σ) can vary with the platform, and the extent of information on the present condition and past loading experiences of the particular structure. The safety index can vary as a function of the non-hurricane related hazards (fire, explosions, blow-outs, collisions), that can threaten the safety of a platform. The influence of variations in these parameters on the acceptability criteria is studied in detail in the next section.

Note that the RSR in expression 5.3 is based on the median values of load and strength and thus all the biases associated with the nominal estimates of load and strength should be included when using this formulation. The biases could be included by the following expression:

$$RSR = (RSR_n)\ (BR_{RSR}) \qquad ...(5.22)$$

where RSR_n is obtained by the methods described in Chapter 3. BR would vary with the method used in evaluation of RSR_n and on the characteristics of a platform. RSR obtained would represent the true RSR of a platform, and it should be used in making a decision on the safety level of a platform.

Example: An example representation of equation 5.3 is shown in Fig. 5.5. In this example the load uncertainty and the strength uncertainty have been assumed as 0.75 and 0.25 respectively. Therefore, the total uncertainty level (σ) is 0.79. It has been assumed in this example that 20% of the total risk is associated with the hurricane load and rest 80% of risk is associated with the other load sources which have not been considered in this study. Therefore, in this example $(P_f)_{storm} = 0.2\ (P_f)_{total}$. Corresponding to load uncertainty of 0.75, the force ratio, FR would be 0.17 by equation 5.5. In this example, the consequences (C) have been considered in monetary terms and are represented as products of present value function (PVF) and cost ratio (CR). Then utilizing the formulations presented for acceptability criteria (acceptable and marginal safety index), the acceptable and marginal RSR values for a particular consequence level are determined. These results have been plotted in Fig. 5.5.

Based on the availability of the improved data for a platform at higher screening cycles and due to reduction in the load-strength-consequence uncertainty level, equation 5.3 would change. Therefore, the positioning of the two lines shown in Fig. 5.5, which defines the acceptable and marginally acceptable safety level of platforms to the public regulatory bodies, would also change. In addition, the width of the marginal safety band would also change with uncertainty level.

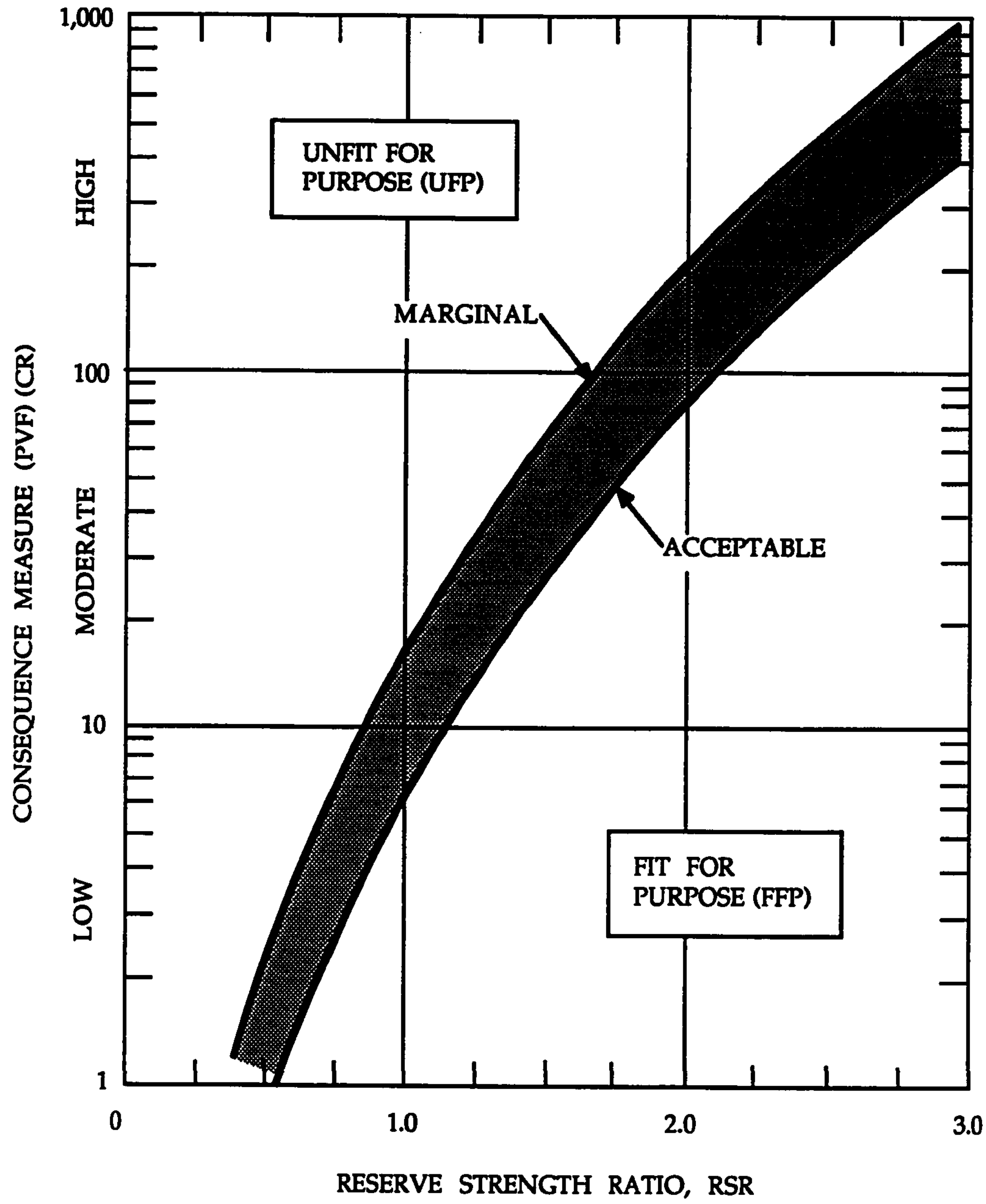


Figure 5.5: Fitness For Purpose (FFP) as Function of Consequences, RSR, and Uncertainties Associated

The platforms which fail to qualify as FFP at a screening cycle would need one of the following actions:

a) to carry out an evaluation of RSR at a higher screening cycle with improved techniques, with associated reduced uncertainties;

b) to take the steps to increase the RSR level or reduce the consequence level;

c) to decommission the platform.

In the next section, a parametric study performed for the influence of variations in load, strength, and consequence parameters on making a decision on safety of a platform is presented.

5.4 Parametric Study

In this section, a parametric study of the characterization (equation 5.3) used for fitness for purpose evaluation of a platform has been made. This formulation is given as below:

$$\mathbf{RSR = FR\ exp\,(\beta\sigma)}$$

where, RSR represents desired level of reserve strength of a platform, **FR** is the ratio of the median annual wave force to the reference level wave force, β represents the safety index for the platform acceptable to the public-regulatory bodies, and σ includes the variance in the estimates of load, strength, and consequence for a platform.

The median annual force corresponds to a 2-year return period force level. The standard deviation associated with load effects varies between 0.50 to 1.00 for the Gulf of Mexico [*Olufsen and Bea, 1990*]. Therefore, FR would vary between 0.1 to 0.3.

The safety index could be expressed as proportional to the inverse of the probability of failure of a platform. Therefore, it would vary with the probability of failure for a category of platforms in a given geographical region acceptable to the public-regulatory bodies. The consequence level of a platform embedded in the expressions of acceptability criteria would vary for a similar platform but in different geographical region.

Total P_f of a platform is dependent upon various sources of failure of a platform, such as environmental overload, collision, dropped objects, blowouts, fire, explosions, war (missiles), etc. In this study, the sole hazard source considered is wave storms or hurricanes. From previous database available on failure of the platforms related to various hazard sources, it has been observed that approximately 20% of the failure or damages occurred from wave storms or hurricanes [*Bea, 1990*]. However, it would vary with the structural characteristics of a platform, design criteria acceptable to regulatory bodies, geographical region, and various other factors discussed earlier. Therefore, in this section, a range of 20% to 100% has been considered for the risk level associated with wave storms or hurricanes.

The standard deviation (an uncertainty measure) varies for different screening cycles due to the state of knowledge of platform and environment, and the methods involved in computations of load, strength, and consequence. The standard deviation would be the highest for screening cycle-1 and lowest for screening cycle-4. At screening cycle-4, the type-II uncertainties would reduce significantly and the standard deviation would be closer to the type-I standard deviation. In order to demonstrate the influence of variation in standard deviation on FFP evaluation, a range of σ from 0.0 to 1.0 has been taken.

The parametric study presented in the following sub-sections is based on standard deviation in load and strength only. This study has been demonstrated on an example platform taken from published literature [*Bea et al, 1988*].

As the consequence level influence β or P_f, the uncertainties associated with the evaluation of the consequence level would also influence the σ and the allowable and marginal RSR.

In this study, the fitness for purpose of a platform is evaluated against the environmental overload. In addition, the emphasis is on the suitability of the structure instead of the cost effectiveness to produce the remaining reserves. An important point to note here is that, by following this methodology, the adequacy of the platform structure to sustain the environmental overload can be evaluated. Thereafter, a decision to produce or not-produce may also require a cost-benefit evaluation of the risk mitigation measures or of the measures needed for salvage of the platform by an operator. Alternatively, an operator may modify the acceptability criteria and more detailed consequence evaluation for that particular platform and make a decision on the basis of methodology presented in this study.

The parametric study in this section is based on an assumption that σ depends upon the variances in load and strength estimates only. An example platform has been taken from the published literature to describe the results.

5.4.1 Influence of Variation in σ_S:

The standard deviation (σ_r) of the strength is kept constant at 0.25 and the standard deviation of the load effects, σ_S corresponding to the reference level force is varied from 0.5 to 1.0. Hence, the combined standard deviation (σ) would vary from 0.56 to 1.03. The force ratio, FR would vary from 0.10 to 0.31 for the load effect uncertainty varying from 1.0 to 0.5 respectively.

The FFP expressions for acceptable and marginal β values obtained by **utility-cost benefit evaluation have** been used. For different consequence values between a range of 1 to 1,000, which represents utility effect in this case, the acceptable and marginally acceptable values of RSR have been obtained using expression 5.3 and substituting values of β, σ_S, σ. Such curves were developed for σ_S values as 0.50, 0.75, 1.0 and are shown in Fig. 5.6. From this Figure, the following observations could be made:

By variation of the standard deviation of load effect the width, location, and slope of marginal zone would change as shown in Fig. 5.6. The curvature of the marginal zone increases with an increase in the standard deviation of the load effects and its width would be more for high consequence level than due to the low consequence level. The marginal band tends to move towards nearly linear relationship between RSR and consequence levels, with reduction in variance associated with load effects. From computations, it is found that for σ_S equal to 0.25, a nearly linear relationship exists within ranges of RSR as 1.0 to 2.0, and of consequence as 1 to 1,000.

For illustration purpose, for σ_S varying between 0.5 to 1.0, the ranges of RSR are measured corresponding to ranges of consequence measure (PVF x CR). In case of high consequence level (between 100 to 1,000), the required RSR would range from 1.6 to 3.0. In case of moderate consequence level (between 10 to 100), the required RSR would range from 0.85 to 2.4. For low consequence level (between 1 to 10), the required RSR would range between 0.25 to 1.2.

For a platform falling in high consequence zone, a significantly high RSR would be required to satisfy the acceptability criteria required by public-regulatory bodies, when the uncertainties associated with load effects are high. For example in Fig. 5.4, when consequence level for a candidate

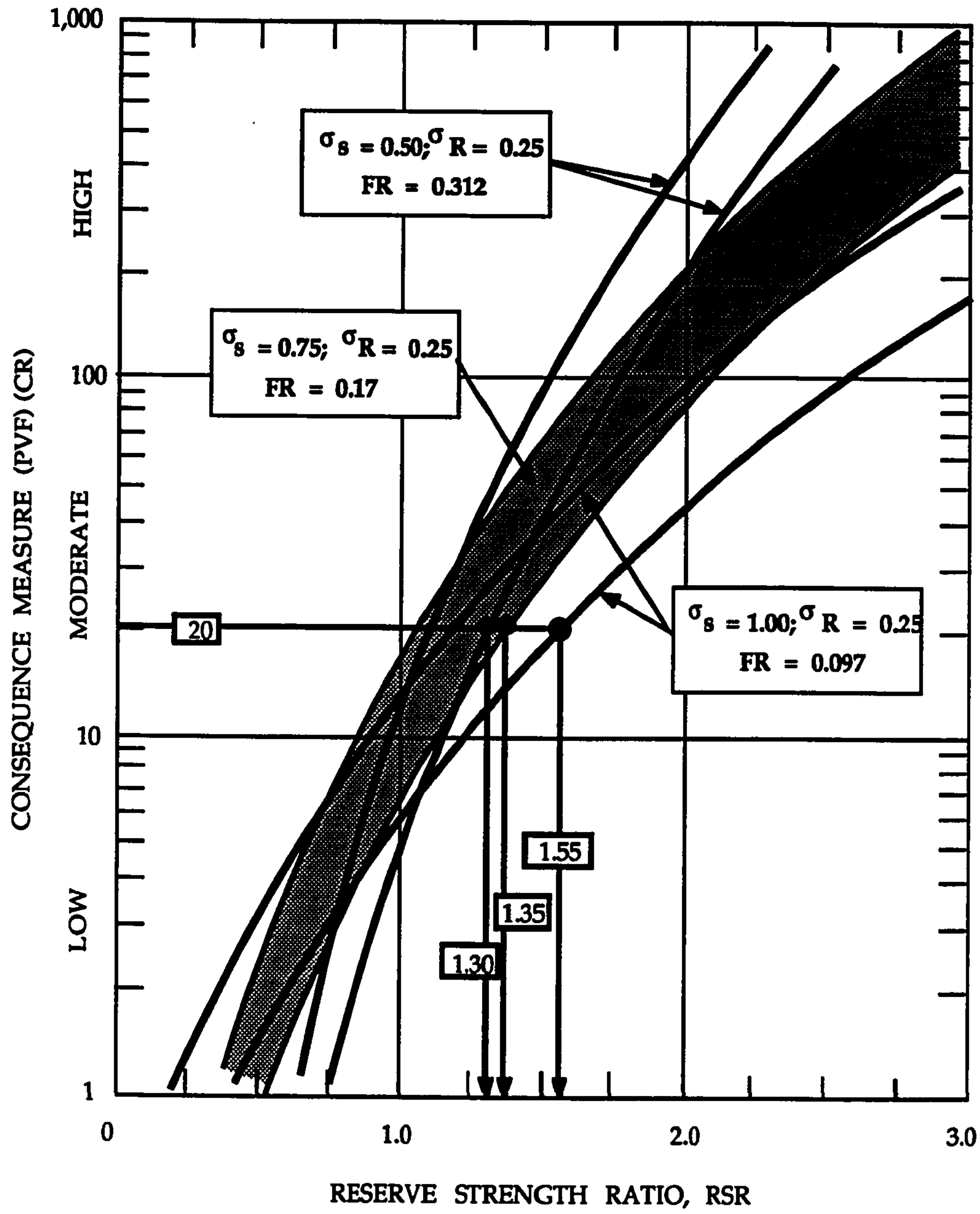


Figure 5.6: Influence of Variation in Uncertainty Level in Load Effects on Fitness For Purpose (FFP) Criteria

platform is 20, the acceptable RSR would be 2.0, 2.50, or 3.05 for load effect uncertainty of 0.5, 0.75, or 1.0 respectively. Therefore, implementation of measures to reduce uncertainty in load effects could be a cost-effective measure for such a platform.

For a platform falling in the medium consequence zone, the location of the marginal band and its width do not vary significantly for the different σ_S levels. For example, for a platform with consequence level of 20, the acceptable RSR to meet the safety criteria by regulatory bodies would be 1.3, 1.35, or 1.55 for σ_S of 0.5, 0.75, or 1.0 respectively. Hence, reduction in load effect uncertainty may not necessarily make a platform fit for purpose.

At low consequence level, the required values of acceptable and marginal RSR increase with a reduction in σ_S. It is observed that when the standard deviation is very high, a platform with even very low RSR could be classified as FFP.

From this parametric study, it is observed that use of sophisticated methods for evaluation of load effects would be desirable for platforms with high consequence levels. For such platforms, an investigation to nearly eliminate type-II uncertainties (modelling uncertainties) could be a cost effective measure.

5.4.2 Influence of Variation in σ_R:

The uncertainty level associated with the ultimate strength of a platform, σ_R is assumed in this section to vary from 0.05 to 0.55, and the load uncertainty, σ_S is assumed constant at 0.75. It implies that force ratio, FR is equal to 0.17 and total uncertainty level would vary between 0.75 to 0.90. The FFP criteria equations are plotted on Fig. 5.7 for different values of

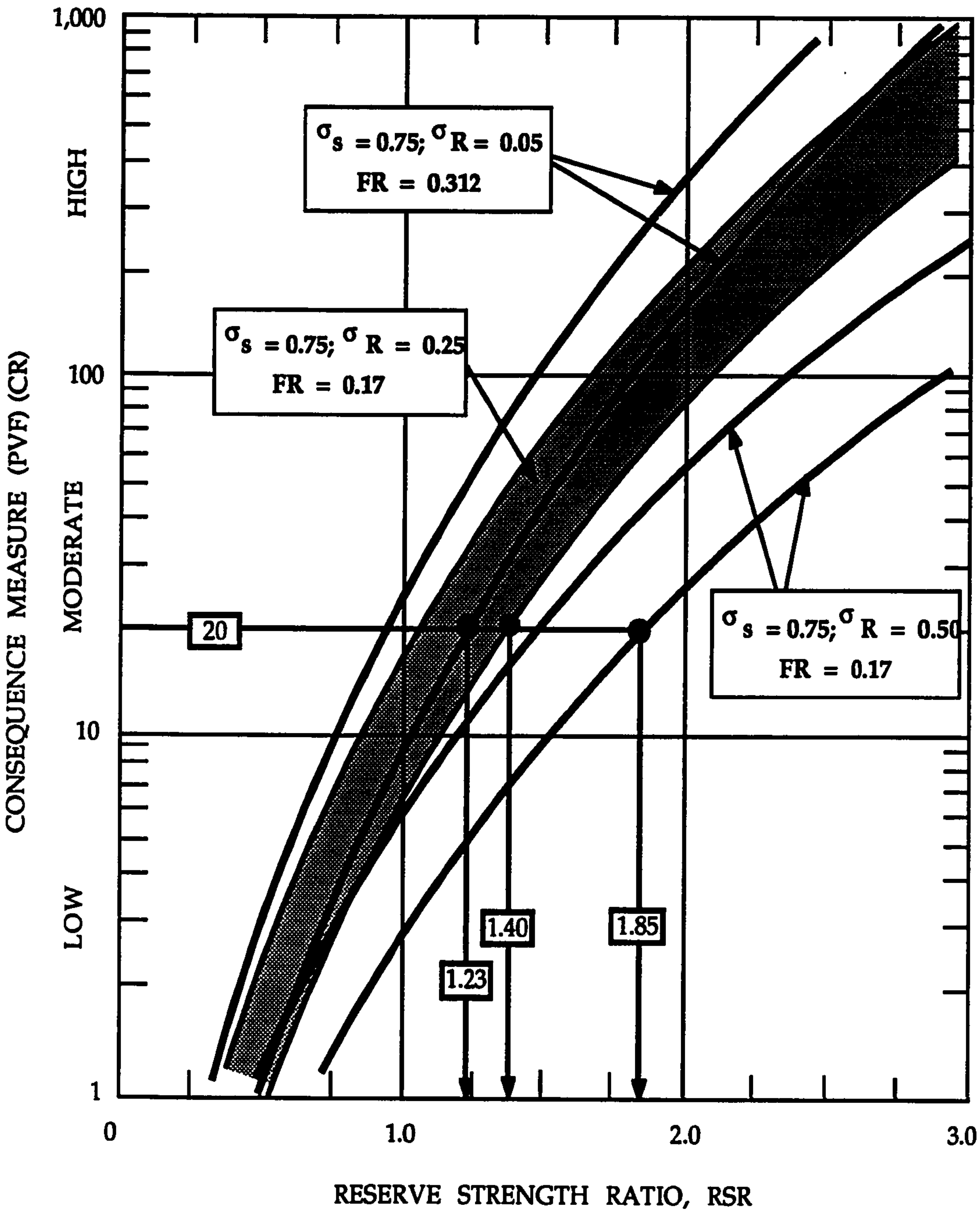


Figure 5.7: Influence of Variation in Uncertainty Level in Ultimate Strength on Fitness For Purpose (FFP) Criteria

consequence level and strength uncertainty level as 0.05, 0.25. and 0.75 respectively.

From Fig. 5.7, it is observed that the width of marginal safety band does not change significantly for a particular level of strength uncertainty due to changes in RSR or consequence levels of a platform. Whereas, it was found significant in case of load uncertainty. The reason for this difference is that FR is determined on the basis of load uncertainty level.

For a particular consequence level for a platform, the acceptable RSR values would differ and a consistent variation in RSR is noted.

From Fig. 5.7, it is seen that the variation is nearly linear in case of low consequence platforms, and the relationship (RSR vs. consequence) becomes curvilinear with increase in consequence level. In case of high to very high consequence platforms, a significant benefit could be achieved by reduction in strength uncertainty level.

From Fig.5.7, it is observed that for a consequence level equal to 20, the RSR should exceed 1.23, 1.45, and 1.85 for strength uncertainty levels as 0.05, 0.25, and 0.50 respectively. to meet the public-regulatory combined. For a consequence level equal to 200, the RSR should exceed 2.1, 2.45, or 3.5 for strength uncertainty level of 0.05, 0.25, or 0.50 respectively.

From this study, it is found that a reduction in strength uncertainty of members would be a cost effective alternative to improve safety of a platform. Note that the strength uncertainty level of 0.50 could occur for a platform with damaged members. For a platform with medium consequence level of 20 and with an increase in strength uncertainty level from 0.25 to 0.5, the required RSR would be approximately 32 % higher than would be needed for a case with strength uncertainty of 0.25. On the contrary at this consequence level, the required RSR would be only 14 % less for a case with σ_R of 0.05 when compared to the base case with σ_R of 0.25.

The old existing platforms, which in some cases are corroded, cracked at joints or welds (fatigue effect), deteriorated, etc., the strength uncertainty level is likely to be high. Therefore, in such cases, a significant benefit could be achieved by a more detailed determination of the as-is damaged state of the platform. Improved models for strength evaluation of damaged members could be applied in order to reduce uncertainty level and thus improve the likelihood of FFP classification of the platform.

5.4.3 Influence of Variation in risk level associated with storm loads:

The base case shown in Fig. 5.3 was based on an assumption that the wave storm loads correspond to only 20% of the total risk associated with the platform. The rest 80% of the risk may be associated with likelihood of failure of the platform due to other sources such as: earthquake, boat/ ship/ submarine collision, dropped objects, fire, etc.

The percentage risk associated with wave storm influence the acceptability criteria as demonstrated in section 5.3. In this sub-section, a parametric study is presented for a case, when measures have been taken to eliminate risk to the platform due to all other sources except the risk due to wave storms. Therefore, in such a case the risk level associated with storm loads could be taken as 100% ($P_{f\ storm} = 1.0\ P_{f\ total}$).

In such a case, the safety index equations for acceptable and marginal β would change as follows:

$$(\beta_a)_{storm} = [-\ln \{0.915 /(PVF \times CR)\}]^{0.625}$$

$$(\beta_m)_{storm} = [-\ln \{1.83 /(PVF \times CR)\}]^{0.625}$$

Based on these, the FFP characterization is plotted as shown in Fig. 5.8 and compared to the base characterization with only 20% risk level associated with storms.

From this figure, it is observed that for a platform with consequence level equal to 50, the required RSR would reduce to 1.1 for 100% storm risk case compared to 1.8 for the 20% storm risk case. It corresponds to 40% reduction in required RSR for the platform to be classified as fit for purpose at a consequence level of 50.

Therefore, in case of old, damaged platforms with reduced crude flow rates and reduced remaining reserves in the reservoir, a cost-benefit evaluation of the various measures which could reduce or eliminate other risks associated with the operation of the platform and the measures required to upgrade the platform to increase its RSR could become a fruitful exercise. In some cases, it may turn out to be cost effective to reduce operations on a platform than to carry out extensive underwater operations to increase strength of the platform.

5.5 Summary

Reliability based method for fitness for purpose evaluation of a platform in its existing state has been presented. The fitness for purpose evaluation is based on risk assessment and risk acceptability phases. The risk level associated with a platform in its existing intact, deteriorated, or damaged state is determined by evaluation of its likelihood of failure and the consequences associated with its failure. The likelihood of failure of a platform is dependent upon its capacity level. The capacity level and consequence level for a platform could be determined by qualitative and quantitative methods discussed in Chapter 3 and Chapter 4 respectively. The

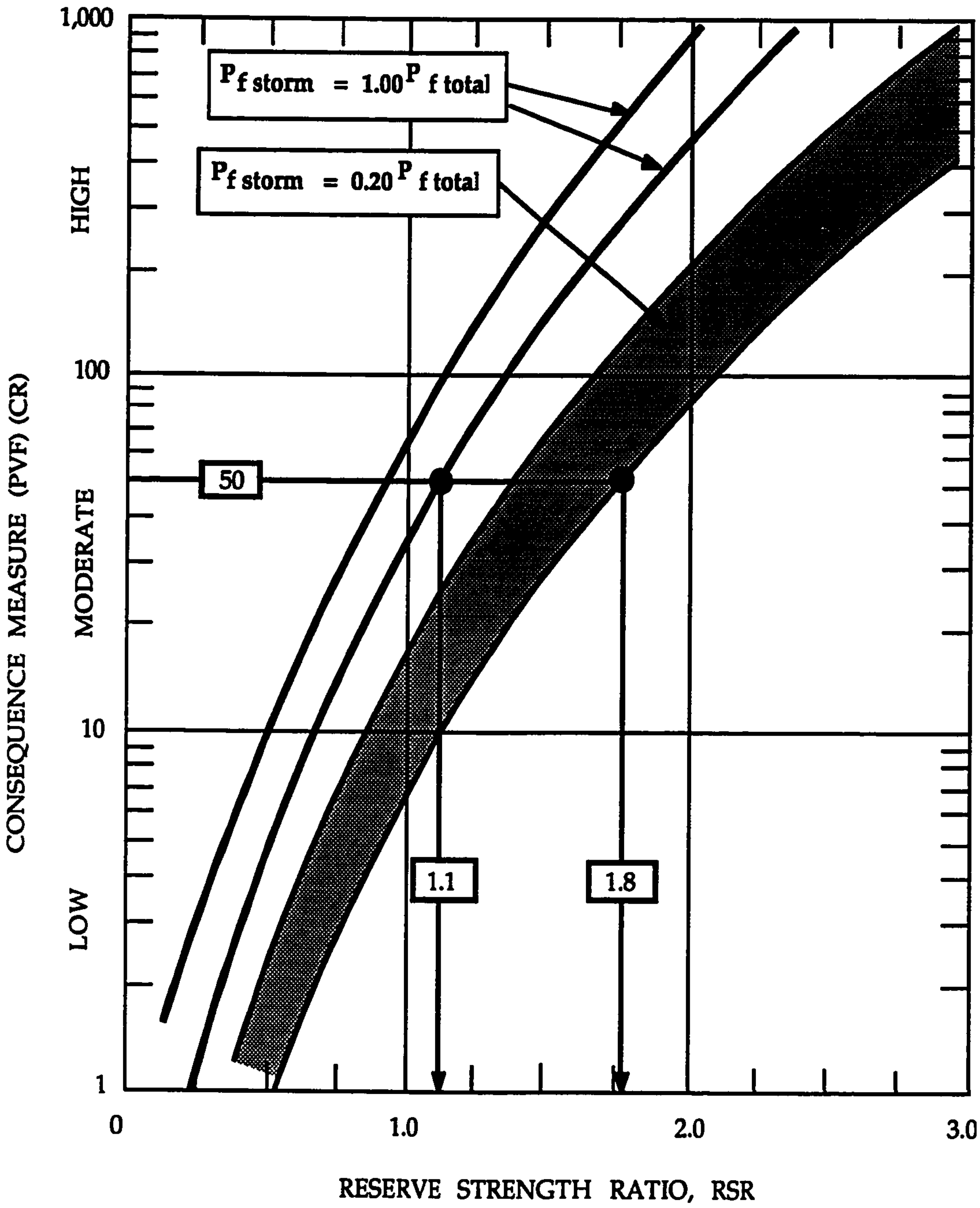


Figure 5.8: Influence of Variation in Risk Level Associated with Wave Storm on the Fitness For Purpose (FFP) Criteria

corresponding methods for qualitative and quantitative assessments of fitness for purpose of a platform were presented in this chapter.

The assumed ranges based on qualitative evaluations of capacity and consequence levels are presented, which could change with preferences of a regulator, operator, or user for a particular geographical region and type of platform.

The characterization of the quantitative method for fitness for purpose evaluation originates from well known safety index formulation used in conventional reliability analysis. Its derivation is given in full detail, and the primary variable parameters are identified. The parameters are: reserve strength ratio (RSR), uncertainties (bias, variances) in load and strength estimates, consequences, and acceptable annual risk level.

The risk acceptability to the public-regulatory bodies depends on several factors and could be obtained by comparison with the historical risk acceptable to the society for offshore platforms in a particular geographical region, or by utility (cost-benefit) evaluations.

The characterization was demonstrated for an example platform. A parametric study was presented for variations in important parameters. It was demonstrated that significant advantages could be achieved by reduction in uncertainty level associated with load effects and by reduction in the risk level associated with other hazards.

CHAPTER 6

RISK MANAGEMENT PROGRAM

A risk management program is needed for a platform to maintain it in a fit-for-purpose (FFP) state over its remaining life to perform the originally planned or modified operations. In order to undertake such a program on a continuing basis, the following tasks are the most important:

1) to periodically obtain the details of a platform in its "as-is" state;

2) to obtain and evaluate the effectiveness of the proposed Inspection, Maintenance, and Repair (IMR) program for a platform and to check that it meets the regulatory guidelines;

3) to evaluate the influence of defects, damages, and modifications (physical and operational) made to a platform on its capacity and consequence levels;

4) to evaluate the various means to upgrade the safety level of the platform and to improve its safety level from "UFP" and "Marginal" zones to the "FFP" zone;

5) to select an optimum program to maintain the safety level of a platform based on its cost-benefit evaluation.

These important tasks could be considered to fall into "Risk Identification," "Risk Assessment," "Risk Mitigation," and "Risk Maintenance" phases. These tasks are important for several reasons: to prevent economic losses through accidents liable to cause a production shutdown or loss of operation; to meet the requirements of government regulatory, insurance companies and certifying bodies; for owners benefits;

and to ensure a provision of the required measures for safety of the site personnel.

The "**Risk Identification**" phase requires implementation of a periodic survey/ inspection program and maintenance of records. The recommendations for in-place surveys to monitor the adequacy of corrosion protection system and to determine the condition of a platform are covered in API-RP-2A Section 14. The different survey levels, frequency, and important considerations are covered in these guidelines, which are followed by the regulatory bodies [MMS]. Some important considerations in establishing the condition of a platform are discussed in Section 2.2 of this study.

The "**Risk Assessment**" phase involves determination of the effects of changed conditions of a platform on its capacity and consequence levels. The changed conditions are obtained from review of as-installed drawings, design basis, and inspection records for a platform. In this phase, the influence of the changes in a platform on the attributes covered in Sections 3.2.2 and 4.2, which determines the capacity and consequence levels respectively, are evaluated. The effect of some of the deterioration and damage types on the capacity of a platform are covered in Section 2.4.

The "**Risk Mitigation**" phase focuses on identification and selection of the suitable techniques to reduce the risk level associated with a structure. The risk level can be reduced in several ways as discussed in Section 6.4.

The "**Risk Maintenance**" phase involves the measures needed to implement the selected risk-mitigation measures and to make a decision on the period for implementation of the next risk management cycle. It also includes planning for unforeseen events for a platform.

These four phases in a risk-management program for a platform have been studied by various researchers and in many recent joint industry projects. The important considerations related to this study have been discussed in the following sections in this chapter. Some of the prominent works have also been reviewed and discussed in these sections. In the literature, the tasks in the four phases have been sometimes referred to as IMR (Inspection, Maintenance, and Repair) program.

6.1 Inspection, Maintenance, and Repair (IMR) Programs:

IMR program for a platform constitutes a proposed procedure which an operator intends to implement to maintain it in a safe state and as suitable for continued service over its remaining productive life. Such a program should meet the regulatory guidelines in force. The most important roles of an IMR program are: to determine the condition of the platform; to assess the influence of changes in a platform condition from last inspection or its as-installed state on its capacity and consequence levels; to evaluate the need for upgrading; to evaluate alternate schemes for upgrading.

IMR programs are very important to maintain a platform within acceptable safety level. I (Inspection) improves the state of knowledge of the components and helps in establishing unanticipated flaws and defects. M (Maintenance) is intended to keep the structure in its safe state, to perform safe operations, and to fulfil its intended purpose. R (Repair) is intended to draw attention to the necessity to restore the capacity of a structure considering the future damages and defects possible. Fig. 6.1 presents a flow chart covering the major steps and components in an IMR program.

An IMR program may constitute of the following (Fig. 6.1):

1) Inspection criteria.

2) Methods for interpretation of significance of variation on the capacity and consequence levels.

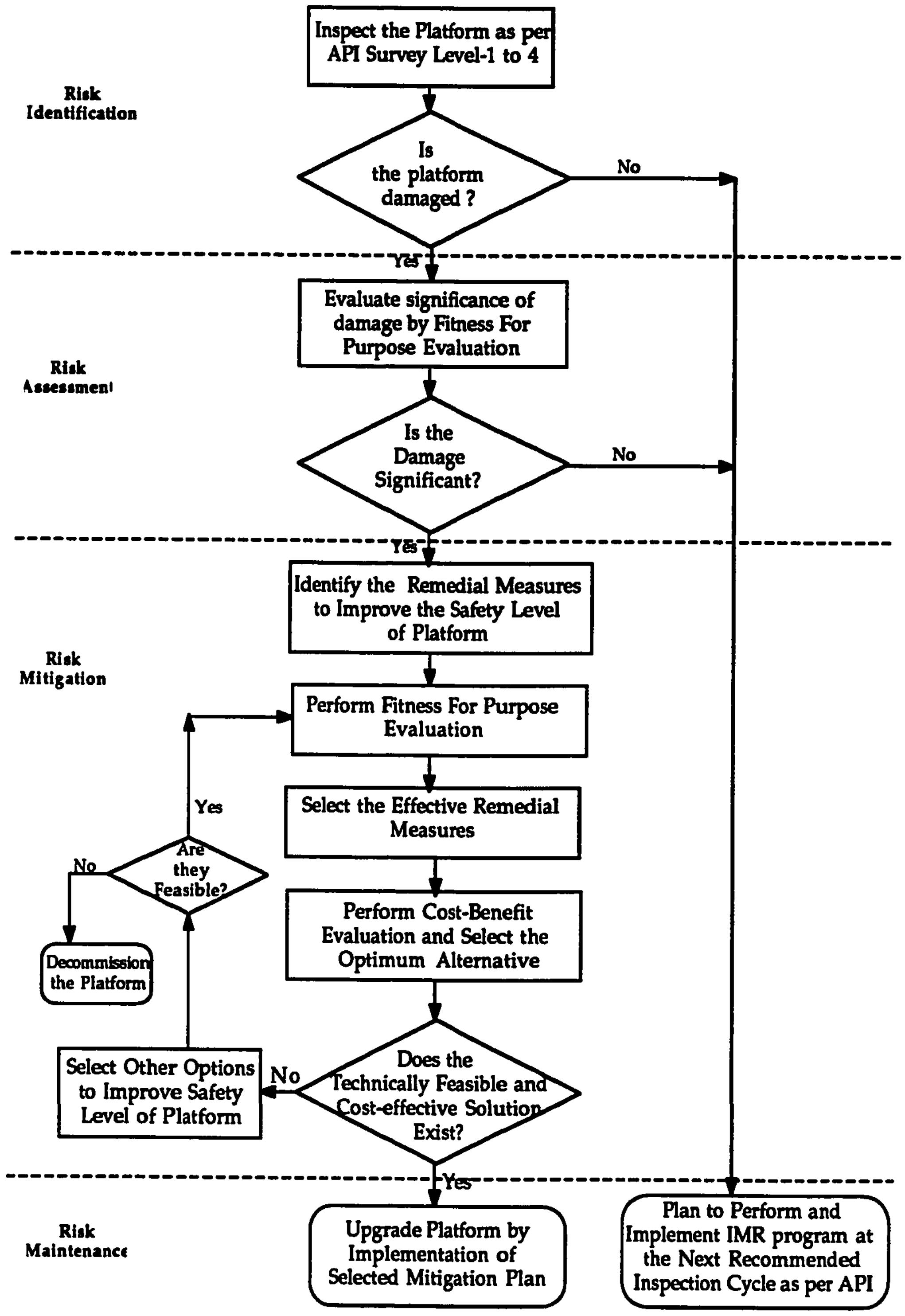


Figure 6.1: Steps in an Inspection, Maintenance, Repair (IMR) Program

3) Method to evaluate the need for upgrading (repair or retrofitting) of a platform. This method should be based on using the capacity-consequence diagrams developed.

4) Methods for techno-economic evaluation of different upgrading alternatives for the platform.

5) Methods to make a decision on implementation of the program.

Steps in an IMR program:

Inspection Criteria: In general the guidelines given in Section 7 of API-RP-2A are to be followed for inspection of platforms. The primary goal of inspection is to improve the state of knowledge of components. The different techniques used were covered in Section 2.2 and the API survey guidelines were summarized in Table 2.1.

Damage assessment: Engineering interpretation of the inspection data, determination of the type of damage, the degree of damage, and significance of damage on continued operations and the structural and foundation integrity of a platform are the key steps in the damage assessment process. The influence of damage on loads, capacity, and consequence levels of a platform are assessed. The effect of the various types of damages and deteriorations of the components on their strength were discussed in Section 2.4. The various attributes influencing consequence evaluation were covered in Section 4.1 and 4.2.

Evaluation of Significance of changes on FFP Evaluation: The capacity and consequence levels for a platform are re-evaluated in accordance with the procedures described for the respective screening cycles, and the FFP of the platform is re-evaluated on the basis of capacity-consequence evaluation as described in Chapter 5.

Upgrading measures: The platforms whose safety against the reference level lateral loads have been evaluated as "Marginal" or "UFP" are then considered for various remedial measures and upgrading techniques to improve their safety. The various techniques developed in the past to increase the capacity or reduce the consequences are discussed in Section 6.3.

Selection of the optimum upgrading technique: A selection of the optimum upgrading technique for a platform requires a techno-economic evaluation of the various upgrading schemes. Such evaluation should consider the potential for upgrading the safety level and the cost involved with the alternatives. The optimum upgrading scheme for a platform would require a cost-benefit evaluation for the different alternatives. Such an economic (cost-benefit) evaluation should consider the expected remaining life of the field, the cost of repairs, remaining safe life of the platform, etc.

Repair planning: The detailed engineering of the optimal repair procedure would be required and a detailed repair and inspection plan would be developed. An appropriate weather window would have to be selected for implementation of the repair. The maintenance strategy should consider updating of the IMR program based on evaluations made at different cycles and a decision on the next implementation of the IMR program.

6.2 Risk Identification

Inspection programs are needed for identification of risk of failure of a platform and its consequences to human life, environment, property, and other sources. The Inspection part is needed primarily to confirm the integrity of the different parts of a structure, which are normally hidden from view. It should provide an accurate characterization of each component of interest in a platform in their intact or damaged state. At the higher screening cycles, the inspection effort may be focussed in the "weak zone" to develop an accurate linear or non-linear structural analyses. By increasing the state of knowledge of components, the capacity and consequence levels could be evaluated more accurately and the upgrading of a platform required based on lower screening cycles may not be required and it would result in substantial cost and time savings.

In recent years, various researchers in academia and industry have pursued development of methods for planning and execution of inspection, maintenance, and repair techniques for offshore steel platforms. Substantial work has been done to optimize the inspection cycles, with consideration of crack initiation and propagation. Some of the prominent works are reviewed here.

The *Underwater Engineering Group (UEG) of U.K. (1989)* have recently completed a Joint Industry Project with emphasis on making a decision on inspection planning. UEG approach is based on inspection priority ranking of components according to their degree of importance (in terms of consequences upon failure to the structural integrity), susceptibility to damage, failure modes, inspection history to date, certification requirements, cost associated, and reliability of inspection. Their approach recommends substantial pre-inspection work at the engineering office including sophisticated engineering analyses to determine the importance of a component to the structural system.

Bea et al (1988) described a process to develop the AIM (Assessment, Inspection, and Maintenance) cycle for a platform in a Joint Industry Project. AIM process focuses on development of comparative cost-benefit evaluation of various AIM alternatives to maintain or upgrade safety of structures. This approach was applied on several platforms and the cost-benefit effectiveness of various AIM alternatives were tested. In the evaluation of platforms, non-linear analyses was performed to evaluate their ultimate strength. *(Bea et al 1989)*

Bourgeois and Gernhardt (1987) describes the computerized inspection planning system to plan and track the inspection program. This system utilizes Computer Aided Inspection Reporting System (CAIRS) which consists of inspection planning and inspection database, with a facility for graphic representation. Chevron Oil Company uses CAIRS to plan the IMR cycle for their approximately 1,000 structures in the Gulf of Mexico.

Marshall (1979) presented the inspection procedure followed by Shell Oil company for its more than 1,450 platforms in the Gulf of Mexico. Their procedure is based on underwater visual inspection of certain platforms depending on their age, condition, interval since last inspection, and service history. The platforms subjected to overloads during hurricane are visually inspected for major damages after the storm. The above water inspection is performed annually on all the platforms.

Buslov et al (1987) have presented a system approach for IMR operations for a platform. Their method is based on a decision tree type approach. They explain the way in which such an effort can be executed by a contracting organization.

Lotsberg and Kirkemo (1989) have developed a method for optimization of in-service inspection using probabilistic analysis and resource allocation techniques.

In addition substantial work has been done on inspection of fatigue cracks in the welds and the different ways to inspect, monitor, and control their propagation. More recently, probability based decision making approaches have been proposed by various researchers.

6.2.1 Condition Assessment:

The first important part of an IMR program is to determine the "as-is" condition to improve the state of knowledge of the platform. The condition of a platform can be obtained in the following ways:

1) Condition survey Periodic inspection by diver(s) or by remotely operated vehicles (ROV).

2) Continuous monitoring of structural integrity of a platform by vibration monitoring or by acoustic emission (AE) techniques.

3) Performance monitoring by strain gauges and inclinometers. Cathodic protection monitoring and foundation monitoring.

The API has recommended a 4-level survey program for platforms in the U.S. [*API, 1991*]. MMS has also included these survey levels as a requirement for all the platforms under its jurisdiction. These requirements focus on the need to monitor the adequacy of corrosion protection system and determination of the condition of the structure. Table 2.1 summarized the frequency of survey, the methods, and the items which need to be evaluated at each survey level.

Continuous monitoring of platforms is done by sensors installed on the platform, and it has been mostly done for the platforms located in deepwater and in harsh environment. Such monitoring involves huge cost in data acquisition and interpretation and is not conventionally used on the Gulf of Mexico platforms. On some of the deepwater platforms in the Gulf of Mexico, it has been done to monitor their behavior and for research purposes. For more details of underwater inspection and defect assessment methods reference is made to a new approach developed by UEG in a joint industry project [*UEG, 1989*]. UEG work addresses the philosophy behind the method and provides practical guidance for under-water inspection of steel platforms.

6.2.2 Inspection

The underwater survey is mostly done by visual inspection techniques. In some cases they are carried out by remotely operated vehicles. The following methods for underwater inspection are employed:

- measurement of potential;
- photography: 35 mm and 70 mm;
- video system: black and white or color;
- ultrasonic thickness measurement;
- ultrasonic fault detection;
- magnetic particle inspection;
- temperature measurement;
- gamma ray radiography;

In order to perform some of the above inspection techniques, it is essential to prepare the surface to be examined. The surface can be prepared by brushing or blasting, and by use of very high pressure water jets. The high pressure water jetting may generally include injection of calibrated sand as an abrasive. The other method of cleaning is by pneumatic or hydraulic tools such as needle hammers, chisels, rotary brushes, etc. This takes a considerable length of time and requires highly specialized divers, and is extremely expensive to perform.

To remedy this, in recent past effort has been made to develop new inspection techniques. Some of the NDT methods which have been developed are given as below: vibro-detection; pressure detection; acoustic emission; vibration examination; modal analysis, and flexibility measurement.

The most simple form of non-destructive test is visual inspection, and remains the most widely used method for inspection of offshore platforms.

Visual inspection of a platform carried out to ascertain the integrity of the primary and secondary members, in the case of routine inspection comprise the following:

1) A check on the general configuration of structural members, riser clamps, conductor guides, boat landings, mudmats, etc.

2) A general inspection of the platform to ascertain any settlement, buckling, cracking, etc., recording lengths, widths, depths and positions. Only the external faces of legs, members, risers and riser supports are inspected in the above investigation. In the event of buckling of a major structural member, the connecting welds of the defected member are inspected to determine the presence or otherwise of perforation, shear or cracking.

3) General inspection of the platform, particularly in the zone of heavy marine cover, to find any pitting corrosion, determining the size, depth and frequency of pitting using a pit gauge, especially at welds and in heat affected zones. The location of pitting is recorded for subsequent ultrasonic examination and electrical potential measurement.

4) Survey of the state of marine growth and its classification according to hard or soft. The rate of marine growth is established by measuring the thickness of four points at right angles at every 5 m from floor to surface on each platform pile and riser.

5) Check on the integrity of cathodic protection system. In general, the position, number, and state of fixity of all sacrificial anodes on the platform is reported.

6) Rubbish survey, identifying and locating all ferrous rubbish within 2m of the platform. If electrical contact is established, this reduces the effectiveness of the cathodic protection system.

7) Check on the floor profile to determine scour around and in the vicinity of the piles, legs, conductors, risers, and flowlines. A floor profile is also produced and reported for each face of the platform, measuring the distance from the horizontal bracing to the floor every meter. The length, width, depth and position of any distortion in the floor potentially evidencing horizontal displacement of the legs is measured. In particular, the state of the riser support bed is reported.

6.2.3 Proposed IMR Program: The proposed IMR scheme by an operator to maintain the structural integrity of a platform over its remaining period of productive life should be evaluated to meet the requirements of API and MMS. In case there is a discrepancy, the remedial actions proposed by an operator to improve the safety level of the platform be evaluated.

6.3 Risk Assessment:

For the damaged and deteriorated members, different models for capacity evaluation of elements would be used. These models have been based on model tests and calibration with analysis. However, due to increased uncertainties involved with the inspection and capacity assessment of the damaged and deteriorated members, the uncertainty level associated with the strength level used in equation 5.3 would increase. Therefore, a detailed evaluation of the strength uncertainty be made.

If the strength reduction of a platform is represented by a damage factor ϕ, the mean safety margin would become $[\ln(\phi R_m) - \ln(S_m)]$. The safety index formulation presented in equation 5.2 could be written as follows:

$$\text{Safety index, } \beta = \frac{\text{Mean Safety Margin}}{\text{Uncertainty Level}} = \frac{\ln\,[\phi\, R_m / S_m]}{[(\sigma_{\ln \phi R_m})^2 + (\sigma_{\ln S_m})^2]^{1/2}} \quad ...(6.1)$$

where, R_m = Median ultimate strength of a platform

ϕ = Damage factor for ultimate strength of a platform

S_m = Median load effect on a platform

$(\sigma_{\ln \phi R_m})^2$ = Variance of log of the median ultimate strength of the damaged member.

$(\sigma_{\ln S_m})^2$ = Variance of log of the median load effect

Note that the variance represented by $(\sigma_{\ln \phi R_m})^2$ would be higher than the strength variance considered earlier, because it would include the variance associated with the parameters considered in the model used for capacity of a damaged and deteriorated member. The equation 6.1 upon reformulation for RSR could be written as follows:

$$RSR = [FR / \phi] \exp (\beta\, \sigma) \quad ...(6.2)$$

where FR is a force ratio, β is a safety index (a measure of the platform reliability), σ expresses combined standard deviation in the loadings and damaged capacities of the platform, and RSR is the **true (mean) reserve strength ratio in undamaged state.** By substituting FR value given in equation 5.5, the equation 6.2 would become as follows:

$$RSR = [1 / \phi] \exp (\beta\, \sigma - 2.33\ \sigma_s) \quad ...(6.3)$$

Therefore for a damage factor of 0.8 for a platform, the mean damaged RSR for a given consequence level in Fig. 5.5 would be 0.8 times RSR. The variance associated with strength of the platform due to damaged components would increase. The effect of increase in variance in strength was demonstrated in Fig. 5.7, where it was noted that the marginal band shifts and bends towards increased RSR for the same level of consequence level.

Therefore, shifting of marginal band would occur and the fitness for purpose criteria would become more conservative.

Hence for a damaged platform, the marginal and acceptable RSR would be higher than compared to an undamaged platform. Therefore, in case of a damaged and deteriorated platform, when the capacity evaluation is done by detailed quantitative methods, the conservativeness introduced due to the fitness for purpose criteria could be reduced.

6.4 Risk Mitigation:

Upon failure to qualify a platform at a screening cycle as FFP, a decision may be made to do one of the following:

a) to evaluate RSR at a higher screening cycle with improved techniques.

b) to take steps to upgrade the RSR or reduce the consequence level.

c) to decommission the platform.

d) to reduce risk associated with other sources. (see Fig. 5.8)

e) to reduce the uncertainties associated with load, strength, and consequence levels. (see Fig. 5.6 and Fig. 5.7)

For the platforms whose safety turns out to be "Marginal", a detailed study would be required to evaluate the possible ways of upgrading the safety level. As mentioned in previous sections, during the operational life of a platform, a number of conditions tend to reduce the RSR level of a platform. The consequence level of a platform may also have changed due to modifications in production facilities and functions of the platform. The reduction in RSR level of a platform may tend to make the platform "Marginally FFP' or UFP. The RSR of a platform could be brought in the FFP zone by development and implementation of a suitable "platform upgrading" program. The three possible paths, which can be undertaken to develop a feasible platform upgrading program to achieve an acceptable safety level are shown in Fig. 6.2.

Path-A involves increasing the RSR level, and it can be done by either reducing loads on the platform or increasing the strength of the platform. The strength improvement measures may be undertaken in the critical bay(s) only. The second path-B involves reduction of consequences of a platform, i.e., to reduce the risk level of a platform to people, environment, property, and loss of natural resources upon its failure (collapse). The third path-C, the more promising may involve both RSR improvement and consequence reduction measures. Thus, a cost-effective program can be developed by such considerations.

Thus by implementing the techniques listed above, the suitability of a platform could be improved from "Marginal" and UFP categories to the FFP category. The best (optimum) solution will depend upon a comparative techno-economic evaluation of the capacity and consequence levels of a platform.

In addition, another approach based on reduction of uncertainties associated with the acceptability (FFP) criteria can be used. The acceptability criteria for a platform can be changed by variation in acceptable risk level and by variation in the uncertainties (biases and variances) associated with the methods used in its determination of load, strength, and consequence levels.

The above mentioned approaches for risk mitigation can be significantly improved by establishing the condition of a platform with the improved inspection techniques. By such improved survey techniques, the uncertainties associated with the capacity and consequence evaluations arising from the condition of a platform can be reduced, and an improved

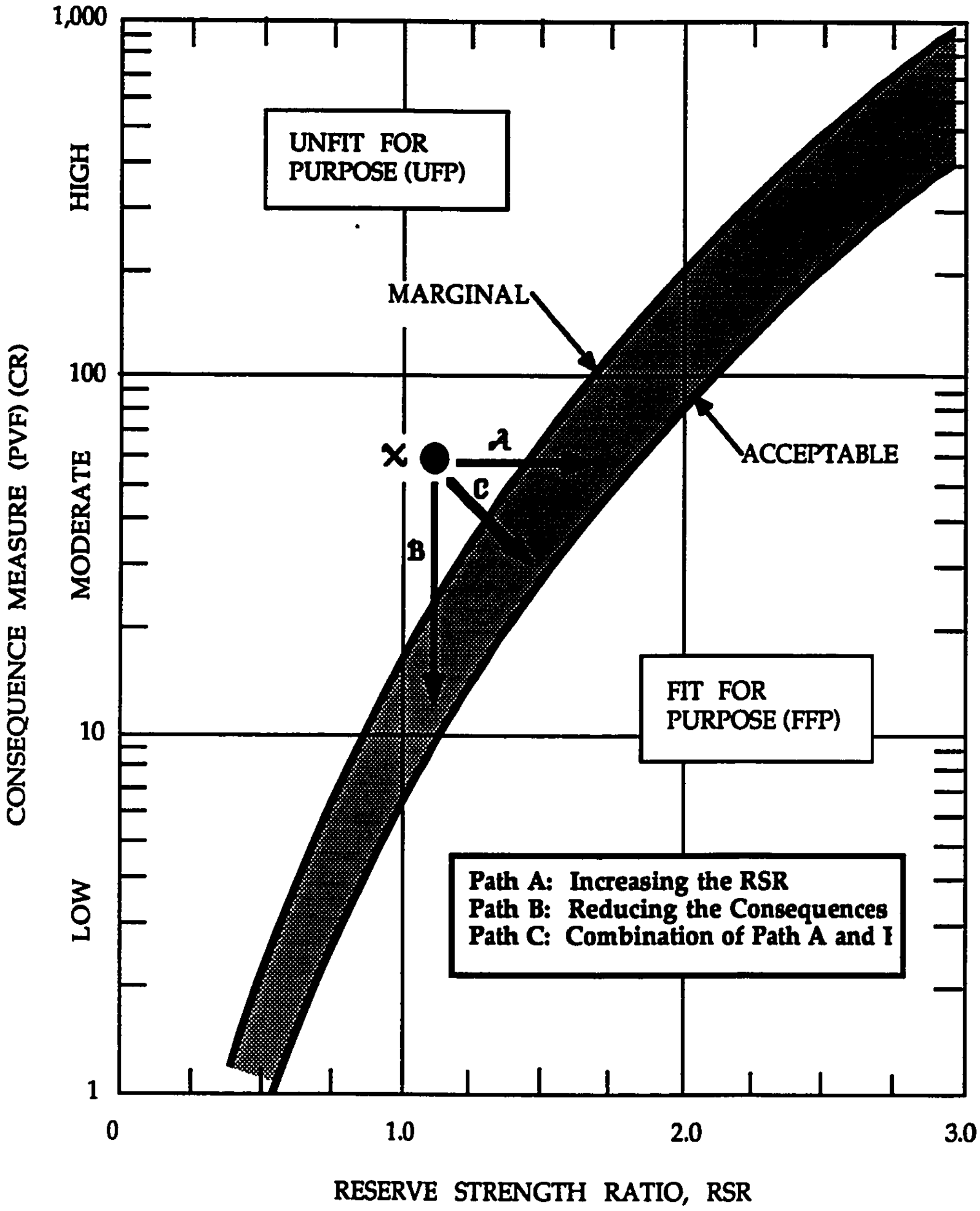


Figure 6.2: Alternative Ways for Upgrading of Platforms

assessment is made. The significance of the effect of reduction in uncertainties were described in Figures 5.6 and 5.7.

In the following sub-sections, the alternative techniques for upgrading physical and operational conditions, and acceptability criteria are described.

6.4.1 Upgrading Physical and Operational Conditions:

The major attributes influencing capacity (RSR) and consequences for the platforms were described in detail in Sections 3.2.2 and 4.2. During the in-service life of a platform, a number of conditions tend to alter the RSR and consequence levels of a platform, as were discussed in Section 1.1. RSR is most influenced by change in reference design criteria, damage and deterioration of ageing structure, modifications to platform structure. The consequence level is most influenced by the modifications made to the topside facilities and in the operational philosophy.

The risk mitigation for a platform could be achieved in various ways. In recent years several innovative applications have been developed to mitigate risk on many platforms. It can be achieved through individual or a combination of techniques under the categories of structural modifications or non-structural techniques [*Shinners et al, 1988, UEG, 1983, Smith et al, 1988*]. The structural risk mitigation is achieved by various techniques to reduce loads and/or to increase the strength of a platform. The non-structural risk mitigation techniques are related to modifications in the operational philosophy, reduction in the topside facilities, and by installation of safety valves and other safety measures.

"for number sequence only"

6.4.1.1 Structural Risk-Mitigation Techniques:

1) Load reduction by reducing the projected area for wave loads:

a) Wave does not hit the deck: It is done by clean up of the minor structural systems, appurtenances, and other material in the wave zone. The items in the wave zone, which can be removed are typically marine growth; minor structural systems such as intermediate decks, boat landings, stairs, walkways; and appurtenances such as barge bumpers, casings, and caissons. The removal of marine growth helps due to reduction in the projected area and the drag coefficient due to reduced roughness.

b) Wave hits the deck: In case wave hits the deck, the wave load can be reduced by removal of equipment from cellar deck, by streamlining the deck structure, and by raising of the deck. The decks of six steel platforms with the interconnected bridges in the Ekofisk field were recently raised together by 20 ft. to overcome the effects due to seabed subsidence of the field.

In addition, the wave loads can be reduced by provision of a barrier to reduce impact of waves. A circular concrete barrier (140 meter in diameter, 105 meter in height, and with weight of 250,000 tons) has been recently installed around the Ekofisk storage tank, to safeguard the tank due to seabed subsidence.

2) Strength Increase by Maintenance and Repair Measures:

a) By maintaining corrosion resistance and fatigue endurance of members and welds: The corrosion resistance of steel components is maintained by provision of adequate cathodic protection provided by anodes. The anodes are periodically inspected and may need to be replaced. The fatigue endurance of welds and joints is maintained in tolerance limits by grinding.

a) Upgrading the evacuation facilities

b) Demanning and operating remotely from adjacent platform

c) Removal of storage tanks

d) Providing sub-surface shut-in safety valves

e) Implementing more frequent IMR program

f) Installing emergency shut-down and back-flow valves in risers.

2) De-rating of Operations: The magnitude of operations carried out on the platform can be reduced to achieve a reduction in its consequence level by modifications to the topside facilities . The consequences of a platform are directly associated with the operations performed on the platform. Several alternatives are possible: de-manning to reduce potential for loss of life and injuries; reducing topside equipment or operations; reducing functions of the platform; reducing connections of a platform with other platforms.

3) By Introduction of Human Safety Measures

a) By provision of an Early Warning System (EWS) for storms to check functioning of life-saving equipment needed to evacuate people, to confirm functioning of tele-communication facilities, to shut-off the emergency shut-down valves for wells and risers to avoid risk of pollution and loss of resources.

b) Training of Crew: By training the crew, the panic and fear at the evacuation time is reduced with the knowledge of the alternate evacuation routes and schemes. In addition, the potential for human error is reduced.

6.4.2 Upgrading the Acceptability Criteria for a Platform:

The acceptability criteria discussed in detail in Chapter 5 depends upon the acceptable safety index and uncertainties in parameters. The positioning of the boundaries of the marginal band is based on the formulation given in equation 5.3 in Section 5.3. This equation has σ which represents the overall uncertainty (variance) in the parameters, and has a major influence on RSR.

The acceptability criteria developed in Fig. 5.3 can be upgraded by reduction in the uncertainties in the parameters and the methods used for evaluation of capacity and consequence levels. By such variation in uncertainties, the width of the marginal band will change, and its boundaries will be repositioned. The variation in the FFP criteria due to change in the uncertainty level was demonstrated in Figures 5.6 and 5.7.

The magnitude of type-II uncertainties (bias and variance) i.e., modelling uncertainties, can be reduced by obtaining improved knowledge of the "as-is" state of a platform and by making more accurate evaluation of the RSR based on detailed quantitative methods at the higher screening cycles. The magnitude of σ can also be reduced by using more accurate techniques to estimate the operational and environmental loadings. Thus, the safety level of a platform could be upgraded by reduction in or management of the uncertainties in the parameters.

The IMR program which would input improved information of the platform is likely to reduce the uncertainties associated with the capacity and consequence parameters, which may change the position of the two lines as shown in Figures 5.6 and 5.7.

6.5 Risk Maintenance

The risk level of a platform is maintained by implementation of the platform upgrading techniques identified in the risk mitigation phase or planning of interval for the next IMR cycle. This effort would be required at different stages of each of the four screening cycles, which was presented in Fig. 1.2.

An interval for the next cycle of implementation of IMR program depends upon the required interval by MMS, likelihood of reduction in strength of platform below a preselected threshold level, and occurrence of other phenomenon. In case there is a collision with the platform or the hurricane passes the region, the IMR program should be implemented. The decision analysis may be applied to select an appropriate inspection interval.

The requirements from inspection would vary for the different screening cycles. A brief discussion of the requirements at the different cycles is given further.

At screening cycle-1, its is most important to establish the "as-is" characteristics of the components of a platform. The data available for a platform, the API inspection level performed for a platform need to be checked for an accurate characterization of the state of components.

At the beginning of screening cycle-2, the major problems possible in a platform are known through the assessment at screening cycle-1. The inspection efforts should then be focussed in the critical region identified at screening cycle-1 and an improved characterization of the components in that zone is obtained. In this way, the uncertainties in the capacity evaluation would reduce. The effort should focus on identification of the various damages and deteriorations in the platform in order to improve the capacity estimate.

At screening cycle-2, through the coarse quantitative method, the weak zones in the platform are identified. Thus, the inspection effort at screening cycle should focus on these weak zones. NDT survey of selected members may be needed at this stage. The capacity evaluation for these zones could be significantly improved or more refined computer models could be developed for these zones. In this way, a more accurate estimate of RSR could be made by the linear elastic analysis. At this cycle, the complete IMR program could be implemented.

At screening cycle-4, a local inspection of members or joints would be needed for specific bays of the platform. The platforms which require implementation of large scale strengthening and other costly measures to improve their safety level should be evaluated at this cycle by the non-linear analysis techniques.

The implementation of the risk mitigation program may become timely and costly affair, and it must be carefully done. A detailed study of the maintenance plan be developed and incorporated during a suitable weather window. A significant amount of money could be saved by a well studies, planned, and implemented program.

It must be ensured that the benefits sought by the risk mitigation plan against one source of hazard does not increase the risk level against other hazard source. For example, strengthening of platform against hurricane by adding braces or repair of joints could reduce the fatigue life of the members, if proper care was not taken. Therefore, the process should involve an evaluation of the influence of the repair plan on the safety of structure against other load sources.

6.6 Summary

The important steps and tasks in an Inspection, Maintenance, and Repair (IMR) program for a platform to maintain it in acceptable safe state have been presented. An IMR program consists of four phases: risk identification, risk assessment, risk mitigation, and risk maintenance.

API-RP-2A recommends type of inspections and the inspection intervals for different type of platforms, which are based on their functions.

A decision on significance of the damage on safety of a platform is made in this methodology on the basis of capacity-consequence evaluation presented in Chapter 5.

The various means available to improve the safety level of a platform were discussed. These fall under structural and non-structural categories. The likely benefits to be achieved based on such methods were demonstrated. In many cases a combination of several means to upgrade safety level of a platform would prove cost-effective.

The implementation of risk maintenance program may become a timely and costly affair. Therefore, a detailed study of the maintenance plan be developed and incorporated in a suitable weather window. A significant amount of money could be saved by a well studies, planned, and implemented program. It must be ensured that the benefits sought by the risk mitigation plan against one source of hazard does not increase the risk level against other hazard source. Therefore, the process should involve an evaluation of the influence of the repair plan on structural safety against other load sources.

CHAPTER 7

EXAMPLE PLATFORMS

The screening methodology developed in this study has been applied on 3 example platforms. These example platforms correspond to the real structures used in the Gulf of Mexico. They have different structural configurations and are located in different water depths. Hence, the validity of this methodology is tested for its suitability for safety assessment of a large number of platforms with variations in their physical properties, environmental, and geotechnical parameters.

This chapter contains a summary of the results obtained for the three platforms, as listed below:

Name	Water Depth	No. of Main Legs
Platform-A	271 ft.	8-leg
Platform-B	140 ft.	4-leg
Platform-C	52 ft	8-leg

The structural, environmental, and geotechnical details of these platforms are summarized in the following sections. The detailed calculations for the screening cycle-2 for platform-A & B are included in Appendix-E & -F, to demonstrate the application of the coarse quantitative method.

7.1 Platform - A

7.1.1 Description of Platform

Platform-A consists of an eight-leg steel template jacket located in 271 ft. water depth in the Gulf of Mexico *(Fig. 7.1)*. It represents a typical 8-legged platform operating in the Gulf of Mexico. It is a continually manned, self-contained drilling platform, with 24 oil producing wells, and has been in operation for more than 20 years. Three risers (1-16"ϕ , and 2-10"ϕ) are provided for transportation of crude to and from the platform. In addition, three pump casings of 16"ϕ, two boat landings, and barge bumpers are provided. The total weight (dead + live) of topside facilities is 13,450 Kips (=6,100 Tons). The center line elevation of the lower deck is (+)46' and of the sump deck is (+)35'-6". The bottom of steel elevation of the lower deck is (+)45'-3".

The site characteristics are: medium sands overlying stiff clays. The foundation of the platform constitutes of 8-42"ϕ piles with a maximum penetration of 270 ft., and are grouted to the jacket legs.

The original design criteria of platform is based on the design wave height of 58 ft. with 16 sec. period and 100-year return period considered in 1969. The marine growth was taken as 2" on diameter for all members between mean sea level and (-) 100'. The nominal sizes of members (as per the as-installed drawings) have been used in this study, except where the platform records were updated based on post-installation inspection reports.

All the steel members have a nominal yield stress of 36 ksi. In this study an expected (mean) yield stress of 45 ksi has been used, to account for bias in the yield strength and the strain rate effect. This value reflects the actual in-service steel strength rather than the allowable design value.

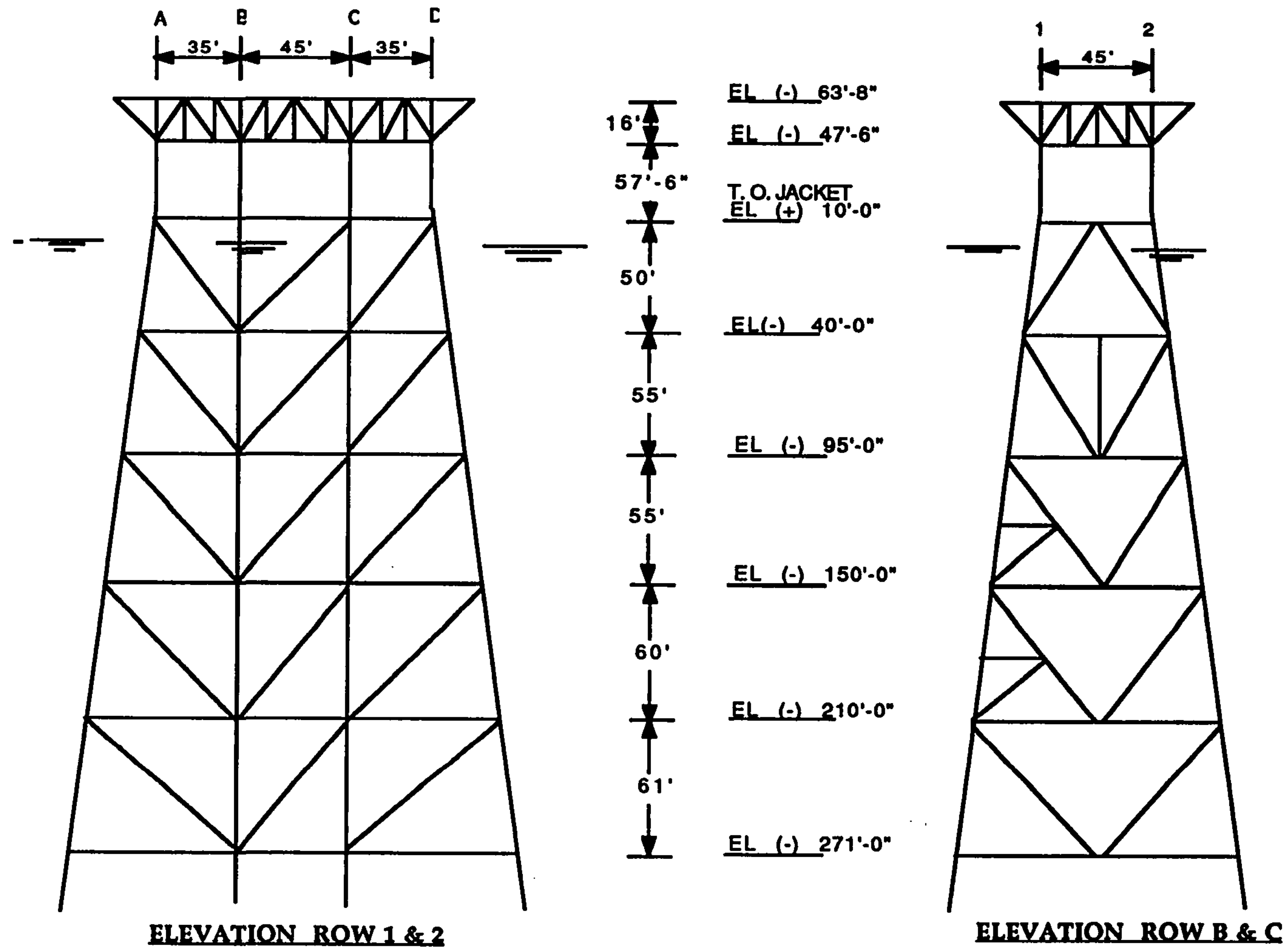


Figure 7.1: Structural Configuration - Platform A

7.1.2 Screening Cycle-1

At screening cycle-1, a coarse qualitative evaluation of capacity and consequence levels is done by following the procedure described in Section 3.3. In this cycle, a 2-phase process is applied, with evaluation of the platform at phase-1 by a pre-defined cut-off criteria. In the second phase, a cumulative effect of various attributes on likelihood of increase or decrease of capacity and consequence levels. The detailed evaluation is described in this section.

Screening Cycle- 1A; Cut-off criteria: The platform is evaluated for major conditions to determine the likelihood of significant increase in load and/or strength levels. The process followed is as given in Fig. 3.8 for load level evaluation and Fig. 3.9 for strength level evaluation. More detailed schematic diagrams have been developed for each of these primary conditions and are given in Chapter 8 (Figs. 8.5 to 8.11). The as-design characteristics of the platform are described in the previous section.

The platform has not been designed for 25-year return period. The API reference level wave height at this water depth is 69 ft. and the design wave height is 58 ft. Therefore, the reference level wave height is higher than original design wave by approximately 19% (= 69/58). Therefore as per the criteria set in Section 3.3 and Section 8.3, the wave load on the platform is likely to increase significantly over the design wave load. Thus, the platform exits screening cycle-1A and the platform is further evaluated at screening cycle-1B.

Screening Cycle-1B: The platform is evaluated to determine the variations in the capacity (RSR) for each bay of the platform. For each bay of the platform, the strength and load factors (R, S) are estimated by simple comparison of the original design criteria with the latest API reference level criteria. Then a numerical value for RSR is obtained, which gives an approximate idea of variation in RSR from a central figure of say 1.25. The following factors are determined for load and strength for the damaged bay for demonstration purpose.

Deck Bay:			
Material factor	R1	1.25	due to Fy
Platform condition factor	R2	1.0	when no damage
		0.75#	when the deck legs are corroded in the splash zone
Platform modifications factor	R3	1.0	None
Structural configuration Factor	R4	1.0	D/t < 60, so full Mp would be realized
Design criteria variation factor	S1	1.42*	(API reference level wave = 1.19 x Design wave)
Deck elevation factor	S2	1.0**	Wave does not hit deck
Platform modification factor	S3	1.0	None

The deck legs are considered to be corroded in the splash zone from 42" x 1" (original size) to 41.5" x 0.75" (corroded size). Therefore wall thickness reduces by 0.25" from original of 1.0". Therefore R2 ≈ 0.75.

* Design criteria variation would be due to wave heights and drag coefficients used in the original design and the as-is state of the platform. Drag coefficient used is similar (= 0.6) but current API reference level wave height is 19% (=69/58) higher. Therefore the wave load increase would be proportional to 2.0 (= α) power of ratio of wave heights (= 1.19). Therefore S1 would be equal to 1.42.

** The bottom of steel elevation of lower deck is (+)45'-6". Total of storm tide and surge is 3.5'. Wave crest height could be approximately evaluated as 0.55 H to 0.60 H, i.e., 38' to 41.4'. The crest elevation from m.s.l. would be 41.5' to 44.9'. These elevations are lower than 45.5'. Therefore wave will not hit the lower deck and S2 = 1.0.

Therefore variation in RSR $= [1.25 \times 1 \times 1 \times 1 / (1.42 \times 1 \times 1)$

$= 0.88$, when the platform has no damage i.e., according to qualitative classification assumed, RSR would be LOW (0.8 to 1.0) for the platform in undamaged state.

Therefore variation in RSR $= [1.25 \times 0.75 \times 1 \times 1 / (1.42 \times 1 \times 1)$

$= 0.66$, when the platform has damage i.e., when the platform is damaged (corroded), the qualitative classification of RSR would change to VERY LOW (<0.8).

Consequence level: The platform is continually manned, self-contained drilling type with 24 oil-wells and 3-risers provided to transport crude to and from the platform. Therefore, with this platform the likelihood of loss of lives, environmental pollution, loss of production and resources, influence on operation of other platforms, and significant property loss exists. Based on the criteria assumed in Chapter 4, this platform would fall in High Consequence (HC) category.

Fitness For Purpose (FFP) Evaluation: The qualitative formulation of FFP evaluation presented in Fig. 5.1 is used. For this platform based upon screening cycle-1B, the capacity measure (RSR) has been evaluated as LOW for undamaged case and VERY LOW for damaged (corroded) case, and its consequence measure has been identified as HIGH. These qualitative measures are compared in Fig. 7.2 for both damaged and undamaged cases. For both states, the platform fall in the Unfit For Purpose (UFP) zone.

This platform is then screened out for further evaluation by the coarse quantitative method (screening cycle-2).

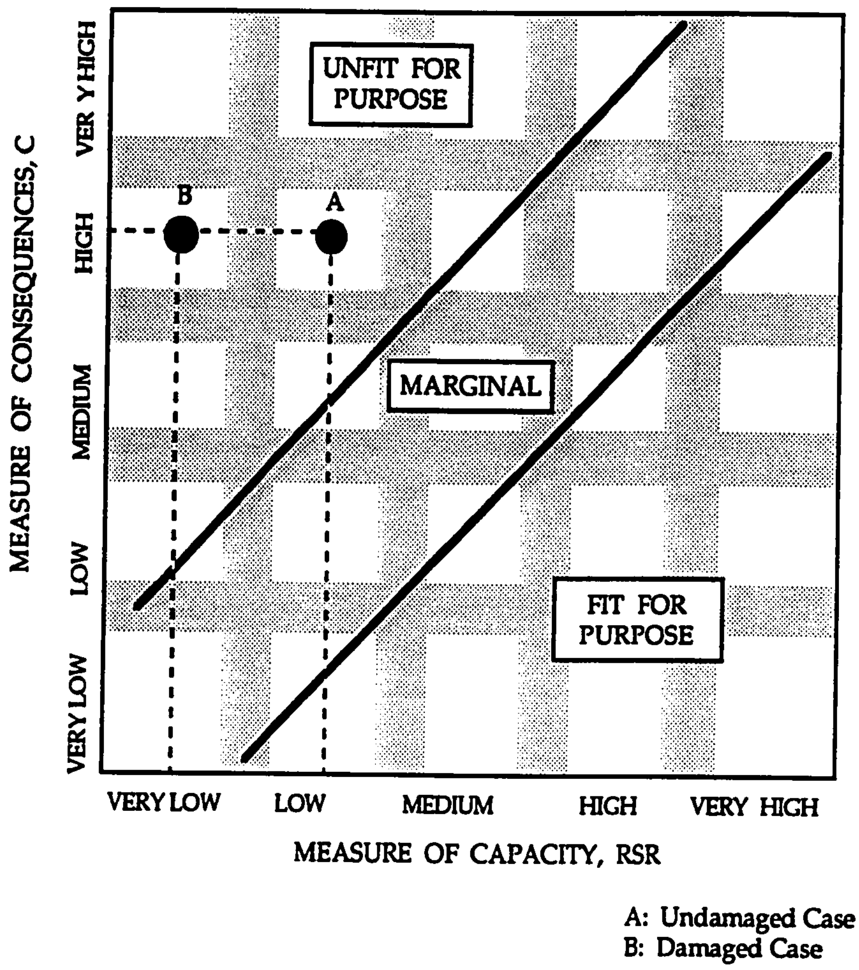


Figure 7.2: Fitness For Purpose assessment for Platform - A (Screening Cycle -1B)

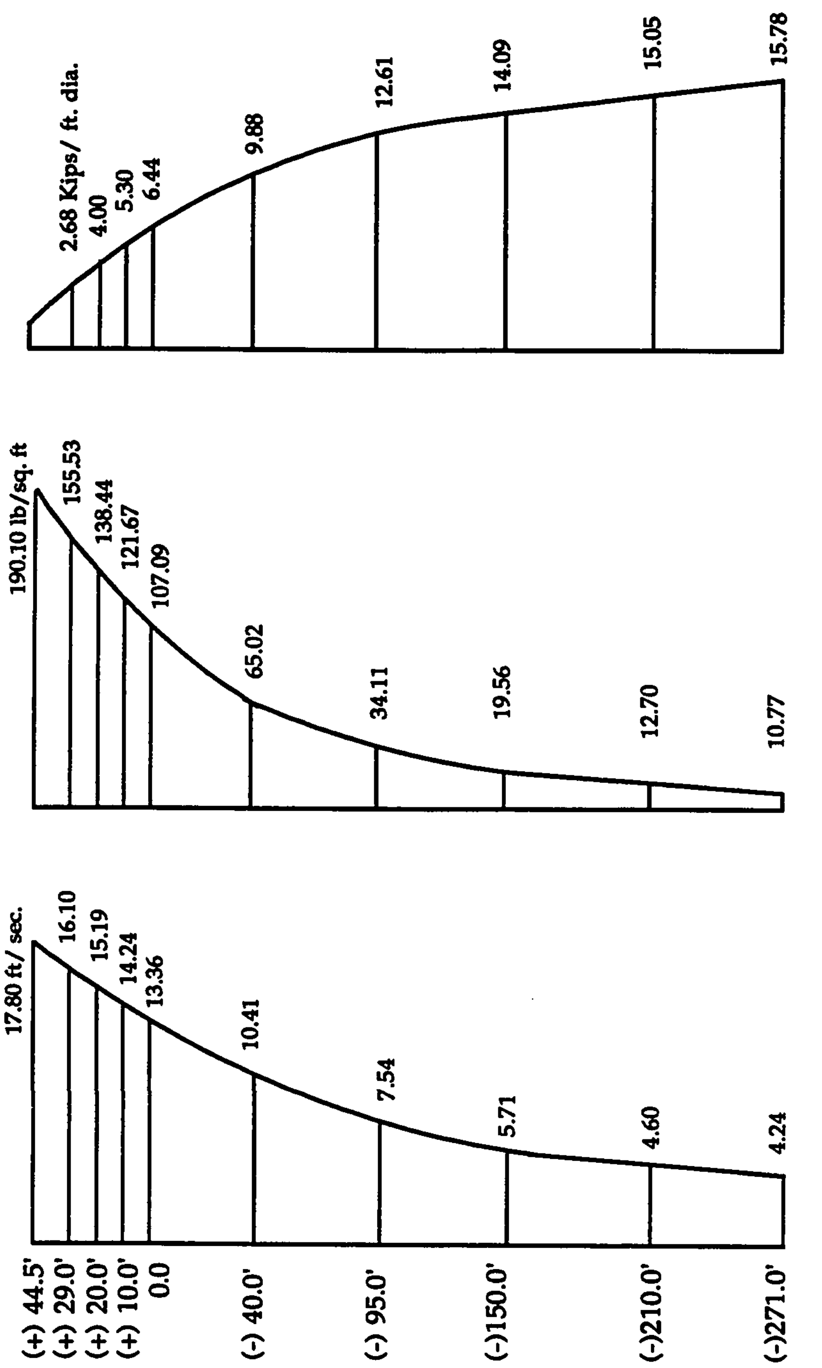
7.1.3 Screening Cycle-2

At screening cycle-2, a coarse quantitative evaluation of the lower bound of RSR is done by following the procedure described in detail in Section 3.4. In this cycle, the load and strength patterns for the platform are developed, by determination of the loads and strengths for the different bays on an individual basis. The nominal estimates of the load and strength for the different bays, and the uncertainties (biases and variances) in their estimates are determined. The detailed calculations for their estimation are given in Appendix-E and the results are summarized and discussed in this section.

Development of the lateral load pattern:

The procedure described for Option-B in Section 3.4.1.1 is followed to develop the lateral load pattern. The wind load on the deck and the wave loads on the jacket part contribute to the reference level lateral load for this platform. A wave kinematics profile is developed for a unit diameter vertical pile extending from the seabed to above the mean sea level, for wave height (H) of 69 ft. with wave period (T) of 12.8 sec. and no associated current. The wave kinematics is based on Airy's theory stretched to the crest elevation i.e., the wave kinematics at the mean sea level (m.s.l.) is considered at the wave crest. The horizontal particle velocity at different horizontal framing elevations, the crest elevation, and mudline are indicated in Fig. 7.3.

The unit wave loads at these elevations and the overall load (base shear) for a vertical pile are shown in Fig. 7.3. Based upon the values given for a unit diameter vertical pile in Fig. 7.3, the wave load pattern for the complete platform has been developed and the lateral load for different components are given in Table 7.1. The basic assumption made in the development of the wave load pattern is that a virtual vertical pile is located at the middle of the jacket and all the wave kinematics developed for it is valid for all of the members of the jacket. Alternately, all the members of a jacket are assumed to be appropriately lumped at the center of the jacket and the wave kinematics developed for a unit diameter vertical pile at this location is applied. Hence, the phase effect or spatial variation in wave loads with distance is neglected and conservative results have been obtained.



a: Horizontal Particle Velocity, u vs. depth

b: Drag Force/ unit area

c: Cumulative Load Pattern

Figure 7.3: Wave Kinematics and Forces on a Unit Diameter Vertical Pile - Platform A

Table 7.1: Lateral Loads Based on Components				
	End-on Direction		Broad-side Direction	
Item	Load	Percentage	Load	Percentage
Wind Load	80 Kips	4.4	160 Kips	7.9
Wave Load:				
Main legs	465 Kips	25.3	465 Kips	23.0
Conductors	736 Kips	40.1	736 Kips	36.3
Risers	53 Kips	2.9	53 Kips	2.6
Vertical Braces	271 Kips	14.8	292 Kips	14.4
Horizontal Braces	120 Kips	6.5	211 Kips	10.4
Appurtenances	110 Kips	6.0	110 Kips	5.4
Total Wave Load	1755 Kips		1867 Kips	
Total Lateral load	1835 Kips		2027 Kips	

In addition, the topside load of 13,450 Kips has been considered to be equally distributed on the eight legs, i.e., 1,681 Kips on each leg.

From the above results, it is noted that for this platform approximately 62 % to 68 % of the base shear (total lateral load) is from the main legs, conductors, and risers, whereas the vertical and horizontal braces contribute only 22 % to 25 % load. The jacket appurtenances such as boat landings, barge bumpers, casings, etc. have been assumed here to contribute 6 % and the wind loads on the deck structure roughly 4 % to 8 %.

The summary of wave loads for each bay is given in Table 7.2. From Table 7.2, it is noted that about 60% of the base shear is due to the wind and wave loads in the deck bay and the jacket bay-I, i.e., up to El. (-) 40'. Another 18% contribution to the base shear is from jacket bay-II, which is from El. (-) 40' to El. (-) 95'.

The overturning moment values at the top (deck-jacket connection) and bottom (mudline) of the jacket are also required. Based on the results obtained in Table 7.2, in a simple analysis the lever arm for obtaining the base moment could be assumed equal to the water depth.

Table 7.2: Lateral Loads Based on Bay

Item	End-on Direction		Broad-side Direction	
	Load	Percentage	Load	Percentage
Wind Load	80 Kips	4.4	160 Kips	7.9
Wave Load:				
Deck Bay	385 Kips	21.0	385 Kips	19.0
Jacket Bay-I	638 Kips	34.8	683 Kips	33.7
Jacket Bay-II	328 Kips	17.9	363 Kips	17.9
Jacket Bay-III	117 Kips	6.4	190 Kips	9.4
Jacket Bay-IV	120 Kips	6.5	126 Kips	6.2
Jacket Bay-V	98 Kips	5.3	101 Kips	5.0
Mud Level Framing	16 Kips	0.9	18 Kips	0.9
Total Wave Load	1755 Kips		1867 Kips	
Total Lateral load	1835 Kips		2027 Kips	

In Section 2.3 this aspect was discussed in more detail. In this way, the overturning moments have been approximately evaluated as follows:

@ top of jacket:	End-on Direction:	10,953 Kips-ft.
	Broad-side Direction:	15,753 Kips-ft.
@ bottom of jacket:	End-on Direction:	502, 085 Kips-ft.
	Broad-side Direction:	558,917 Kips-ft.

The resultant lateral wave load for reference wave parameters would act at higher elevation than the resultant lateral load based on the original design parameters. Hence, the overturning moment is likely to be higher than determined at the design stage. Therefore, the foundation piles would be subjected to increased loads.

The bias and variance in these estimates would be closer to the bias and variance associated with the base shear and overturning moment determined in Section 2.3 for a single vertical pile. The additional bias in results would exist due to the various simplifications made in the evaluation of lateral loads for the platform for Option-B.

The additional bias in these results could be determined by comparison with the results obtained by Dean's Stream Function theory by using a computer software. For this platform when subjected to wave height of 69' with wave period of 12.8 sec. and no current, and Dean's Stream Function theory is applied, the base shear is obtained as 1,600 Kips and 1,850 Kips for the end-on and broad-side directions respectively [*AIM, 1988*].

$$\text{Bias} = \frac{\text{Actual or measured value}}{\text{Predicted or nominal value}}$$

Bias in lateral load (end-on direction) = 1,600/ 1,835 = 0.872

Bias in lateral load (broad-side direction) = 1,850/ 2,027 = 0.913

Note that these bias estimates would differ for different platforms, due to variations in the percentage contribution of the different members. The above bias would exist due to several reasons in the lateral loads evaluated in this way, which are primarily due to reduced wave forces on the outer legs and vertical frames, and on the other members.

In the end-on case the bias is higher, because the effect of large distance between the outer legs on the wave kinematics has been neglected in our simplified procedure, by considering that the wave kinematics and the unit wave load profiles developed at the center of the jacket are valid for all the jacket members. The well conductors are located in the one half portion of this platform. Therefore, the wave loads on conductors evaluated above would reduce, when the kinematics at their exact location are considered.

Development of the bay strength pattern:

The strength of each bay (or sub-structure) is independently evaluated at failure of the first component (a brace, a leg, or a joint), by the procedure described in Section 3.4.2. The detailed calculations are given in Appendix-E, and a summary of the results is given in this section.

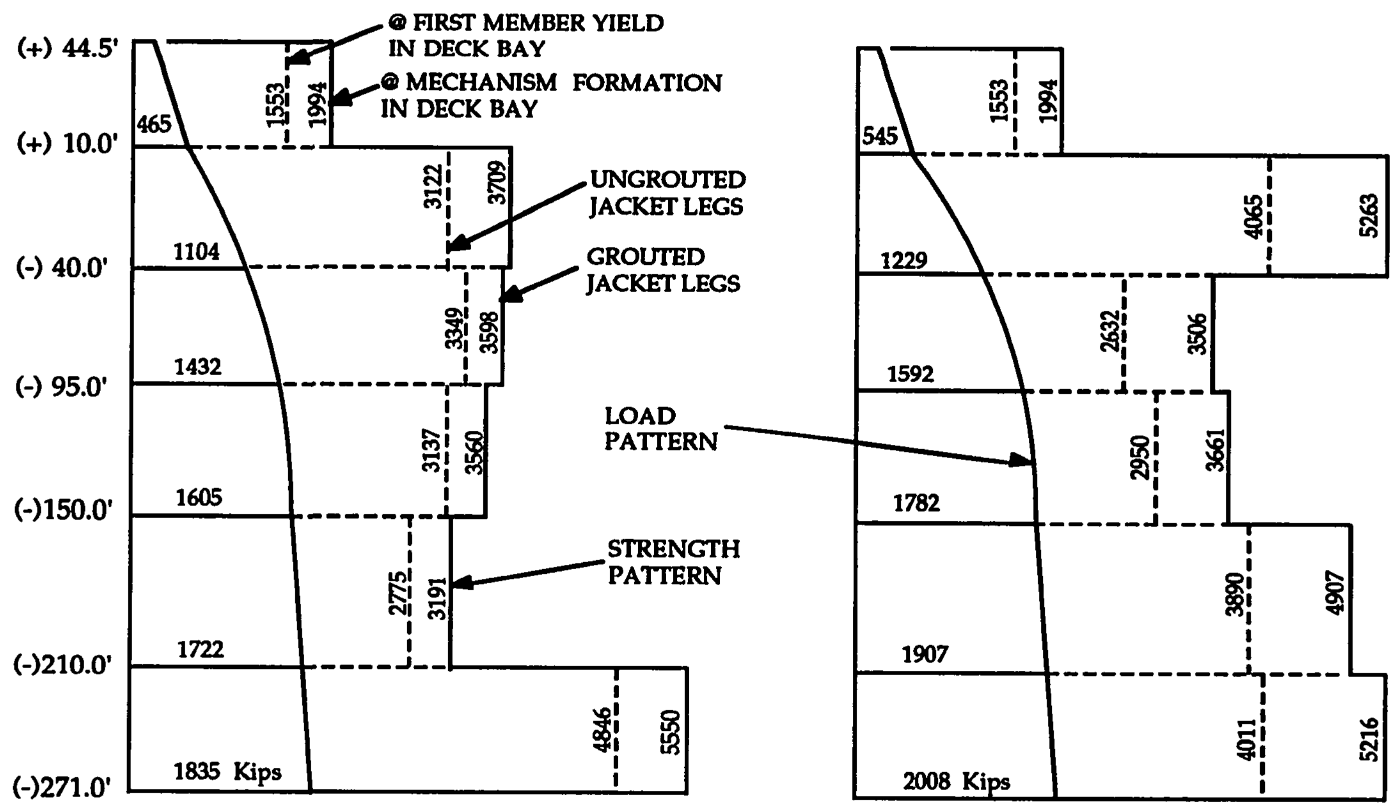
In evaluation of the ultimate axial load capacity of a brace, it is important that the phenomenon which would reduce the axial capacity are considered in an accurate way. The axial capacities of the members would reduce due to: imperfections in the members; and the local lateral bending loads from the wave, current, and hydrostatic pressure acting on the members.

End-on load case: When the wave approaches from the positive X-direction, the two out of three braces along the rows A and B would be in tension. The strength pattern for the platform has been developed on the basis of the first member failure (lower bound) and is shown in Fig. 7.4 [refer Appendix-E for detailed computations]. The upper bound strength levels for the deck and foundation bays are also shown.

Broad-side load case: The strength pattern for the broadside lateral load case is also shown in Fig. 7.4.

These patterns show an irregularity in the bay strengths at the failure of first member in the individual bays. The lower bound estimate of the deck bay is related to the load at yielding of the first section, and the upper bound represent the load level at the formation of a mechanism. The strength estimates for the jacket bays have been determined for the two cases of ungrouted and grouted leg-pile annulus.

In the end-on load case, the jacket bay between elevations (-)150' and (-)210' governs with the minimum bay strength as 2,775 Kips for the ungrouted case and as 3,191 Kips for the grouted case. In the broadside load case, the jacket bay between elevations (-)40' and (-)95' governs with the



a: End-on Load Cases

b: Broadside Load case

Figure 7.4: Load and Strength Patterns- Platform A

minimum bay strength as 2,632 Kips for the ungrouted case and as 3,506 Kips for the grouted case.

In some bays the strengths of the horizontals (outer) is lesser than the horizontal components of the primary vertical braces *[refer Appendix-E]*. Note that upon failure of the very first compression brace in a bay, the load transfer from the failed compression brace to the other vertical braces in a bay will be possible through the primary (outer or diagonal) horizontals. Therefore, the bays with lower strength of the primary horizontals may not carry additional load beyond load level at failure of the first brace and the other members are likely to fail in a sequence.

Determination of the Component Strength at failure of the first member: The component strength (CS_1) or the lower bound of RSR at failure of the first member in the platform is determined by comparison of its load and strength patterns. The load pattern is extrapolated in this example to determine the base shear at which it will meet any point on the strength pattern.

Note that an accurate evaluation of the base shear, which will cause the first member to fail, would require development of load patterns for increased wave height(s). Such load patterns can be developed easily with the use of computer softwares. A rough estimate of the base shear at failure of the first member in any bay could be made as follows:

End-on load case:

when legs are ungrouted, load ≈ 2,775 + (1,835 - 1,722) = 2,888 Kips.

when legs are grouted, load ≈ 3,191 + (1,835 - 1,722) = 3,304 Kips.

Broad-side load case:

when legs are ungrouted, load ≈ 2,632 + (2,008 - 1,591) = 3,049 Kips

when legs are grouted, load ≈ 3,506 + (2,008 - 1,591) = 3,923 Kips.

From the static pushover analysis (refer Section 3.6), the **upper bound** of the ultimate capacity of this platform for the end-on load case is obtained as 2,747 Kips, which correspond to failure of the first two components in compression (braces) between Elevations (-) 150' and (-) 210'. In case of the broadside load case, the upper bound estimate is obtained as 2,755 Kips by the static pushover analysis and the first component failure occurs in the jacket bay between elevations (-)40' and (-)95'. The first member failure occurs at a load level of about 2,700 Kips for the end-on case, and 2,755 Kips for the broadside case *[AIM,1988]*.

Note that for this platform, we are getting higher strength level at screening cycle-2 than that by the static pushover analysis at screening cycle-4. One of the reason for this difference is that we are not considering the upward movement of the elevation of the resultant lateral load when the wave hits the deck. Such consideration will be more important for the broadside load case, where the most likely to fail (MLTF) member is in the second bay of the jacket. In this case, with upward shifting of the resultant load elevation, the contribution to the base shear of the reference level wave loads below this bay will reduce substantially from that shown in Fig. 7.4.

By this simplified method, we are able to accurately identify the most likely to fail (MLTF) member in a bay of the platform. Therefore, with little computations and without necessarily using the computers, we are able to locate the "weak zone" of the platform. At screening cycles 3 and 4, an advantage of this is taken by developing a more refined model for the "weak zone" identified at screening cycle-2.

The biases introduced in the component strength (CS_1) by screening cycle-2 are determined as follows:

End-on case:	Ungrouted legs,	Bias = 2,747 / 2,888 = 0.95.
	Grouted legs,	Bias = 2,747 / 3,304 = 0.83
Broad-side case:	Ungrouted legs,	Bias = 2,755 / 3,049 = 0.904
	Grouted legs,	Bias = 2,755 / 3,923 = 0.70

The load at failure of the first member is equal to 0.87 of the ultimate strength (R_u). This shows low redundancy level for the platform due to the K-bracing pattern in the vertical framing and due to the weaker horizontals. The horizontal braces are unable to transfer the overload from the yielded tension brace or the load shed by the failed compression member to the other braces in a bay. These horizontals failed subsequently to the failure of the first or the second compression braces, without redistributing the loads from the compression to the tension braces.

The nominal estimate of the RSR for the two directions is determined as follows:

End-on case:

Ungrouted legs, RSR = 2,888/ 1,835 = 1.57 with a bias of 0.95 / 0.872 = 1.09

Grouted legs, RSR = 3,304 / 1,835 = 1.80 with a bias of 0.83 / 0.872 = 0.95

Broad-side case:

Ungrouted legs, RSR = 3,049 / 2,027 = 1.50 with a bias of 0.904/0.913 = 0.99

Grouted legs, RSR = 3,923 / 2,027 = 1.94 with a bias of 0.70 / 0.913 = 0.77

For this example, the results obtained at screening cycle-2 are higher for the component strength (lower bound of RSR). Hence, the RSR obtained in this way is also higher and the associated bias is lower due to reduced bias in the component strength.

These bias estimates could be taken as the first estimate of the general (or mean) biases associated with RSR computed by using the screening cycle-2 method for the 8-legged platforms. This approach must be applied on more number of 8-legged platforms to obtain a range of bias values under varying conditions. In this way, a more accurate estimate of mean bias, and variance in the mean bias for 8-legged platforms could be obtained.

The mean value of RSR for this platform is then evaluated for use in the fitness for purpose (FFP) evaluation, in the following way:

$$(RSR)_{mean} = (RSR)_{nominal}\ (Bias)_{RSR}$$

End-on load case: $(RSR)_{mean} = 1.71$

Broadside load case: $(RSR)_{mean} = 1.49$

Note that these mean RSR would have associated uncertainty in their estimates, incorporated in terms of variance. The variance in RSR would be the combination of variances in load and strength.

Consequence Level: The platform is continually manned, self contained drilling platform with 24 oil-wells and 3 risers provided for transportation of crude to and from the platform.

Therefore, for this platform, there is likelihood of loss of lives, environmental pollution, loss of production and resources, influence on operation of other platforms connected by risers, significant loss of property. According to the definition provided in Section 4.3, this platform would be classified as a High Consequence (HC) platform.

Fitness For Purpose Evaluation: The fitness for purpose formulation given by equation 5.3 is generated for the uncertainties (standard deviations, or variances) associated with this platform.

As discussed in an Chapter 2, the uncertainties in load and strength could be taken as 0.66 and 0.25 respectively. Therefore the total uncertainty would be 0.71.

The capacity - consequence diagram is then developed for σ_s of 0.66 and σ of 0.71 as shown in Fig. 7.5. From this figure, a judgement on safety of the platform could be made.

The mean RSR evaluated for the end-on load case was 1.71 and its consequence level is designated as "High." This RSR in conjunction with High consequence level of the platform is located by bar-X in Fig. 7.5. Based on this location, the platform could be termed as marginal or unfit for purpose.

The mean RSR evaluated for the broad-side load case was 1.49 and given consequence level as "High consequence," the platform is located by bar-Y on the capacity-consequence diagram. Therefore, the platform could be termed as unfit for purpose for this level of reference force.

The safety level of this platform could be improved by taking measures to reduce the its consequence level upon its failure, by reducing the uncertainties associated with load and strength, or by reducing or eliminating the risk level associated with failure of the platform from sources other than storm wave.

From Fig. 7.6, it is observed that in case the COV of load effects could be reduced from 0.75 to 0.5, the safety level of the platform against end-on load case would become marginal. In addition, if strength uncertainty (COV) could also be reduced than a more optimum solution could be achieved.

Fig. 7.7 demonstrates the shifting of marginal safety band with variation in the probability of failure associated with the storm loading. Given that the probability of failure associated with the storm wave is the same as the total probability of failure the platform could be classified as fit-for-purpose.

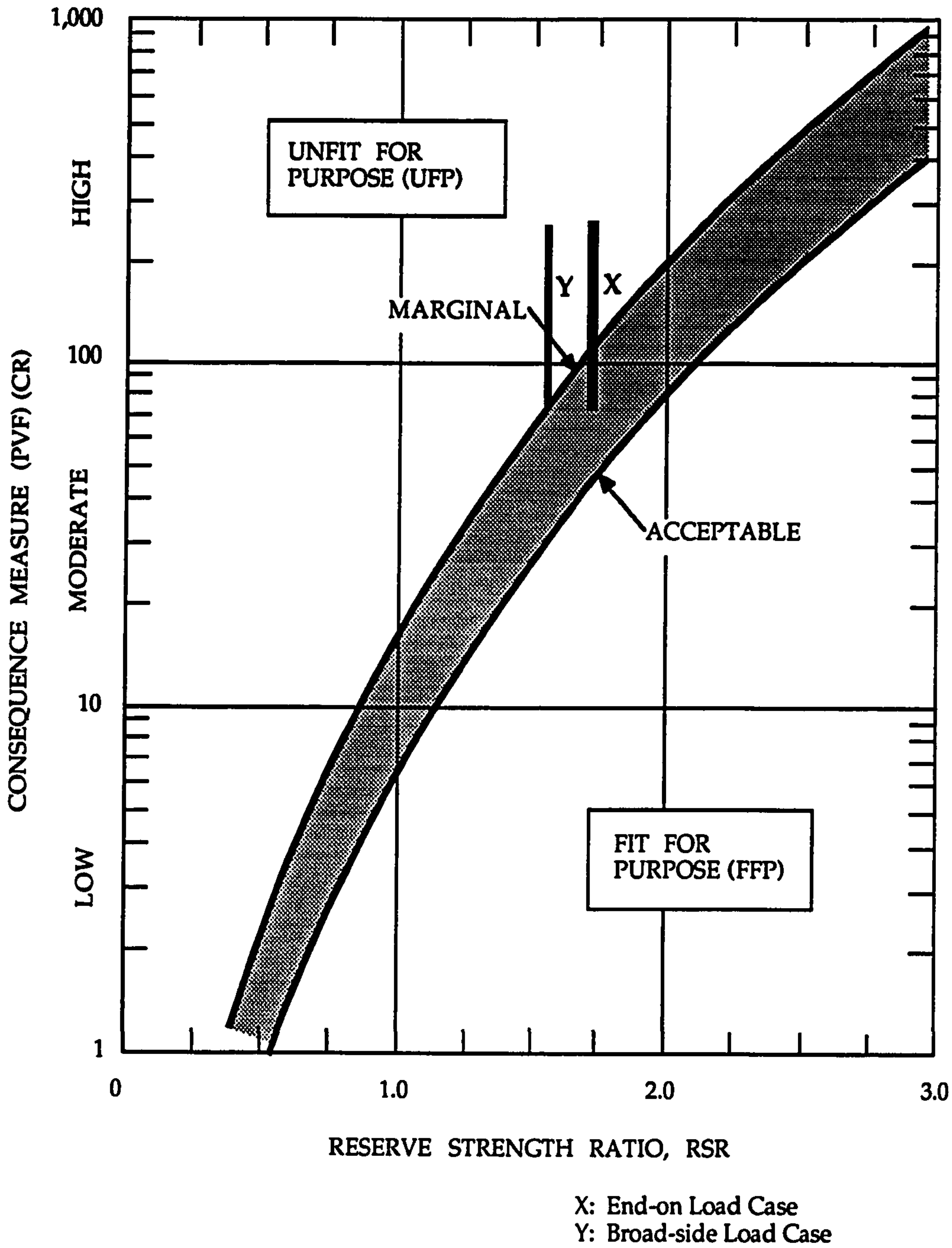
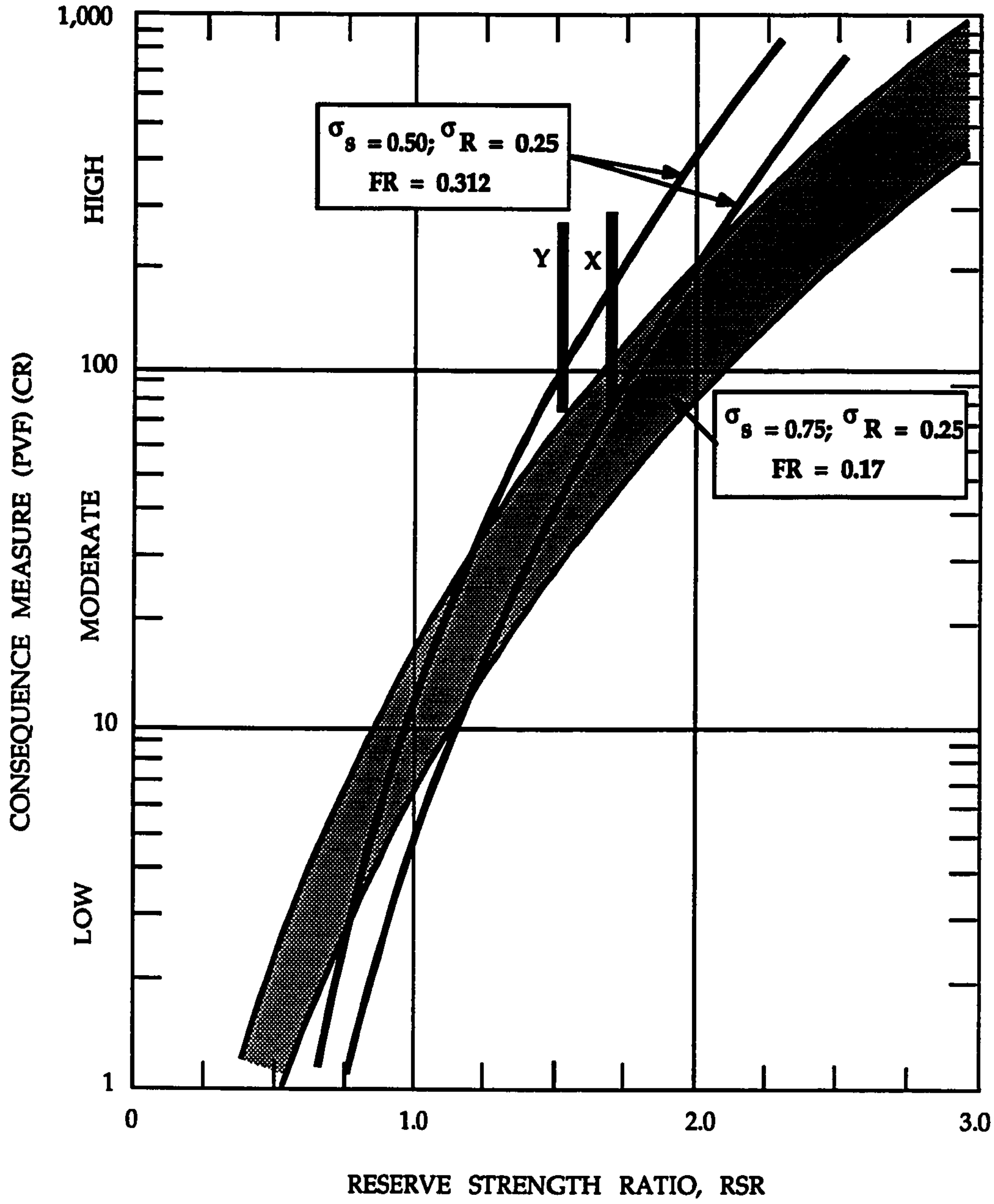
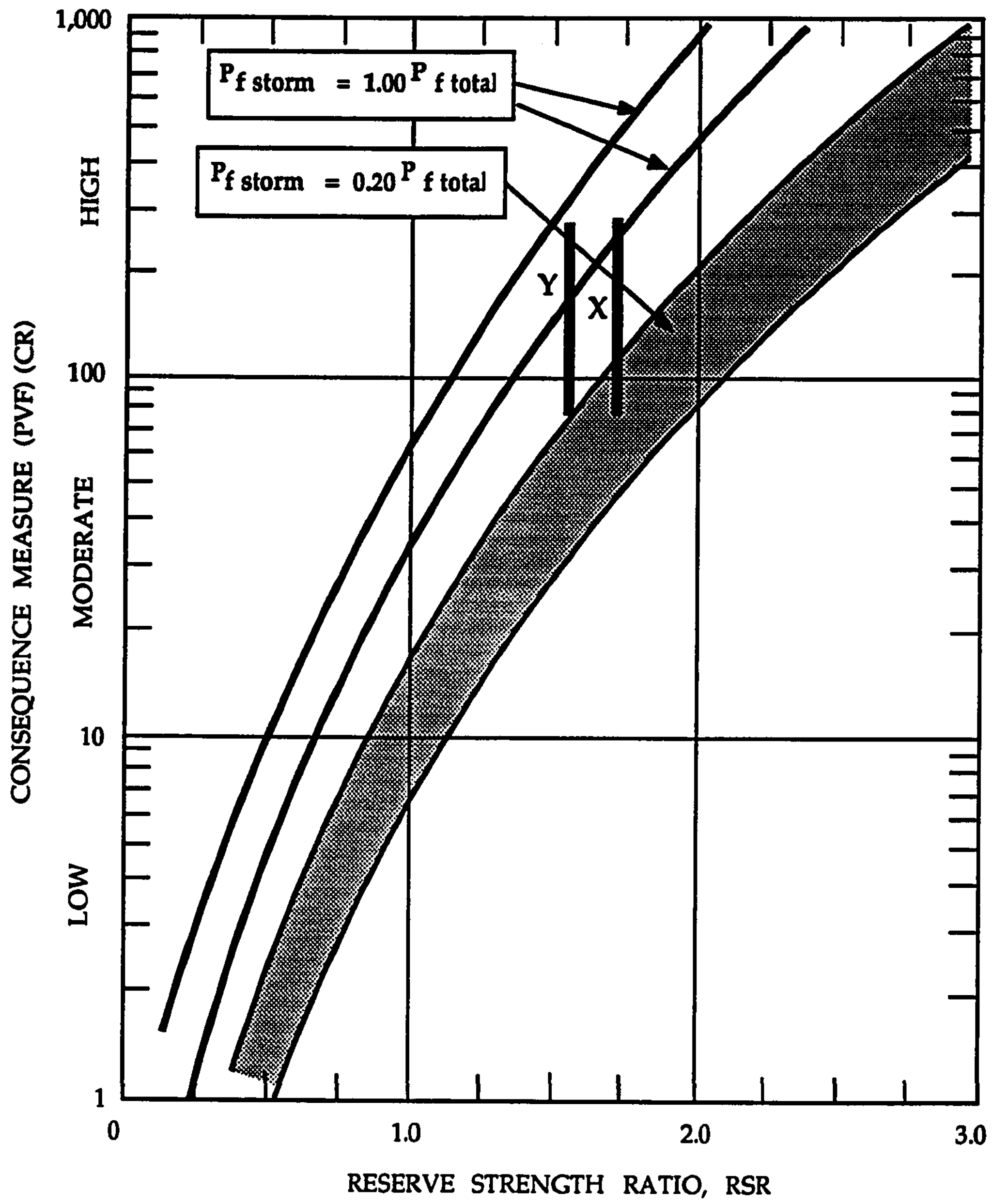


Figure 7.5: Fitness For Purpose (FFP) Evaluation of Platform A



X: End-on Load Case
Y: Broad-side Load Case

Figure 7.6: Alternative Way to Improve FFP of Platform-A



X: End-on Load Case
Y: Broad-side Load Case

Figure 7.7: Alternative Way to Improve the Fitness For Purpose (FFP) Criteria for Platform -A

7.2 Platform - B

7.2.1 Description of Platform

Platform-B consists of a four-leg steel template jacket located in 140 ft. water depth in the Gulf of Mexico (Fig. 7.8). It represents a typical 4-legged aged platform operating in the Gulf of Mexico. It is an unmanned platform with 9 gas producing wells, and has been in operation for more than 25 years. Two risers are provided for transportation of gas to other platforms. In addition, sump caisson, pump casings, two boat landings, and four barge bumpers are provided.

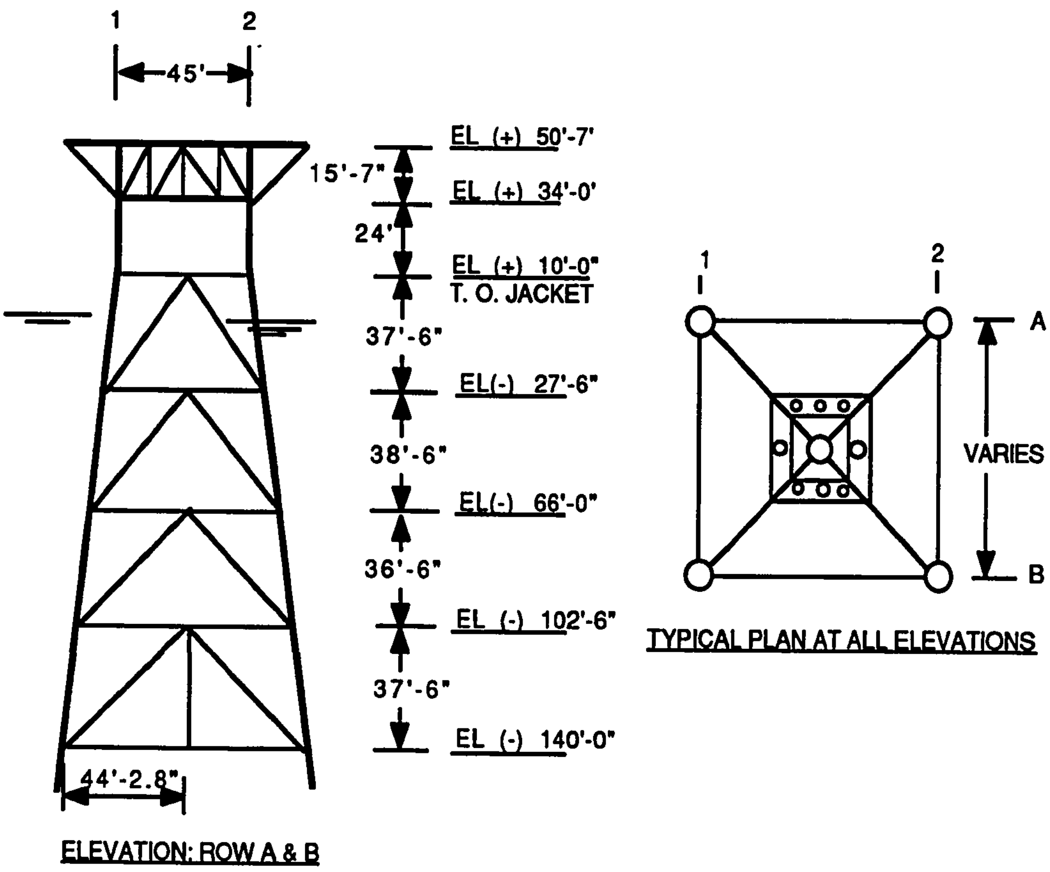


Fig. 7.8: Structural Configuration - Platform B

The total weight (dead + live) of topside facilities has been assumed as 3,500 Kips. The elevations of the upper deck are (+)50'-7" (center line) or (+)52' (Top of steel), of the lower deck is (+)34' (center line), and of the sump deck is (+)25'-6". The bottom of steel elevation of the lower deck is (+)33'-1.5", and the deck floor is grated. The deck legs have been connected with the pile-jacket structural system at Elev. (+)17'.

The general soil characteristics at the platform site are 10 ft. thick layer of soft clay overlying stiff clays. The foundation of the platform constitutes of 4-36"ϕ piles and a central pile/ conductor of 30"ϕ with a maximum penetration of 170 ft., and the pile-leg annulus is ungrouted.

This platform was designed in 1962 based on the 25-year return period wave with 46 ft. wave height. The marine growth is taken as 2.5" on diameter for all members between mean sea level and (-)28' and tapers off below (-)28' to the mudline. The nominal sizes of members (as per the as-installed drawings) have been used in this study, except where the platform records were updated based on post-installation inspection reports.

All the steel members have a nominal yield stress of 36 ksi. In this study an expected (mean) yield stress of 45 ksi has been used, to account for bias in yield strength and strain rate effect.

The structural configuration of the platform is symmetric, with ungrouted legs, and no joint cans on the legs. The strength evaluation due to loads from the diagonal direction may be needed for the deck and foundation bays.

Damages have been reported in this platform. The prominent damages reported are in the vertical frames comprising of one vertical brace each in the second and third bays of the jacket. The influence of these damages on evaluation of RSR and safety assessment is demonstrated.

7.2.2 Screening Cycle-1

The evaluation at screening cycle-1, based on Section 3.3 is described in this section.

Screening Cycle- 1A: Cut-off criteria: The platform is evaluated for the major conditions to determine the likelihood of significant increase in load and/or strength levels. The process followed is as given in Fig. 3.8 for load level evaluation and Fig. 3.9 for strength level evaluation. The as-design characteristics of the platform are described in the previous section.

The platform has been designed for 25-year return period. Therefore, based on Fig. 3.8, the first condition is met and the platform is identified to have likelihood of significant increase in lateral load. The platform is screened out for further evaluation at screening cycle-2. However, for demonstration purpose, its evaluation at screening cycle-1B is presented.

Screening Cycle-1B: The evaluation is aimed at determination of the variations in the capacity (RSR) for each bay of the platform. For each bay of the platform, the strength and load factors (R, S) discussed in Section 3.3.3 are estimated by comparison of the original design criteria with the latest API reference level criteria. Then a numerical value for RSR is obtained, which gives an approximate idea of variation in RSR from a central figure of say 1.25 (as assumed in this study). The following factors are determined for load and strength for the critical (damaged) jacket bay.

Jacket Bay:

Material factor	R1	1.25	due to Fy
Platform condition factor	R2	1.0	when no damage
		0.75#	when one brace is damaged
Platform modifications factor	R3	1.0	None
Structural configuration Factor	R4	0.8	Inverted V-brace in vertical frames
Design criteria variation factor	S1	2.04*	(API reference level wave = 1.43 x Design wave)
Deck elevation factor	S2	1.2**	Wave hit the deck
Platform modification factor	S3	1.0	None

One vertical brace out of total of 4 in a jacket bay in the frames along one direction is damaged. Therefore, the bay strength would reduce by approximately 25 %. i.e., R2 ≈ 0.75.

* Design criteria variation would be due to wave heights and drag coefficients used in the original design and the as-is state of platform. Drag coefficient used is similar (= 0.6) but current API reference level wave height is 143% (=65.7/46) higher. Therefore the wave load increase would be proportional to 2.2 (= α) power of ratio of wave heights (= 1.43). Therefore S1 would be equal to 2.04.

** The bottom of steel elevation of lower deck is (+)33'-1.5". Total of storm tide and surge is 4.4'. Wave crest height could be approximately evaluated as 0.55 H to 0.60 H, i.e., 36' to 39'. The crest elevation from m.s.l. would be 40.4' to 43.4'. Therefore, wave will be approximately 7 to 10 ft. into lower deck and S2 ≈ 1.2.

Therefore variation in RSR ≈ [1.25 x 1 x 1 x 0.8 / (2.04 x 1.2 x 1)

= **0.41**, when the platform has no damage

Therefore variation in RSR = [1.25 x 0.75 x 1 x 0.8 / (2.04 x 1.2 x 1)

= **00.31**, when the platform has damage i.e., according to the assumed qualitative classification of RSR, this platform in its damaged and undamaged states would be termed with LOW RSR (0.8 to 1.0)

Consequence level: The platform is 4-legged unmanned, well platform with 9 gas wells and 2 risers provided to transport gas to other platforms. This platform has been in-service for more than 25 years.

Therefore, with this platform, there is no likelihood of loss of lives, minimal environmental pollution due to gas leakage, loss of production and resources will be lower, influence on operation of other platforms connected by risers will be minimal because gas is transformed from this platform to others, moderate loss of property in case the platform need to be replaced in 140 ft. water depth. According to the definition provided in Section 4.3, this platform would be classified as a Very Low Consequence (VLC) platform.

Fitness For Purpose (FFP) Evaluation: The qualitative formulation of FFP evaluation presented in Figure 7.9. For this platform based upon screening cycle-1B, the capacity measure (RSR) has been evaluated as VERY LOW for damaged and undamaged cases and the consequence measure has been identified as VERY LOW. These measures are evaluated in Figure 7.9 for both damaged and undamaged cases, they fall in the Marginal zone.

This platform is then screened out for further evaluation by the coarse quantitative method (screening cycle-2).

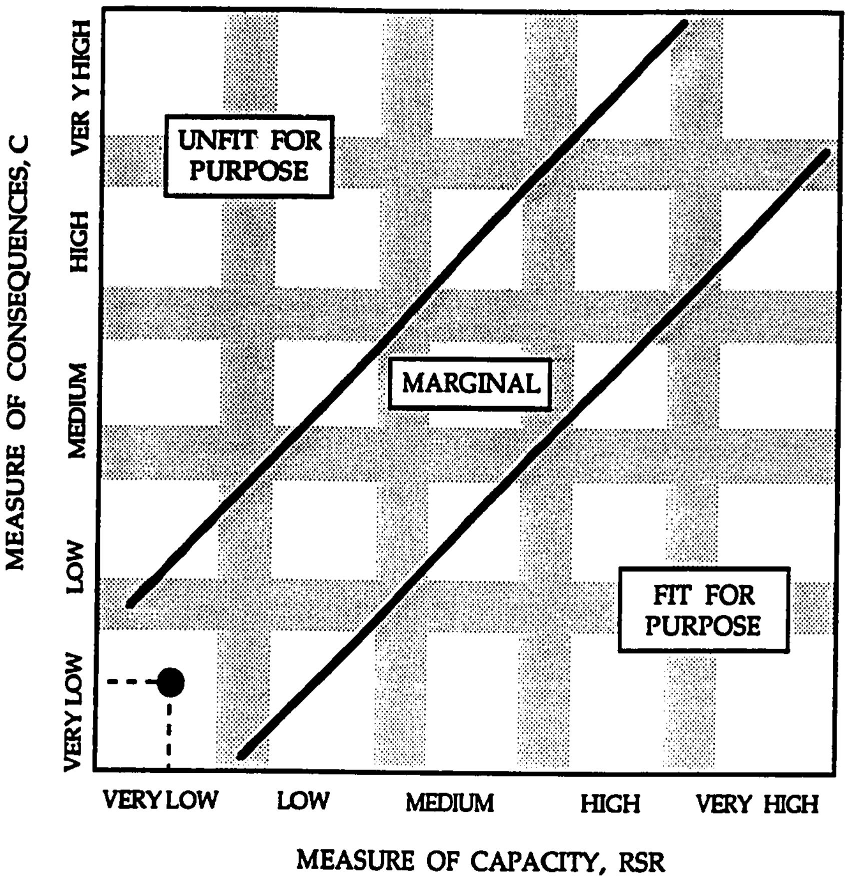


Figure 7.9: Fitness For Purpose assessment for Platform - B (Screening Cycle -1B)

7.2.3 Screening Cycle-2

At screening cycle-2, a coarse quantitative evaluation of the lower bound of RSR is done by following the procedure described in detail in Section 3.4. In this cycle, the load and strength patterns for the platform are developed, by determination of loads and strengths for the different bays on an individual basis. The nominal estimates of the load and strength for the different bays, and the uncertainties in their estimates are determined. The detailed computations were performed to estimate the loads and strength, and the results are summarized in this section.

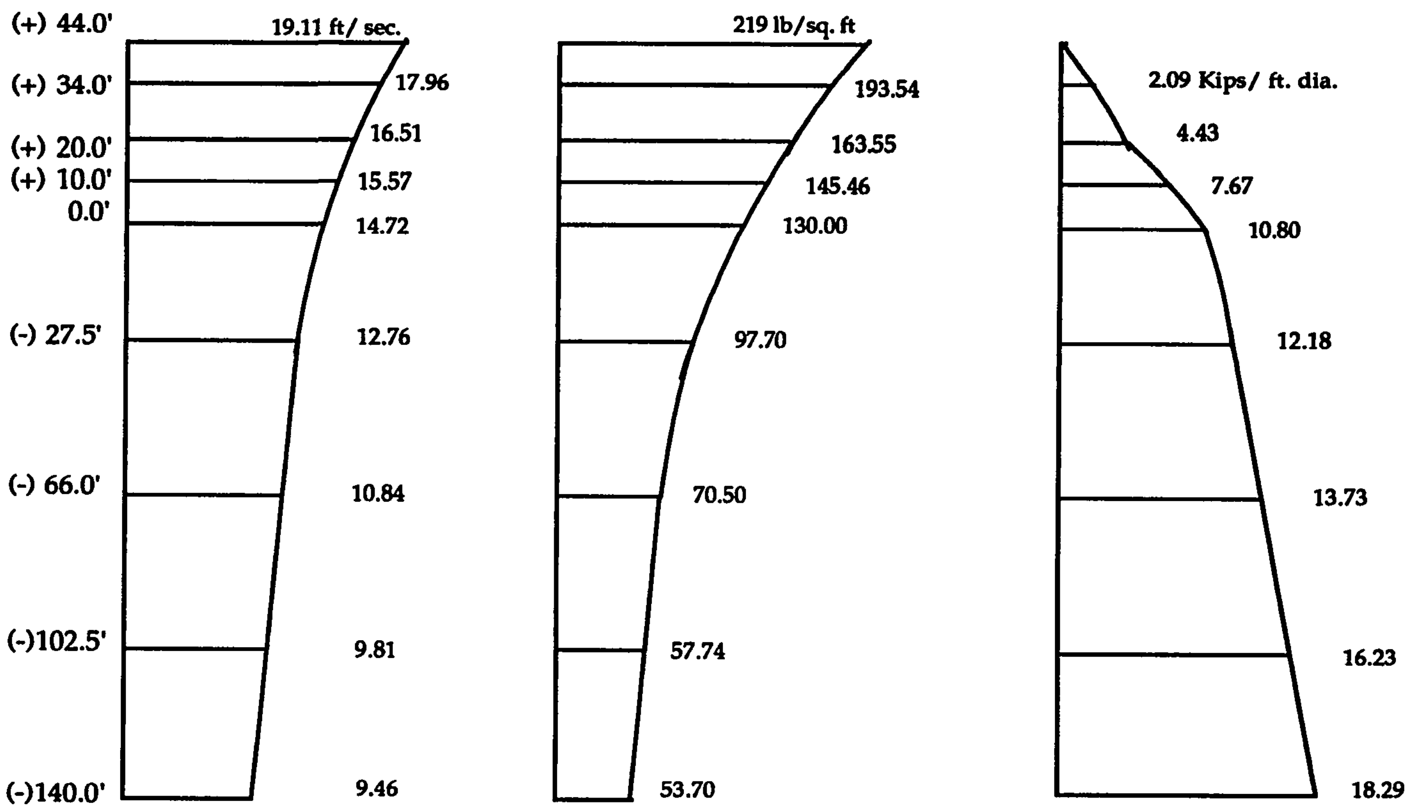
Development of the lateral load pattern:

The procedure described for Option-B in Section 3.4.1.1 is followed to develop the lateral load pattern. The wave crest elevation is approximately at (+) 44', which means that 11' of the the lower deck will be hit by the 100-year return period wave. The reference level lateral load will be summation of the wind and wave loads on the deck structure and equipment, and the wave loads on the jacket.

A wave kinematics profile is developed for wave height (H) of 65.7 ft. with time period (T) of 12.4 sec. on a unit diameter vertical pile extending from the seabed to above the mean sea level. Airy's linear wave theory stretched to the crest elevation has been used, considering the wave kinematics at wave crest same as at the m.s.l. The horizontal particle velocity at different horizontal framing elevations, at crest level, and at mudline are indicated in Fig.7.10.

The unit wave loads at these elevations and the overall load (shear) on the unit diameter vertical pile are also shown in Fig. 7.10. Based upon the unit load values given in Fig. 7.10, the wave load pattern for the complete platform is developed. The basic assumption made in determination of the wave load pattern is that the vertical pile is located at the middle of the jacket and the wave kinematics developed for it will be valid for all of the members of the jacket. Hence, the phase effect or spatial variation in the wave load with distance is neglected.

The wave loads in the two orthogonal directions would be nearly same due to symmetric structural configuration, and are obtained by following the procedure given in Sections 2.3 and 3.4. The results are summarized below:



a: Horizontal Particle Velocity, u vs. depth

b: Drag Force/ unit area

c: Cumulative Load Pattern

Figure 7.10: Wave Kinematics and Forces on a Vertical Unit Diameter Pile - PlatformB

Table 7.3: Lateral Loads Based on Components

Item	Load	Percentage Load
Wind Load	50 Kips	3.2
Wave Load:		
Deck	556 Kips	36.0
Main legs (4 No.)	213 Kips	13.8
Central leg	46 Kips	3.0
Conductors (8 No.)	229 Kips	14.8
Risers (2 No.)	35 Kips	2.3
Vertical Braces	137 Kips	8.9
Horizontal Braces	178 Kips	11.5
Appurtenances	100 Kips	6.5
Total Wave Load	1,494 Kips	
Total Lateral load	1,544 Kips	

In addition, it is assumed that the vertical load from the topside facilities is 3,500 Kips and that it is equally distributed on the four legs, i.e., 875 Kips on each leg.

From the above results, it is noted that for this platform the wind and wave loads on the deck structure are roughly 39% of the base shear (total lateral load); the wave loads on main legs, conductors, and risers are 34% of the base shear; and the vertical and horizontal braces contribute only 20% to the base shear. The jacket appurtenances such as boat landings, barge bumpers, casings, etc. have been assumed here to contribute 6.5%.

The summary of wave loads on each bay is given in Table 7.4. From Table 7.4, it is noted that about 69% of the base shear is due to the wind and wave loads in the deck bay and the jacket bay-I, i.e., up to El. (-) 27'-6". The bottom three jacket bays contribute a8-12% to the base shear.

Table 7.4: Lateral Loads Based on Bay

Item	Load	Percentage Load
Wind Load	50 Kips	3.2
Wave Load:		
Deck bay	678 Kips	43.9
Jacket bay-1	336 Kips	21.8
Jacket bay-2	186 Kips	12.1
Jacket bay-3	137 Kips	8.9
Jacket bay-4	124 Kips	8.0
Mud Level Framing	33 Kips	2.1
Total wave load	Σ = 1,494 Kips	
Total Lateral Load	1,544 Kips	

The overturning moment values at the top (deck-jacket connection) and bottom (mudline) of the jacket are also required. Based on the results obtained in Table 7.4, in a simple analysis the lever arm for obtaining the base moment could be assumed equal to the water depth. In this way, the overturning moments have been approximately evaluated as follows:

@ top of jacket: 29,000 Kips-ft.

@ bottom of jacket: 241,650 Kips-ft.

The bias in these results could be determined by comparison with the results obtained by the Dean's Stream Function theory by using a computer software. In this way, the base shear has been obtained as 1,825 Kips for Cd = 0.7 *[Bea et al, 1988]* or approximately base shear will be 1,630 Kips for Cd = 0.6.

Bias in lateral load = Actual / Predicted = 1,630/ 1,544 = 1.06

In case the elevation of the lower deck is higher than the wave crest elevation and the reference level wave passes below the deck, the reference level force reduces to 1,050 Kips by Option-B approach and will be approximately equal to 1,029 Kips for C_d = 0.6 by the Dean's Stream Function theory. Therefore, bias will be 0.98 (= 1,029/ 1,050).

Note that these bias estimates would differ for different platforms, due to variations in the percentage contribution of the different members. The bias is lower than 1.0, when only the jacket structure and its appurtenances are considered, because the reduced wave forces on the outer legs and vertical frames, and on the other members have not been considered. In the first case, when the wave hits the deck, the bias is higher than 1.0, which is due to the simplified method considered in evaluation of the wave load on the deck.
In our estimate, we have not considered the effects of wave run-up and draw-down, when the wave hits the deck and its equipment. At screening cycle-2, such simplifications may be accepted, whereas at the higher screening cycles the wave loads on the deck should be evaluated more accurately.

Development of the bay strength pattern:

The strength of each bay (or sub-structure) is independently evaluated at the failure of the first component (a brace, a leg, or a joint), by the procedure described in Section 3.4.2. The results obtained from detailed calculations are summarized in this section.

The axial capacities of the members would reduce due to: imperfections in the members; local lateral bending loads from the waves, current, and hydrostatic pressure acting on the members.

The structural configuration of the 4-bay jacket, as seen in Fig. 7.8 shows inverted V-braces in the vertical framings. The sizes of the braces vary from 14"ϕ x 0.375" to 18"ϕ x 0.375". The horizontal braces are also of similar sizes. The load transfer from an upper bay to the lower bay in this structure would typically be from the horizontal brace to the vertical-diagonal brace (inverted V-brace). Therefore, the load at the failure of first component in a bay would be based on the strengths of the leg-horizontal brace joint, outer horizontal brace, inverted V-vertical brace. The minimum strength of these three along the load direction will determine the lower bound strength of the bay. In each bay of this jacket, one vertical brace will be in tension and the other will be in compression. Therefore, the compression capacity of the brace, being lower than its tensile capacity will determine the bay lateral load at its failure.

The strength pattern for the platform has been developed based on the lateral load at first member failure (lower bound) and is shown in Fig. 7.11. The upper bound strengths for deck and foundation bays are also shown.

The lower bound estimate of the deck bay is related to the load at yielding of the first section, and the upper bound at the formation of a mechanism. The strengths for jacket bays have been determined for the grouted and ungrouted leg-pile annulus cases. A summary of the strength estimate is given in Table 7.5.

From this Table and Fig. 7.11, it is observed that in the ungrouted leg case, the joint failure by punching of the horizontal brace in the leg governs for all of the jacket bays. For the grouted leg case, the yielding of deck legs seems to govern the load carrying capacity.

Table 7.5: Bay Strength Evaluation:

Item	Horizontal brace size	Pum	Distribution Factor (Brace)	Pbay
Ungrouted Annulus:				
Jacket Bay -I	14" dia. x 0.375"	116	0.198	586 Kips
Jacket Bay -II	16" dia. x 0.375"	127	0.209	608 Kips
Jacket Bay-III	18" dia. x 0.375'	138	0.212	751 Kips
Jacket bay -IV	20" dia. c 0.375"	119	0.217	687 Kips
Grouted Annulus:				
Jacket Bay-I	14" dia. x 0.375"	247	0.148	1673 Kips
Jacket Bay-II	14" dia. x 0.375"	324	0.163	1987 Kips
Jacket Bay-III	14" dia. x 0.375"	345	0.169	2046 Kips
Jacket Bay IV:	14" dia. x 0.375"	402	0.177	2269 Kips

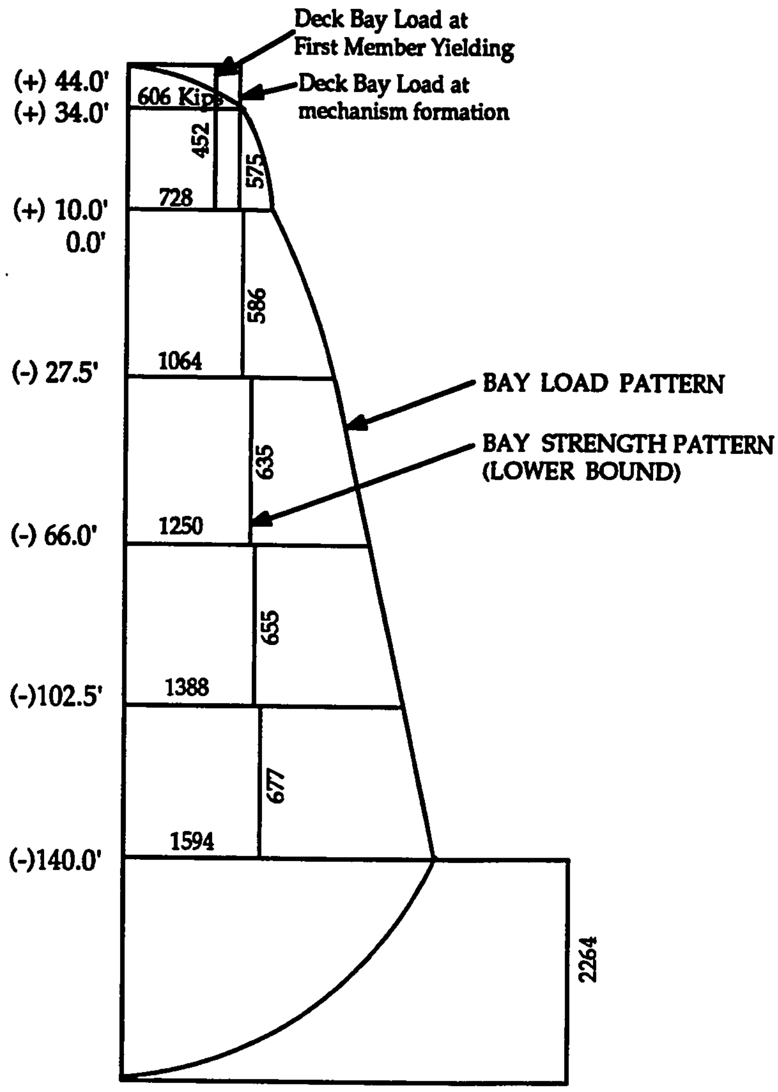


Figure 7.11: Load and Strength Patterns - Platform B

Determination of the Component Strength at failure of the first member: The component strength (CS_1) or the lower bound of RSR at failure of the first component (a joint, a brace, or a leg) in the platform is determined by comparison of the load and strength patterns. The load pattern is interpolated to determine the base shear at which it will meet any point on the strength pattern.

Note that an accurate evaluation of the base shear, which will cause the first member to fail, would require development of load patterns for reduced wave height(s). By use of computer software, if available, the load pattern could be developed easily for selected load cases.

The lower bound strength in the ungrouted case will be **677 Kips.** For the grouted leg case, the deck bay seems to govern the minimum load carrying capacity of the platform. A rough estimate of the base shear at failure of the first member in the deck bay could be made as follows:

$$\text{Lateral Load at Failure of the First Component} \approx 452 + (1{,}544 - 728) = 1268 \text{ Kips.}$$

From the static pushover analysis, the **upper bound** of the ultimate capacity of this platform in the ungrouted case is obtained as 1,060 Kips, which correspond to failure of leg-horizontal brace joint, vertical compression braces, yielding of deck legs sections with the increase in lateral loads and when the wave hits the deck. The failure of the components is concentrated in the deck bay (legs) and the jacket bay-I. The first member failure occurs at a load level of about 830 Kips.

Therefore, the redundancy factor (RF) is equal to 1.28 (= 1,060/ 830) for the ungrouted case. This redundancy is observed in this platform, due to the difference in the load at failure of the leg-brace joint and the load at yielding of the leg sections.

The upper bound of the ultimate strength for the grouted case has been determined as 1,155 Kips.

The biases introduced in the component strength (CS_1) estimates by screening cycle-2 are determined as follows:

Ungrouted Case: Bias = 1,060 / 677 = 1.57

Grouted Case: Bias = 1,155 / 1,268 = 0.91.

The nominal estimates of RSR for the two cases is determined as follows:

Ungrouted Case: RSR = 677 / 1,544 = 0.44 with a bias of 1.57 / 1.06 = 1.48

Grouted Case: RSR = 1,268 / 1,544 = 0.82 with a bias of 0.91 / 1.06 = 0.86

The mean values of RSR are then evaluated for use in the fitness for purpose (FFP) decision, in the following way:

$(RSR)_{mean} = (RSR)_{nominal}\ (Bias)_{RSR}$

Ungrouted case: $(RSR)_{mean} = 0.65$

Grouted case: $(RSR)_{mean} = 0.71$

Consequence Level:

The platform is 4-legged, unmanned, well platform with 9 gas-wells and 2 risers provided for transportation of gas to other platforms. This platform has been in production for more than 25 years.

Therefore, with this platform, there is no likelihood of loss of lives, minimal environmental pollution due to gas leakage, loss of production and resources will be lower, influence on operation of other platforms connected by risers will be minimal because gas is transformed from this platform to others, moderate loss of property in case the platform need to be replaced in 140 ft. water depth. According to the definition provided in Section 4.3, this platform would be classified as a Very Low Consequence (VLC) platform.

Fitness For Purpose Evaluation:

The fitness for purpose formulation given by equation 5.3 is generated for the uncertainties (standard deviations, or variances) associated with this platform. As discussed in Chapter-2, the uncertainties in load and strength could be taken as 0.66 and 0.25 respectively. Therefore the total uncertainty would be 0.71. The capacity consequence diagram is then developed for σs of 0.66, σ of 0.71 as shown in Fig. 7.12. From this figure, a judgement on the safety of the platform could be made.

The reference load on the platform would be the same for the two directions. The mean RSR evaluated for the grouted case was 0.71 and for the ungrouted case was 0.65, and its consequence level was designated as "Very Low." These values of RSR in conjunction with the consequence level of the platform are located as bar-X and bar-Y in Fig. 7.12. Based on the locations of the acceptance criteria (acceptable and marginal lines), the platform could be termed as fit for purpose or marginal for this level of reference force.

7.3 Platform - C

7.3.1 Description of Platform

Platform-C consists of an eight-leg steel template jacket located in 52 ft. water depth in the Gulf of Mexico (Fig. 7.13). It is more than 30-year old and represents an unconventional 8-legged platform which was designed during the early stage of offshore development, and it is currently operating in the Gulf of Mexico.

It is an unmanned tender drilling and production platform with 7 gas producing wells, and has been in operation for more than 30 years. Two risers are provided for transportation of gas to and from the platform. In addition, sump caisson, pump casings, one boat landing, and two barge bumpers are

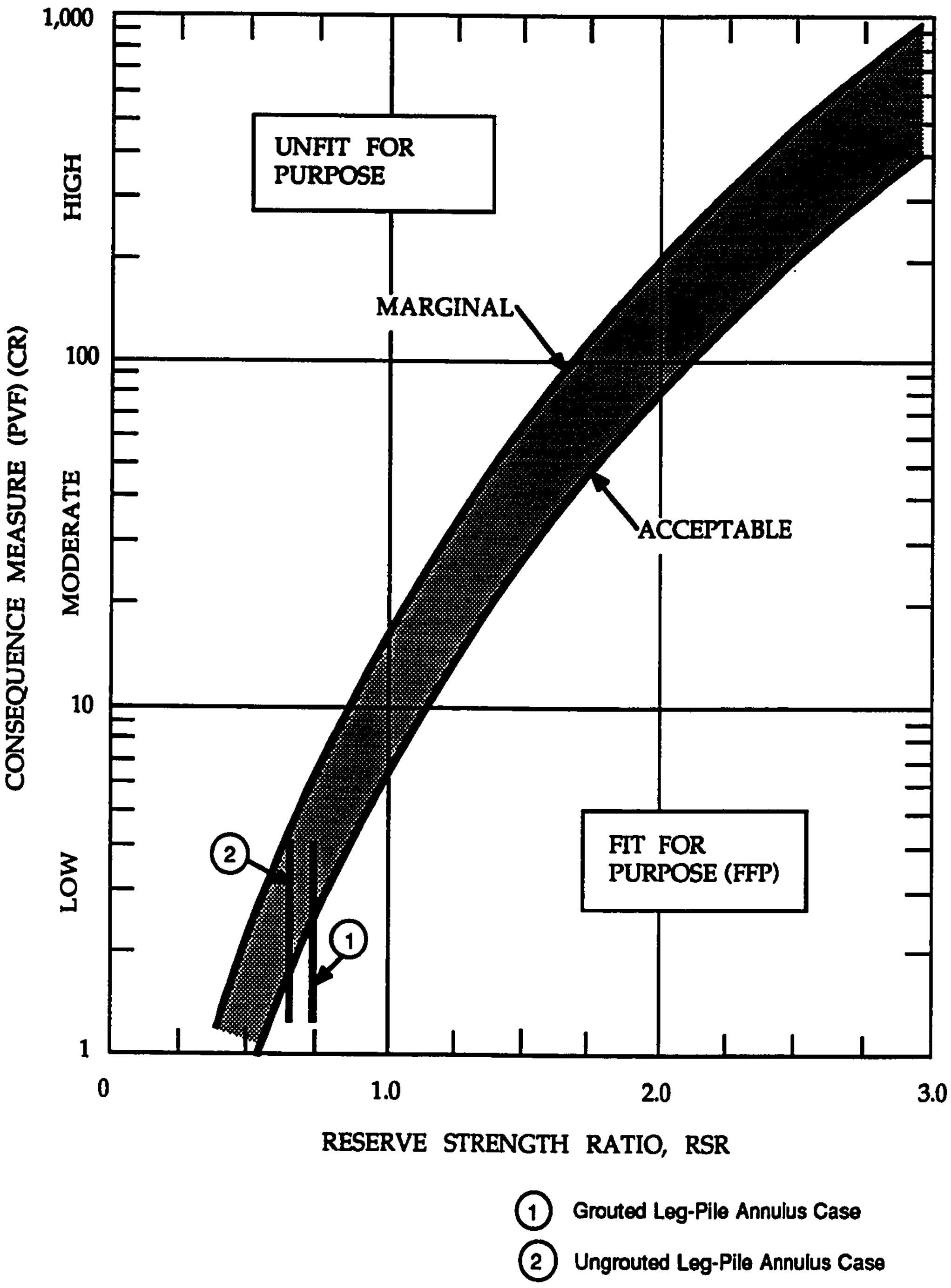


Figure 7.12: Fitness For Purpose (FFP) Evaluation of Platform B

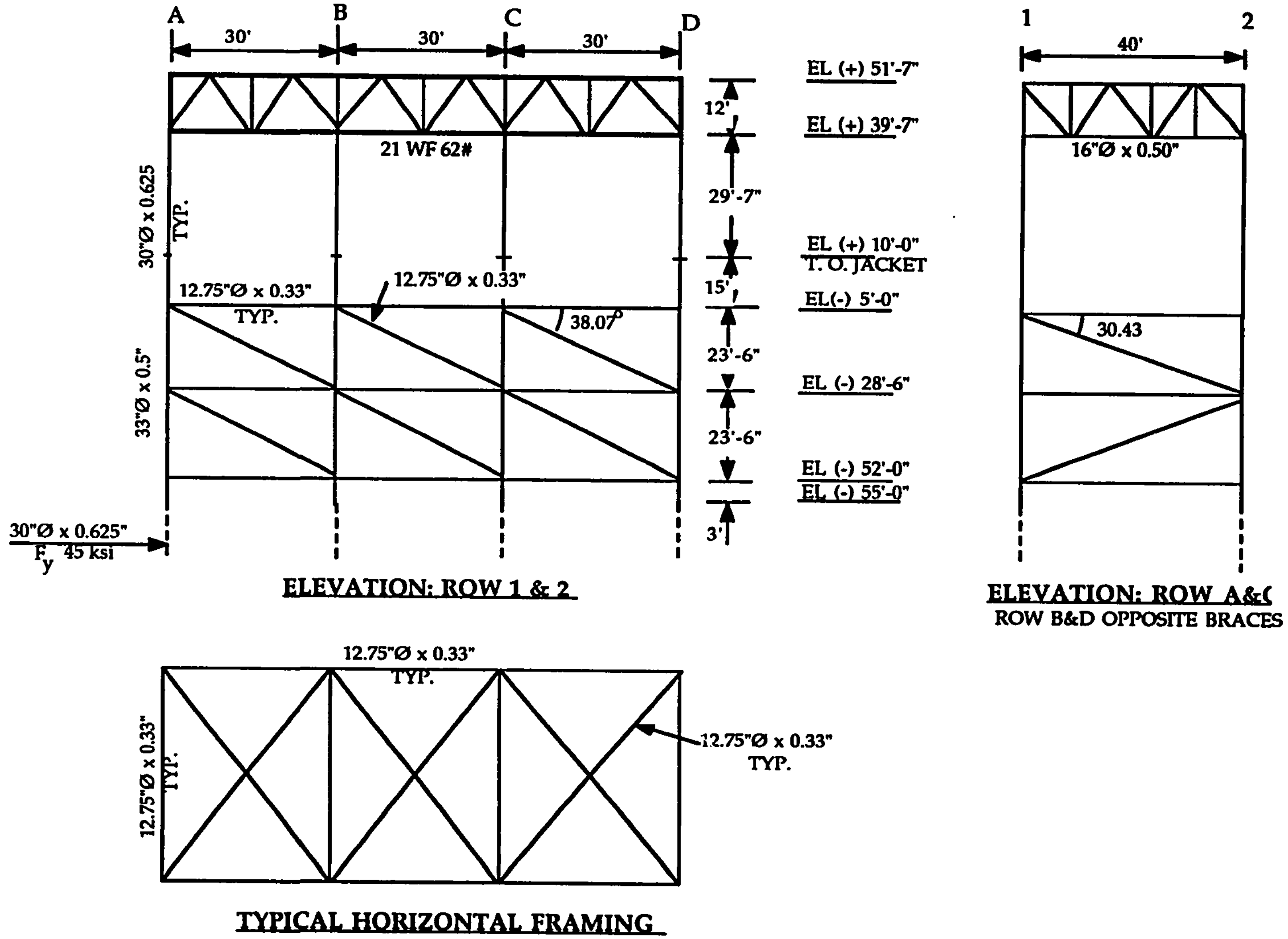


Figure 7.13: Structural Configuration - Platform C

provided. The total weight (dead + live) of topside facilities has been assumed as 500 Kips.

The elevation of the lower deck is (+)39'-7", of the upper deck is (+)51'-7". The bottom of steel elevation of the lower deck is (+)39', and is grated. The deck leg-pile connection is at elevation (+)10'.

The soil conditions at the site are characterized by 5 ft. of soft clay underlain by stiff clay to 115 ft, underlain by dense sand. The foundation of the platform constitutes of 8-30"ϕ piles and the conductor of 30"ϕ with a maximum penetration of 150 ft., and are grouted to the jacket legs.

The design of the platform is based on the 1959 design wave criteria based on 25-year return period wave with wave height (H) of 38 ft. and wave period (T) of 10 sec. The marine growth is taken as 2" on diameter for all members between mean sea level to the mudline. All the steel members have a nominal yield stress of 36 ksi. In this study an expected (mean) yield stress of 45 ksi has been used, to account for bias in yield strength and strain rate effect. The platform has no joint cans on legs.

Damages have been reported for this platform at the top horizontal framing of the jacket due to separation of one primary brace and dent and cracks formation in three other primary braces.

7.3.2 Screening Cycle-1

The evaluation at screening cycle-1, based on Section 3.3 is described in this section.

<u>Screening Cycle- 1A; Cut-off criteria</u>: The platform is evaluated for the major conditions to determine the likelihood of significant increase in load and/or strength levels. The process followed is as given in Fig. 3.8 for load

level evaluation and Fig. 3.9 for strength level evaluation. The as-design characteristics of the platform are described in the previous section.

The platform has been designed for 25-year return period. Therefore, based on Fig. 3.8, it is identified to have likelihood of significant increase in lateral load. Thus, it is screened out for further evaluation at screening cycle-2. But for demonstration purpose, this platform has also been evaluated at screening cycle-1B.

Screening Cycle-1B: The platform is evaluated to determine the variations in the capacity (RSR) for each bay of the platform. For each bay of the platform, the strength and load factors (R, S) are estimated by simple comparison of the original design criteria with the latest API reference level criteria. Then a numerical value for RSR is obtained, which gives an approximate idea of variation in RSR from a central figure of say 1.25 (as assumed for this study). The evaluation of likely variation in RSR for one critical jacket bay is given below:

Jacket Bay:			
Material factor	R1	1.25	due to Fy
Platform condition factor	R2	0.9#	Reduction due to damage to horizontal braces.
Platform modifications factor	R3	1.0	None
Structural configuration Factor	R4	1.0	
Design criteria variation factor	S1	1.38*	(API reference level wave = 1.16 x Design wave)
Deck elevation factor	S2	1.1**	Wave hit the deck
Platform modification factor	S3	1.0	None

The braces in the top horizontal framing in the jacket are damaged. These contribute to strength essentially upon failure of one diagonal brace in the vertical frame. Therefore, reduction in strength due to

their failure would be lower and it is assumed here as 10 %. Therefore $R2 \approx 0.90$.

* Design criteria variation would be due to wave heights and drag coefficients used in the original design and the as-is state of platform. Drag coefficient used is similar (= 0.6) but current API reference level wave height is 16% (=44/38) higher. Therefore the wave load increase would be proportional to 2.2 (= α) power of ratio of wave heights (= 1.16). Therefore S1 would be equal to 1.38.

** The elevation of lower deck is (+)39'-7". Total of storm tide and surge is 9'. Wave crest height could be approximately evaluated as 0.75 H, i.e., 33'. The crest elevation from m.s.l. would be 42'. These Therefore wave will hit the lower deck and S2 could be taken as 1.1

Therefore variation in RSR = [1.25 x 0.9 x 1 x 1 / (1.38 x 1.1 x 1)

= **0.74**, when the platform has damages.

i.e., according to qualitative classification assumed, RSR would be VERY LOW (<0.8) to LOW (0.8 to 1.0) for the platform in damaged state.

Consequence Level: The platform is 8-legged, unmanned, tender drilling and production platform with 7 gas-wells and 2 risers provided for transportation of gas to and from other platforms. This platform has been in production for more than 30 years.

Therefore, with this platform, there is no likelihood of loss of lives, minimal environmental pollution due to gas leakage, loss of production and resources will be lower, influence on operation of other platforms connected by risers will be moderate because gas is transported to and from this platform, moderate loss of property in case the platform needs to be replaced in 52 ft. water depth. According to the definition provided in Section 4.3, this platform would be classified as a Low Consequence (LC) platform.

Fitness For Purpose (FFP) Evaluation: The qualitative formulation of FFP evaluation presented in Fig. 7.14 is used. For this platform based upon screening cycle-1B, the capacity measure (RSR) has been evaluated as VERY LOW to LOW for damaged case, and the consequence measure has been identified as LOW. These measures are evaluated in Fig. 7.14 and they fall in the Marginal zone.

This platform is then screened out for further evaluation by the coarse quantitative method (screening cycle-2).

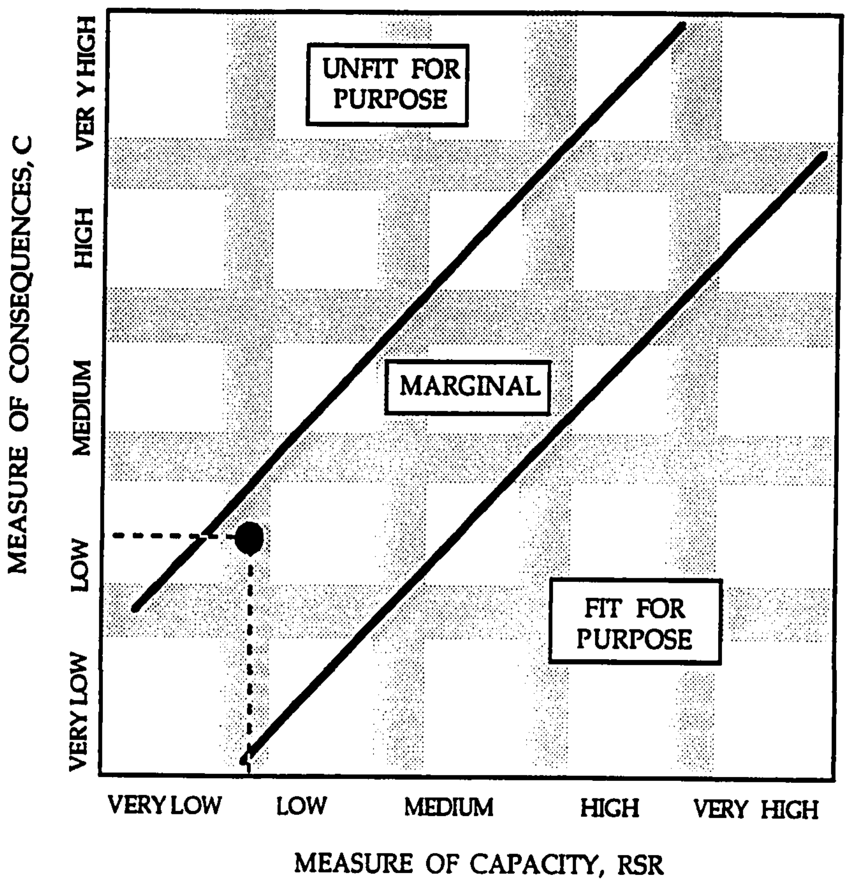


Figure 7.14: Fitness For Purpose assessment for Platform - C (Screening Cycle -1B)

7.3.3 Screening Cycle-2

At screening cycle-2, the load and strength patterns for the platform are developed to evaluate the lower bound of RSR, by following the procedure described in detail in Section 3.4. The detailed calculations for their estimation have been performed and the results are summarized and discussed in this section.

Development of the lateral load pattern: The procedure described for Option-B in Section 3.4.1.1 is followed to develop the lateral load pattern. The wave crest elevation is (+)42.44', which means that 3.5' of the lower deck will be hit by the 100-year return period wave. The 100-year return period API-reference level wave for this water depth is 44'. The reference level lateral load will be the summation of wind and wave loads on the deck structure and equipment, and the wave loads on the jacket.

A wave kinematics profile is developed for 44 ft. wave height with 10 sec. period on a unit diameter vertical pile extending from the seabed to above the mean sea level, based on Airy's theory. The kinematics at the mean sea level (m.s.l.) is stretched to the crest elevation. The horizontal particle velocity at different horizontal framing elevations, the crest elevation, and mudline were developed in a similar way as for Platforms A and B.

The unit wave loads at these elevations and the overall load (base shear and overturning moment) for a vertical pile were developed, and based on these the wave load pattern for the complete platform has been developed. The results obtained for the lateral loads on the complete platform are summarized in Table 7.6 and discussed here.

Table 7.6: Lateral Loads Based on Components

Item	End-on Direction Load	End-on Direction Percentage	Broad-side Direction Load	Broad-side Direction Percentage
Wind Load	50 Kips	5.2	100 Kips	9.0
Wave Load:				
Lower deck	126 Kips	13.1	284 Kips	25.6
Main legs	244 Kips	25.3	244 Kips	21.9
Conductors	219 Kips	22.7	219 Kips	19.7
Risers	24 Kips	2.5	24 Kips	2.2
Vertical Braces	63 Kips	6.5	66 Kips	6.0
Horizontal Braces	162 Kips	16.9	99 Kips	8.9
Appurtenances	75 Kips	7.8	75 Kips	6.8
Total Wave Load	913 Kips		1011 Kips	
Total Lateral load	963 Kips		1111 Kips	

In addition, the topside load of 500 Kips has been considered to be equally distributed on eight legs. This load is very low, and is neglected in further evaluations

From the above results, it is noted that for this platform the wind and wave loads on the deck structure are roughly 18% to 35% of the base shear (total lateral load); the wave loads on the main legs, conductors, and risers are 44 % to 51 % of the base shear; and the vertical and horizontal braces contribute 15 % to 25 % load. The jacket appurtenances such as boat landings, barge bumpers, casings, etc. have been assumed here to contribute 8 %.

Table 7.7 Lateral Loads Based on Bay

Item	End-on Direction Load	End-on Direction Percentage	Broad-side Direction Load	Broad-side Direction Percentage
Wind Load	50 Kips	5.2	100 Kips	9.0
Wave Load:				
Deck Bay	384 Kips	39.6	541 Kips	48.8
Jacket Bay-I	286 Kips	29.7	265 Kips	23.9
Jacket Bay-II	194 Kips	20.1	174 Kips	15.7
Mud Level Framing	49 Kips	5.0	30 Kips	2.7
Total Wave Load	913 Kips		1011 Kips	
Total Lateral load	963 Kips		1111 Kips	

The summary of wave loads for each bay is given in Table 7.7. From Table 7.7, it is noted that about 45% to 58% of the base shear is due to the wind and wave loads in the deck bay. The jacket bay-I, i.e., up to El. (-) 28.5', contributes about 24% to 30% to the base shear, whereas the jacket bay-II contributes between 16% to 20%.

The bias in these results could be determined by comparison with the results obtained by Dean's Stream Function theory by using a computer software. In this way, the base shear has been obtained by others as 943 Kips and 1,329 Kips for the end-on and broad-side directions respectively.

Bias in lateral load (end-on direction) = 943/ 963 = 0.98

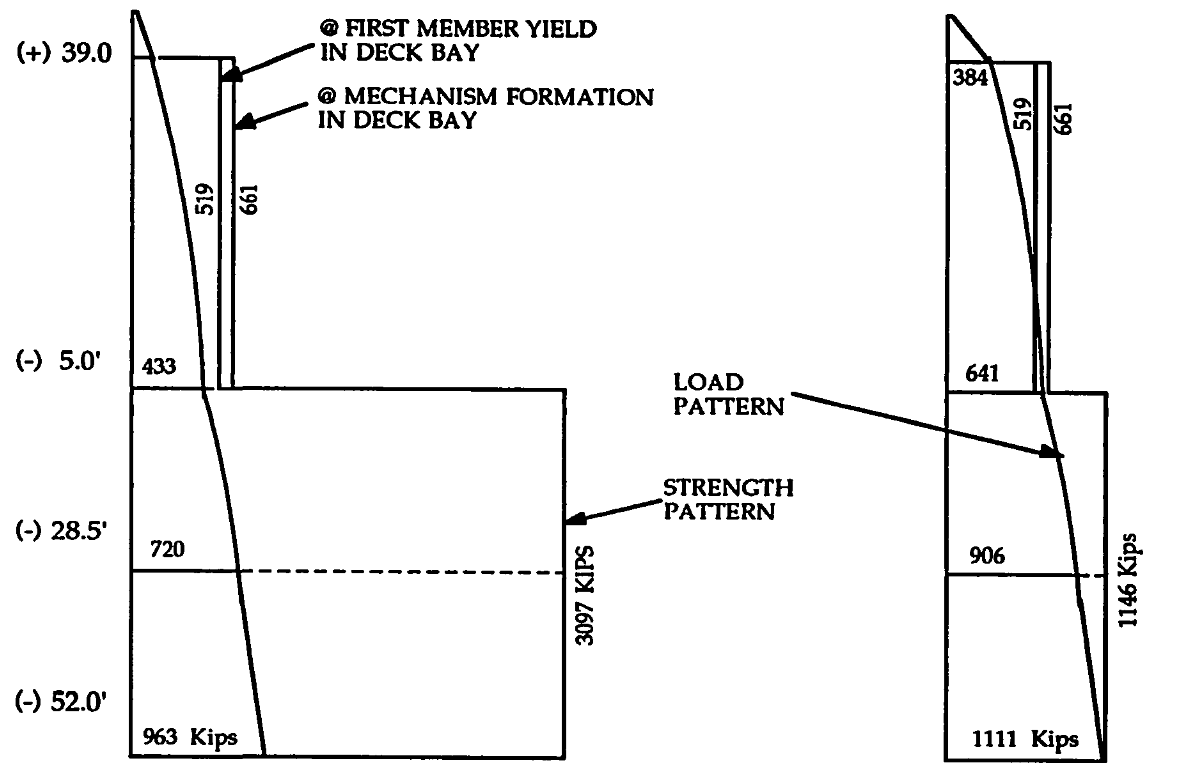
Bias in lateral load (broad-side direction) = 1,329/ 1,111 = 1.20

The above bias would exist due to several reasons in the lateral loads evaluated in this way, which are primarily due to wave forces on the deck, the reduced wave forces on the outer legs and vertical frames, and on the other members. In the computations of the wave loads on the deck, the wave run-up has not been included. In real situation, there will be wave run-up on the equipment, and the nominal wave loads will be higher.

Development of the bay strength pattern:

The strength of each bay (or sub-structure) is independently evaluated at failure of the first component (a brace, a leg, or a joint), by the procedure described in Section 3.4.2. The summary of the results is given in this section.

End-on load case: When the wave approaches from the positive X-direction, all of the three braces along the rows A and B would be in tension. The strength pattern for the platform has been developed on the basis of the first member failure (lower bound) and is shown in Fig. 7.15. The upper bound strength levels for the deck bay is also shown.



a: End-on Load Cases

b: Broadside Load case

Figure 7.15: Load and Strength Patterns- Platform C

Broad-side load case: The strength pattern for the broadside lateral load case is also shown in Fig. 7.15.

These patterns show an irregularity in the bay strengths at the failure of first member in the individual bays. The lower bound estimate of the deck bay is related to the load at yielding of the first section, and the upper bound represent the load level at the formation of a mechanism.

In the end-on load case, the strength of deck bay at first yield of a section is 519 Kips and at formation of a mechanism is 661 Kips. The bay strength of the jacket bays I and II would be same and it is 3,097 Kips, when the tensile strength of all of the vertical braces in a bay governs. Therefore, from Fig. 7.15, it is observed that the deck bay will govern the load level at failure of first component and the ultimate load at collapse of the jacket.

In the broadside load case, the strength of the jacket bays I and II is 1146 Kips. From Fig. 7.15, it is observed that sections in the deck bay and the compression vertical braces in the jacket bay-II are likely to fail.

Determination of the Component Strength at failure of the first member: The component strength (CS_1) or the lower bound of RSR at failure of the first member in the platform is determined by comparison of its load and strength patterns. The load pattern is extrapolated in this example to determine the base shear at which it will meet any point on the strength pattern.

Note that an accurate evaluation of the base shear, which will cause the first member to fail, would require development of load patterns for increased wave height(s). Such load patterns can be developed easily with the use of computer softwares. A rough estimate of the base shear at failure of the first member in any bay could be made from Fig. 7.15 as follows:

End-on load case: $\approx 519 + (963 - 433) = 1{,}049$ Kips.

Broad-side load case: $\approx 519 + (1{,}111 - 641) = 989$ Kips

In this example, the base shear at formation of plastic mechanism in the deck bay can be determined as:

End-on load case: $\approx 661 + (963 - 433) \quad = 1{,}191$ Kips.

Broad-side load case: $\approx 661 + (1{,}111 - 641) \quad = 1{,}131$ Kips

From the static pushover analysis (refer Section 3.6), the lower bound estimate at failure of first leg section is 960 Kips for the end-on load case, whereas the **upper bound** of the ultimate capacity is obtained as 1,130 Kips. The upper bound estimate correspond to the yielding of the 8-leg sections. The nominal estimates obtained above are close to these results.

In case of the broadside load case, the results obtained by the static pushover analysis are lower than compared with the nominal estimates.
By static pushover analysis, the first member failure occurs in the vertical compression brace in the jacket bay-II at base shear of 650 Kips and the ultimate capacity of the platform is about 760 Kips. The nominal estimates obtained above identify the weak links in the structure as the deck bay and the jacket bay-II.

Note that for this platform, we are getting a higher strength level at screening cycle-2 than that by the static pushover analysis at screening cycle-4. One of the reason for this difference is that we are not considering the upward movement of the elevation of the resultant lateral load when the wave hits the deck. Such consideration will be more important for the broadside load case, where the most likely to fail (MLTF) member is in the second jacket bay.

The biases introduced in the component strength (CS_1) by screening cycle-2 are determined as follows:

End-on case: Bias = 1130 / 1,049 = 1.08.

Broad-side case: Bias = 760 / 989 = 0.77

The load at failure of the first member is equal to 0.85 of the ultimate strength (R_u). This shows low redundancy level for the platform.

The nominal estimate of the RSR for the two directions is determined as follows:

End-on case: RSR = 1,049/ 963 = 1.09 with a bias of 1.08 / 0.98 = 1.10

Broad-side case: RSR = 989 / 1111 = 0.89 with a bias of 0.77/ 1.20 = 0.64

For this example, the the results obtained at screening cycle-2 are higher for the component strength (lower bound of RSR) for the broadside load case. Hence, the RSR obtained in this way is also higher and the associated bias is lower due to reduced bias in the component strength.

The mean value of RSR is then evaluated for use in the fitness for purpose (FFP) decision, in the following way:

$(RSR)_{mean} = (RSR)_{nominal}\ (Bias)_{RSR}$

End-on load case: $(RSR)_{mean} = 1.20$

Broadside load case: $(RSR)_{mean} = 0.57$

Note that these mean RSR would have associated uncertainty in their estimates, incorporated in terms of variance. The variance in RSR would be the combination of variances in load and strength.

Consequence Level:

The platform is 8-legged, unmanned, tender drilling and production platform with 7 gas-wells and 2 risers provided for transportation of gas to and from other platforms. This platform has been in production for more than 30 years.

Therefore, with this platform, there is no likelihood of loss of lives, minimal environmental pollution due to gas leakage, loss of production and resources will be lower, influence on operation of other platforms connected by risers will be moderate because gas is transported to and from this platform, moderate loss of property in case the platform need to be replaced in 52 ft. water depth. According to the definition provided in Section 4.3, this platform would be classified as a Low Consequence (LC) platform.

Fitness For Purpose Evaluation:

The fitness for purpose formulation given by equation 5.3 is generated for the uncertainties (standard deviations, or variances) associated with this platform. As discussed in Chapter-2, the uncertainties in load and strength could be taken as 0.66 and 0.25 respectively. Therefore the total uncertainty would be 0.71. The capacity consequence diagram is then developed for σs of 0.66, σ of 0.71 as shown in Figure 7.16. From this figure, a judgement on safety of the platform could be made.

The reference load on the platform would be same for the two directions. The mean RSR evaluated for the end-on case was 1.20 and designated as "Very Low." This RSR in conjunction with very low consequence level of the platform is located at point X in the Figure 7.16. Based on this location, the platform could be termed as fit for purpose.

The mean RSR evaluated for the broadside case was 0.57 and given consequence level as "very low consequence," the platform is located by Y on the capacity-consequence diagram. Therefore, the platform could be termed as marginal or unfit for purpose for this level of reference force.

7.4 Summary

The screening cycle methodology developed in this study has been applied on three example platforms located in water depths from 52 ft. to 271 ft. and their structural configurations vary from 4-leg to 8-legs. The

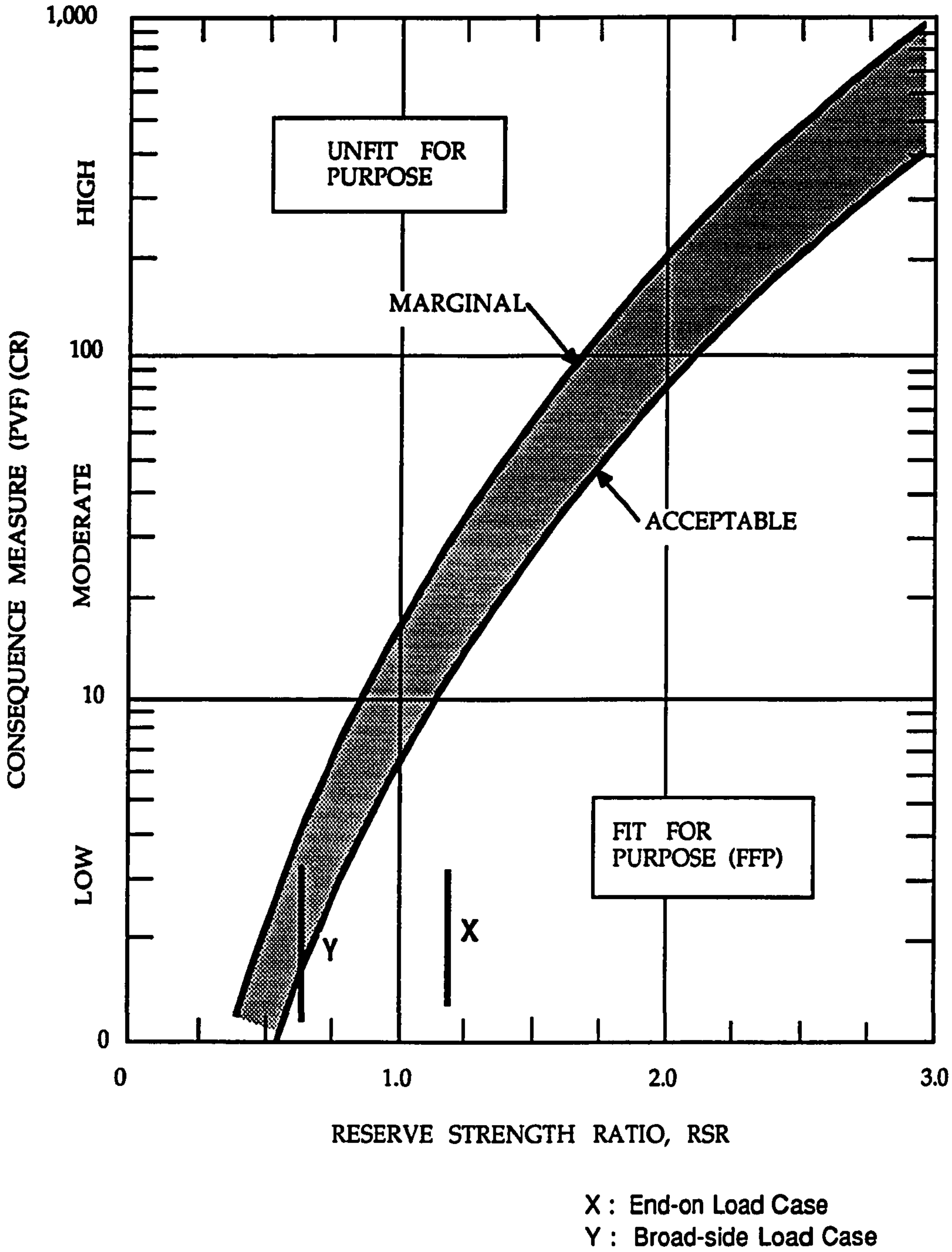


Figure 7.16: Fitness For Purpose (FFP) Evaluation of Platform C

operational criteria for these platforms varied from drilling platform to integrated drilling cum production platform.

Screening cycle-1A cut-off criteria screens out Platform B and Platform C at the first condition considered in the process (Whether the platform was designed for 25 year return period wave condition?) for further evaluation at screening cycle-2. However, screening cycle-1B was applied to all three platforms and based on this, Platform A was classified as Unfit For Purpose, Platform B and Platform C as Marginal. Based upon the overall methodology presented in Fig. 1.3, all the three platforms required further evaluation at screening cycle-2.

At screening cycle-2, the load and strength patterns for each platform were developed to evaluate the nominal estimate of RSR. To this a bias factor, established for a class of platform is multiplied to obtain the mean value of RSR. In these examples, the bias factor has been determined based on the differences in the load and strength values obtained by the method developed in this study for screening cycle-2 and the results obtained by computer based static pushover analysis.

The mean values of RSR and consequence estimates obtained for these platforms are then compared in the fitness for purpose formulation presented in this study. From this, we obtained approximate estimates of fitness for purpose for these platforms. Platform A was identified as Unfit for Purpose, and Platform B and Platform C were identified as Marginal.

The influence of selected alternatives to improve FFP of Platform A were studied. The results indicate that Platform A could be classified as Marginal, in case the standard deviation associated with the load effects could be reduced from 0.75 to 0.50, and it could be classified as Fit For Purpose in case the risk associated with other hazard sources is eliminated and the risk associated with storm waves could be increased from current 20% to 100%.

From these examples, it is proved that the methodology developed at screening cycles 1 and 2 is pragmatic and useful for periodic screening of a very large number of platforms existing in the Gulf of Mexico. These methods are simpler and quicker to apply, and they incorporate the historical experience available with these platforms. It may be noted, that these methods could be refined over a period of time upon their application on a larger number of platforms. Hence, it is believed that the bias in the results obtained by these methods could be reduced over time and with further establishing biases for different categories and sub-categories of platforms, e.g., one sub-category could be 8-legged platforms in 200' to 300' water depth and located in storm dominated region.

The efficiency in application of these methods could be further improved with the development of computer programs. For screening cycle-1, due to its dependence on heuristic knowledge, the development of an expert system is recommended. The basis in the development of such an expert system for screening cycle-1A is discussed in the next chapter. Screening cycle-1B and screening cycle-2 could also be developed in an expert system form, with development of interfaces for suitable wave load evaluation and structural analysis packages.

CHAPTER 8

BASIS FOR DEVELOPMENT OF AN EXPERT SYSTEM

An effort has been initiated in this study to develop a basis for preparation of a knowledge-based expert system based upon this methodology. The expert system development in this methodology is suggested for screening cycle-1. The two phases of screening cycle-1 were described in Chapters 3 and 4 of this study and they have provided a basis for such a development.

In this chapter, first the major components and the different stages in preparation of an expert system are discussed. Then a demonstration of the concepts required for preparation of a knowledge base have been presented. In essence, the **knowledge elements** required to make an assessment of the capacity level of a platform have been identified and described through knowledge trees. Based on these knowledge trees, the knowledge could be transferred into rules by the use of commercially available expert system shells.

8.1 Major Components of an Expert System

The major components of an expert system are the **knowledge-base, inference mechanism,** and **user interface,** as shown in Fig. 8.1.

An expert system would provide a limited computational capability within itself due to the nature of their development. The computation capability can be provided more efficiently in an expert system by integrating it with other databases and software packages. Such an integration would require the development of efficient interfaces. The platform information

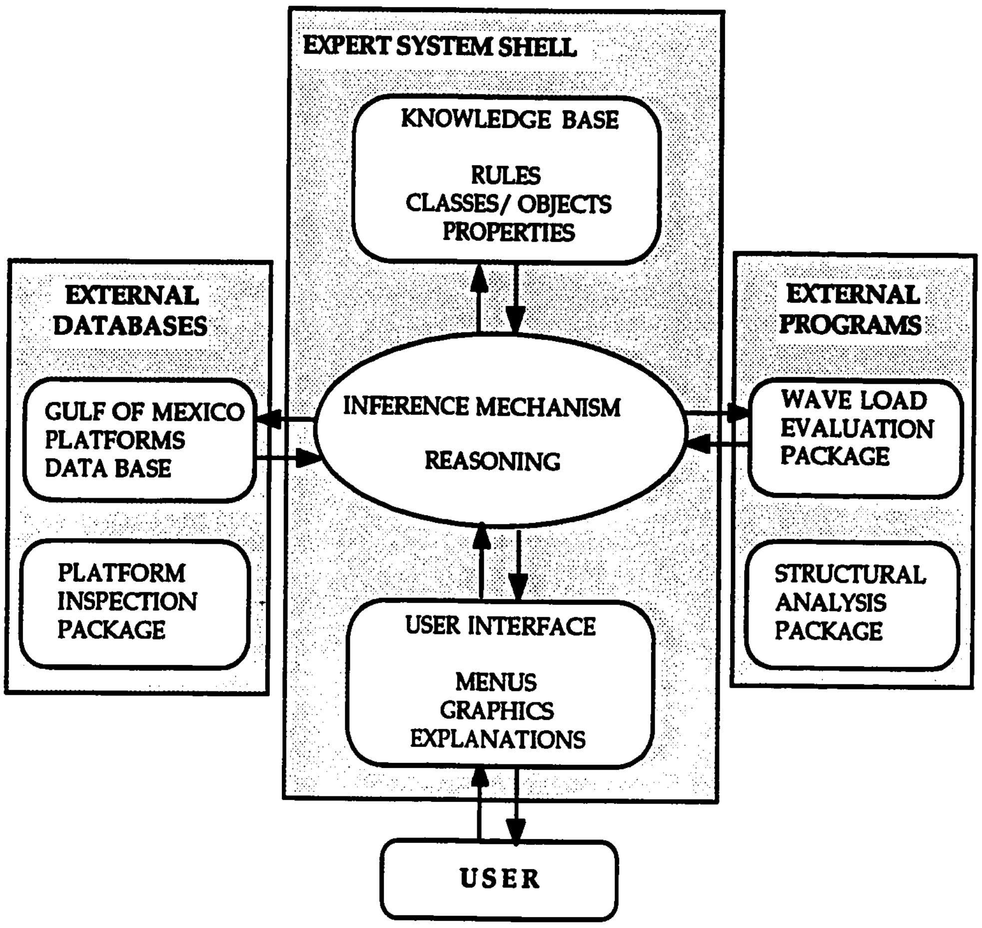


Figure 8.1: Organization of an Expert System

database could be similar to that prepared for the Gulf of Mexico platforms [*Dodson, 1987*]. A platform inspection package may also be integrated, which could be similar to the CAIRS package [*Frisbie, 1987*]. In addition, appropriate structural analysis and wave load evaluation packages could be integrated if such a system is planned for screening cycle 2, e.g., SACS, STRUCAD.

The sophistication of an expert system shell depends upon the capability of its inference mechanism, which provides a framework for editing and structured development of rules. The characteristics of an inference mechanism and the procedure and logic it follows in making decisions direct the development and chaining of rules, knowledge base, and user interface. The effectiveness of an expert system shell depends upon its inference mechanism. By the use of an expert system shell, the development effort required for a prototype expert system reduces to identification and preparation of knowledge base and user interface for a specific application.

A knowledge base is developed by identification of the rules and by description of the objects. It is done by selection of the basic elements in the process and their relationship among each other. The object-oriented development is represented by the different objects, classes, and methods. The rules in a knowledge base are the representation of the knowledge elements and they link the various objects through time. The class describes the structure of a set of objects. The properties and methods for the classes and objects are described by using the in built editors in an expert system.

The safety assessment process involves a very large number of parameters, and some of them are inter-related and have common properties, as described in Chapters 2, 3, and 4. By use of classes and objects, the problem can be structured in a manageable form. Also in this way, the individual classes, their methods, and objects can be updated without affecting the other classes.

Knowledge acquisition is done by: interviewing the experts; questionnaires sent to the experts; review of the technical literature; and performing specific studies. The knowledge base thus acquired will have uncertainties associated with the various rules, which can be represented by certainty factors or by probability theory.

An inference engine is a control mechanism which incorporates the reasoning process for selection of the rules in a knowledge base. Expert system shells are usually based on the forward-chaining (data-driven) of rules, backward-chaining (goal-driven) of rules, or a combination of both by their inference engines. The **forward-chaining** starts with examining (matching) of the premises of the rules with the available data for a platform, and with firing of those rules that immediately can be concluded. The new knowledge obtained from the conclusions arrived upon firing of a rule is then used to evaluate the premises of other rules. In this manner, with execution of several rules in a logical fashion, a final conclusion is obtained. In an inference engine based on **backward chaining**, the process starts with the goal (conclusion) part of a rule and it moves backward to the lower level by matching the platform information with the conclusions. In some shells, a mixed chaining strategy is incorporated and is found very useful due to an easier display of execution status and when a rule will be considered.

A user interface includes questions, statements, menu-sequences, graphic representations, and explanation statements for the different parameters. The explanation statements inform the user of the reasoning path the system is taking to solve a specific problem. An efficient user interface developed in this way would also form a useful training tool for less experienced engineers. This will be of great importance due to the limited resources of regulators and operators.

The ultimate system should be easy to integrate with external programs and should be portable on different hardware.

8.2 Stages in Building of an Expert System

The five stages in building of an expert system are shown in Figure 8.2. The various activities required under each of these five stages are briefly described here:

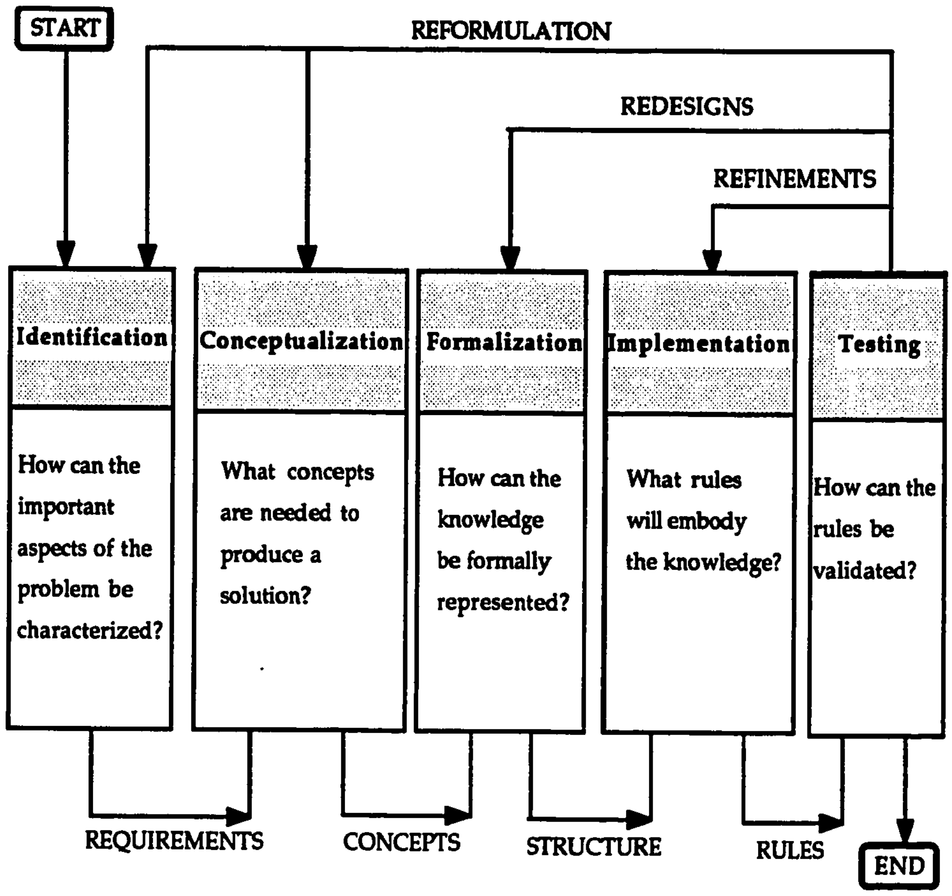


Figure 8.2: Development Phases for Building an Expert System

Identification: At this stage, the important features of the problem are identified and an evaluation of the alternative ways in which the different aspects of the problem can be characterized is made.

The important items here are: identification of the problem; type of problem; scope of the work; participants in the project; identification of the required resources; and the goals and objectives.

The goal in this study is to demonstrate the feasibility of preparation of a full-blown expert system for use by the regulatory bodies in re-assessment of the safety of the existing steel jacket offshore platforms. This demonstration aims at evaluation of the feasibility of development of an expert system for this methodology and to identify the knowledge elements which may be followed by future researchers for development of an eventual expert system.

As a first step, the expert system development may be attempted for screening cycle-1A cut-off criteria (refer Section 3.3).

Conceptualization: At this stage, the goal is to determine which concepts are needed to produce a solution. Thus, the emphasis is on the concepts, relations among themselves, and the control mechanism. Then the sub-tasks are determined, the strategies to solve the problem are formulated, and the constraints related to problem solving activity are explored.

The knowledge representation is normally initiated at an abstract level at this stage. An analysis of the complete problem is avoided before starting implementation of the program. The representation of knowledge would essentially follow the summary Figures 3.8 and 3.9 in Chapter 3 and are reproduced here as Fig. 8.3 and 8.4. A general discussion of the concepts involved is presented in section 3.3. Specific development of knowledge trees for organization of knowledge is given in the next section of this chapter,

Formalization: This phase emphasizes at the formal representation of the knowledge. The key concepts and relations are expressed within the framework developed in an expert system shell. The following are important in formalization of the knowledge trees and concepts:

a) Appropriate tools for the problem. (Knowledge engineer's work)

Appropriate tool for representation of the concepts and knowledge trees are needed. At the initial stage of an expert system development, a rule-based framework could be attempted. The uncertainty associated with the different rules could be represented by "certainty factors." Alternately, an object oriented programming mode may be selected. An expert system shell suited to the needs of the problem is then selected.

b) Output of this stage: At this stage, a decision is made on development of explicit domain knowledge i.e., the data structure, inference rules, and control strategies.

Implementation: At this stage in the development of an expert system, the formalized knowledge in rules form is developed in the form of a working computer program. The requirements of such a program are as follows:

a) It should proceed rapidly to check the decisions made during the earlier phases of development.

b) There is a high probability that the program code would need an updating in several iterations.

At the first stage in development of an expert system a rapid prototype be made to demonstrate the formal representation of knowledge in rule form and the user interaction in making decisions. In this way, the process of development and the chaining of rules and decision making process would become clearer to the identified panel of experts and an improved input could be obtained from them. At the second stage, the more elaborate expert system be developed.

Testing: During this stage, the performance and utility of the prototype is evaluated and revisions are planned, if necessary. The important items at this stage are listed as below:

a) The software is tested on some example platforms.

b) During this stage, the following problems may be uncovered:

- missing concepts and relations;
- knowledge represented at the wrong level of details;
- unwieldy control mechanisms.

Based upon testing of the rapid-prototype, the process of development may need re-formulation.

c) Important questions to be answered from testing: This phase is very important, because the feasibility of the ultimate system is dependent on the success in testing of the prototype model. During this phase, the focus is to evaluate the following:

- Does the system make decisions that experts generally agree are appropriate?
- Are the inference rules correct, consistent, and complete?
- Does the control strategy allows the system to consider items in a natural order?
- Are the system's explanations adequate for describing how and why the conclusions are being reached?
- Do the test problems cover the domain, handling the example cases and probing limitations in its use for expected hard cases?
- Does the solution of the problem help the user in a significant way?
- Are the conclusions appropriately organized, ordered, and presented at the right level of detail?
- Is the system fast enough to satisfy the user?
- Is the user interface friendly enough?

d) User's expectations of an expert system: The system should meet the user's expectations, if feasible. A user may expect the eventual expert system to show high quality performance and that it is fast, reliable, easy to use and understand, and provide default options to overcome mistakes.

8.3 Development of a Knowledge Base

In this study, a demonstration of the feasibility of production of an expert system based on the cut-off criteria at screening cycle-1A is evaluated. An expert system could also be developed for the screening cycle-1B due to the minimal computations required at this cycle. The preparation of an expert system for screening cycle-2, which requires substantial computations, could be achieved by development of interfaces to integrate independent analysis software as is shown in Fig. 8.1.

The overall organization of an expert system and description of its components were discussed in sections 8.1 and 8.2. The commercially available expert system shells could be selected to significantly reduce the time needed for preparation of an expert system. However, the flexibility of such a system developed in this way would be limited due to the capabilities and limitations of the shell. Some of the expert system shells available commercially are: VP-Expert shell; NEXPERT *OBJECT*; ART; Level5 etc.

In this section, the initial conceptualization which would form a basis for development of a knowledge base of an expert system for screening cycle 1-A is attempted. The work reported in this section is limited and aims to provide a basis for further development of a prototype expert system. The overall assessment of a platform at screening cycle-1A was based on Figures 3.8 and 3.9. These figures are reproduced in this Section as Figures 8.3 and 8.4. In the following sub-sections, details of the knowledge elements on which the knowledge acquisition process should focus are discussed.

Based on these researchers would be able to select the goals and sub-goals, categories and sub-categories; identify objects and classes; and formulate rules. These will also help in making the selection of a suitable expert system shell.

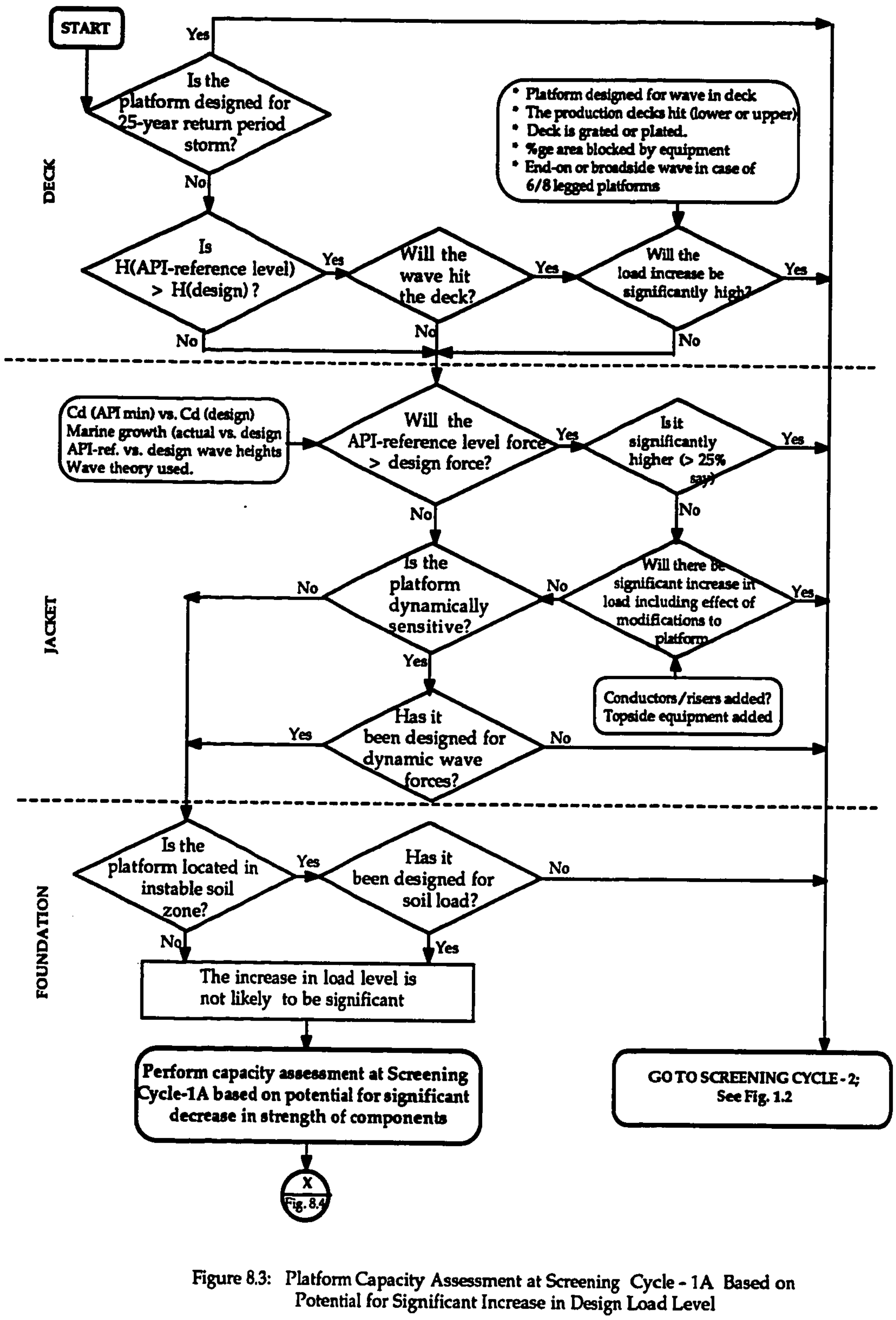


Figure 8.3: Platform Capacity Assessment at Screening Cycle - 1A Based on Potential for Significant Increase in Design Load Level

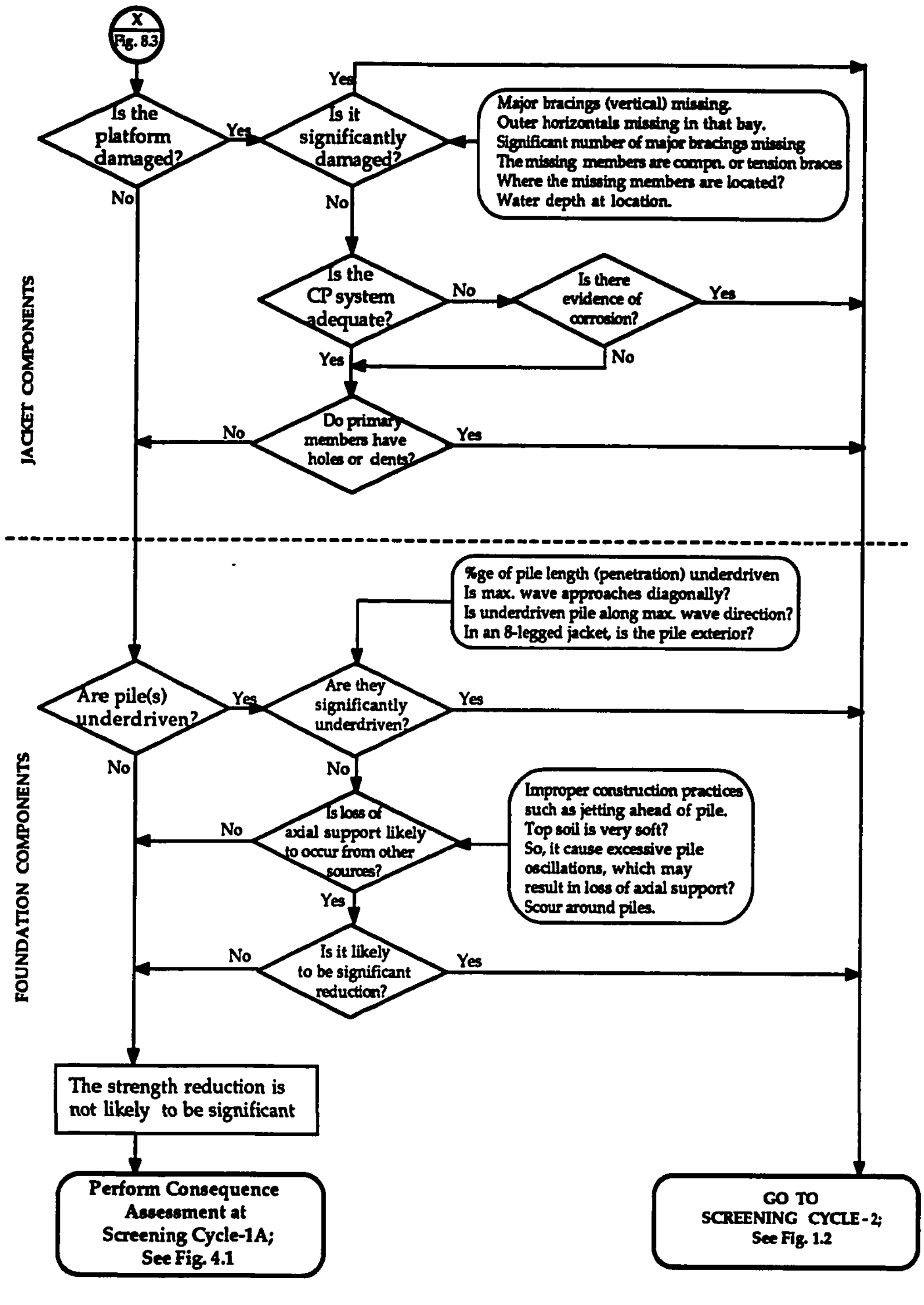


Figure 8.4: Platform Capacity Assessment at Screening Cycle - 1A Based on Potential for Significant Decrease in Strength Level of Components

8.3.1 Conceptualization at Screening Cycle-1A:

The details of the screening cycle-1 process are given in Section 3.3. At this screening cycle a qualitative assessment is made on the basis of a cut-off criteria summarized in Figures 8.3 and 8.4. The cut-off criteria is based on an evaluation of the significance of variations in the important conditions and parameters of a platform between its as-is (current) state and its as-designed (original design) and installed state. The focus of evaluation is on the determination of the potential for a significant increase in the loads and load effects or a significant decrease in strength due to a single major attribute, in the as-is (current) state of the platform from its original designed and installed state.

A knowledge tree is given in Fig. 8.5 for making a qualitative assessment of the capacity level of a platform. Capacity is related to the RSR of the platform. Six conditions are evaluated for the likelihood of significant increase in loads and load effects, or the likelihood for significant reduction in strength level. **In this study in Section 3.3, a significant variation in capacity (RSR) of a platform was assumed to occur when the loads acting on the platform are expected to increase by more than 25% or the strength level of its components is expected to decrease by more than 20% compared to the original design loads and strength.**

Figures 8.6 to 8.11 present knowledge trees for making decisions for each of these six conditions on the basis of the above basic assumption. Detailed descriptions of these criteria were presented in Section 3.3. In this section, importance of selected decision modes presented in these figures are discussed briefly.

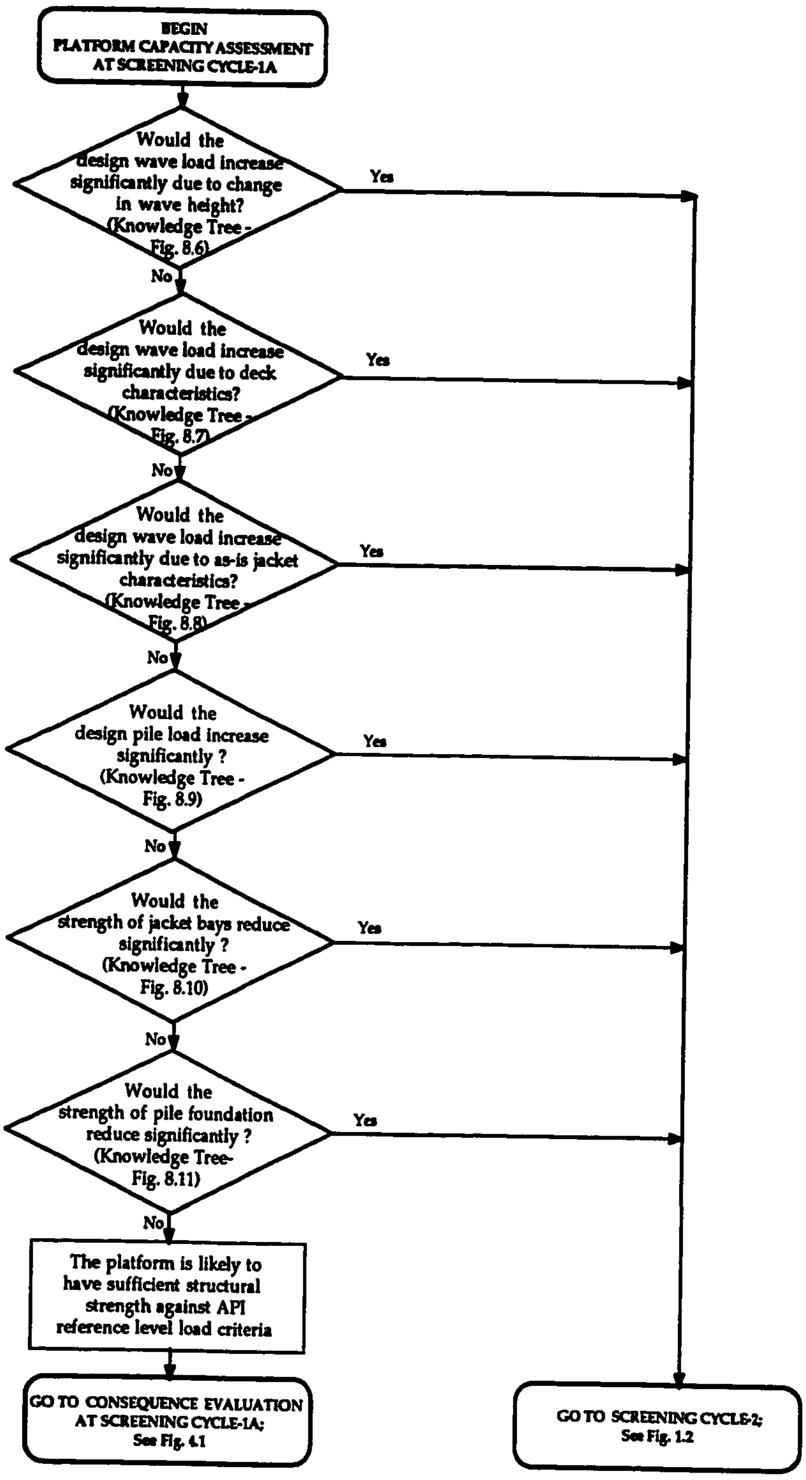


Figure 8.5: Knowledge Tree for Qualitative Assessment of Platform Capacity at Screening Cycle- 1A

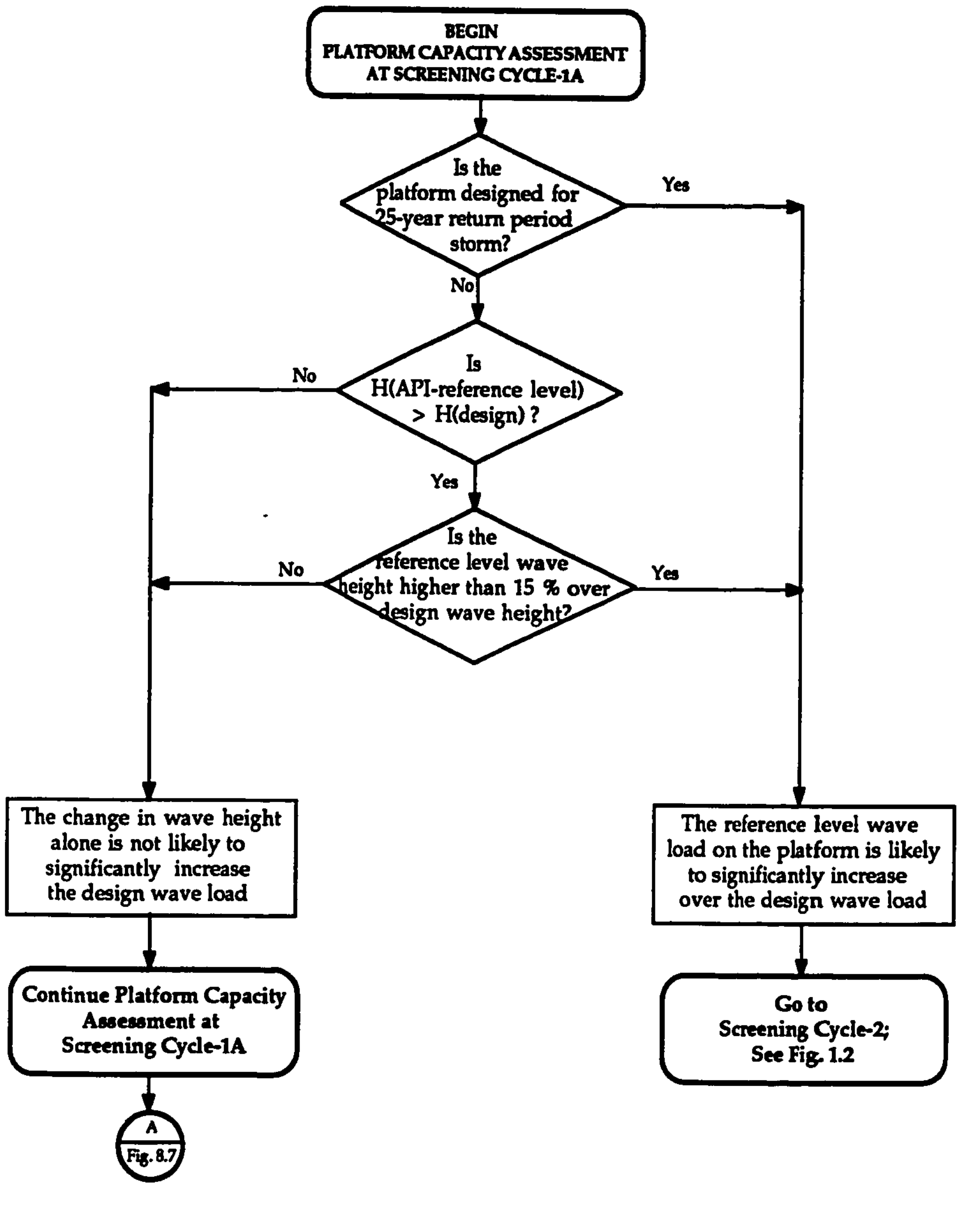


Figure 8.6: Knowledge Tree for Likelihood of Increase in Design Wave Load due to Change in the Wave Design Criteria

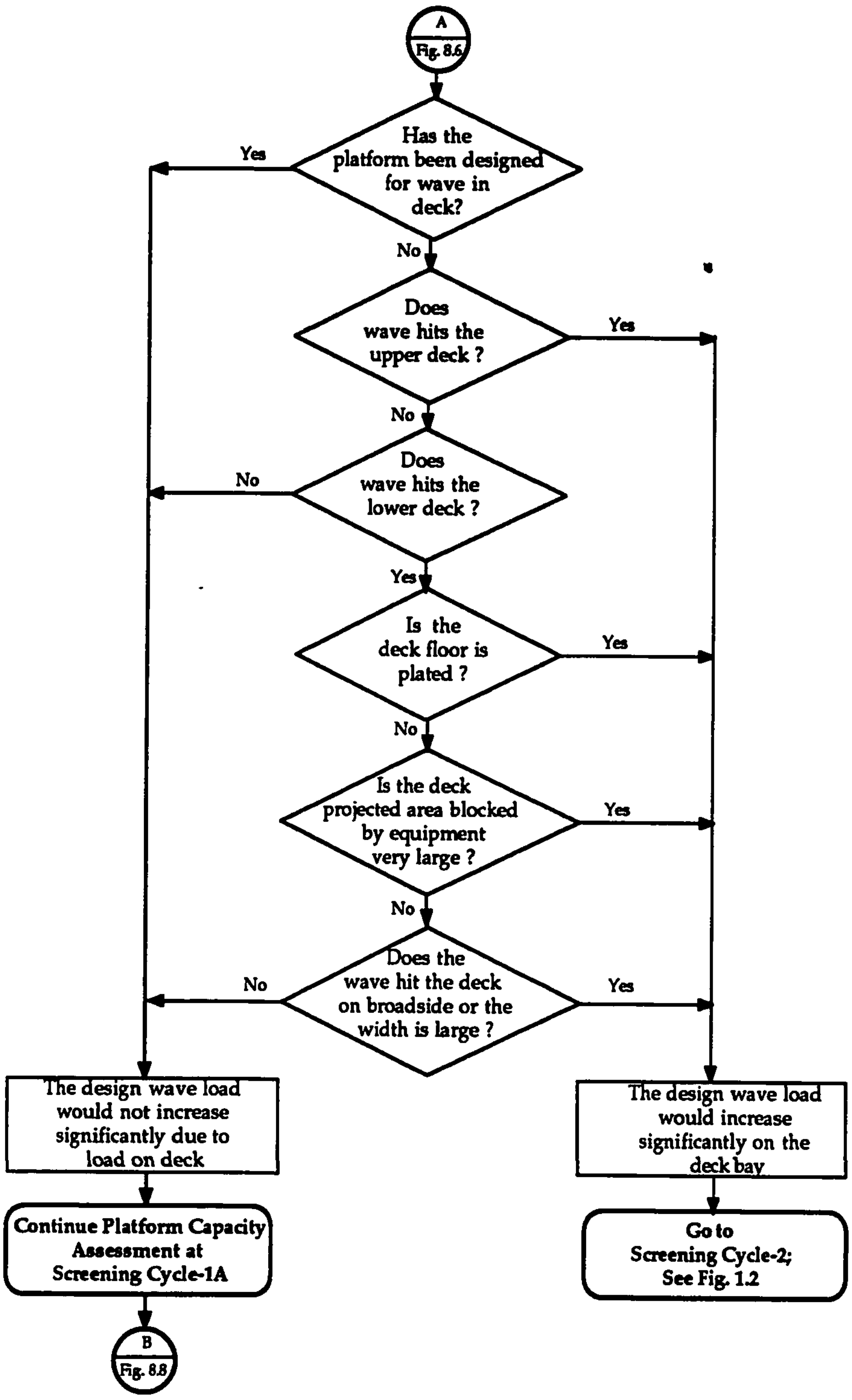


Figure 8.7: Knowledge Tree for Likelihood of Increase in Wave Load due to Deck Characteristics

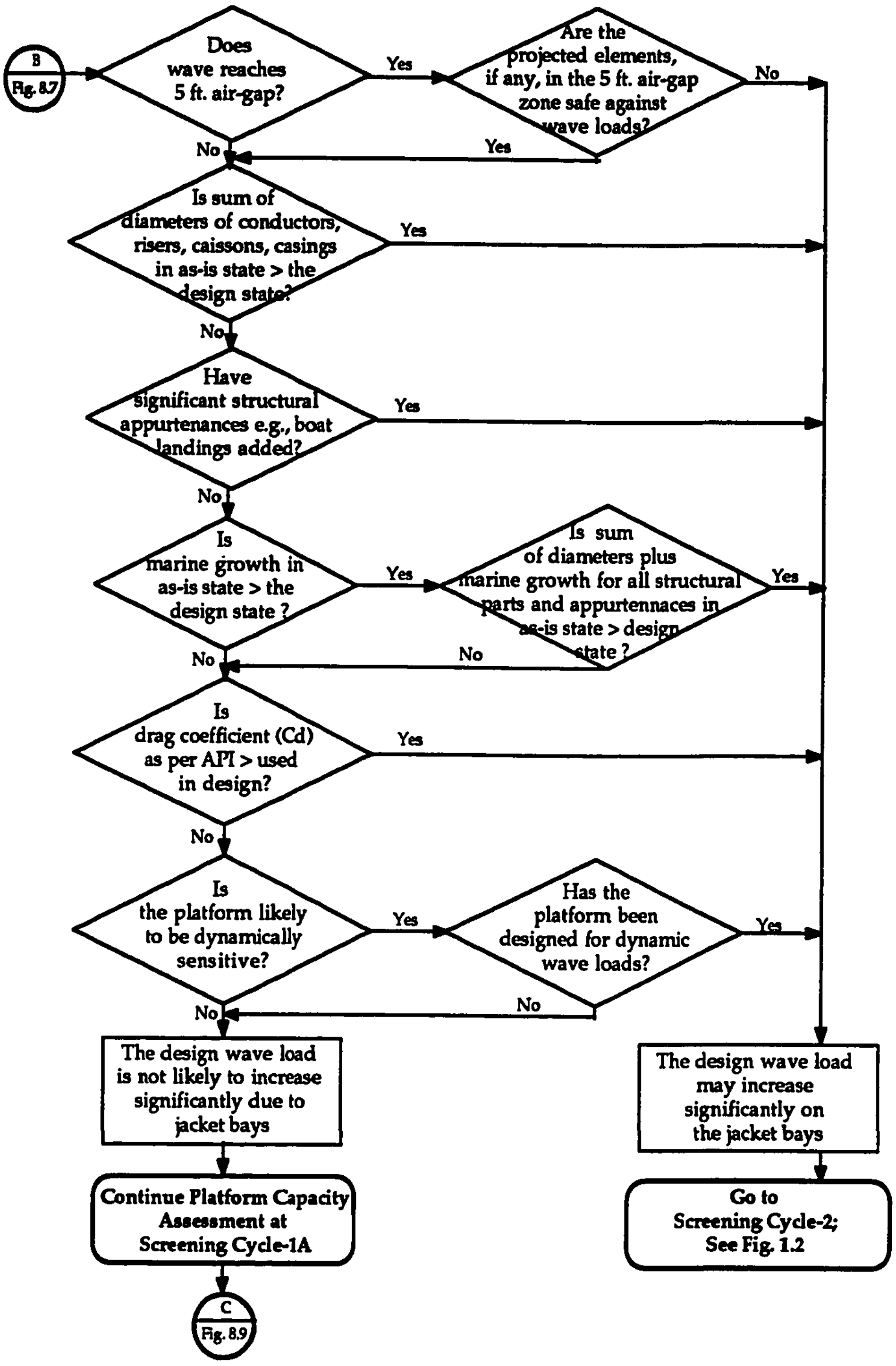


Figure 8.8: Knowledge Tree for Likelihood of Increase in Design Wave Load due to As-Is Jacket Characteristics

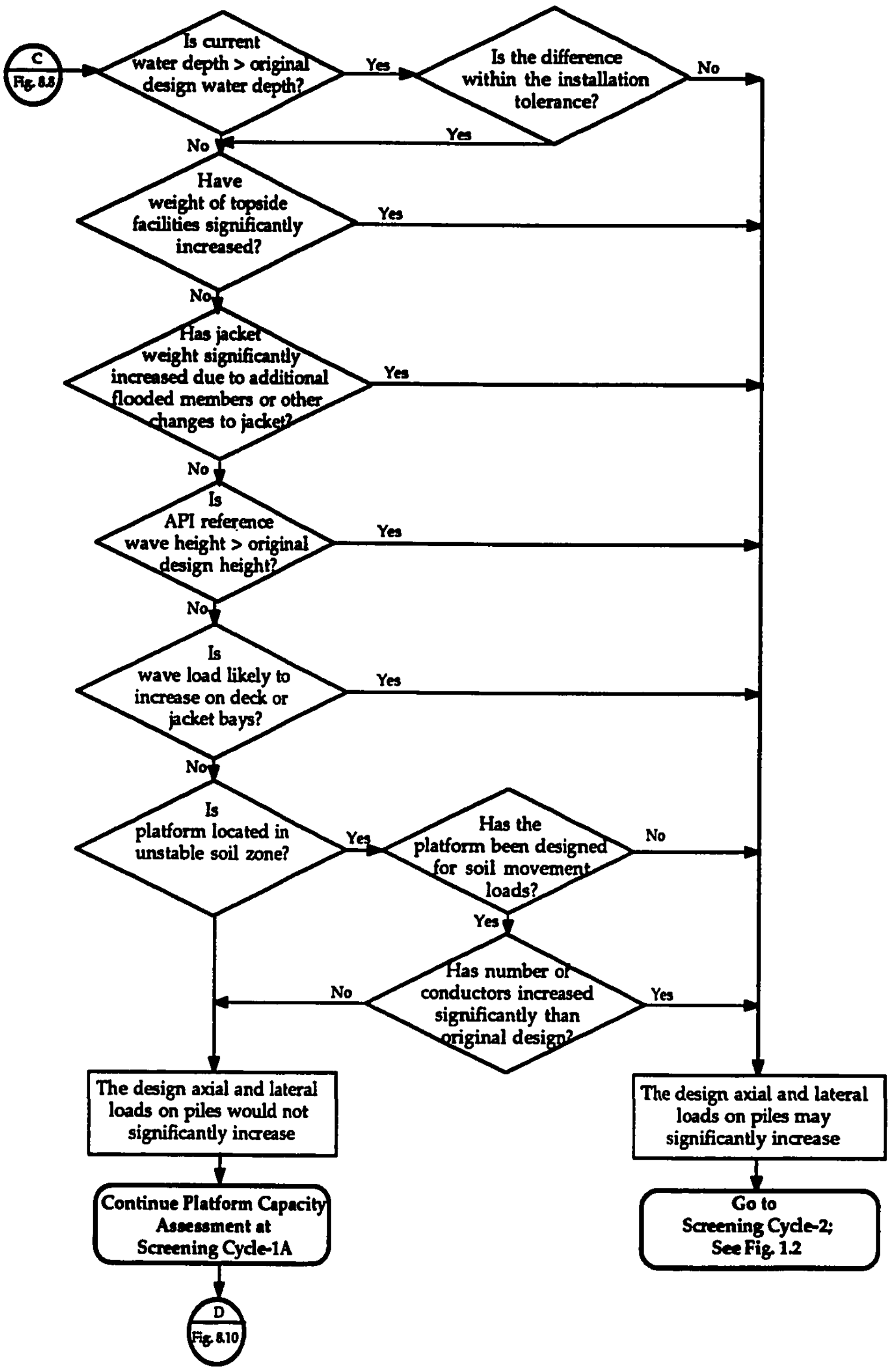


Figure 8.9: Knowledge Tree for Likelihood of Increase in Design Loads on the Piles

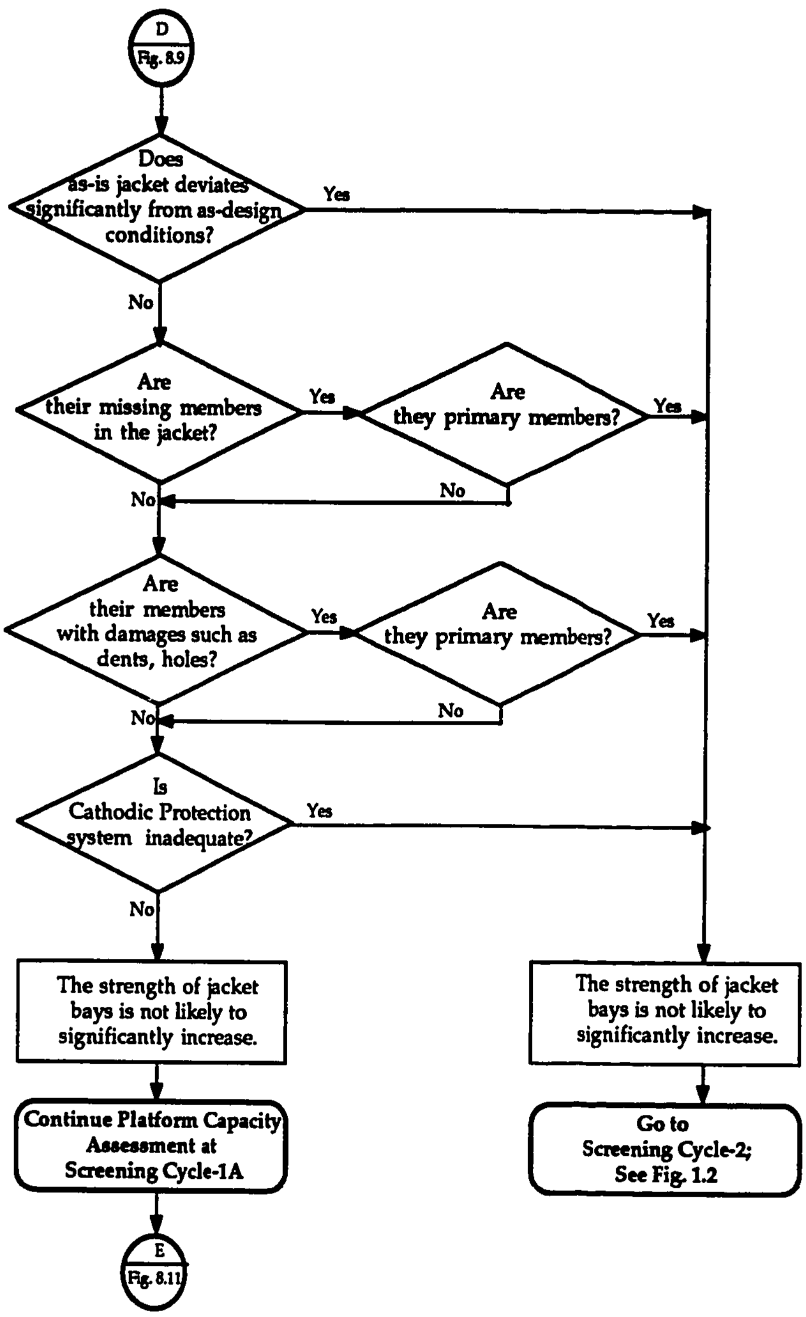


Figure 8.10: Knowledge Tree for Likelihood of Reduction in Strength of Jacket Bays

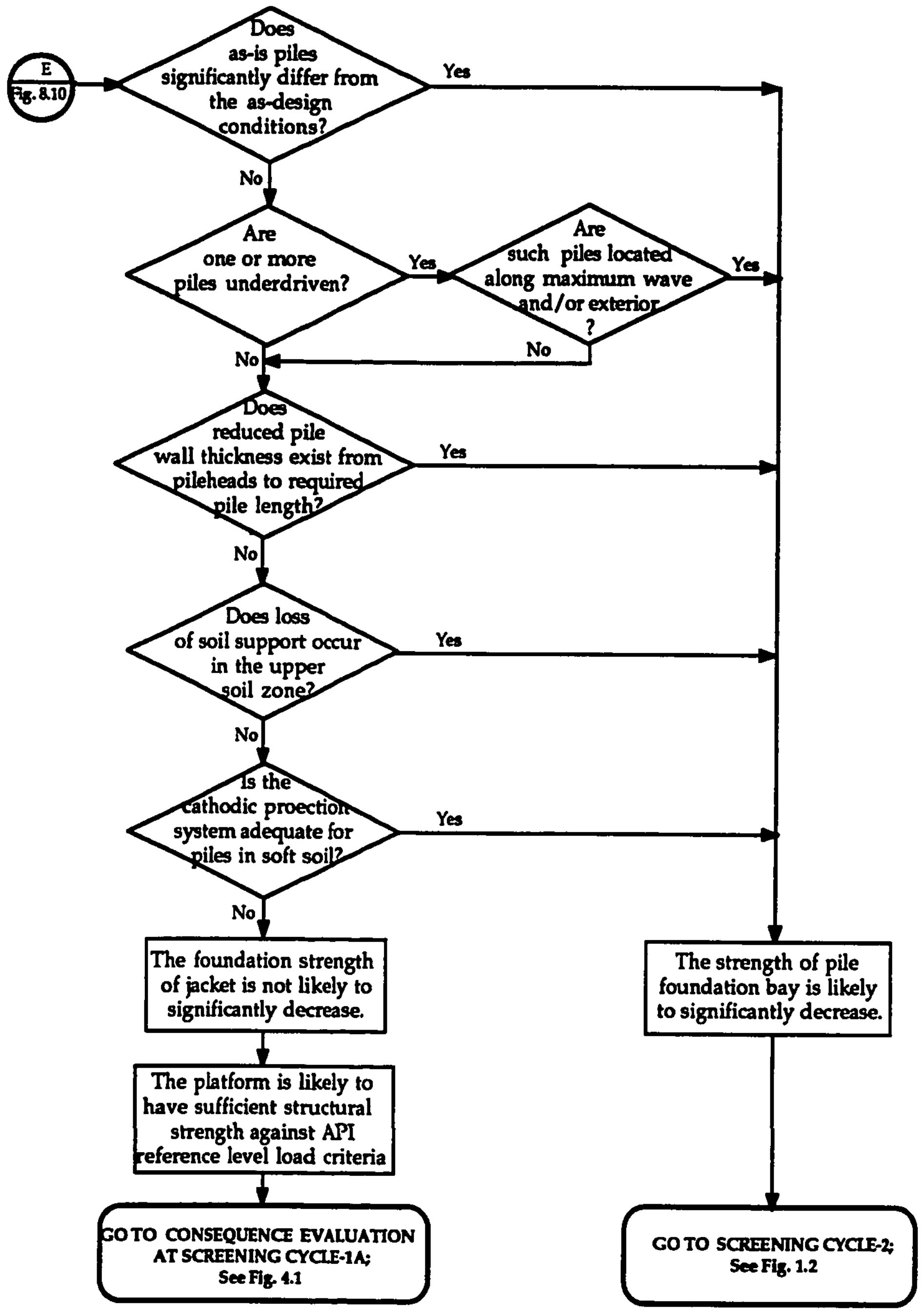


Figure 8.11: Knowledge Tree for Likelihood of Reduction in Strength of Pile Foundation

8.3.2 Knowledge Trees:

Knowledge trees (Figures 8.6 to 8.11) have been developed for the six major criteria to determine the likelihood of significant increase in wave load or of significant reduction in strength for a steel jacket offshore platform.

Change in Wave Design Criteria:

The wave force is proportional to the square of the wave height. Therefore variation in the design wave height would have an increased effect on the design wave loads. Many of the platforms in the Gulf of Mexico before 1965 were designed for the 25 year return period waves, which were significantly smaller than the current criteria based on the 100-year return period waves. Therefore, the reference level load computed according to the current API criteria would be significantly higher than the original design load level. In case the API reference level wave height is approximately 15% higher than the original design wave height, the original design wave load would increase by more than 25%. Hence, it forms an important cut-off criteria.

Influence of Deck Characteristics on increase in the design wave load:

If the current API reference level wave hits the deck(s) and the deck(s) was not designed for the loads due to the wave in the deck(s), then the design wave loads would significantly increase. In case the reference wave hits only the lower deck, on which usually the minor equipment are placed, the increase in the load level will depend upon the type of deck plate, type of equipment, projected area of deck and equipment, and on which side (long or short) of the deck the wave hits. Other basis for evaluation would include: whether profiling of the deck girders has been done to reduce the wave loads; whether additional secondary structural systems project below the cellar deck.

Influence of the as-is characteristics of the jacket on increase in the design wave load: The wave loads on jacket are evaluated by Morison's equation, which was discussed in detail in Chapter 2. The wave load is directly proportional to the diameter of members and the drag coefficient, and is proportional to the square of wave height. In case boat landings are provided it is proportional to approximately 2.2 power of wave height. The major part of the total base shear on the jacket comprises of the wave load on vertical members (jacket legs, conductors, risers, caissons, casings) and other members in the wave zone.

The variation in the wave load from the design state could be evaluated by comparison of the total diameter of the members in the wave zone which differ from the original design criteria. In case, the existing marine growth is higher than the summation of diameters of members plus marine growth in the two states be compared to evaluate the likelihood of increase in the load level.

The drag coefficient on tubular members vary from 0.6 to 1.2 due to member properties, flow parameters, and type of marine growth. The reference level force is evaluated based on drag coefficient of 0.6.

The natural period of more slender platforms in deeper water depths become closer to the wave period, thus the wave forces increase could significantly due to dynamic behavior. If such platforms have not been designed for dynamic loads, the force level would differ.

Influence on the Pile loads:

If the water depth differs from the original design state, or the wave height is higher than in the original design, or if the wave load is likely to increase on the jacket or the deck, the overturning moment at the base of the jacket will increase. By increase in loads on the superstructure and the substructure, the axial load on piles would increase.

The lateral load on the piles will increase when the platform is subjected to additional forces, such as unstable soil loads, which were not considered in the original design.

Influence on Strength of Jacket Bays:

The strength of jacket would reduce compared to the as-design state, when during installation operation the design assumptions were not fully realized. For example, if grouting of leg -pile annulus or skirt sleeve-pile annulus were considered but due to packer failure or other reasons, the required grout level and density were not achieved. Other installation errors may include that the jacket was oriented beyond tolerance level, jacket was tilted in one direction, piles were installed in the wrong sequence or in different legs resulting in reduced pile wall thickness in highly stressed zones. Residual stresses may have been induced during jacking operations, if required for levelling of jacket. There may also be damage incurred to members during installation.

If the primary members are missing or are damaged, the strength level of that jacket bay would reduce. The location (elevation, bay number), orientation (along or across waves), number of such members in a bay, and force type (compression or tension) of such members are important. If adequate anodes for cathodic protection were not provided or if they had deteriorated, the strength of jacket bays would reduce due to corrosion effects. A general degradation of structural members would occur due to inadequate cathodic protection system.

Influence on Strength of Foundation Bay:

The strength of piles would differ, when their physical parameters is between the as-is and the original design states. In case pile penetration is lower than as-designed, the location of underdriven pile, type of loading (tension, or compression) is important. In some cases, the piles may have reserve strength when higher pile penetration has been provided than required to maintain symmetry. If the pile wall thickness at mudline and its vicinity is lower than design, then the piles may yield.

If scour occurs at a platform site and the resulting loss of soil support to the pile is significant, then the stresses due to lateral loads would be higher and the increased pile wall thickness may be required for a larger depth. During installation of piles, if the as-desired state of platform is not achieved and residual stresses remain, or grouting is not adequate, or pile sections differ, then the strength of piles would reduce in case remedial measures were not considered

Based on the knowledge trees described in Figures 8.6 to 8.11, the platforms which do not meet any of the basic conditions, would be termed as the platforms whose capacity (RSR) is likely to remain approximately 1.25 or higher. The platforms which meet these conditions, and there is likelihood of either increase in the loads or reduction in the strength, their capacity (RSR) is likely to be lower than 1.25 and they must be evaluated in more detail at screening cycle-1B or screening cycle- 2.

The knowledge presented in these figures could be formalized by the development of rule base. With these knowledge trees available, the rules could be developed in a rational way. The user interface would include a large number of "explain" statements, to inform the user about the meaning of the different terms, explanation of various theories, and formulas. The platform data sheets included in Appendix-A could form a good starting point for the development of a user interface.

8.4 Summary:

The major components and various stages in the development of an expert system have been discussed in the context of re-qualification of existing steel jacket offshore platforms. The development of a knowledge base and a user interface are the key components, when an expert system shell is used.

The expert system is recommended to be developed for the qualitative screening cycle-1, because the evaluation criteria at this cycle is based on judgement and experience, and it needs to be applied periodically on a very large number of platforms.

The schematic diagrams portraying the "knowledge elements" have been developed for qualitative evaluation of the capacity of a platform. They are based on the cut-off criteria developed in this research and they identify the knowledge that is needed to evaluate the performance of a jacket against storm wave loads. These schematic diagrams would be of help to researchers in the development of a knowledge base. These diagrams would need further work by identification of the next level of reasoning and need to be developed for the evaluation of the consequence level of a platform. Then on this basis, the rules can be formulated in a simpler way by use of a suitable expert system shell. Significant work will be required in development of the user interface, which should explain to the users the various terms, theories, reasons in making such an assessment.

CHAPTER 9

SUMMARY AND CONCLUSIONS

The assessment of the structural safety of existing steel jacket offshore platforms has been investigated in this study. The work carried out focuses on: development of a methodology for screening of the platforms according to their safety levels, development of a simplified method for evaluation of the capacity of a platform, and demonstration of the feasibility of preparation of an expert system based on this methodology.

The safety assessment of ageing offshore platforms, in order to ensure their safe operation and continued production of crude oil and gas, has become of increasing importance in the recent years in the United States and worldwide. This has created a dilemma for the government regulatory bodies, owners, and operators in the United States, where a major number of platforms exist. In recent years, their interest to maintain the safety of platforms against loss of life, environmental pollution, and loss of resources and property has increased due to the awareness of the public towards consequences of their failure.

The regulatory bodies, such as the Minerals Management Services (MMS) established in the United States, have a role to play to ensure that the structures operating in the offshore waters are safe to public life, environment, property and loss of production. However, they do not have a methodology to make routine assessments of the safety of 4,500 platforms operating in the Gulf of Mexico. Hence, the primary focus of this study has been on its use by the regulatory bodies to assess the safety of the offshore platforms located in the Gulf of Mexico.

In this Chapter, first a summary of the methodology and the processes involved are presented. Then the conclusions, derived based on the details presented in the previous chapters, are discussed. The areas where more work is required for safety assessment of platforms in the Gulf of Mexico and other geographical locations are focussed.

9.1 Summary

A steel jacket platform is typically constructed of three main parts: deck structure (air-above water); jacket structure (water medium); and foundation piles (soil medium). The jacket structure consists of a 3-D tubular space frame, supported vertically and laterally by steel tubular piles driven through jacket legs into foundation soils. The deck structure supports the production, drilling, living, transportation, safety and other facilities.

Such platforms have been in use for more than 45 years in the Gulf of Mexico and for a lesser period in other offshore areas around the world.
A significant difference in structural configurations of platforms is noted, because of variations in their design criteria from: water depth; environmental and geotechnical parameters; construction and installation equipment selected; fabrication technology of the period; operational criteria; and the difference in design philosophy.

There are more than 4,400 offshore platforms in the United States. Many of these platforms (37 %) have been operating beyond their usual design life of 20 years *[Dyhrkopp, 1990]*, and some have suffered damages due to corrosion, fatigue, dropped objects, and collision with boats. Thus, they do not necessarily meet the acceptability criteria (safety standards) existing today.

The various regulatory bodies *[MMS, 1988]* and operators are responsible for the safety of these platforms in continued operations, for which "safety" includes safety of life, safety against pollution, safety against loss of resources and property. Thus a well defined methodology is needed to provide consistency in periodic assessment of the safety of a large number of existing platforms.

A platform should be structurally sound and should not pose undue hazards to the personnel, environment, and property to be classified as suitable for service. A four-cycle safety assessment methodology has been developed, as shown in Fig. 9.1, which is based on a comprehensive evaluation of the suitability for service of the candidate platforms at one or more of the 4-cycles. From such an evaluation, the platforms are categorized as "Fit For Purpose (FFP)," "Marginal," and "Unfit For Purpose (UFP)."

To evaluate the suitability for service of a platform, the loadings imposed and induced in the structure, strength and capacity characteristics of the structure, and the potential consequences, if the structure fails to perform satisfactorily, are the three most important evaluation criteria. The loading on the structure and the strength of the platform are combined and expressed through a term, Reserve Strength Ratio (RSR= Ultimate Capacity/Minimum Reference Force), which represents the overload capacity of the platform.
The consequence level of a platform essentially depends upon the functions of the platform, the operational philosophy followed, and the importance of the platform to functioning of other platforms in the vicinity. An overall decision on the suitability for service of a platform at a level is then based on a comprehensive assessment of its capacity (expressed by RSR) and consequences. An evaluation of feasibility of IMR program to maintain the safety level or to upgrade the safety level of a platform is then done.

The first step in the safety assessment process (screening cycle-1) is to screen the platforms according to the need for an in-depth investigation of

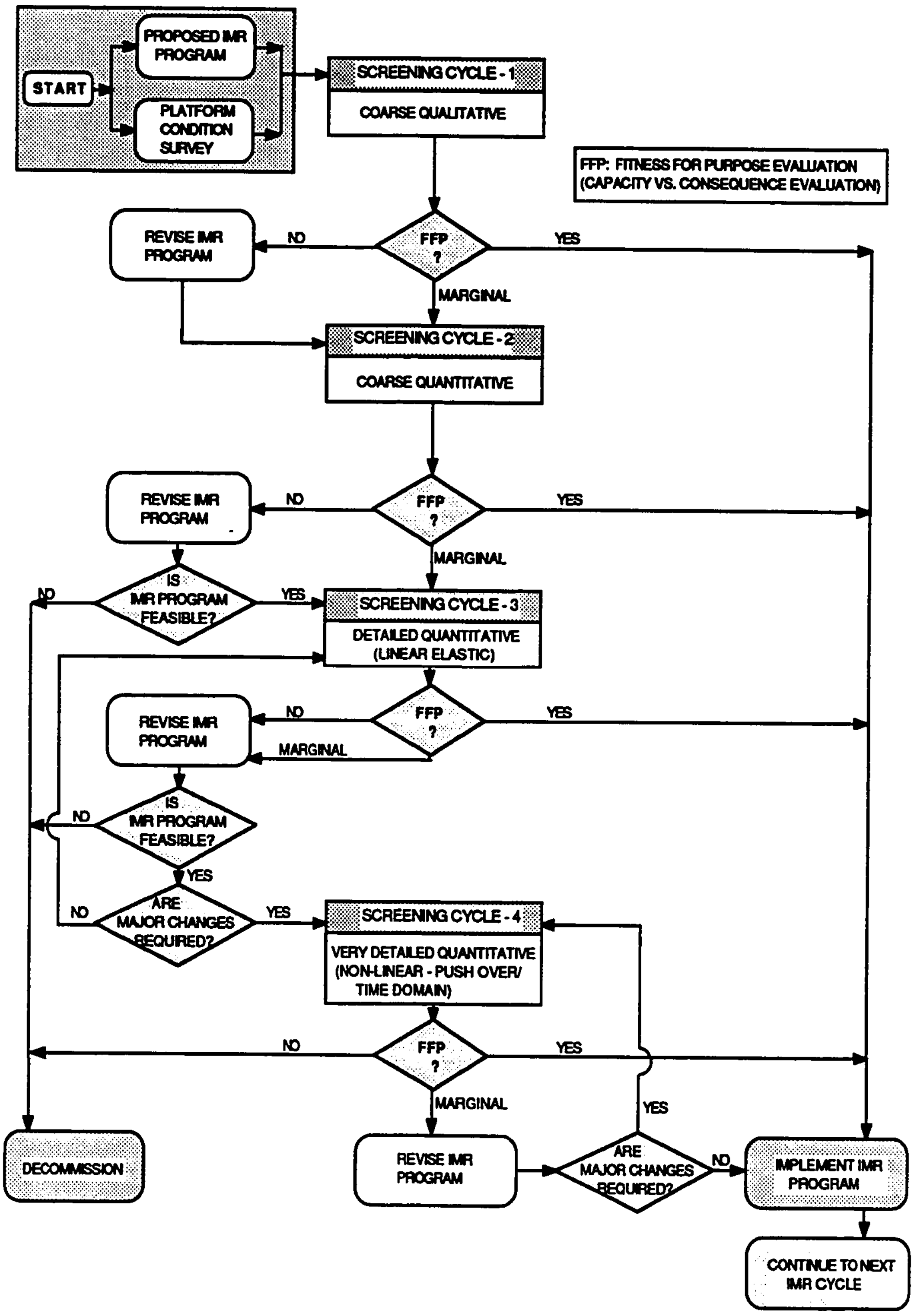


Figure 9.1: Algorithm for Safety Assessment of Offshore Platforms

their suitability for service in continued use. At this step, it is an attempt to approach such a decision using the same factors that would be evaluated by an experienced offshore engineer, yet the process is organized so that it can be applied by a competent engineer in a methodical manner.

In Phase-A of the screening cycle-1, the emphasis is to look at the major factors which individually may significantly affect the structural integrity and safety of a platform, so as to require further investigation. The major criteria which influence load and strength levels of a platform are evaluated in a logical fashion. In case, there is likelihood of a significant increase in load level or a significant decrease in strength level of a platform by a single criteria, the platform exits the process and is further evaluated at screening cycle-2. In Phase-B of screening cycle-1, a cumulative influence of the various major factors on the likelihood of reduction in capacity or increase in consequence level of a platform is evaluated. The platforms which have a significant effect on their structural integrity and safety due to a cumulative effect of various factors, are screened out for further investigation. This phase is based on a qualitative assessment of the capacity and consequence levels. Then based on these two parameters a subjective judgement of the safety of a platform is done. The comprehensive capacity and consequence levels obtained at the screening cycle-1 evaluation are subjectively categorized as: very low, low, medium, high, or very high. Then these qualitative levels are compared in a capacity-consequence diagram, to make a decision on overall safety of a platform.

Screening cycle-2 of the process is based on application of simplified (coarse) quantitative evaluation to determine if the platform has sufficient reserve structural and foundation capacity, when compared against the reference level forces. The emphasis is to check the occurrence of possible failure modes and mechanisms in the three parts of a platform. The failure modes which occurred in the past were identified from a detailed review. The coarse quantitative evaluation of capacity is aimed at determination of its "lower bound" estimate. The lower bound estimate is based on component strength of members and ignores the "system effect," i.e., the structural redundancy in the platform. The simplified evaluation is based on comparison of load and strength patterns for a platform, which are developed by determination of load and ULS strength levels for each bay of the platform.

At screening cycle-2, the consequence level of a platform is evaluated based on "utility theory-decision analysis" approach. The potential consequences are assessed in monetary terms and converted to a utility scale. The Fitness for Purpose evaluation at this screening cycle is based on quantitative characterization, which includes: uncertainties in loading, capacity, and consequences; acceptance criteria; expected maximum force and reference level force on a platform. It permits the establishment of acceptable, marginal, and unacceptable combinations of RSR and consequences for given force ratios and loading-capacity-consequences uncertainties.

Screening cycle-3 applied on a reduced number of platforms is based on essentially the evaluation process which would be employed by a verification agent for a new platform. The capacity level at this cycle is evaluated by conventional linear structural analysis. Upon evaluation of RSR by linear structural analysis, a decision on overall safety of a platform is made based on capacity-consequence diagram developed for the platform parameters.

Screening cycle-4 is the application of a system analysis, based on a non-linear analysis, which determines the redundancy level of structure and post-ultimate capacities of members to evaluate the possibility of progressive collapse of the structure. At this stage, the degree of damage tolerance of the system is emphasized. At the simplest, the analysis can be done by performing member-replacement "static push-over analysis," by monotonically increasing the lateral load on the platform and determining member response. The static push-over RSR does not give accurate results due to neglect of transient, dynamic, and cyclic nature of wave loads, and due to neglecting the potential degradation in capacity of elements during intense cyclic loading. These deficiencies can be addressed through the use of time-history non-linear analysis. However, such analysis is extremely difficult and time consuming, and thus is used for selected platforms.

By such a 4-cycle screening process, a detailed evaluation of reserve strength and consequence levels for the platforms is required only for a few most likely to fail platforms. Upon classification of a platform as Marginal or UFP, at each of the 4-cycles, the next step in the process is to make a decision on the feasibility of the IMR program proposed for the candidate platform. The IMR program plays a key role in maintaining the safety of a platform in continued operations. A platform classified as Marginal or UFP at any level will need revision of its IMR program and evaluation of its feasibility. If the revised IMR program is found to be adequate, then the fitness of the platform in its upgraded condition would be evaluated at the same or the next screening cycle. If the revised program is insufficient, then the platform may be considered for decommissioning to avoid the negative consequences.

The safety level of a platform could be upgraded by several alternatives to achieve desired variation in the mean capacity (RSR), reduction in uncertainties in load, capacity, and consequences. The alternatives include load reduction, structural strengthening, more refined estimates of load and strength, and upgrading of the acceptability criteria due to availability of improved data for a platform, and reduction in risk level for storm loading.

The best solution depends upon the characteristics of the particular platform considered.

An expert system provides an effective medium to implement the methodologies developed at screening cycle-1 for screening of a very large number of platforms according to their safety levels.

9.2 Conclusions

The four cycle safety assessment process developed in this study would provide a cost and time effective solution for assessment of the safety of a very large number of platforms against storm wave loads. The cost and time involved in its implementation on a platform would vary with its characteristics.

Simplified techniques developed in this study, for assessment at screening cycle-1 and screening cycle-2 are of special importance for identification of the critical platforms by the government regulatory bodies. Hence, these evaluation cycles would provide a mean with the regulatory bodies or operators to make a routine assessment of very large number of platforms operating in the Gulf of Mexico, in order to ensure the safety of structures to public life, environment, property and loss of production.

The processes developed for the first two cycles are simpler but involve heuristic knowledge of steel offshore platforms in the Gulf of Mexico and worldwide, and the decision making incorporates reliability analysis aspects. A detailed review of literature was done to form the basis, which also involved establishing the behavior characteristics, and failure modes and failure mechanisms of jackets against storm wave loads. The "experience factor" has been exploited to develop this approach, which gives sufficiently good first estimate of the safety level of a platform. This has been demonstrated by three example platforms in Chapter 7.

By the simplified method for capacity (RSR) evaluation at screening cycle-2, the most likely to fail (MLTF) member in a bay of the platform could be identified in many cases, as was demonstrated in Chapter 7. Therefore, with little computation effort and without necessarily using the computers, the "weak zone" of the platform could be approximately located. The results achieved by the simplified method could be further improved by formation of categories and sub-categories of platforms and establishing bias factors for load and strength for each of these categories. Such bias factors could be derived by implementation of this method on a large number of platforms.

The platforms which are identified as "Marginal" and "UFP (Unfit for Purpose)" at the first two screening cycles would require either detailed quantitative evaluation of their capacity and consequence at higher screening cycles or upgrading their capacity level or downgrading their consequence level. Before moving on with more complex analyses, in some cases it may be cost-effective to improve the state of knowledge of the platform by implementation of an IMR (Inspection, Maintenance, and Repair) program. It will also help to ensure that the assumptions made in more complex analyses are correct and results are reliable. This effort may be focussed on the "weak zone" of the platform which were identified in screening cycle-2, for cost effectiveness. In addition, screening cycle -2 would act like a preprocessor due to aid in data acquisition for the platforms to be evaluated at higher screening cycles.

The selection of an optimum alternative for upgrading of capacity or downgrading of consequence would require a cost-benefit evaluation. The safety level of a platform could also be improved by change in the acceptability criteria used in the fitness for purpose evaluation, as presented in Chapter 5. The acceptability criteria could be changed by reduction in the uncertainty level or the risk level associated with the storm waves. The risk level could be reduced by elimination of structural risk associated with other load sources by removal of some equipment, or provision of additional safety measures, or restriction of ship movement in proximity of platforms. The level of benefit achieved were presented in Chapter 5. In Chapter 7, the benefit achieved by these measures were demonstrated.

Screening cycle-1 and -2 have the potential for development in the form of an expert system. The basis for development of such an expert system has been explored for screening cycle-1A in Chapter 8. The knowledge trees developed provide a first step to prepare the rule base for expert system. The maximum benefit of the process presented at the first two cycles could be achieved in such a way, as to make the process relatively simple to apply on a very large number of platforms on a routine basis.

The methods presented at the first two screening cycles should be considered as evolving in nature with time due to the heuristic knowledge involved, varying characteristics of the platforms, variations in the acceptability criteria due to preferences of the parties involved. It is believed that at the initial implementation of this methodology, a lesser number of platforms would pass screening cycles -1 and -2 due to lack of improved state of knowledge of the platforms. With continued implementation, the safety assessment of a larger number of platforms could be achieved at these screening cycles. Therefore, these methods would provide a cost effective approach in time.

9.3 Recommendations for Future Work

The methodology developed in this study needs to be tested on a greater number of example platforms. At the next level of implementation, the various platforms could be classified in categories and sub-categories on the basis of their operational functions and structural characteristics, as were presented in Chapter 1. Upon implementation of this methodology at screening cycles-2 to -4 on the different categories of platforms, bias factors associated with the capacity (RSR) assessment at screening cycle-2 and screening cycle-3 could be established. This work will improve the confidence level in this methodology.

Screening cycle-1 should be applied on a very large number of platforms to determine its effectiveness and to identify the next level of heuristic factors involved. A working expert system should be developed for screening cycle-1 as demonstrated through the knowledge trees. This expert system will facilitate application of the process on a very large number of platforms in a routine manner.

The method for capacity evaluation at screening cycle-3 as described in Chapter 3 should be applied on additional example platforms, and the bias factors associated with the nominal RSR, which is evaluated in an approximate way, be established. Note that these bias factors would also vary with different categories of platforms.

The consequence evaluation needs further work to develop consensus on application between the regulatory bodies and operators. The basis for evaluation of the consequence levels for different categories of platform should be further refined.

Similar basis for evaluation of safety of platform against other load sources (e.g., earthquake, ice, fatigue, accidental loads) and in different geographical regions (e.g., California, Cook Inlet, Alaska) should be developed.

APPENDIX-A

PLATFORM DATA SHEET

(To be filled up by the evaluator using the data supplied by the operator)

Evaluator name:	Date:

PLATFORM IDENTIFICATION:

Name : ____________

Location: *Gulf of Mexico/S.California/Cook's Inlet* Block : ________

Latitude : ____________ Longitude : ________

Water Depth : ____________*ft..* *(Shallow/Intermediate/Deep)*

Installation Date: ____________ Age(Years in service): ____

Owner : ____________ Orientation : _____N__

DOCUMENTS AVAILABILITY CHECKLIST:

Jacket & Deck Drawings: Basic: *Yes/No* Detailed : *Yes/No*

Design criteria : *Yes/No* Topside Facilities Details : *Yes/No*

API Code used : *Yes/No* Design Report : *Yes/No*

Design Review Records : *Yes/No* Computer Models : *Yes/No*

Soil Borelogs Report : *Yes/No* As-Installed Report : *Yes/No*

Pile/Conductor Driving Report : *Yes/No*

Periodic Survey Records : *Yes/No* Damage Reports : *Yes/No*

Platform Modifications Records :*Yes/No*

Designed by:__________ Constructed by: ________

Installed by : _________ Operated by : ________ Period: ______

Period: ______

Inspected by:__________ Repaired by : ________

OPERATIONS OF PLATFORM:

Functions :..*Drilling/Production/Living Quarters/ Compressor/PDQ/Support*

Number of Men: *Unmanned (Permanently)/ Manned (Periodically)*

Manned: *0-5/ 6-15 / 16-25 / more*

Production Level: Crude Production: *0-5,000 BOPD/ 5,000-10,000 BOPD/*

Processing Capability (own) : *0-10,000 BOPD/*

Process Support to Other Platforms :

Importance to Other Platforms:

Independent Platform	: *Yes/No*
Production Piped to a Central Platform	: *Yes/No*
Supporting Platform for Manifolds	: *Yes/No*
Production Piped to it from Other Platforms	: *Yes/No*
Number of Platforms Connected to it	: *0-1 / 2-5 / >5 (Specify)*

TOPSIDE / PRODUCTION FACILITIES:

Number of Wells: As-Designed : ______ As-Present : ________

% Difference : ______ % Size : ________

No. of Risers/Flowlines: As-Designed: ________ As-Present : ________

% Difference : ________ % Size : ________

Operating Payload : ________ No. of Men on Board : ______

Storage on Board : *Nil / <20,000 bbl / 20,000 - 100,000 bbl / > 100,000 bbl*

Deck Layout Changes : *Yes/No*

Equipment Added : *Yes/ No* Equipment removed : *Yes/No*

Tonnage Added : ________ Tonnage Removed : ________

Evaluator **Date** **Page 2 of 6**

Safety Facilities: Boat Landings : *nil / one / two* @Elev.:____ *(T.O.S.)*

Early Warning System Tested : *Yes/No*

Safety Facilities Provided : *Life Boats/Life Rafts*

Life Boats Operational : ________

Survival Capsules : ________

TOPSIDE/STRUCTURAL DETAILS:

Deck Weight : ________ Deck Elev. : ____ *(T.O.S.)*

Deck Dimensions : ________ Deck Type : *Grated/Plated*

Equipment Size on Lower Deck : *Small/Medium/Large*

Deck Structural Changes : *Yes/No*

Changes in Truss Structure : ________ Changes in Plating : ________

Cantilever Added : ________ New Braces to Legs: ________

ENVIRONMENTAL LOADING:

Water Depth: "As-Design": ________ "As-Installed": ________

% Difference in "As-Design" and "As-Installed": __%

Wave Height: "As-Design": ________ Latest API : ________

% Difference in "As-Design" and "Latest API" : __%

Wave Crest (Best Estimate): ____ B.O.S. Elevation of Lower Deck: ____

% Overlap: *Nil (Airgap>5 ft.)/ Nil (Airgap <5 Ft.)/ 0-25% /25-50% / 50-100% / >100%*

Wave Theory: As-used in Design : ________ Ideal Wave Theory: ________

% Difference in Wave Loading: ________

Evaluator Date Page 3 of 6

On-Bottom Loads (100-year Design Loads):

Base Shear : ________ Overturning Moment: ________

Wind Speed : ________ Current : ________

Storm Tide : ________

Possibility of Wave Load Increase during the Operating Period: ______

Increase in No. of Risers & Conductors : ________

Wave Hitting the Deck : ________

% Increase in Wave Load : ________

FOUNDATION SUITABILITY:

Soil Type-Near Seabed (General): *Very Soft/ Soft / Medium/ Hard / V. Hard*

Pile Size: Diameter : ________ Thickness : ________

Pile Makeup : ________ Steel Grade : ________

No. of Piles: Main Piles : ________ Skirt Piles : ________

Pile Tonnage : ________

Pile Penetration: As-Design: ________ As-Installed : ________

% Difference : ________

Pile Jacket Connection: *Connected at Top / Grouted / Grouted & Welded*

Conductors Considered as Axial Piles : *Yes/No*

Mudmat Provided : *Yes/No*

Pile Loads: Axial: ______ Shear: ________ Moment: ________

Special Geotechnical Conditions : *Yes/No*

Mudslide Zone : ______ Bad Soil Zone: ________

Liquefaction Potential : ______

Evaluator Date

PLATFORM CONFIGURATION:

Jacket Structural Configuration: *Standard / Non-Standard Design*

No. of Legs: Main Legs: ______ Skirt Legs: __________

Leg Cans : Yes/No Can Steel : A36/Z-Property Steel

Jacket Tonnage: _______

Bracing Pattern:

Vertical Framings: *Longitudinal only / Transverse only / Both Directions*

Bracing Between Top Horz. & Lower Deck: *Yes/No*

Diagonal / K-Bracing / X-Bracing

D/t Ratio : _________

Kl/r Ratio : _________

Horizontal Framings: Number of Levels : _______

In Splash Zone : *Yes / No*

Outer Bracing : *Yes / No / Only in One Direction*

Inner Bracing : *Diagonal / K / X*

Supports for Conductors and Risers : *Adequate / Inadequate / Damaged*

Splash Zone Protection : *Yes/No*

Extra Thickness Provided : *Yes/No* Wrap Plate : *Yes/No*

Paint System : _________

CONSTRUCTION/LOADOUT/INSTALLATION:

Any Unusual Conditions or Occurrences Noted: *Yes/No*

Evaluator **Date** **Page 5 of 6**

INSPECTION, MAINTENANCE, AND REPAIR LEVEL:

Records Available : *Yes / No*

Last Inspection Done (Years) : *None/0-1 / 1-2 / 3-5 / >5 (Specify)*

Level of Inspection (API) : *I / II / III / IV / Other (Specify)*

Next Inspection Planned (Years): ________

Post-Hurricane Survey : *Yes/No* Report Available: *Yes/No*

General Appearance of Platform: *Stable/ Damaged / Deflected / Rotated*

Any Damages Reported : Yes / No Damages Repaired: Yes / No

Damage in/of : *Splash Zone / Brace Damaged at / Leg Damaged/*

Level of Maintenance : *Regular/Intermittent/None*

Repairs Recommended?: *Yes/No* Repairs Done?: *Complete/Partial/None*

PERFORMANCE HISTORY OF PLATFORM:

Hurricane Passed by the Platform Location: *Yes / No*

Any Unusual Responses Noted : ________

Any Damages Reported : ________

Corrosion of Members Reported : ________ In Splash Zone : *Yes/No*

Corrosion Protection System : *CP/Impressed Current*

No. of Anodes : ________

CP- Anodes Replaced : *Yes / No* When : ________

Marine Growth Accumulation : *Heavy/Medium/Light* Cleaning: ___

Evidence of Settlement of Platform: ________

Deflection of Platform : *Bending / Torsion*

Condition of Riser Clamps : *Intact/Broken/Loose*

Evaluator **Date** **Page 6 of 6**

APPENDIX - B

WAVE FORCE COMPUTATION FACTORS FOR BRACES

In this appendix, approximate factors are computed for wave load evaluation for the diagonal braces in the vertical and horizontal frames. The wave force for the diagonal braces in the vertical frames in line with the wave direction could be approximately evaluated as follows:

Force /unit length of member, $F_p = 0.5\ \rho\ C_D\ D\ (u \operatorname{Cos} \phi)^2\ h / \operatorname{Cos} \phi$, represents the force perpendicular to the member.

The unit force in X-direction $= (Fp) \operatorname{Cos} \phi$

$= [0.5\ \rho\ C_D\ D\ u^2 h] \operatorname{Cos}^2 \phi$

$=$ [unit force on a vertical pile] $\operatorname{Cos}^2 \phi$

where ϕ is the angle in degrees between the brace axis and the vertical axis.

ϕ (in degrees)	$\operatorname{Cos}^2 \phi$
30	0,75
45	0.50
60	0.25

The wave force for the diagonal braces in the vertical frames perpendicular to the wave direction could be approximately evaluated as follows:

Force /unit length of member, $F_p = [0.5\ \rho\ C_D\ D\ u^2]\ h / \operatorname{Sin} \gamma$,

$= (Fp)\ [1/ \operatorname{Sin} \gamma]$

$=$ [unit force on vertical pile] $[1/\operatorname{Sin} \gamma]$

represents the force perpendicular to the member. ϕ is the angle in degrees between the brace axis and the vertical axis.

γ	$1/\sin\gamma$
30	2.000
45	1.414
50	1.305
55	1.220
60	1.155

The wave force in the diagonals in the horizontal frames could be evaluated by the same expression given above for the diagonal braces in the vertical frames. The angle ϕ represents the angle between outer horizontal and the brace.

APPENDIX-C

COMPONENT STRENGTH

The methods used for the evaluation of the ultimate strength of tubular members, interconnecting joints, and welded connections are reviewed in this Appendix. These methods are used for the evaluation of component and system strengths, and the uncertainties in the strength estimates made in Section 2.4 of this report. The component strength of tubulars against the following failure modes is required, in order to evaluate the strength of a bay or of the complete platform:

1. Axial compression strength of a tubular.
2. Axial tensile strength of a tubular.
3. Ultimate moment capacity of a tubular.
4. Strength of damaged members.
5. Joint strength: leg-brace and vertical brace-horizontal brace joints.
6. Tensile strength of welded connection: deck-jacket connection.

C-1 Strength of Intact Tubular Members:

Ultimate Capacity under Primary Axial Loading: The ultimate capacity of the members subjected primarily to the axial load is given by P_u or P_{cr} for the members in compression and by P_y for the members in tension. The following empirical formulation has been established based on the numerical data obtained by inelastic analyses of the members with residual stresses, 1% out-of-roundness, and 0.1% out-of-straightness *[Toma & Chen, 1987]*.

$$P_u / P_y = 1.0 - 0.091\,\lambda - 0.22\,\lambda^2 \quad \text{for } 0 < \lambda < 1.41 \qquad \text{...(C-1)}$$

$$= 0.015 + 0.834 / \lambda^2 \quad \text{for } 1.41 < \lambda < 2.0 \qquad \text{...(C-2)}$$

where: $$\lambda = (1/\pi)\,(\sigma_y / E)^{0.5}\,(Kl/r) \qquad \text{...(C-3)}$$

where, λ = Modified slenderness ratio

$$P_y = A\,\sigma_y \qquad \text{...(C-4)}$$

The following formulation is given for the members with residual stresses, 2% out-of-roundness, and 0.2% out-of-straightness *[Toma & Chen, 1987]*.

$$P_u / P_y = 1.0 - 0.25\lambda - 0.13\,\lambda^2 \qquad \text{for } 0 < \lambda < 1.41 \qquad \text{...(C-5)}$$

$$= 0.052 + 0.67/\lambda^2 \qquad \text{for } 1.41 < \lambda < 2.0 \qquad \text{...(C-6)}$$

where: $\lambda = (1/\pi)\,(\sigma_y / E)^{0.5}\,(Kl/r)$

Fig. C.1 show the curves A & B based on the above formulations for the ultimate compression load by *Toma and Chen (1987)* and compares them with the CRC curve. The curves A & B would give conservative results compared to the CRC curve.

API-RP-2A (1989) recommended practices specify the fabrication tolerance limits as finite values instead of percentage of the member lengths. As per API, beam-columns should not deviate from straightness more than 3/8 inch (10 mm) in total or 1/8 inch per 10 foot increment in length. All of the braces in the horizontal plane should be held vertically within ± 1/2 in (13 mm) tolerance of the drawing dimensions. All other braces, where the end points are dimensioned should be erected so that such points are within ± 1/2 in (13 mm) of the planned dimensions. Thus, the ultimate capacity expressions obtained by curve-A would meet the API fabrication tolerance limits and would be typical for the fabricated tubular columns and braces used in offshore platforms. Following are noted from curve-A for A36 steel (F_y = 45 ksi and E = 29,000 ksi. Hence, λ = (Kl/r) / 80):

Kl/r	λ	P_u / P_y
40	0.5	0.900
80	1.0	0.689
120	1.5	0.386
160	20	0.224

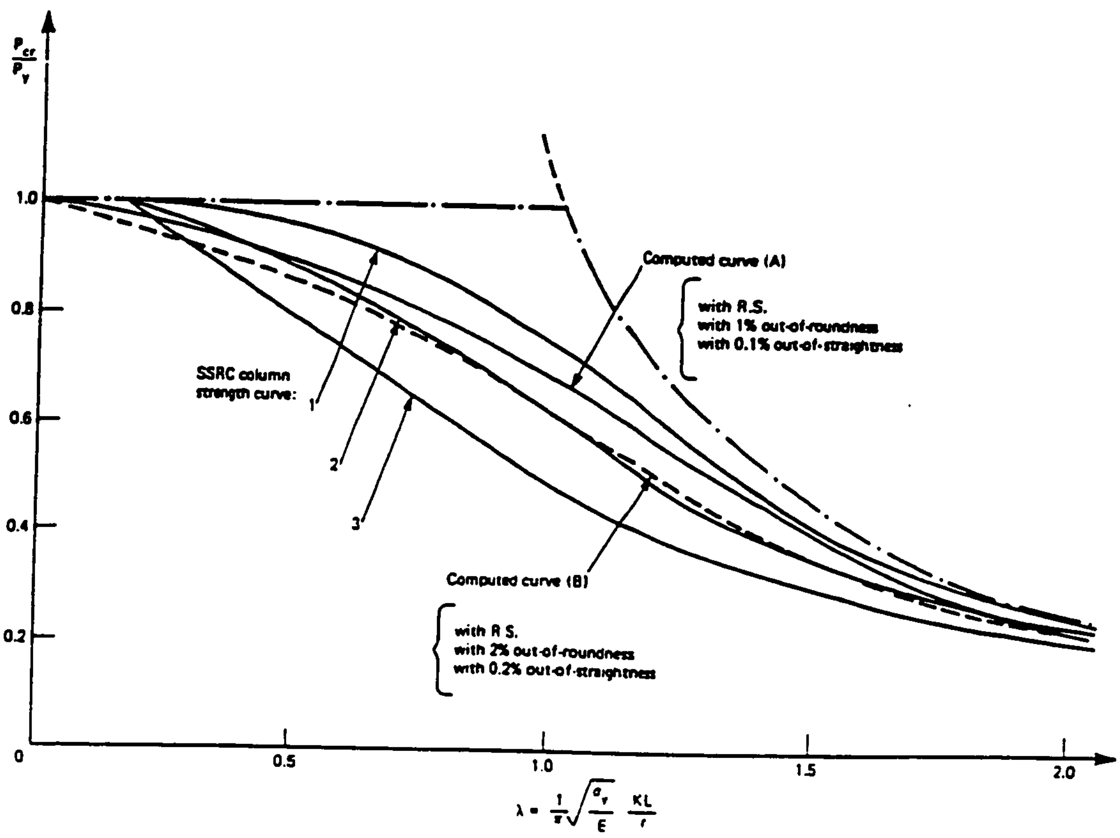


Figure C.1: Calculated Column Strength Curve Compared with SSRC Multiple Column Curve

From this table, a significant reduction in the ultimate compression capacity of a member is noted for an increase in its slenderness ratio (Kl/r). In general, for the Gulf of Mexico platforms the Kl/r ratio is about 80-100 for the braces and about 40 for the legs and piles.

For the less slender members, i.e., the members with Kl/r ratio less than 60, the reduction in the post-buckling compression load is lesser and the buckling load would be closer to the compressive yield load.

$Kl/r < 20$	: Failure by compression yielding.
$20 < Kl/r < 120$	: Failure by inelastic buckling. Post-buckling behavior of the member is of degrading type.
$Kl/r > 120$	: Failure by elastic buckling or post-buckling behavior.

Note that the ultimate compression strength of the tubulars is affected by a number of parameters: yield strength, Young's modulus (material stress strain curve), residual stresses, geometrical parameters, out-of-straightness and out-of-roundness of the members, slenderness ratio, degree of restraint at the ends and along the member, effect from other loads (flexural loads), and hydrostatic pressure. The ultimate tensile strength (yield) is affected only by the geometrical parameters and the yield strength.

The stress-strain relationship for the structural steel is assumed as elastic-perfectly plastic. However, due to the presence of residual stresses and other material-related nonlinearity and inhomogeneity, the buckling load of the members with intermediate slenderness ratios would occur at loads lower than Pcr. This happens because the presence of residual stress causes the fibers, with an initial compressive stress, to yield before the applied stress reaches the yield strength of the material.

The members which are not straight would have deflection even before the load is applied or at zero load state. A perfectly straight member (column) would show deflections only beyond the Euler load, whereas an imperfect member would start to bend as soon as the load is applied. Thus, a slender column with large initial imperfections will fail at loads considerably below the Euler load, while the relatively slender columns will support the axial loads only slightly less than the Euler load.

The out-of-roundness of a member would reduce its local buckling strength, and would initiate the yielding process at lower load.

The slenderness ratio (Kl/r) has the maximum influence on the buckling strength of a member. The buckling strength is inversely proportional to the square of slenderness ratio.

The degree of restraint provided at the end of a member by the adjacent members reduces the effective length of a member for evaluation of the critical buckling load i.e., it would change the behavior of a brace from usually considered pinned end state (K = 0.8) towards that for the fixed end state (K = 0.5). Hence, the degree of restraint is important in the determination of the critical buckling strength for a member.

Deflection of a member would increase when along with the axial load, the lateral loads along its length or moment at its ends are also acting on a member. With an increase in the axial load, plasticization would occur in the critical regions of the member and would cause its failure at a load considerably below the Euler load.

A uniform hydrostatic pressure field would not reduce the theoretical elastic buckling load. However, only the slender columns will buckle elastically. Most members used in the steel jacket platforms have intermediate slenderness ratios and they usually buckle beyond the elastic limit of the cross-section. Thus, the bending rigidity (EI) and the buckling strength of tubular would change.

Hydrostatic pressure in deep-water would have similar effect on the buckling strength of a member as for a member subjected to the lateral load or end moments. Hydrostatic pressure, Q, on a tubular member in the deep water jackets induces compressive hoop stresses. The effect of hydrostatic pressure, Q, on the maximum additional non-hydrostatic axial load capacity, P_u, of an imperfect long column, can be incorporated by the following expression *[Toma and Chen, 1987]:*

$$P_{uq} / P_y = [P_u / P_y]\,[1 + 0.125\,\lambda\,Q / Q_{cr}]\,[1 - (8.6 / 33)\,(Q / Q_{cr})^{1.2}] \quad \text{...(C-7)}$$

where, $$Q_{cr} = [2\,E / (1 - v^2)]\,(t/D)^3 \quad \text{...(C-8)}$$

Hydrostatic pressure would have no effect on the theoretical elastic buckling strength of a perfectly tubular column. However, due to imperfect nature of most of the tubular members, the above relationship should be considered to evaluate effect of the hydrostatic pressure on their ultimate axial capacity. The effect of hydrostatic pressure on the strength curve of a tubular column is shown in Fig. C-2.

As discussed above, in a tubular member the hydrostatic pressure, geometric imperfections, and residual stresses would have the similar effect as noted for the initial stresses in a member. These will induce moment and the axial load capacity of a member would reduce.

Evaluation of the Yield Moment for a Beam-Column:

The capability of a member to allow development of a full plastic hinge depends upon its D/t ratio, which represents the ductility ratio of a member and its susceptibility to local buckling. The relatively stocky and compact sections would have higher ductility and would be less vulnerable to local

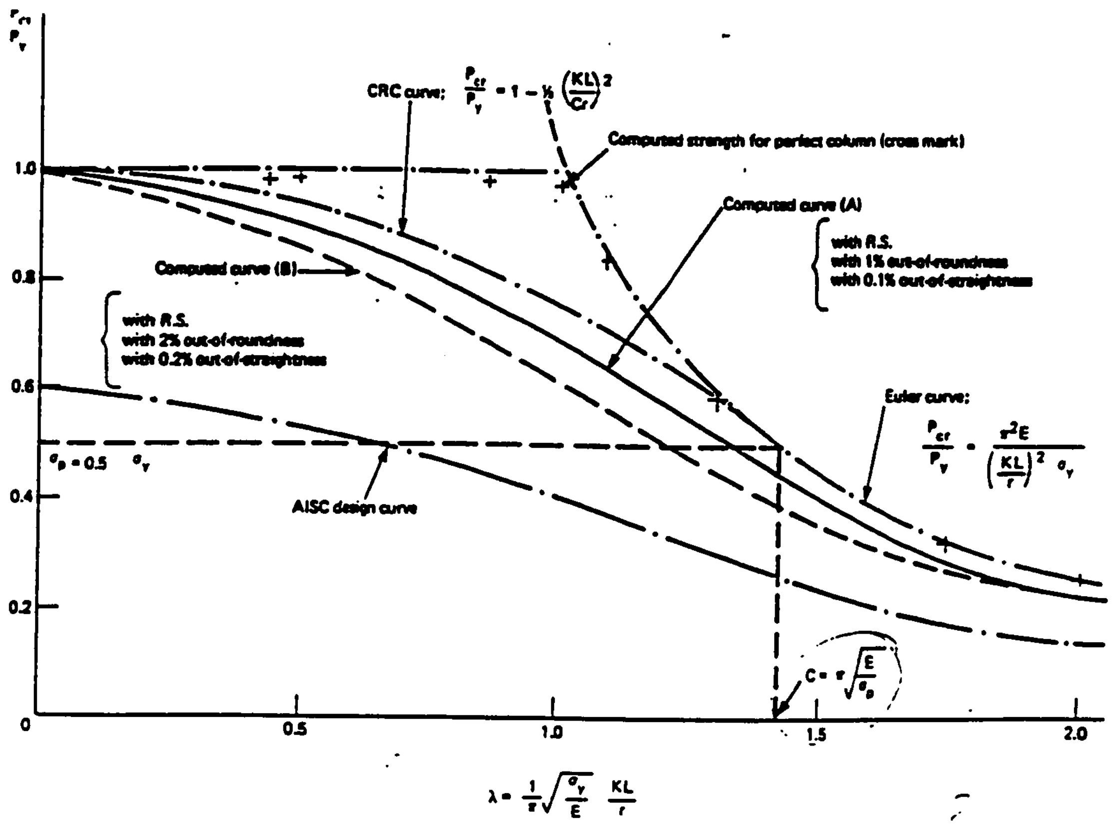


Figure C.2: Effect of Hydrostatic Pressure on the Strength Curve buckling. The load carrying capacity of a tubular member decreases sharply when an elastic local buckling occurs. Fig. C-3 shows inelastic behavior of the tubular sections for different D/t ratios. The detailed description of the behavior of tubular sections for different D/t ratios is given below:

D/t ratio	Type of Section	Remarks
<8	Very stocky section	* Local buckling would not occur
8 to 25	Compact section	* Full M_p can be reached.
		* Section possess sufficient rotation capacity to redistribute moments and to form a plastic mechanism.
		* Failure mode: Plastic collapse
25 to 60	Semi-compact section	* Limited curvature and rotation capacity.
60 to 190		* Failure in the plastic buckling range.
		* Negligible plastic rotation capacity.
> 190		* Failure in the elastic buckling range.
		* Very sensitive to imperfections.

When D/t > 60, the possibility of buckling should be evaluated and the allowable F_a and F_b be based on the allowable local buckling stress, F_{xc} instead of the yield stress, F_y in the AISC stress interaction formulae. The value of plastic moment, M_p for the member will reduce its first yield capacity, M_y.

The following expressions have been recommended for evaluation of the maximum moment capacity of tubulars *[Sohal and Chen, 1987]*.

$$M_{cr} / M_p = 0.919 + 0.393\,(D/t)/100 - 0.545\,[(D/t)/100]^2 \quad \text{for } 36 < D/t < 72 \quad \text{...(C-9)}$$

$$M_{cr} / M_p = 1.037 - 0.163\,(D/t)/100 \quad \text{for } 72 < D/t < 96 \quad \text{...(C-10)}$$

where, D/t is the diameter/thickness ratio and M_p denotes the plastic moment capacity of a tubular.

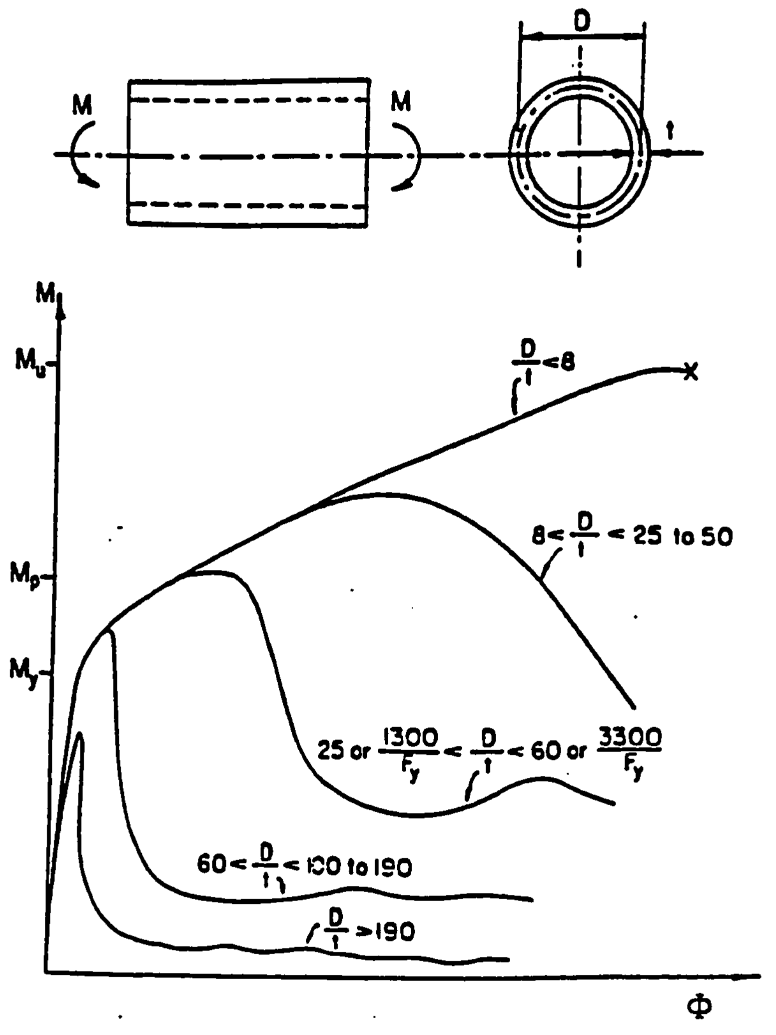


Figure C.3: Inelastic Behavior of Tubular Sections

The plastic moment capacity, M_p, of thin walled tubulars could be approximated by the following expression. M_y, indicates the moment at first yield of the member.

$$M_p \approx 4\,\sigma_y\,t R^2 \quad \text{...(C-11)}$$

$$M_y = \pi \sigma_y t\,(R^2 - 1.5\,R\,t) \approx \pi \sigma_y t R^2 \quad \text{...(C-12)}$$

$$M_y / M_p = 0.785 \quad \text{...(C-13)}$$

D/t	36	48	60	72	84	96
M_{cr}/M_p	0.99	0.98	0.96	0.92	0.90	0.88

Full plastic moment capacity of a tubular member would reduce due to the presence of external hydrostatic pressure, Q, and is given by the following relationship [*Chen and Han, 1985*]:

$$[M_{pq} / M_p] = 1 - [(i_R + 3) / 20]\,[Q / Q_{cr}] \quad \text{...(C-14)}$$

where: M_{pq} = Plastic moment capacity reduced due to the external hydrostatic pressure

i_R = Value of the out-of-roundness of a cross-section and expressed in percent.

$$Q_{cr} = [2\,E / (1 - \nu^2)]\,(t/D)^3 \quad \text{...(C-15)}$$

For $i_R = 1.0$ (i.e., 1% out-of-roundness)

Q/Q_{cr}	0.0	0.20	0.40	0.60	0.80	1.0
M_{pq}/M_p	1.0	0.96	0.92	0.88	0.84	0.80

Interaction of Axial and Bending Loads:

The load carrying capacity of a circular tubular section under combined action of the axial and bending loads can be determined by the following exact formula *[Chen and Han, 1985]*.

$$[M / M_{cr}] = \cos[(\pi/2)\,(P / P_{cr})] \quad \text{...(C-16)}$$

where P_{cr} and M_{cr} are determined from the equations given earlier. The interaction diagrams for the columns with different slenderness ratios are shown in Fig. C-4, which are based on accurate theoretical analysis. In the offshore platforms, the normal range of L/r ratio is 80-120 for the braces and about 40 for the legs and piles. For the members in these ranges, the interaction diagrams in simplified analyses can be approximately considered as straight lines.

The lateral loads acting on a platform are primarily carried by the inclined braces in the vertical frames. These braces are predominantly loaded in compression or tension along their axial direction and the flexure loading on them are from the local transverse loads and the moment due to relative bending stiffness of the member compared to the adjacent members in the plane of the frame. The major vertical braces would normally have a moment of the order of 0.05 to 0.10 of their plastic moment capacity, M_p. The members in the wave zone would have higher moments. Fig. C-5, shows the effect of moment on reduction in the axial capacity of members.

The deck-legs, jacket-legs, and piles carry vertical loads from the deck and the vertical load component of the overturning moment due to the lateral loads. They also carry local lateral loads due to waves and winds, which result in high bending moments. These members are usually of large diameter and have high axial and flexural load carrying capacities. For the Gulf of Mexico platforms with normal topside loads, the deck legs would have higher stresses due to the bending moments than due to the axial loads. Therefore, the ultimate moment capacity of the deck legs will not reduce much. Fig. C-6, shows the variation in M_u/M_p for the different values of axial loads.

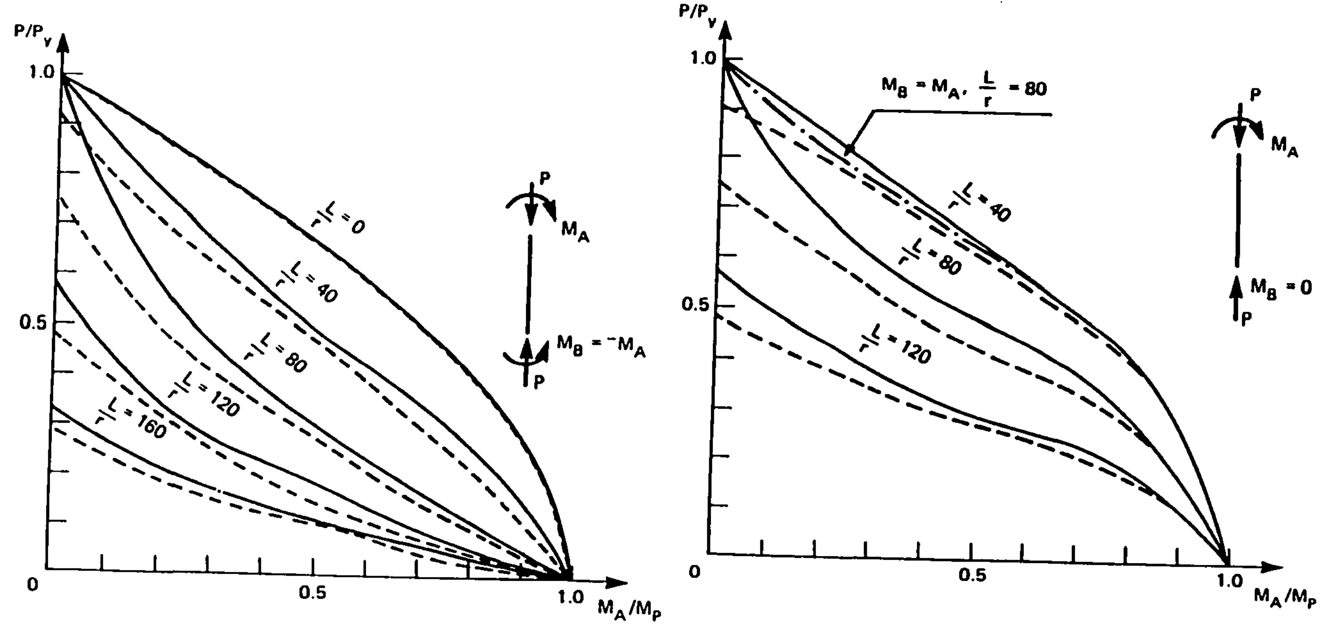


Figure C.4: Interaction Curves for Different Slenderness Ratios

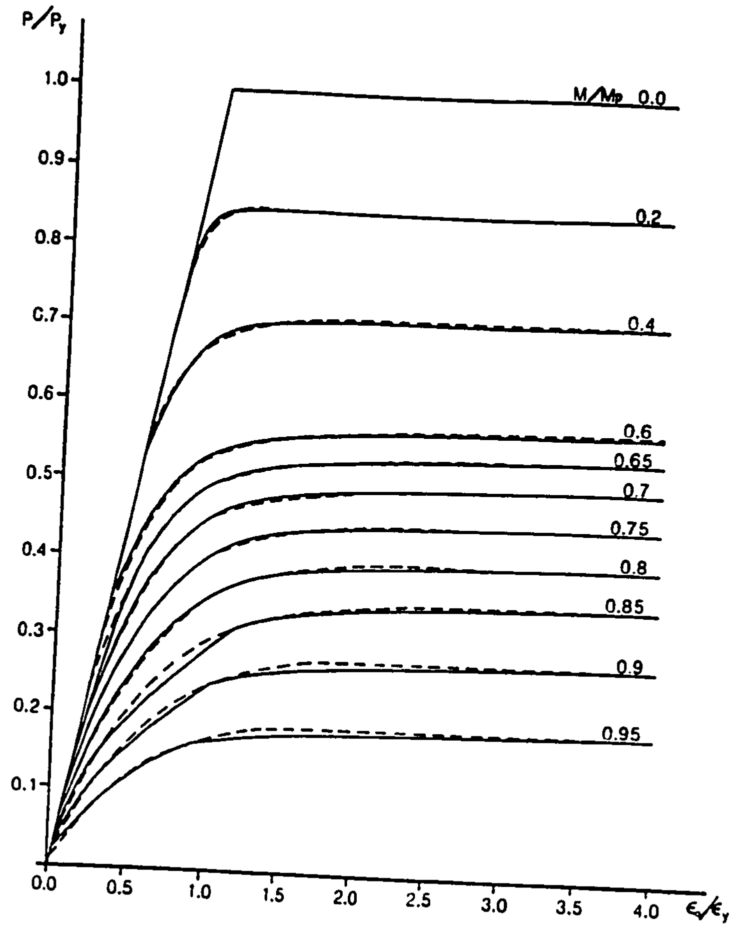


Figure C.5: Effect of Moment on Reduction in the Axial Capacity of Tubular Members

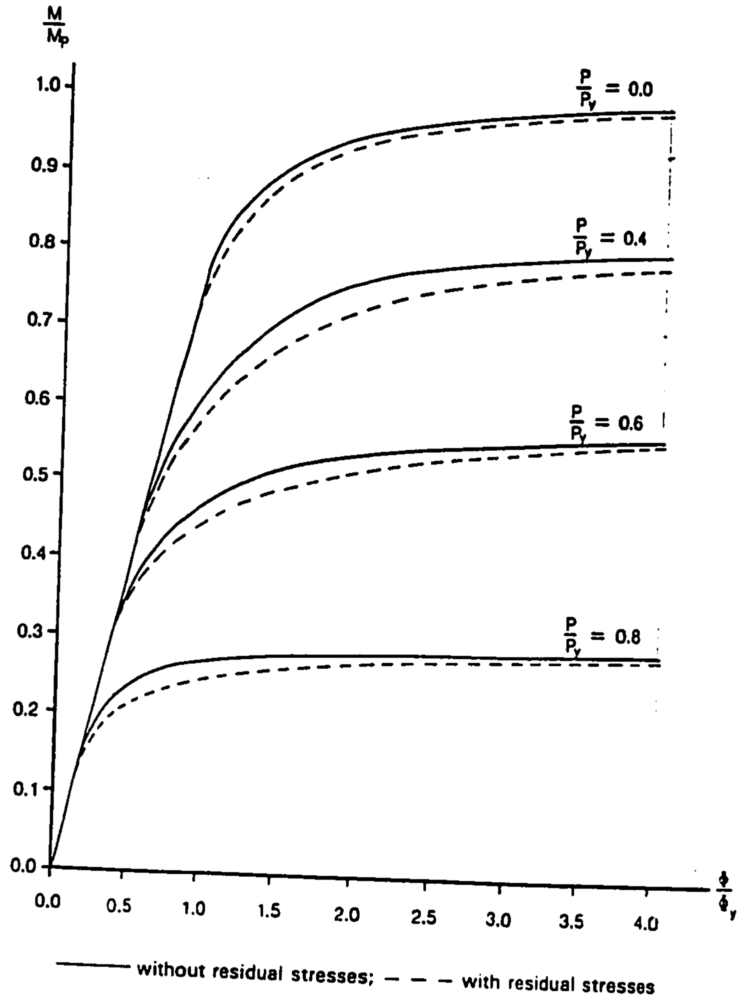


Figure C.6: Effect of Axial Load on Reduction in the Moment Capacity of Tubular Members

Thus, depending on the relative magnitude of axial and bending loads on the members, and their ultimate load capacity the interaction diagram shown in Fig. C-4 would change.

The strength interaction relation for the bending moment, M and an external non-hydrostatic axial load, P, would be non-linear, as given by the following expressions. The effect of external hydrostatic pressure on the members is also included in these expressions.

$$M / M_{pq} = 1 - 1.18 [P / P_{yq}]^2 \quad \text{for } 0 \le [P / P_{yq}] \le 0.65 \qquad \text{...(C-17)}$$

$$M / M_{pq} = 1.43 [1 - P/P_{yq}] \quad \text{for } 0.65 \le P/P_{yq} \le 1.0 \qquad \text{...(C-18)}$$

where, M_{pq} is Full plastic moment capacity reduced for the presence of external hydrostatic pressure, Q; P_{yq} is a non-hydrostatic axial load at the full yield condition, reduced for the presence of external hydrostatic pressure, Q. P_{yq} is given by the following expression:

$$P_{yq} / P_y = 1 - [\{i_R + 7.6\} / 33] \; [Q / Q_{cr}]^{1.2} \qquad \text{...(C-19)}$$

C-2 Evaluation of the ultimate strength of a joint:

The ultimate strength of a joint would be the lower of the nominal brace load, at which a joint is likely to fail, and the ultimate strength of brace. The ultimate strength of a joint in terms of the nominal brace loads are given as follows.

$$P_u = Q_u \, Q_f \, F_y \, T^2 / \sin\theta \qquad \text{... (C-20)}$$

where, $Q_u = 3.4 + 19\beta$ $\qquad T$ = Chord thickness

$\beta = d/D$ $\qquad D$ = Chord diameter

$Q_f = 1 - \lambda \gamma A^2$ $\qquad \gamma = D/(2T)$

$A = [f_a^2 + f_{b1}^2 + f_{b1}^2] / F_y$

λ = 0.03 for brace axial stress

= 0.045 for brace in-plane bending

= 0.021 for brace out-of-plane bending

$$M_u = [Q_u \, Q_f \, F_y \, T^2 / \sin\theta] \, (0.8 \, d) \qquad \text{... (C-21)}$$

where, d = Brace diameter

In the above formulas, Q_u is the non-dimensional factor for loads in the individual braces and Q_f is the non-dimensional factor for stresses in the chord. Fig. C.7 presents the various configurations of the simple joints used in jacket platforms. The load deflection behavior of T and double-T joints in the cases of punching and pullout loads are shown in Fig. C.8. From this figure, it is noted that these joints have a significantly high ultimate strength, an order of magnitude 2 or more, under the axial pullout loads than for the punching load case.

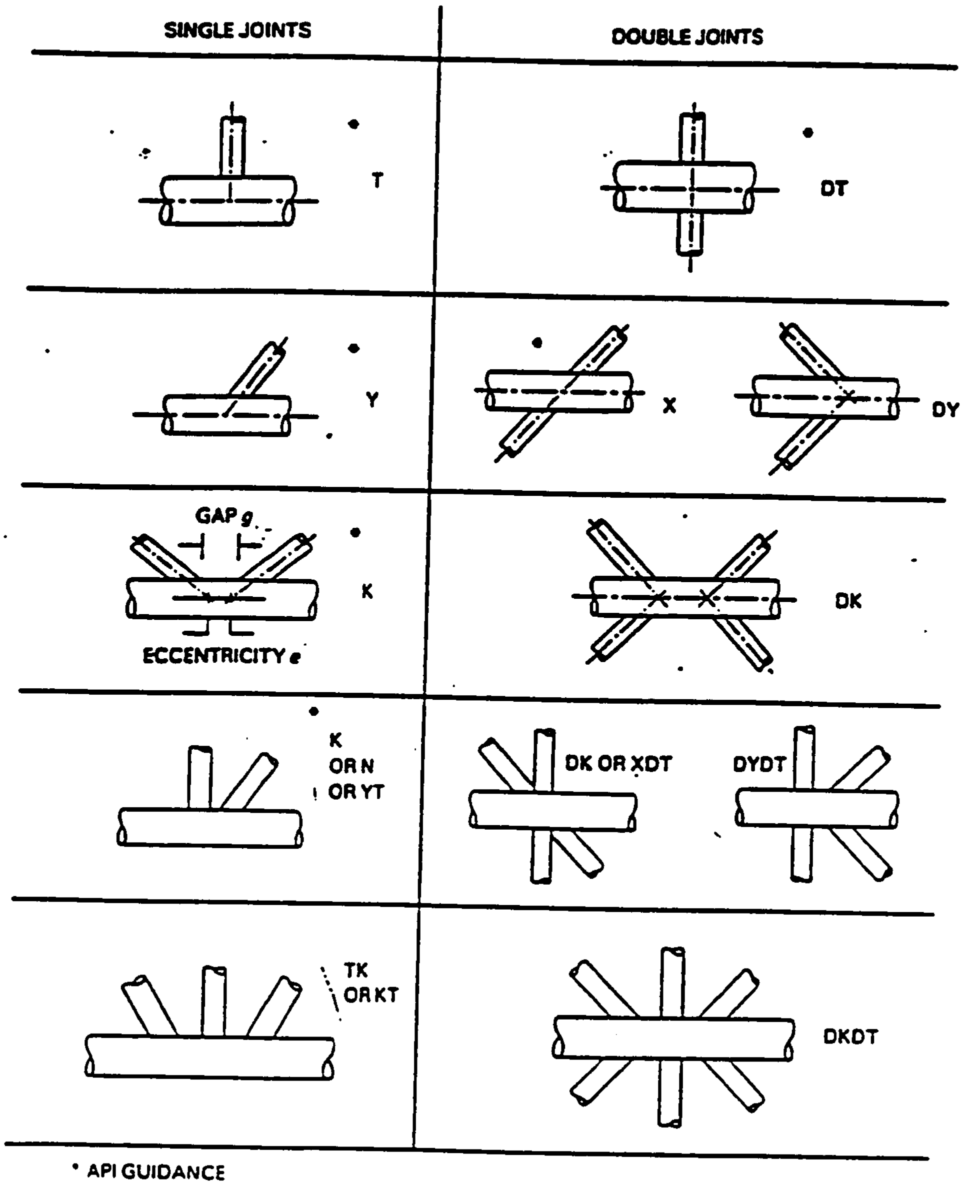


Figure C.7: Simple Joint Configurations

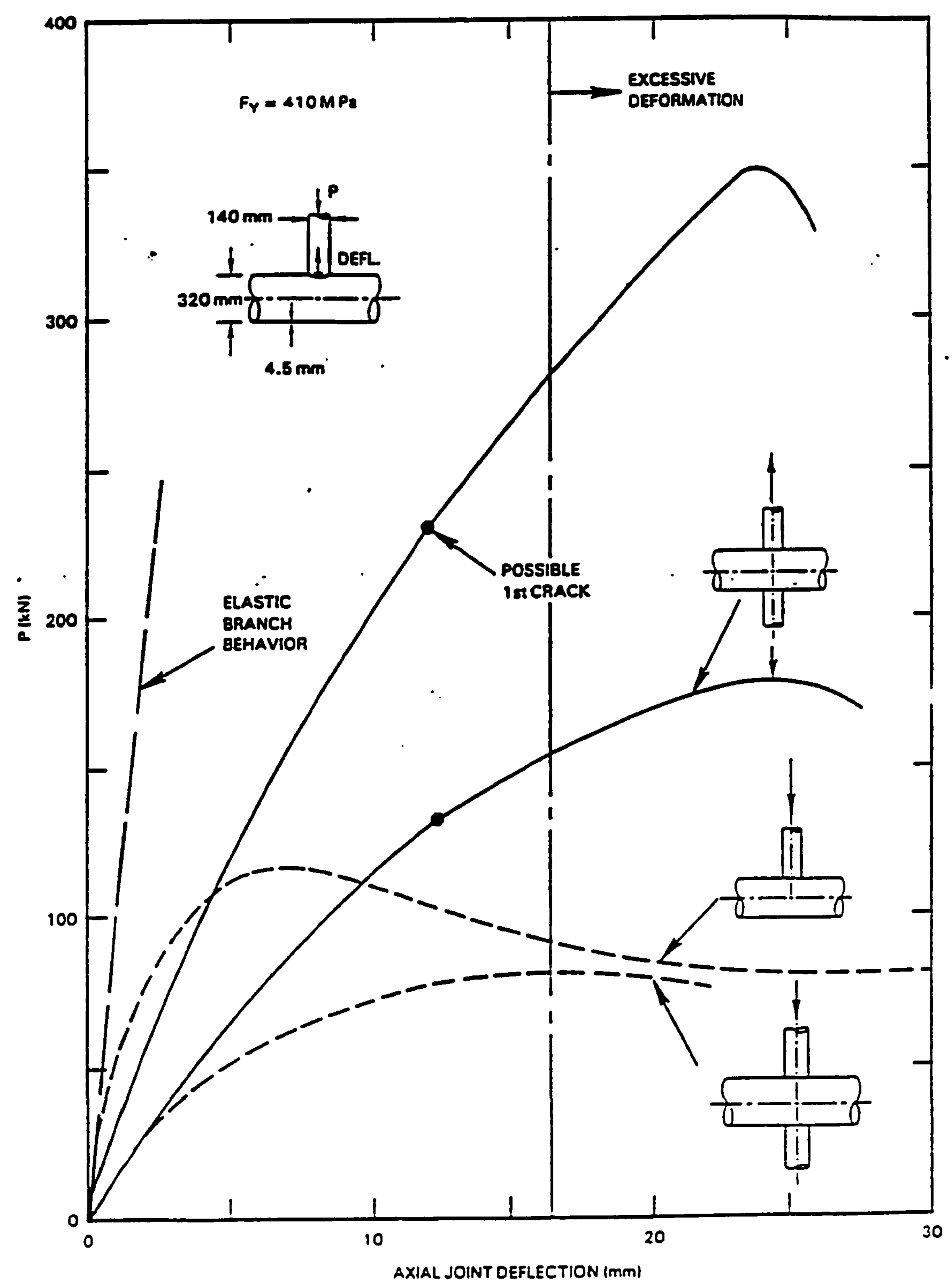


Figure C.8: Load-Deflection Behavior of Axially Loaded T and DT Joints

APPENDIX- D

COMPLEMENTARY STANDARD NORMAL TABLE

This Table gives the N(0,1) distribution; $\phi(-\beta) = 1 - \phi(\beta)$

β	$\Phi(-\beta)$	β	$\Phi(-\beta)$	β	$\Phi(-\beta)$
0.00	0.5000	0.40	0.3446	0.80	0.2119
0.01	0.4960	0.41	0.3409	0.81	0.2090
0.02	0.4920	0.42	0.3372	0.82	0.2061
0.03	0.4880	0.43	0.3336	0.83	0.2033
0.04	0.4841	0.44	0.3300	0.84	0.2005
0.05	0.4801	0.45	0.3264	0.85	0.1977
0.06	0.4761	0.46	0.3228	0.86	0.1949
0.07	0.4721	0.47	0.3192	0.87	0.1922
0.08	0.4681	0.48	0.3156	0.88	0.1894
0.09	0.4642	0.49	0.3121	0.89	0.1867
0.10	0.4602	0.50	0.3085	0.90	0.1841
0.11	0.4562	0.51	0.3050	0.91	0.1814
0.12	0.4522	0.52	0.3015	0.92	0.1788
0.13	0.4483	0.53	0.2981	0.93	0.1762
0.14	0.4443	0.54	0.2946	0.94	0.1736
0.15	0.4404	0.55	0.2912	0.95	0.1711
0.16	0.4364	0.56	0.2877	0.96	0.1685
0.17	0.4325	0.57	0.2843	0.97	0.1660
0.18	0.4286	0.58	0.2810	0.98	0.1635
0.19	0.4247	0.59	0.2776	0.99	0.1611
0.20	0.4207	0.60	0.2743	1.00	0.1587
0.21	0.4168	0.61	0.2709	1.01	0.1563
0.22	0.4129	0.62	0.2676	1.02	0.1539
0.23	0.4091	0.63	0.2644	1.03	0.1515
0.24	0.4052	0.64	0.2611	1.04	0.1492
0.25	0.4013	0.65	0.2579	1.05	0.1469
0.26	0.3974	0.66	0.2546	1.06	0.1446
0.27	0.3936	0.67	0.2514	1.07	0.1423
0.28	0.3897	0.68	0.2483	1.08	0.1401
0.29	0.3859	0.69	0.2451	1.09	0.1379
0.30	0.3821	0.70	0.2420	1.10	0.1357
0.31	0.3783	0.71	0.2389	1.11	0.1335
0.32	0.3745	0.72	0.2358	1.12	0.1314
0.33	0.3707	0.73	0.2327	1.13	0.1292
0.34	0.3669	0.74	0.2297	1.14	0.1271
0.35	0.3632	0.75	0.2266	1.15	0.1251
0.36	0.3594	0.76	0.2236	1.16	0.1230
0.37	0.3557	0.77	0.2207	1.17	0.1210
0.38	0.3520	0.78	0.2177	1.18	0.1190
0.39	0.3483	0.79	0.2148	1.19	0.1170

β	Φ(−β)	β	Φ(−β)	β	Φ(−β)	β	Φ(−β)	β	Φ(−β)	β	Φ(−β)
1.20	0.1151	1.80	0.3593E-01	2.40	0.8198E-02	3.00	0.1350E-02	3.60	0.1591E-03	5.00	0.2869E-06
1.21	0.1131	1.81	0.3515E-01	2.41	0.7976E-02	3.01	0.1306E-02	3.61	0.1531E-03	5.05	0.2203E-06
1.22	0.1112	1.82	0.3438E-01	2.42	0.7760E-02	3.02	0.1264E-02	3.62	0.1473E-03	5.10	0.1694E-06
1.23	0.1094	1.83	0.3363E-01	2.43	0.7550E-02	3.03	0.1223E-02	3.63	0.1417E-03	5.15	0.1299E-06
1.24	0.1075	1.84	0.3289E-01	2.44	0.7344E-02	3.04	0.1183E-02	3.64	0.1363E-03	5.20	0.9935E-07
1.25	0.1057	1.85	0.3216E-01	2.45	0.7143E-02	3.05	0.1144E-02	3.65	0.1311E-03	5.25	0.7582E-07
1.26	0.1038	1.86	0.3144E-01	2.46	0.6947E-02	3.06	0.1107E-02	3.66	0.1261E-03	5.30	0.5772E-07
1.27	0.1020	1.87	0.3074E-01	2.47	0.6756E-02	3.07	0.1070E-02	3.67	0.1212E-03	5.35	0.4384E-07
1.28	0.1003	1.88	0.3005E-01	2.48	0.6569E-02	3.08	0.1035E-02	3.68	0.1166E-03	5.40	0.3321E-07
1.29	0.9853E-01	1.89	0.2938E-01	2.49	0.6387E-02	3.09	0.1001E-02	3.69	0.1121E-03	5.45	0.2510E-07
1.30	0.9680E-01	1.90	0.2872E-01	2.50	0.6210E-02	3.10	0.9676E-03	3.70	0.1077E-03	5.50	0.1892E-07
1.31	0.9510E-01	1.91	0.2807E-01	2.51	0.6037E-02	3.11	0.9354E-03	3.71	0.1036E-03	5.55	0.1423E-07
1.32	0.9342E-01	1.92	0.2743E-01	2.52	0.5868E-02	3.12	0.9042E-03	3.72	0.9956E-04	5.60	0.1067E-07
1.33	0.9176E-01	1.93	0.2680E-01	2.53	0.5703E-02	3.13	0.8740E-03	3.73	0.9569E-04	5.65	0.7985E-08
1.34	0.9013E-01	1.94	0.2619E-01	2.54	0.5543E-02	3.14	0.8447E-03	3.74	0.9196E-04	5.70	0.5959E-08
1.35	0.8851E-01	1.95	0.2559E-01	2.55	0.5386E-02	3.15	0.8163E-03	3.75	0.8837E-04	5.75	0.4436E-08
1.36	0.8692E-01	1.96	0.2500E-01	2.56	0.5234E-02	3.16	0.7888E-03	3.76	0.8491E-04	5.80	0.3293E-08
1.37	0.8535E-01	1.97	0.2442E-01	2.57	0.5085E-02	3.17	0.7622E-03	3.77	0.8157E-04	5.85	0.2438E-08
1.38	0.8380E-01	1.98	0.2385E-01	2.58	0.4940E-02	3.18	0.7363E-03	3.78	0.7836E-04	5.90	0.1800E-08
1.39	0.8227E-01	1.99	0.2330E-01	2.59	0.4799E-02	3.19	0.7113E-03	3.79	0.7527E-04	5.95	0.1326E-08
1.40	0.8076E-01	2.00	0.2275E-01	2.60	0.4661E-02	3.20	0.6871E-03	3.80	0.7230E-04	6.00	0.9716E-09
1.41	0.7927E-01	2.01	0.2222E-01	2.61	0.4527E-02	3.21	0.6636E-03	3.81	0.6943E-04	6.10	0.5220E-09
1.42	0.7781E-01	2.02	0.2169E-01	2.62	0.4397E-02	3.22	0.6409E-03	3.82	0.6667E-04	6.20	0.2778E-09
1.43	0.7636E-01	2.03	0.2118E-01	2.63	0.4269E-02	3.23	0.6189E-03	3.83	0.6402E-04	6.30	0.1463E-09
1.44	0.7494E-01	2.04	0.2068E-01	2.64	0.4146E-02	3.24	0.5976E-03	3.84	0.6147E-04	6.40	0.7636E-10
1.45	0.7353E-01	2.05	0.2018E-01	2.65	0.4025E-02	3.25	0.5770E-03	3.85	0.5901E-04	6.50	0.3945E-10
1.46	0.7215E-01	2.06	0.1970E-01	2.66	0.3907E-02	3.26	0.5570E-03	3.86	0.5664E-04	6.60	0.2018E-10
1.47	0.7078E-01	2.07	0.1923E-01	2.67	0.3793E-02	3.27	0.5377E-03	3.87	0.5437E-04	6.70	0.1023E-10
1.48	0.6944E-01	2.08	0.1876E-01	2.68	0.3681E-02	3.28	0.5190E-03	3.88	0.5218E-04	6.80	0.5130E-11
1.49	0.6811E-01	2.09	0.1831E-01	2.69	0.3573E-02	3.29	0.5009E-03	3.89	0.5007E-04	6.90	0.2549E-11
1.50	0.6681E-01	2.10	0.1786E-01	2.70	0.3467E-02	3.30	0.4834E-03	3.90	0.4804E-04	7.00	0.1254E-11
1.51	0.6552E-01	2.11	0.1743E-01	2.71	0.3364E-02	3.31	0.4664E-03	3.91	0.4610E-04	7.10	0.6107E-12
1.52	0.6426E-01	2.12	0.1700E-01	2.72	0.3264E-02	3.32	0.4500E-03	3.92	0.4422E-04	7.20	0.2946E-12
1.53	0.6301E-01	2.13	0.1659E-01	2.73	0.3167E-02	3.33	0.4342E-03	3.93	0.4242E-04	7.30	0.1407E-12
1.54	0.6178E-01	2.14	0.1618E-01	2.74	0.3072E-02	3.34	0.4189E-03	3.94	0.4069E-04	7.40	0.6654E-13
1.55	0.6057E-01	2.15	0.1578E-01	2.75	0.2980E-02	3.35	0.4040E-03	3.95	0.3902E-04	7.50	0.3116E-13
1.56	0.5938E-01	2.16	0.1539E-01	2.76	0.2890E-02	3.36	0.3897E-03	3.96	0.3742E-04	7.60	0.1445E-13
1.57	0.5821E-01	2.17	0.1500E-01	2.77	0.2803E-02	3.37	0.3758E-03	3.97	0.3588E-04	7.70	0.6636E-14
1.58	0.5706E-01	2.18	0.1463E-01	2.78	0.2718E-02	3.38	0.3624E-03	3.98	0.3441E-04	7.80	0.3017E-14
1.59	0.5592E-01	2.19	0.1426E-01	2.79	0.2635E-02	3.39	0.3494E-03	3.99	0.3298E-04	7.90	0.1359E-14
1.60	0.5480E-01	2.20	0.1390E-01	2.80	0.2555E-02	3.40	0.3369E-03	4.00	0.3162E-04	8.00	0.6056E-15
1.61	0.5370E-01	2.21	0.1355E-01	2.81	0.2477E-02	3.41	0.3248E-03	4.05	0.2557E-04	8.10	0.2673E-15
1.62	0.5262E-01	2.22	0.1321E-01	2.82	0.2401E-02	3.42	0.3131E-03	4.10	0.2062E-04	8.20	0.1169E-15
1.63	0.5155E-01	2.23	0.1287E-01	2.83	0.2327E-02	3.43	0.3017E-03	4.15	0.1659E-04	8.30	0.5058E-16
1.64	0.5050E-01	2.24	0.1255E-01	2.84	0.2256E-02	3.44	0.2908E-03	4.20	0.1332E-04	8.40	0.2167E-16
1.65	0.4947E-01	2.25	0.1222E-01	2.85	0.2186E-02	3.45	0.2802E-03	4.25	0.1067E-04	8.50	0.9197E-17
1.66	0.4846E-01	2.26	0.1191E-01	2.86	0.2118E-02	3.46	0.2700E-03	4.30	0.8524E-05	8.60	0.3864E-17
1.67	0.4746E-01	2.27	0.1160E-01	2.87	0.2052E-02	3.47	0.2602E-03	4.35	0.6794E-05	8.70	0.1608E-17
1.68	0.4648E-01	2.28	0.1130E-01	2.88	0.1988E-02	3.48	0.2507E-03	4.40	0.5402E-05	8.80	0.6623E-18
1.69	0.4552E-01	2.29	0.1101E-01	2.89	0.1926E-02	3.49	0.2415E-03	4.45	0.4285E-05	8.90	0.2701E-18
1.70	0.4457E-01	2.30	0.1072E-01	2.90	0.1866E-02	3.50	0.2326E-03	4.50	0.3391E-05	9.00	0.1091E-18
1.71	0.4363E-01	2.31	0.1044E-01	2.91	0.1807E-02	3.51	0.2240E-03	4.55	0.2677E-05	9.10	0.4363E-19
1.72	0.4272E-01	2.32	0.1017E-01	2.92	0.1750E-02	3.52	0.2157E-03	4.60	0.2108E-05	9.20	0.1728E-19
1.73	0.4182E-01	2.33	0.9903E-02	2.93	0.1695E-02	3.53	0.2077E-03	4.65	0.1656E-05	9.30	0.6773E-20
1.74	0.4093E-01	2.34	0.9642E-02	2.94	0.1641E-02	3.54	0.2000E-03	4.70	0.1298E-05	9.40	0.2629E-20
1.75	0.4006E-01	2.35	0.9387E-02	2.95	0.1589E-02	3.55	0.1926E-03	4.75	0.1015E-05	9.50	0.1011E-20
1.76	0.3921E-01	2.36	0.9138E-02	2.96	0.1538E-02	3.56	0.1854E-03	4.80	0.7914E-06	9.60	0.3847E-21
1.77	0.3836E-01	2.37	0.8894E-02	2.97	0.1489E-02	3.57	0.1784E-03	4.85	0.6158E-06	9.70	0.1450E-21
1.78	0.3754E-01	2.38	0.8657E-02	2.98	0.1441E-02	3.58	0.1717E-03	4.90	0.4780E-06	9.80	0.5408E-22
1.79	0.3673E-01	2.39	0.8424E-02	2.99	0.1395E-02	3.59	0.1653E-03	4.95	0.3701E-06	9.90	0.1998E-22

APPENDIX-E

DETAILED COMPUTATIONS FOR EXAMPLE PLATFORM-A

The platform-A is a 8-legged platform located in 271 ft. water depth, and was designed and installed to the practices followed during 1969. The main legs of the platform are sized to provide support to the 42"Φ piles. The physical features of Platform-A are given below and in Section 7.1.1.

The **salient features** of this platform were summarized in Section 7.1.1 in the main report, and are listed below:

- Grouted leg/pile annulus
- No joint cans on legs
- Severely damaged platform with 4-members ineffective
- Self-contained drilling platform
- Penetration of the 8-42" Φ piles up to 270' below seabed
- Anodes used for cathodic protection system
- Grated deck used.
- 24-conductors of 22" Φ x 0.5" and -risers
- 2-boat landings and 8-barge bumpers
- Manned platform with helideck and quarters, evacuated in advance of hurricane.

The structural configuration of the platform-A is given in Fig. E.1.

In this Appendix, the detailed computations for RSR evaluation at the screening cycle-2 are presented, to demonstrate the extent of computations needed to the regulators and users. The results are summarized and discussed in Section 7.1 of this study.

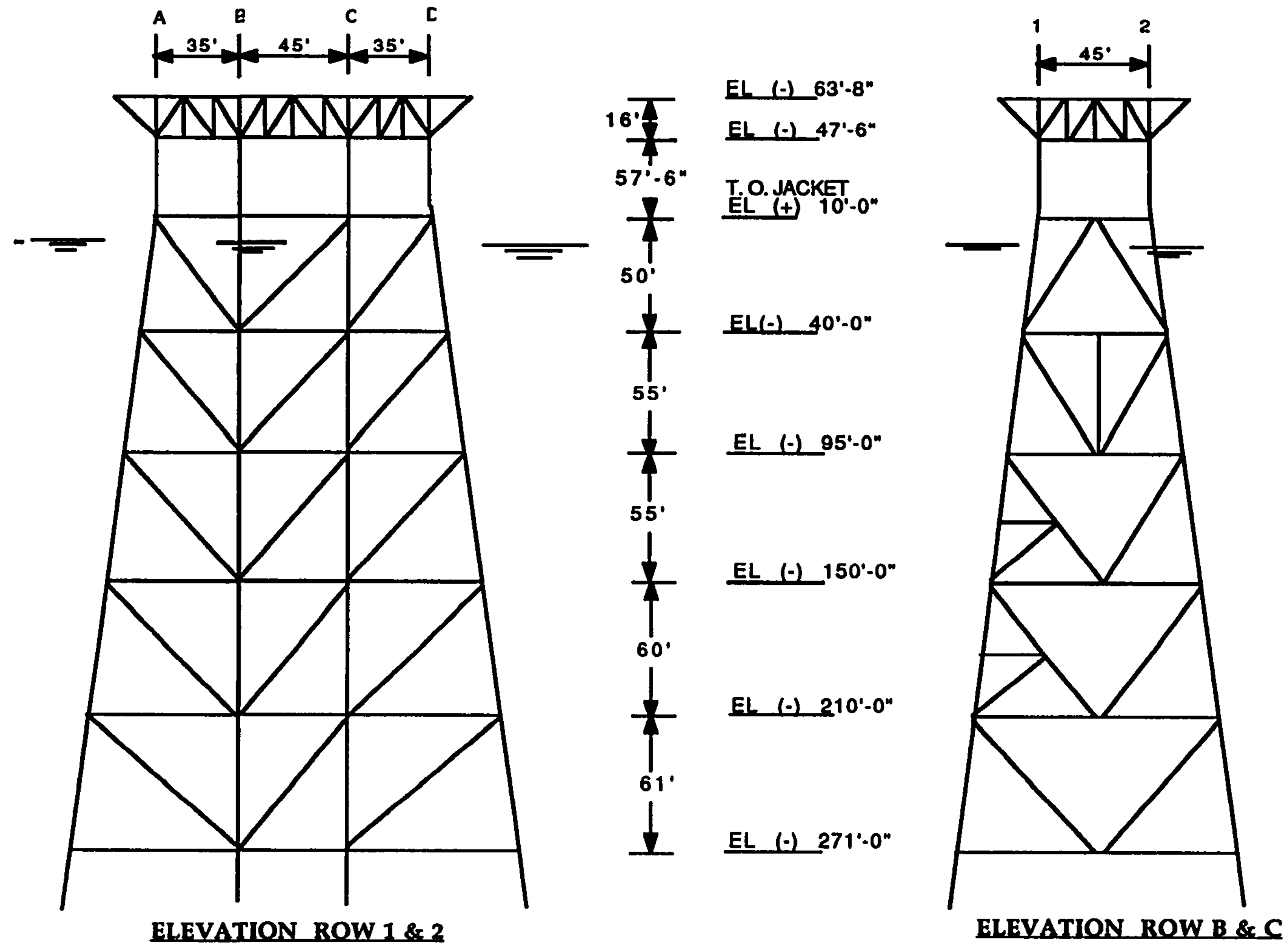


Figure E.1: Structural Configuration - Platform A

E.1 Development of the Lateral Load Pattern

The Airy's linear wave theory, stretched to the crest level has been used to develop lateral load on a unit diameter vertical pile and the platform.

Water depth, d = 271 ft.

Storm tide, s = 2.5 ft.

d + s = 273.5 ft.

$H_{reference}$ = 70 ft. and 1969 H_{max} = 58 ft.

Let S = 1/12, then $T_m = 1.53\ (H_m)^{0.5} = 12.80$ sec.

$L = [g T^2 / 2\pi]\ [\tanh\ (2\pi d / L)] = 839.65\ [\tanh (1719/L)]$; $L = 815.20$ ft.

Let wave crest height = 0.6 H = 0.6 (70) = 42 ft. above m.s.l.

$u = (\pi H/T)\ [\mathrm{Cosh}\ \{(2\pi y/L)\ d\ /(d + 0.6\ H)\}]\ /\ [\mathrm{Sinh}\ (2\pi d/L)]$

$= 17.18\ \mathrm{Cosh}\ [(2\pi/\ 815.20)\ \{273.5\ /\ (273.5 + 42)\}\ y]\ /\ \mathrm{Sinh}\ (\ 2\ \pi\ 273.5\ /\ 815.20)$

$= 17.18\ \mathrm{Cosh}\ (0.0067\ y)\ /\ \mathrm{Sinh}\ (2.108)$

$= 4.237\ \mathrm{Cosh}\ (0.0067\ y)$

$F \approx F_D = K_D\ u\ |u| = [0.5\ \rho\ C_D\ D]\ u\ |u| = [1.99/2]\ (0.6)\ (1)\ u\ |u| \approx 0.6\ u\ |u|$

y (ft)	u_y (ft/sec)	F_D/unit area $= 0.6\,u\,\|u\|$ lb/ft^2	F_D on 1 ft. Φ pile Kips/ft
0.00	4.237	10.77	--
61.0	4.600	12.70	0.720
121.0	5.710	19.560	0.970
176.0	7.540	34.110	1.480
231.0	10.410	65.020	2.730
271.0	13.360	107.090	3.440
281.0	14.240	121.670	1.140
291.0	15.190	138.440	1.300
300.0	16.100	155.530	1.320
315.5	17.800	190.100	2.680
			Σ = 15.780 Kips/ft of Φ

The wave force on a 1 ft Φ vertical pile extending from the seabed to the wave crest elevation is **15.780** Kips, by the Airy's wave theory. In this estimate, the wave crest height has been assumed as 0.6 H above the mean sea level.

The variations of wave velocity and unit wave force with depth, and the shear force on a unit diameter vertical pile are shown in Fig. E.2.

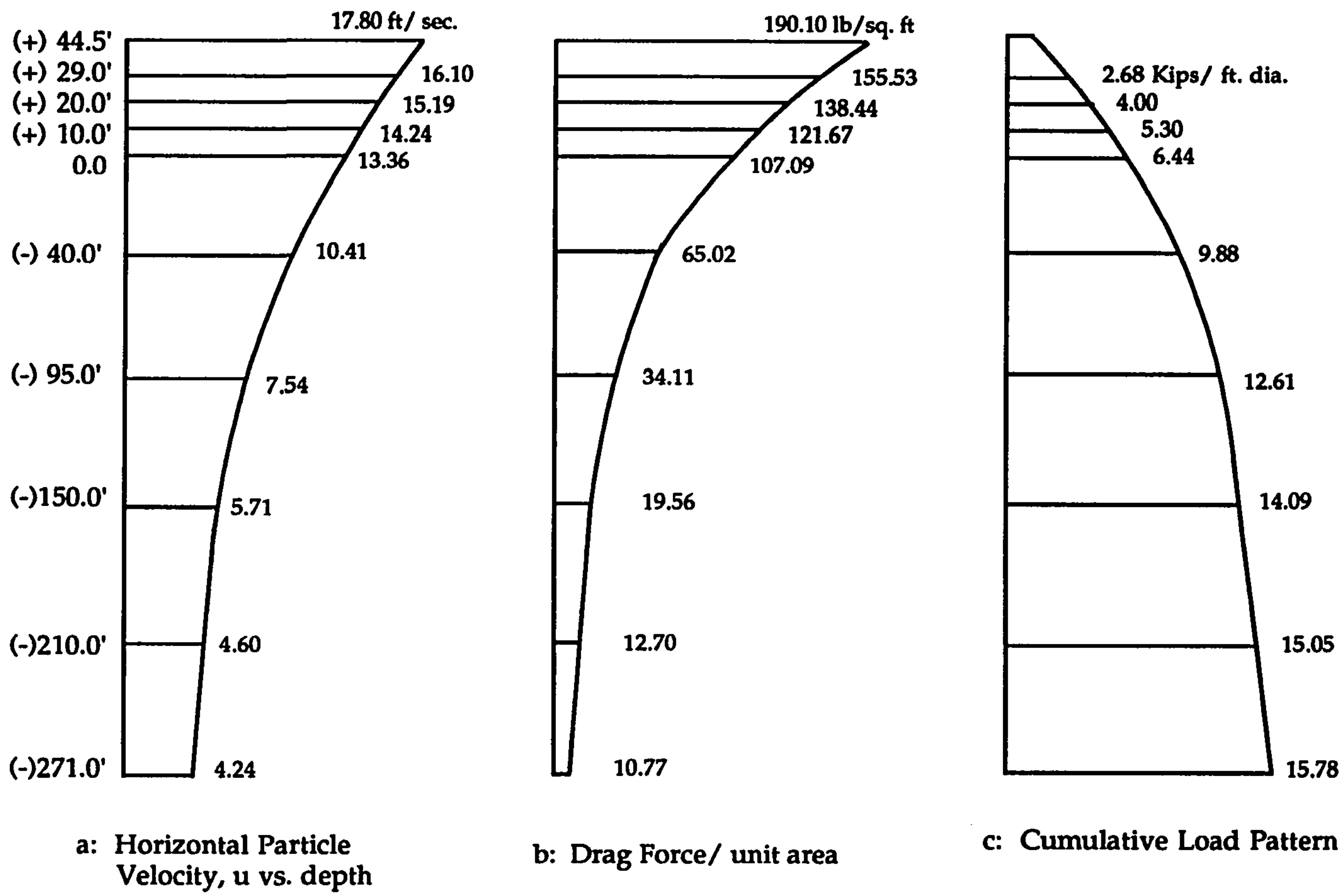


Figure E.2: Wave Kinematics and Forces on a Unit Diameter Vertical Pile - Platform A

I) **Wave load on Deck Bay:**

The deck leg section varies from 36"Φ x 1" to 42"Φ x 1". The deck legs elevation extend from (+)10', topmost jacket horizontal framing, to (+)47'-6", center line elevation of the lower deck. The deck leg section is 42"Φ x 1" from El. (+) 10' to (+) 20', transition from 42"Φ x 0.75" to 36"Φ x 0.75" from El. (+) 20' to (+) 29', 36"Φ x 0.75" from El. (+) 29' to (+) 39', and 36"Φ x 1" from El. (+) 39' to (+) 47'-6".

The wave force on a unit diameter vertical member in a deck bay would be 5.30 Kips (= 2.68 Kips + 1.32 Kips + 1.30 Kips) and it will act at 23.4' elevation.

Force on one deck leg = 2.68 (3) + 1.32 (3.25) + 0.70 (3.5) + 0.60 (3.75) = **17.03 Kips.**

Vertical Members	Diameter	Number	Force in Kips
Deck legs	36"Φ/45"Φ	8	136.24 Kips
Conductors	22"Φ	24	233.20 Kips
Risers	16"Φ	1	7.10 Kips
	10"Φ	2	8.83 Kips
Total			385.37Kips

Wind load on deck (assumed) = 80 Kips.

Total lateral load on deck bay = 80 + 385 = **465Kips.**

The lateral load will act as 15.97' from El. (+) 10'.

Overturning moment @ El. (+) 10' = (80) 60 + (385) 15.97 ≈ **10,953Kips-ft.**

Topside load = **13,450 Kips** (= 6,100 Tons)

If, it is assumed that all the legs are equally loaded, vertical load on the legs = [13,450 / 8] + [10,953 / (2 x 115)] = 1,681 + 48 = **1,729** Kips.

II) Wave Load on Jacket Bays:

The marine growth thickness has been taken as 1" on the members from the mean sea level (m.s.l.) to elevation (-) 100' and to taper off to zero from elevation (-) 100' to the seabed.

a) Jacket Bay-1: El. (+) 10' to El. (-) 40':

The wave force on a unit diameter vertical member in the jacket bay-1, is obtained from Fig. 7.2 as **4.58 Kips** (1.14 Kips from m.s.l. to El. (+)10', and 3.44 Kips from m.s.l. to El. (-) 40'). The marine growth of 2" on diameter is considered for the member parts from m.s.l. to El. (-) 40'.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	45"Φ	8	143.5 Kips
Conductors	22"Φ	24	219.8 Kips
Risers	16"Φ	1	6.90 Kips
	10"Φ	2	9.16 Kips
Appurtenances:			
Caissons/Casings/Barge Bumpers			60.00 Kips
Boat landings			50.00 Kips
Total			489.36 Kips

End-on Direction:

Wave force on vertical braces: 6 no. 20"Φ x 0.438" braces have been provided on the two **side frames**. The formulas developed in Appendix-B have been utilized to evaluate the forces. These vertical braces are at γ (= 55 for 4 braces and 40 for two braces) from the horizontal.

Force on side frames $= 6 \text{ (unit force) (Dia. in ft.) } \sin^2 \gamma$

$\approx 4\,(4.58)\,(1.83)\,\sin^2 55 + 2\,(4.58)\,(1.83)\,\sin^2 40$

$=$ **31.75 Kips.**

8 no. 18"Φ x 0.594" braces have been provided on the four **front frames**. These vertical braces are at γ (= 59.75) from the horizontal.

Force on front frames = 8 (unit force) (Dia. in ft.) (1/ Sin γ)

$\approx$ 8 (4.58) (20 / 12) (1/ Sin 59.75) = **70.69 Kips.**

<u>Wave force on horizontal braces @ El. (+) 10':</u> When wave approaches from the orthogonal directions, the effective horizontal braces would be 4-18"Φ x 0.375" (outer horizontal, 46' long), 4-8.625"Φ x 0.375" (diagonals, 23' width, γ= 37.27), 8-8.625"Φ x 0.375" (12' width, γ = 61.92), 1-8.625"Φ x 0.375" (23' width, γ = 45).

Force on horizontals = (0.1217) [4 (20/16) 46 + (10.625/ 12) {4 (23) Cos^2 37.27 + 8 (12) Cos^2 61.92 + 1 (23) Cos^2 45}]

= 0.1217 (306.67 + 80.60) = **47.13 Kips.**

<u>Total end-on wave load on Jacket Bay-1:</u>

Vertical components	489.36 Kips
Vertical braces	102.44 Kips
Horizontal braces	47.13 Kips
Total force:	638.93 Kips

<u>Broadside Direction:</u>

<u>Wave force on vertical braces:</u> 6 no. 20"Φ x 0.438" braces have been provided on the two **front frames**. These vertical braces are at γ (= 55 for 4 braces and 40 for two braces) from the horizontal.

Force on front frames $\approx$ 4 (4.58) (22 / 12) (1/ Sin 55) + 2(4.58) (22/12) (1/ Sin 48)

= **763.62Kips.**

8 no. 18"Φ x 0.594" braces have been provided on the four **side frames**. These vertical braces are at γ (= 59.75) from the horizontal.

Force on side frames $\approx$ 8 (4.58) (20/ 12) Sin^2 59.75 = **45.57 Kips.**

<u>Wave force on horizontal braces @ El. (+) 10':</u> When wave approaches from the orthogonal directions, the effective horizontal braces would be

4-18"Φ x 0.375" (outer horizontal, 46' long), 4-8.625"Φ x 0.375" (diagonals, 23' width, $\gamma = 37.27$), 8-8.625"Φ x 0.375" (12' width, $\gamma = 61.92$), 1-8.625"Φ x 0.375" (23' width, $\gamma = 45$).

$$\text{Force on horizontals} = (0.1217)\,[2\,(16/12)\,116 + 2\,(14.75/12)\,80 + (12.75/12)\,35.5 + 8\,(10.625/12)\,22.5\,\text{Cos}^2\,26 + 4 = (10.625/12)\,18\,\text{Cos}^2\,51.95]$$

$$= 0.1217\,[309.3 + 196.7 + 37.7 + 128.75 + 24.22] = 84.78 \text{ Kips.}$$

Total load on broadside - Jacket Bay-1:

Vertical components	489.36 Kips
Vertical braces	109.17Kips
Horizontal braces	84.78 Kips
Total force:	683.31 Kips

b) **Jacket Bay-2: El. (-) 40' to El. (-) 95':**

The wave force on a unit diameter vertical member in the jacket bay-2, is obtained from Fig. 7.2 as **2.73 Kips**. The marine growth of 2" on diameter is considered for the member parts from El. (-) 40' to (-) 95'.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	45"Φ	8	85.5 Kips
Conductors	22"Φ	24	131.0Kips
Risers	16"Φ	1	4.11 Kips
	10"Φ	2	5.46 Kips
Total			226.10 Kips

End-on Direction:

Wave force on vertical braces: 6 no. 24"Φ x 0.375" braces have been provided on the two **side frames**. The formulas developed in Appendix-B have been utilized to evaluate the forces. These vertical braces are at γ (= 53 for 4 braces and 50.71 for two braces) from the horizontal.

$$\text{Force on side frames} \approx 4\,(2.73)\,(26/12)\,\text{Sin}^2\,53 + 2\,(2.73)\,(26/12)\,\text{Sin}^2\,50.71 = 22.18 \text{ Kips.}$$

8 no. 18"Φ x 0.457" braces, and 4- 18" have been provided on the four **front frames**. These braces are at γ (= 62.07) from the horizontal.

$$\text{Force on front frames} \approx (2.73)\,(20/12)\,[8\,(1/\text{Sin}\,59.75) + 4] = 59.40 \text{ Kips.}$$

Wave force on horizontal braces @ El. (-) 40': When wave approaches from the orthogonal directions, the effective horizontal braces would be 4-18"Φ x 0.375" (outer horizontal, 46' long), 4-8.625"Φ x 0.375" (diagonals, 23' width, γ = 37.27), 8-8.625"Φ x 0.375" (12' width, γ = 61.92), 1-8.625"Φ x 0.375" (23' width, γ = 45).

Force on horizontals ≈ [0.065 / 0.1217] (47.13) (16/20) = **20.14 Kips.**

Total end-on wave load on Jacket Bay-2:

Vertical components	226.10 Kips
Vertical braces	81.58 Kips
Horizontal braces	20.14 Kips
Total force:	327.82 Kips

Broadside Direction:

Wave force on vertical braces: 6 no. 20"Φ x 0.438" braces have been provided on the two **front frames**. These vertical braces are at γ (= 55 for 4 braces and 40 for two braces) from the horizontal.

Force on front frames ≈ (2.73) [4 (26 / 12) (1/ Sin 53)+2(26/12) (1/ Sin 50.7)] = **63. 12 Kips.**

8 no. 18"Φ x 0.594" braces have been provided on the four **side frames**. These vertical braces are at γ (= 59.75) from the horizontal.

Force on side frames ≈ 8 (2.73) (20/ 12) Sin^2 62.07 = **28.41 Kips.**

Wave force on horizontal braces @ El. (-) 40':

Force on horizontals = (0.065 / 0.1217) (84.78) (14/14) = **45.28 Kips**

Total load on broadside - Jacket Bay-2:

Vertical components	226.10 Kips
Vertical braces	91.53Kips
Horizontal braces	45.28 Kips
Total force:	362.91 Kips

c) **Jacket Bay-3: El. (-) 95' to El. (-) 150':**

The wave force on a unit diameter vertical member in the jacket bay-3, is obtained from Fig. 7.2 as **1.48 Kips**. The marine growth of 2" on diameter is considered for the member parts from El. (-) 40' to (-) 95'.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	45"Φ	8	46.35 Kips
Conductors	22"Φ	24	71.02 Kips
Risers	16"Φ	1	2.22 Kips
	10"Φ	2	2.96 Kips
Total			122.60 Kips

End-on Direction:

Wave force on vertical braces: 6 no. 24"Φ x 0.375" braces have been provided on the two **side frames**. These vertical braces are at γ (= 48.6 for 4 braces and 50.71 for two braces) from the horizontal.

Force on side frames ≈ 4 (1.48) (26/12) $\sin^2$ 48.6 +2 (1.48) (26/12) $\sin^2$ 50.71 = **11.06 Kips.**

8 no. 20"Φ x 0.375" braces have been provided on the four **front frames**. These braces are at γ (= 56.80) from the horizontal.

Force on front frames ≈ (1.48) (22 / 12) [8 (1/ Sin 56.80)] = **25.94Kips.**

Wave force on horizontal braces @ El. (-) 95':

Force on horizontals ≈ [0.0311 / 0.1217] (47.13) = **13.21 Kips.**

Total end-on wave load on Jacket Bay-3:

Vertical components	122.60 Kips
Vertical braces	37.00 Kips
Horizontal braces	13.21 Kips
Total force:	172.81 Kips

Broadside Direction:

Wave force on vertical braces: 6 no. 24"Φ x 0.375" braces have been provided on the two **front frames**. These vertical braces are at γ (= 48.6 for 4 braces and 50.7 for two braces) from the horizontal.

Force on front frames ≈ (1.48) (26 / 12) [4 (1/ Sin 48.6) + 2 (1/ Sin 50.7)] = **25.39 Kips.**

8 no. 20"Φ x 0.375" braces have been provided on the four **side frames**. These vertical braces are at γ (= 56.80) from the horizontal.

Force on side frames ≈ 8 (1.48) (22/ 12) $\sin^2$ 56.80 = **15.20 Kips.**

Wave force on horizontal braces @ El. (-) 95':

Force on horizontals = (0.0341 / 0.1217) (84.78) (18/16) = **26.72 Kips**

Total load on broadside - Jacket Bay-3:

Vertical components	122.60 Kips
Vertical braces	40.59 Kips
Horizontal braces	26.72 Kips
Total force:	**189.90Kips**

c) **Jacket Bay-4: El. (-) 150' to El. (-) 210':**

The wave force on a unit diameter vertical member in the jacket bay-4, is obtained from Fig. 7.2 as **0.97 Kips**.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	45"Φ	8	30.38 Kips
Conductors	22"Φ	24	46.55 Kips
Risers	16"Φ	1	1.47 Kips
	10"Φ	2	1.95 Kips
Total			80.35 Kips

End-on Direction:

Wave force on vertical braces: 6 no. 24"Φ x 0.375" braces have been provided on the two **side frames**. These vertical braces are at γ (= 47.23 for 4 braces and 53.13 for two braces) from the horizontal.

Force on side frames ≈ 4 (0.97) (26/12) $\sin^2$ 47.23 + 2 (0.97) (26/12) $\sin^2$ 53.13 = **7.22 Kips.**

8 no. 24"Φ x 0.375" braces have been provided on the four **front frames**. These braces are at γ (= 54.50) from the horizontal.

Force on front frames ≈ (0.97) (26 / 12) [8 (1/ Sin 54.50)] = **20.65 Kips.**

Wave force on horizontal braces @ El. (-) 150':

Force on horizontals ≈ [0.02 / 0.1217] (47.13) (22/ 20) = **8.52 Kips.**

Total end-on wave load on Jacket Bay-4:

Vertical components	80.35 Kips
Vertical braces	27.87 Kips
Horizontal braces	8.52 Kips
Total force:	**116.74 Kips**

Broadside Direction:

Wave force on vertical braces: 6 no. 24"Φ x 0.375" braces have been provided on the two **front frames**. These vertical braces are at γ (= 47.23 for 4 braces and 53.13 for two braces) from the horizontal.

Force on front frames ≈ (0.97) (26 / 12) [4 (1/ Sin 47.23) + 2 (1/ Sin 53.13)] = **16.71 Kips.**

8 no. 24"Φ x 0.375" braces have been provided on the four **side frames**. These vertical braces are at γ (= 54.50) from the horizontal.

Force on side frames ≈ 8 (0.97) (26/ 12) Sin^2 54.50 = **11.14 Kips.**

Wave force on horizontal braces @ El. (-) 150':

Force on horizontals = (0.02 / 0.1217) (84.78) (20/16) = **17.42 Kips**

Total load on broadside - Jacket Bay-4:

Vertical components	80.35 Kips
Vertical braces	27.85 Kips
Horizontal braces	17.42 Kips
Total force:	125.62 Kips

e) **Jacket Bay-5: El. (-) 210' to El. (-) 271':**

The wave force on a unit diameter vertical member in the jacket bay-3, is obtained from Fig. 7.2 as **0.72 Kips.**

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	45"Φ	8	22.55 Kips
Conductors	22"Φ	24	34.55 Kips
Risers	16"Φ	1	1.08 Kips
	10"Φ	2	1.44 Kips
Total			59.64 Kips

End-on Direction:

Wave force on vertical braces: 6 no. 30"Φ x 0.438" braces have been provided on the two **side frames**. These vertical braces are at γ (= 43.99 for 4 braces and 53.13 for two braces) from the horizontal.

Force on side frames ≈ 4 (0.72) (32/12) Sin^2 43.99 + 2 (0.72) (32/12) Sin^2 53.13 = **6.16 Kips.**

8 no. 24"Φ x 0.375" braces have been provided on the four **front frames.** These braces are at γ (= 50.53) from the horizontal.

Force on front frames ≈ (0.72) (26 / 12) [8 (1/ Sin 50.53)] = **16.17 Kips.**

Wave force on horizontal braces @ El. (-) 95':

Force on horizontals ≈ [0.0127 / 0.020] (8.52) (26/20) (101/ 45) = **15.80 Kips.**

Total end-on wave load on Jacket Bay-5:

Vertical components	59.64 Kips
Vertical braces	22.33 Kips
Horizontal braces	15.80 Kips
Total force:	**97.77 Kips**

Broadside Direction:

Wave force on vertical braces: 6 no. 30"Φ x 0.438" braces have been provided on the two **front frames.** These vertical braces are at γ (= 43.99 for 4 braces and 53.13 for two braces) from the horizontal.

Force on front frames ≈ (0.72) (32 / 12) [4 (1/ Sin 43.99) + 2(1/ Sin 53.13)] = **15.85 Kips.**

8 no. 24"Φ x 0.375" braces have been provided on the four **side frames.** These vertical braces are at γ (= 50.53) from the horizontal.

Force on side frames ≈ 8 (0.72) (26/ 12) Sin^2 50.53 = **7.44 Kips.**

Wave force on horizontal braces @ El. (-) 210':

Force on horizontals = (0.0127 / 0.02) (17.42) (22/20) (171/ 115) = **18.09 Kips**

Total load on broadside - Jacket Bay-5:

Vertical components	59.64 Kips
Vertical braces	23.29 Kips
Horizontal braces	18.09 Kips
Total force:	**101.02 Kips**

Wave load on Mud-level Horizontal Framing:

End on ≈ [0.0108 / 0.0127] (15.80) (117/ 101) = 15.56 Kips.

Broadside = (0.0108/ 0.0127) (18.09) (24/22) (187/ 171) = 18.35 Kips.

SUMMARY OF LATERAL LOADS:

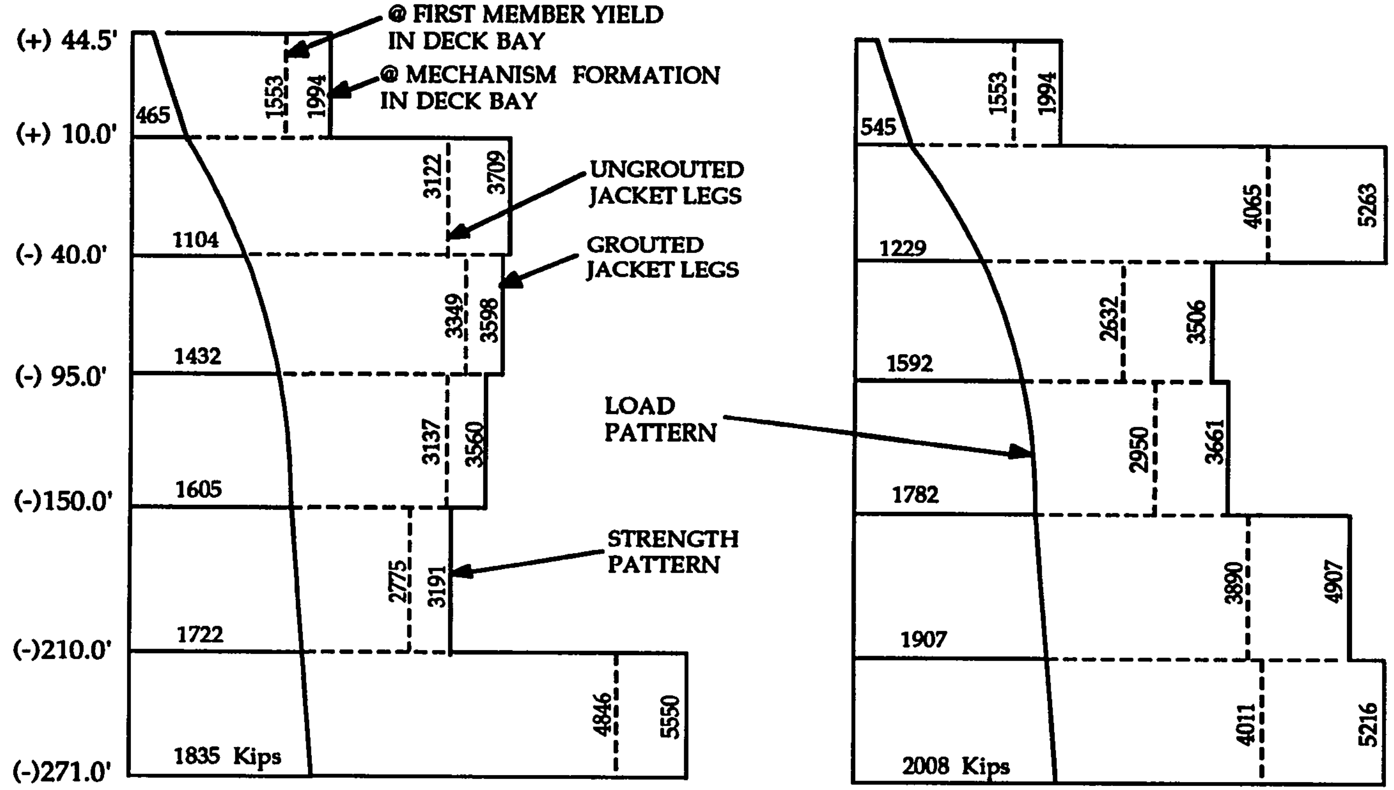
The summary of wave loads for each bay is given below:

	End-on Loading	Broadside loading
Wind loads	80.00 Kips	160.00 Kips
Wave Loads:		
Deck Bay	385.40 Kips	385.40 Kips
Jacket Bay-I	638.93 Kips	683.31 Kips
Jacket Bay-II	327.82 Kips	362.91 Kips
Jacket Bay-III	172.81 Kips	189.90 Kips
Jacket Bay-IV	116.74 Kips	125.62 Kips
Jacket Bay-V	97.77 Kips	101.02 Kips
Mud-Level Framing	15.56 Kips	18.35 Kips
	Σ = 1835.03Kips	2026.50 Kips

The summary of loads on the basis of the loads on different components of Platform-A are given below:

	End-on Loading	Broadside loading
Wind loads	80.00 Kips	160.00 Kips
Wave Loads:		
Main legs (8 No.)	464.60 Kips	464.60 Kips
Conductors (24-No.)	736.15 Kips	736.15 Kips
Risers (4-No.)	52.68 Kips	52.68 Kips
Appurtenances	110.00 Kips	110.00 Kips
Vertical Braces (6-Frames)	271.22 Kips	292.43 Kips
Horizontal Framings	120.36 Kips	210.64 Kips
	Σ = 1835.03Kips	2026.50 Kips

The lateral load pattern for the platform has been developed based on the above and is shown in Fig. E.3.



a: End-on Load Cases

b: Broadside Load case

Figure E.3: Load and Strength Patterns- Platform A

E.2 Evaluation of Platform Strength Pattern

The strength pattern of the platform is obtained by evaluation of the minimum strength of the individual bays. The first step in the determination of the bay strength is to identify the failure modes and mechanisms, which are likely to form in the different bays (deck, jacket, and foundation piles sub-structures) against the imposed loads.

The primary failure modes and mechanisms, which are likely to form under increasing lateral loads are as follows:

1. Deck bay: portal mechanism upon yielding of all the legs; failure of deck leg-pile connection.
2. Jacket bays: individual failure or successive failure of several of the vertical diagonal braces in the vertical frames; or failure of major joint(s) before failure of vertical brace(s).
3. Pile bay: portal mechanism would form due to yielding of all the piles and some conductors; the pile-soil contact may fail due to uplift forces; or piles may fail due to plunging under compressive load.

E.2.1: Deck Bay

The plastic mechanism in the deck bay would form by yielding of the sections at top and bottom of all of the eight main legs with a minimum section of 36"Φ x 1". .

Sump deck elevation:	= (+) 35'
Lower deck Elev.	= (+) 46'
Upper deck Elev.	= (+) 63'
Deck weight	= 1,323 Kips (600 Tons)
Topside weight	= 12,128 Kips (5,500 Tons)
Total load	= 13,450 Kips (6,100 Tons)
Vertical load on one-leg	= 13,450 / 8 = 1,681 Kips.

Size of main legs is 36" Φ x 1" from Elev. (+) 29' to Elev. (+) 46'. The main legs sections transition from 42" Φ x 1" to 36" Φ x 1", from Elev. (+) 15' to Elev. (+) 29'. For section 36"Φ x 1", area (A) is 110 in^2, moment of inertia (I) is 16,851 in^4, and radius of gyration (r) is 12.38 in.

The unsupported length of deck leg, l is 31', and $Kl/r = 24$

$\lambda = (1/\pi)\ (\sigma_y / E)^{0.5}\ (Kl/r) = 0.271$

$P_u / P_y = 0.96$ where, $P_y = \sigma_y A = 4{,}950$ Kips

So, $P_u = 4{,}752$ Kips

$M_p = 4\ \sigma_y\ t\ R^2 = 4{,}860$ **Kips-ft.**

$M_y = \pi\ \sigma_y\ t\ R^2 = 3{,}817$ **Kips-ft.** at first yield of a deck-leg section.

The allowable M_p would depend upon the ratio of (P/P_y) for the deck legs. For this example platform, the actual ratios of the vertical loads on the legs are not known. An approximate estimate of the compression load in the legs can be made as below:

Total topside D.L. and L.L.	= 13,450 Kips
Wind load- End on	= 80 Kips @ El. (+) 65'
Wind Load-Broad side	= 160 Kips @ El. (+) 65'
Wave load on the deck bay	= 385.37 Kips
Overturning moment (OTM)	= 10,953 Kips-ft.for end-on case.
	= 16,554 Kips-ft. for broad-side case.

Assuming that all the legs are equally loaded, the

Total vertical load in the legs = 13,451 / 8 + 10,953/(2x 115) = **1,729 Kips**

Let the vertical load will be approximately 1,800 Kips. The ratio of (P/P_y) is approximately equal to 0.38 and the corresponding ratio of M/M_{cr} would be: $M/ M_{cr} = \text{Cos}\ [0.5\ P/ P_{cr}] = 0.83$

These ratios would vary for the different legs. Therefore, approximate estimates could be obtained by considering the average values. For D/t ratio of 36, M/M_{cr} would be 0.99.

Therefore, M = (0.99) (0.827) 4,860 = 3,979 Kips-ft.

and = (0.99) (0.827) 3,817 = 3,125 Kips-ft. for first yield condition.

The ultimate lateral load capacity, $P_u = 16\ M_p/L = 16\ (3{,}125)/\ 31 \approx$ **1,613 Kips** and will be **2,054 Kips** at formation of a mechanism.

In case of large displacement at the top of the deck bay, the lateral stiffness of the bay would reduce due to the moment induced by the vertical loads on the legs. The reduction in the stiffness of the bay could be determined as the ratio of the vertical load on the leg and the bay height (= 1,800/31 ≈ **60 Kips /ft** of displacement). This value for the stiffness reduction is very less to be of any significance to the strength against formation of a rigid-plastic mechanism discussed earlier.

The net ultimate lateral load capacity at first yield will be 1,613 - 60 = **1,553 Kips for the end -on direction and 1,994 Kips for the broad-side direction.**

E.2.2 Jacket Bays

E. 2.2.1 **End-on Direction:**

Jacket-Bay 1:

Horizontal Braces: For the vertical brace of 14" Φ x 0.375" size, the area (A) is 16.052 in^2, the moment of inertia (I) is 372.76 in^4, and the radius of gyration (r) is 4.82". The local wave loads on the horizontal braces will reduce the axial load carrying capacity of the brace.

Unsupported length of brace, L	35'	45'
$Kl/r = 0.8\ (L \times 12)\ /\ 4.82$	69.71	82.46
$\lambda = (1/\pi)\ (\sigma_y\ /\ E)^{0.5}\ (Kl/r)$	0.8714	1.0308
$P_u\ /\ P_y = 1.0 - 0.091\ \lambda - 0.22\ \lambda^2 = (X)$	0.754	0.6725
$P_y = \sigma_y\ A = 722$ Kips (Tensile Brace)		
Moment in horizontal braces (Kips-ft), $M = (0.122)\ (14/12)\ (L^2/10)$	17.43	28.81
$Mp = 4\ \sigma_y\ t\ R^2 = 275.6$ Kips-ft.		
M/ Mp	0.063	0.105
Corresponding to this, P/ Pcr : (Y)	0.96	0.933
$P_u = X\ Y\ P_y$	523 Kips	453 Kips.

<u>Vertical Diagonal Bracings:</u> The size of vertical diagonals is 20" x 0.438" for which, area (A) is 26.92 in^2, moment of inertia (I) is 1288.2 in^4, radius of gyration (r) is 6.92 in., and D/t ratio is 45.66.

Unsupported length of brace, L	61.03'	67.27'
Angle with horizontal, θ	55	48.01
$Kl/r = 0.8\,(L \times 12) / 6.92$	84.70	93.35
$\lambda = (1/\pi)\,(\sigma_y / E)^{0.5}\,(Kl/r)$	1.058	1.167
$P_u / P_y = 1.0 - 0.091\,\lambda - 0.22\,\lambda^2 = X$	0.6575	0.5942
$P_y = \sigma_y A$ = 1211.4 Kips (Brace 3 in Tension)		
Moment due to local wave loads,		
$M = (0.16)\,(14/12)\,(L^2/ 10)\,\mathrm{Sin}\,\theta$	48.8 Kips-ft.	
$Mp = 4\,\sigma_y\,t\,R^2$	657 Kips-ft.	
M/ Mp	0.074	
Corresponding to this, P/ Pcr : (Y)	0.95	
$P_u\,\mathrm{Cos}\,\theta = X\,Y\,P_y\,\mathrm{Cos}\,\theta$: Compression braces	434 Kips	457 Kips
Tensile braces (Pum)	660 Kips	770 Kips

<u>Jacket legs:</u> The size of the jacket leg is 45" x 0.625" for which, area (A) is 87.13 in^2, moment of inertia (I) is 21,450.7 in^4, radius of gyration (r) is 15.69 in., and D/t ratio is 72. The unsupported length of leg is 50.4'.

The pile section inside the leg is of 42" x 0.1.25" for which, area (A) is 160 in^2, moment of inertia (I) is 33,248 in^4. Therefore, for leg-pile annulus grouted case, the combined section properties be used: area (A) of 247.16 in^2, moment of inertia (I) of 54,698 in^4, and radius of gyration (r) of 14.88 in.

The D/t of the leg is more than 60, therefore the failure of leg will be in the plastic buckling range and the section would have negligible rotation capacity. But the leg-pile annulus is grouted and the local buckling of the leg will not occur.

The moment due to local wave loads, $M = (0.36)\,(L^2/ 10) = 90$ Kips-ft. and $Mp = 4\,\sigma_y\,t\,R^2 =$ 4,746 Kips-ft. Therefore, M/ Mp = 0.02, which is very low.

Two cases of grouted and ungrouted legs have been studies in this example.

		Ungrouted	Grouted
Unsupported length of brace,	$L = 50.4'$		
Angle with horizontal,	$\theta = 82.875$		
Area (A) in^2		87.13	247.16
Moment of Inertia (I) in^4		21,450.70	54,698.00
Radius of Gyration (r) in inch.		15.69	14.88
$Kl/r = 0.8 (L \times 12) / r$		38.55	40.65
$\lambda = (1/\pi) (\sigma_y / E)^{0.5} (Kl/r)$		0.482	0.508
$P_u / P_y = 1.0 - 0.091 \lambda - 0.22 \lambda^2 = X$		0.905	0.897
$P_y = \sigma_y A$ in Kips		3,920.9	11,122
Corresponding to M/Mp, P/ Pcr : (Y)		0.98	1.0
$P_u \cos\theta = X\ Y\ P_y \cos\theta$ in Kips		431	1,237
$P_y \cos\theta$ in Kips		476	1,380

The lateral load carried by the battered jacket legs, and vertical braces could be determined based on the relative stiffness contribution of the legs and vertical braces in a bay.

$$Ko = \Sigma EA/L \cos^2\phi + \Sigma 3EI/L^3$$

$$= 4EA_1/L_1 \cos^2 53.28 + 4EA_2/L_2 \cos^2 82.4 + 3EI_2/(L_2)^3$$

$$= E\ [4\{26.92/(61.03 \times 12)\} \cos^2 55 + 2\{26.92/(67.27 \times 12)\} \cos 48 +$$

$$4\{247.16/(50.4 \times 12)\} \cos^2 82.875 + 8\ \{3 \times 54{,}698/(12 \times 50.4)^3\}]$$

$$= E\ [4\ (0.012) + 2\ (0.0149) + 4\ (0.0063) + 8\ (0.00074)]$$

= 0.103 E for grouted leg case and 0.087 for ungrouted leg case.

Distribution Factors: DF

Vertical braces- outer = 0.012/ 0.103 = 0.117 (grouted) or 0.139 (ungrouted)

Vertical braces- inner = 0.0149/ 0.103 = 0.145 (grouted) or 0.172 (ungrouted)

Jacket legs = 0.0063/ 0.103 = 0.061 (grouted) or 0.025 (ungrouted)

Most Likely to Fail Member (MLTF):

Component	Pum	Pbay = [Ko/ Kom] Pum	
		Grouted	Ungrouted
Compression Brace	**434 Kips**	**3,709 Kips**	**3,122 Kips**
Tensile Brace-1	770 Kips	5,310 Kips	4,477 Kips
Tensile Brace-2	660 Kips	5,641 Kips	4,748 Kips
Jacket leg-	1,237 Kips	49,480 Kips	20,278 Kips

Note that in the above Table, P_{bay} represents the bay load at which that particular component will fail. Therefore, from this Table, the vertical compression brace is the most likely to fail first component. **The minimum strength of the bay in the load terms, at which the very first component of this bay will fail is 3,122 Kips in case the legs are ungrouted and 3,709 Kips in case the legs are grouted.**

In the similar way, the MLTF components for the other bays and the corresponding minimum bay strength (load) are determined. The results are summarized below. In case of the jacket bay- II to jacket bay-V, the most likely to fail member has been the vertical compression brace in each jacket bay.

Jacket Bay	Minimum Bay Strength (Kips)	
	Grouted Leg	Ungrouted Leg
Jacket bay-I	3,709	3,122
Jacket bay-II	3,598	3,349
Jacket bay-III	3,560	3,137
Jacket bay-IV	**3,191**	**2,775**
Jacket bay-V	5550	4,846

E. 2.2.2 Broad-side Direction:

In a similar way as done for the end-on direction, the minimum bay strength, which corresponds to the lateral load level for the bay at which the MLTF component fails, is evaluated for the broad-side load case. In the broadside wave load case, the four-number, 2-leg, parallel frames would resist the load and transfer it to the soil medium.

The results obtained by following the computations as described for jacket bay-I, end-on load case have been performed and the results are summarized below. In case of the jacket bay- I to jacket bay-V, the most likely to fail member has been the vertical compression brace in each jacket bay.

Jacket Bay	Minimum Bay Strength (Kips)	
	Grouted Leg	Ungrouted Leg
Jacket bay-I	5,263	4,065
Jacket bay-II	3,506	2,632
Jacket bay-III	3,661	2,950
Jacket bay-IV	4,907	3,890
Jacket bay-V	5,276	4,011

E. 2.3 Foundation Bay

The overall resistance against the lateral loads or base shear would be provided by the following elements of the jacket and its foundation:

i)	Piles-	8 No.	42 "Φ x 1"
ii)	Conductors	24 No.	20" Φ x 5/8"

a) Evaluation of ultimate resistance of soil, P_u (For Piles; 42 "Φ x 1"):

In soft clays: $P_u = 3c$ to $9c$ for $X = o$ to Xr

$$X_r = 6D / [(\gamma D / c) + 0.5]$$

$$= 6(3') / [(55 \times 3 / 1{,}600) + 0.5] \qquad = 29.84'$$

For $X = 0$ to X_r

$$P_u = 3c + \gamma X + [c / (2D)] X$$

$$= 3c + X[\gamma + c / (2D)]$$

For the extreme wave load condition, the pilehead displacements would be very large. Therefore, the soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation. This will not require to develop the complete p-y curves for a pile.

$P = 0.72\ [X / X_r]\ P_u$

$X_r = 29.84'$, $D = 3$ ft. $P_s = P\ D$

$P_u = 3c + X\ [\gamma + c / (2D)]$

X (ft)	c (Ksf)	γ (Ksf)	P_u Kips/ft	P_u=9c Kips/ft	P Kips/ft	P_s Kips
0	0.4	0.050	1.20	--	0.00	0.00
5	0.4	0.050	1.78	--	0.22	0.66
10	0.4	0.050	2.37	--	0.59	1.77
15	1.6	0.055	--	--	--	--
20	1.6	0.055	11.23	--	5.58	16.74
25	1.6	0.055	12.84	--	8.00	24.00
30	1.4	0.055	--	12.60	9.07	27.00
35	1.4	0.055	14.29	--	9.07	27.00

b) Evaluation of ultimate resistance of soil, pu

(For Conductors: 20 "Φ x 5/8"):

In soft clays: $P_u = 3c$ to $9c$ for X = 0 to Xr

$X_r = 6D / [(\gamma D / c) + 0.5]$

$= 6\,(20 / 12)\ /\ [(55 \times (20 / 12) / 1{,}600) + 0.5]$ $= 18.00'$

For $X = 0$ to X_r

$P_u = 3c + \gamma X + [c / (2D)]\ X$

$= 3c + X[\gamma + c / (2D)]$

For the extreme wave load condition, the pilehead displacements would be very large. The soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation and is given as follows:

$P = 0.72\ [X / X_r]\ P_u$

$X_r = 18.00'$, $D = 1.67$ ft. $P_s = p\ D$

$P_u = 3c + X\,[\gamma + c/(2D)]$

X (ft)	c (Ksf)	γ (Ksf)	P_u Kips/ft	$P_u = 9c$ Kips/ft	P Kips/ft	P_s Kips
0	0.4	0.050	1.20	--	0.00	0.00
10	0.4	0.050	2.90	--	1.16	1.94
18	1.6	0.055	14.44	--	10.37	17.31
25	1.6	0.055	14.40	--	14.40	24.05

c) Evaluation of Ultimate Resistance:

The equilibrium equation at the level of formation of plastic hinge at distance "L" from the pilehead could be formulated as below:

$$2M_p = R_u L - 0.5\,P_s\,(L-10)(L-10)/3 \qquad (1)$$

$$R_u = 0.5\,P_s\,(L-100) \qquad (2)$$

$$\text{or } L = 2R_u/P_s + 10$$

$$\text{or } L-10 = 2R_u/P_s$$

Substituting in equation (1), we get:

$$2M_p = Ru\,[2R_u/P_s + 10] - (1/6)\,Ps\,[2R_u/P_s]^2$$

$$= (2R_u^2/P_s)(1-2/6) + 10R_u$$

$$M_p = (2/3)\,(R_u^2/P_s) + 5R_u$$

$$\text{or } 2R_u^2 + 15P_sR_u - 3P_sM_p = 0$$

$$\text{or } R_u = 0.25\,[-15P_s + \{225P_s^2 + 24P_sM_p\}^{0.5}]$$

$$\boxed{R_u = 0.25\,[-15P_s + \{225P_s^2 + 24P_sM_p\}^{0.5}]}$$

a) Piles: 36 "Φ x 1"

Area, A = 110.0 in^2; Section Modulus, Z_p = 1,225 in^3

Plastic moment, M_p = 4,860 Kips-ft.

Base moment = 106,930 Kips

Thus axial load due to OTM = 106,930 / [2 (86)]

= 621 Kips for wave below deck.

The pile is laterally supported all along its length. Thus, there will not be any reduction in Mp due to axial load of 2,000 kips.

From last section, P_s = 24 Kips/ft.

R_u = 0.25 [-15 (24) + {225 (24)^2 + 24 (24) (4,595)$^{0.5}$] = 326.55 Kips

l = 2 R_u / P_s + 10 = 37.21 ft.

b) Conductors: 20 "Φ x 5/8"

Area, A = 110 in^2; Section Modulus, Z_p = in^3

Plastic moment, M_p = 844 Kips-ft.

From the last section, P_s = 18 Kips/ft.

R_u = 0.25 [-15 (18) ± {225 (18)^2 + 24 (18) (844)$^{0.5}$] = 98 Kips

l = 2 R_u / P_s + 10 = 20.9 ft.

c) Ultimate Lateral Load Capacity of the Foundation Bay:

R_u = 4 (326.55) + 1 (200) + 8 (98)

= 1,306.2 + 200 + 784 = 2,264 Kips

APPENDIX-F

DETAILED COMPUTATIONS FOR PLATFORM-B

The platform-B is a 4-legged platform located in 140 ft. water depth, and was designed and installed as per the practices followed during 1962. The main legs of the platform are sized to provide support to the 36"Φ piles. In addition, a 30"Φ pile/conductor is provided in the middle of jacket to provide sleeve for a well conductor. In addition, 8-conductors are provided near the middle part of the platform *[Bea et al, 1988]*. The age of the platform has been more than 25-years. The salient features of this platform are reproduced here:

Structural Characteristics:

- The leg/pile annulus is ungrouted
- No joint cans have been provided on the legs
- The platform is severely damaged with 4-members completely ineffective. However, in this study first it is assumed that the platform is undamaged.
- The 36"Φ piles penetrate 170' below the seabed.
- The deck floor is grated.

Production characteristics:

- The platform is operated as a tender drilling & production platform
- The platform is used for production of gas and 9-wells and 2-risers have been provided.
- The platform is unmanned, but provided with helideck. No living quarters have been provided on it.
- 2-boat landings and 4-barge bumpers have been fixed to it.

IMR Features:

- The remaining economic life of the platform is 12 years
- Anodes have been used as cathodic protection system

In this Appendix, the detailed computations for evaluation of RSR at the screening cycle-2 are presented, to demonstrate the extent of computations needed by the regulator or users. The results are summarized in Section 7.2 of this study. The configuration of platform is given in Fig. F.1.

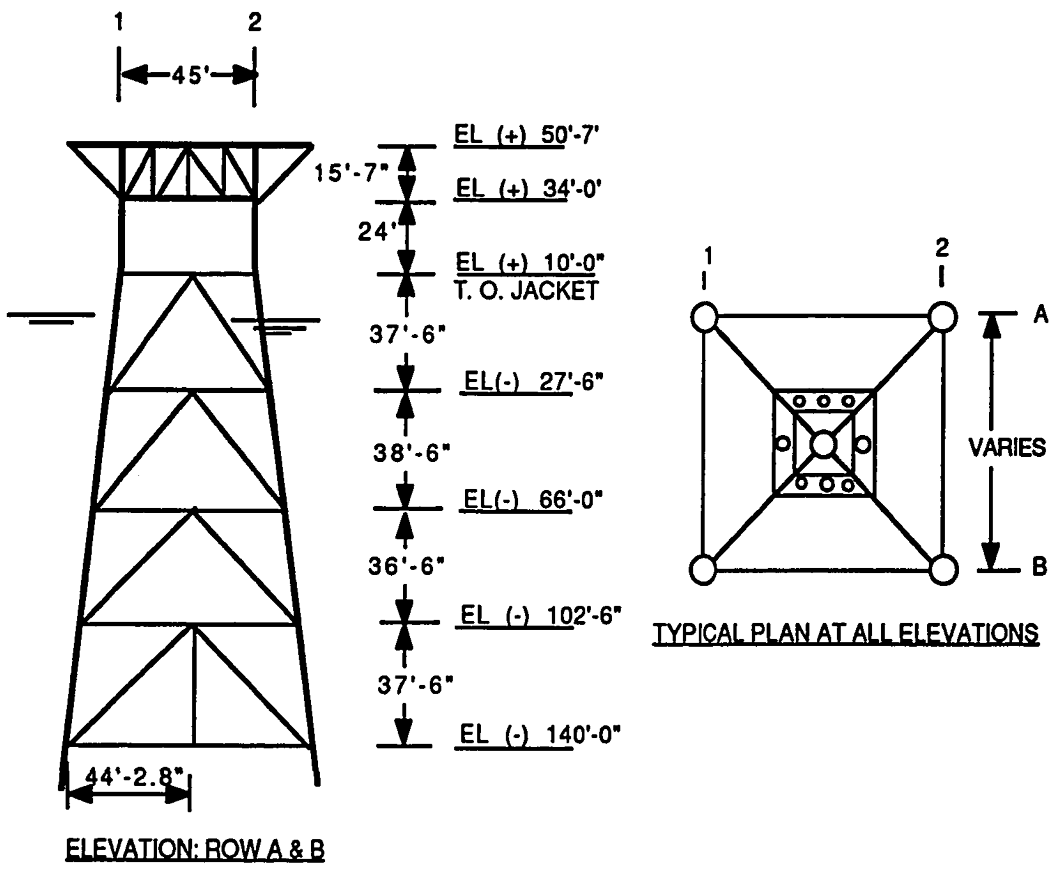


Figure F.1: Structural Configuration - Platform B

F.1 Developemnt of Lateral Load Pattern:

Water depth, d = 140 ft. Storm tide, s = 4.4 ft.

d + s = 144.4 ft. $H_{\text{design-reference}}$ = 65.7 ft.

Let S = 1/12, then Tm = 1.53 $(Hm)^{0.5}$ = 12.40 sec.

$L = [g T^2 / 2\pi] [\tanh (2\pi d / L)] = 788 [\tanh (907/L)]$ = 684.4 ft.

In the computations by Airy's theory, let us assume the wave crest height as 0.6 H. The wave crest elevation would be [s + 0.6 H = 4.4 + 0.6 (65.7)] ≈ 44 ft. The bottom of steel (B.O.S.) elevation of the lower deck is (+) 33'-1.5". Therefore, the wave will hit and inundate the lower deck by 11 ft.

$u_y = (\pi H/T) [\text{Cosh} [(2\pi y/L) d / (d + 0.6 H))] / [\text{Sinh} (2\pi d/L)]$

$= 16.65 \text{ Cosh} [(2\pi/ 684.4) \{145 / (145 + 39.4)\} y] / \text{Sinh} (2\pi\, 145 / 684.4)$

$= 16.65 \text{ Cosh} (0.00722\, y) / \text{Sinh} (1.331)$

$= 9.46 \text{ Cosh} (0.00722\, y)$

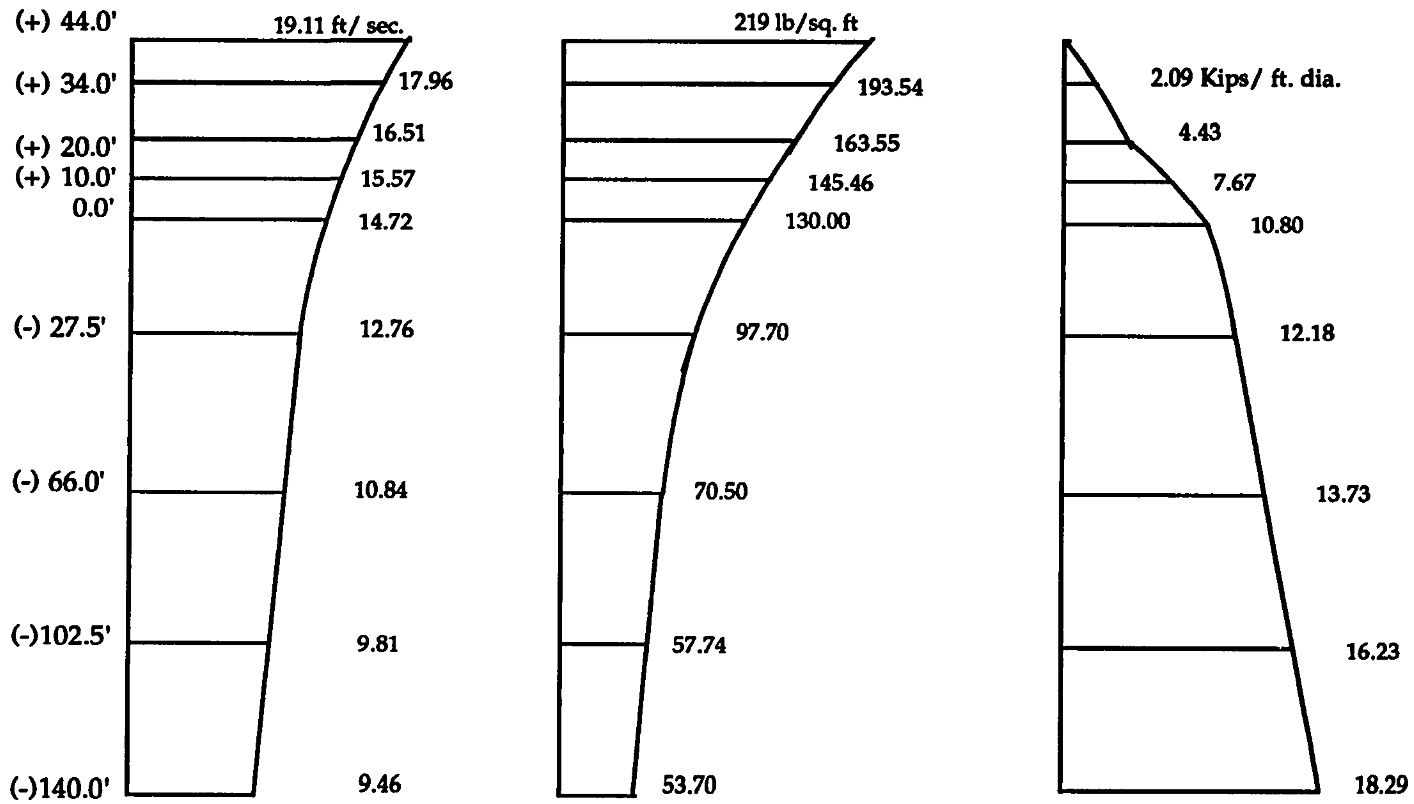
y (ft)	u_y (ft/sec)	F_D/unit area $=0.6 u\|u\|$ lb/ft^2	F_D on 1 ft. Φ pile Kips/ft
0.00	9.46	53.70	--
37.5	9.81	57.74	2.09
74.00	10.84	70.50	2.34
112.5	12.76	97.70	3.24
140.0	14.72	130.00	3.13
150.00	15.57	145.46	1.38
160.00	16.51	163.55	1.55
174.00	17.96	193.54	2.50
184.00	19.11	219.21	2.06
			Σ = 18.29 Kips/ft of Φ

The wave force on 1 ft Φ vertical pile extending from seabed to elevation above the wave crest is 18.29 Kips as shown in Fig. F.2.

I) Wave load on Deck Bay:

Wave load on lower deck: The load evaluation follows the formulas given in Section 2.3 of the main text. The projected area (A_p) of the lower deck, which is hit by the wave is:

A_p = (Deck width) (Deck height in wave) = (55') (11') = 605 sq. ft.



a: Horizontal Particle Velocity, u vs. depth

b: Drag Force/ unit area

c: Cumulative Load Pattern

Figure F.2: Wave Kinematics and Forces on a Vertical Unit Diameter Pile - PlatformB

Total velocity = wave velocity + current velocity = 18.54 + 3 = 21.54 ft/ sec. Note that 18.54 ft/ sec is the average wave velocity on the deck part.

Wave force on deck, $F = 0.5\,\rho\ C\ A_p\,(u_y)^2 = 0.5\,(0.064/32.3)\,(2)\,(605)\,(21.54)^2$

$= 556$ Kips.

<u>Wave load on deck bay:</u> The deck leg section has been assumed as 36"Φx 0.5". The deck legs elevation extend from (+)10', topmost jacket horizontal framing, to (+)33'-1.5", B.O.S. of the lower deck. The wave force on a unit diamter vertical member in a deck bay would be 4.05 Kips (= 2.50 Kips + 1.55 Kips) and it will act at 23.4' elevation.

Force on one deck leg of 36"Φ would be = 3 (4.05) = 12.15 Kips.

Vertical Members	Diameter	Number	Force in Kips
Deck legs	36"Φ	4	48.6 Kips
Central leg	33"Φ	1	11.1 Kips
Conductors	20"Φ	8	54 Kips
Risers	12"Φ	2	8.1 Kips
Total			121.8 Kips

Wind load on deck (assumed)	= 50 Kips.	
Wave load on lower deck	= 556 Kips	
Total wave load on deck bay	= 50 + 556 + 122	= 728 Kips.
Overturning moment @ El. (+) 10'	= (121.8) 23.4 + (556) 41 + (50) 65	
	≈ 29,000 Kips-ft.	

Topside load = 3,500 Kips (say)

If it is assumed that all the legs are equally loaded, the vertical load on the legs = [3,500 / 4] + [29,000 / (2 x 45)] = 875 + 321 = 1,196 Kips.

II) <u>Wave Load on Jacket Bays:</u>

The marine growth on members has been considered as 2.5" on diameter from m.s.l. to El. (-) 28', and it tapers off from El. (-) 28' to zero at the seabed.

a) <u>Jacket Bay-1: El. (+) 10' to El. (-) 27'-6":</u>

The wave force on a unit diameter vertical member in the jacket bay-1, is obtained from Fig. F.2 as 4.51 Kips (1.38 Kips from m.s.l. to El. (+)10', and 3.13 Kips from m.s.l. to El. (-) 27'-6"). The marine growth of 2.5" considered for the member parts from m.s.l. to El. (-) 27'-6".

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	39"Φ	4	61.24 Kips
Central leg	33"Φ	1	13.06 Kips
Conductors	20"Φ	8	65.40 Kips
Risers	12"Φ	2	10.30 Kips
Appurtenances:			
Caissons/Casings/Barge Bumpers			50.00 Kips
Boat landings		2	50.00 Kips
Total			250.00 Kips

<u>Wave force on vertical braces:</u> 4 no. 14"Φ x 0.375" braces have been provided on the two side frames and the two front frames. The formulas developed in Appendix-B have been utilized to evaluate the forces. In this bay, the vertical braces are at γ (= 36.73) from the horizontal.

Force on side frames = 4 (unit force) (Dia. in ft.) $\sin^2 \gamma$

≈ 4 (4.51) (16.5 / 12) $\sin^2$ 53.28 = **15.93 Kips.**

Force on front frames = 4 (unit force) (Dia. in ft.) (1/ $\sin \gamma$)

≈ 4 (4.51) (16.5 / 12) (1/ Sin 53.28) = **30.95 Kips.**

<u>Wave force on horizontal braces:</u> When wave approaches from the orthogonal directions, the effective horizontal braces would be 2-14"Φ, 2-12.75"Φ, and 2-9"Φ. The length of outer horizontal is 45'.

Force on horizontals = (0.146) (2) [(14 + 12.75 + 9) / 12] (45) = **39.2 Kips.**

Total wave load on Jacket Bay-1:

Vertical components	250.00 Kips
Vertical braces	46.88 Kips
Horizontal braces	39.20 Kips
Total force:	336.08 Kips

b) **Jacket Bay-2: El. (-) 27'-6" to El. (-) 66':**

The wave force on a unit diameter vertical member in the jacket bay-2, is obtained from Fig. F.2 as **3.24 Kips**. The average marine growth of 2.08" on diameter of the members in this bay has been considered.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	39"Φ	4	44.40Kips
Central leg	33"Φ	1	9.50 Kips
Conductors	20"Φ	8	47.70 Kips
Risers	12"Φ	2	7.60 Kips
Total			109.20 Kips

Wave force on vertical braces: 4 no. 16"Φ x 0.375" braces have been provided on the two side frames and the two front frames. The formulas developed in Appendix-B have been utilized to evaluate the forces. In this bay, the vertical braces are at γ (= 49.22) from the horizontal.

Force on side frames $= 4$ (unit force) (Dia. in ft.) $\sin^2 \gamma$

$\approx 4\,(3.24)\,(18.08 / 12)\,\sin^2 49.22$ $=$ **11.20 Kips.**

Force on front frames $= 4$ (unit force) (Dia. in ft.) $(1/\sin \gamma)$

$\approx 4\,(3.24)\,(18.08 / 12)\,(1/\sin 49.22)$ $=$ **25.80 Kips.**

Wave force on horizontal braces: When wave approaches from the orthogonal directions, the effective horizontal braces would be 2-16"Φ, 2-12.75"Φ, and 2-9"Φ. The length of outer horizontal is 56'.

Force on horizontals $= (0.098)\,(2)\,[(18.08 + 14.83 + 11.08) / 12]\,(56') =$ **40.20 Kips.**

Total wave load on Jacket Bay-2:

Vertical members	109.20 Kips
Vertical braces	37.00 Kips
Horizontal braces	40.20 Kips
Total force:	186.40 Kips

c) <u>Jacket Bay-3: El. (-) 66' to El. (-) 102'-6":</u>

The wave force on a unit diameter vertical member in the jacket bay-3, is obtained from Fig. F.2 as **2.34 Kips**. The average marine growth of 1.25" has been considered for this bay.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	39"Φ	4	31.40 Kips
Central leg	33"Φ	1	6.70 Kips
Conductors	20"Φ	8	33.20 Kips
Risers	12"Φ	2	5.20 Kips
Total			76.5 Kips

<u>Wave force on vertical braces:</u> 4 no. 16"Φ x 0.375" braces have been provided on the two side frames and the two front frames. The formulas developed in Appendix-B have been utilized to evaluate the forces. In this bay, the vertical braces are at γ (= 43.76) from the horizontal.

Force on side frames = 4 (unit force) (Dia. in ft.) $\sin^2 \gamma$

$\approx$ 4 (2.34) (17.25 / 12) $\sin^2 43.76$ = **6.44 Kips.**

Force on front frames = 4 (unit force) (Dia. in ft.) (1/ $\sin \gamma$)

$\approx$ 4 (2.34) (17.25 / 12) (1/ $\sin 43.76$) = **19.45 Kips.**

<u>Wave force on horizontal braces:</u> When wave approaches from the orthogonal directions, the effective horizontal braces would be 2-18"Φ, 2-12.75"Φ, and 2-9"Φ. The length of outer horizontal is 66'.

Force on horizontals = (0.0705) (2) [(19.7 + 14.45 + 10.7) / 12] (66) = **34.78 Kips.**

<u>Total wave load on Jacket Bay-3:</u>

Vertical members	76.50 Kips
Vertical braces	25.89 Kips
Horizontal braces	34.78 Kips
Total force	137.17 Kips

d) <u>Jacket Bay-4: El. (-) 102'-6" to El. (-) 140':</u>

The wave force on a unit diameter vertical member in the jacket bay-4, is obtained from Fig. F.2 as 2.09 Kips. The average marine growth of 0.42" has been considered for this bay.

Vertical Members	Diameter	Number	Force in Kips
Jacket legs	39"Φ	4	27.50Kips
Central leg	33"Φ	1	5.80 Kips
Conductors	20"Φ	8	28.50 Kips
Risers	12"Φ	2	4.30 Kips
Total			66.10 Kips

<u>Wave force on vertical braces:</u> 4 no. 18"Φ x 0.375" braces have been provided on the two side frames and the two front frames. The formulas developed in Appendix-B have been utilized to evaluate the forces. In this bay, the vertical braces are at γ (= 40.98) from the horizontal.

Force on side frames = 4 (unit force) (Dia. in ft.) $\sin^2 \gamma$

≈ 4 (2.09) (18.42 / 12) $\sin^2 40.98$ = **5.53 Kips.**

Force on front frames = 4 (unit force) (Dia. in ft.) (1/ $\sin \gamma$)

≈ 4 (2.09) (18.42 / 12) (1/ $\sin 40.98$) = **19.57 Kips.**

<u>Wave force on horizontal braces:</u> When wave approaches from the orthogonal directions, the effective horizontal braces would be 2-20"Φ, 2-12.75"Φ, and 2-9"Φ. The length of outer horizontal is 76'.

Force on horizontals = (0.058) (2) [(20.84 + 13.59 + 9.84) / 12] (76) = **32.50 Kips.**

<u>Total wave load on Jacket Bay-4:</u>

Vertical components	66.10 Kips
Vertical braces	25.10 Kips
Horizontal braces	32.50 Kips
Total force:	123.70 Kips

e) **Summary of Lateral Loads:**

i) Summary Based on Bay Loads:

Wind loads		50 Kips
Wave Loads:		
Deck bay	678.00 Kips	
Jacket bay-1	336.08 Kips	
Jacket bay-2	186.40 Kips	
Jacket bay-3	137.17 Kips	
Jacket bay-4	123.70 Kips	
Mud Level Horizontal Framing	33.00 Kips	
Total wave load		Σ = 1,494 Kips
Total Lateral Load		1,544 Kips

ii) Summary Based on Component Loads:

Wind loads		50 Kips
Wave Loads:		
Lower deck	556 Kips	
Main legs (4 No.)	213 Kips	
Central leg	46 Kips	
Conductors (8-No.)	229 Kips	
Risers (2-No.)	35 Kips	
Vertical braces (4-Faces)	137 Kips	
Horizontal braces	178 Kips	
Appurtenances	100 Kips	
Total wave load		Σ = 1,494 Kips
Total Lateral Load		1,544 Kips

Total overturning moment at seabed $\approx$ 50 (140 + 65) + 556 (140 + 40) + 938 (140) = **241,650 Kips-ft** for base shear of 1,544 Kips.

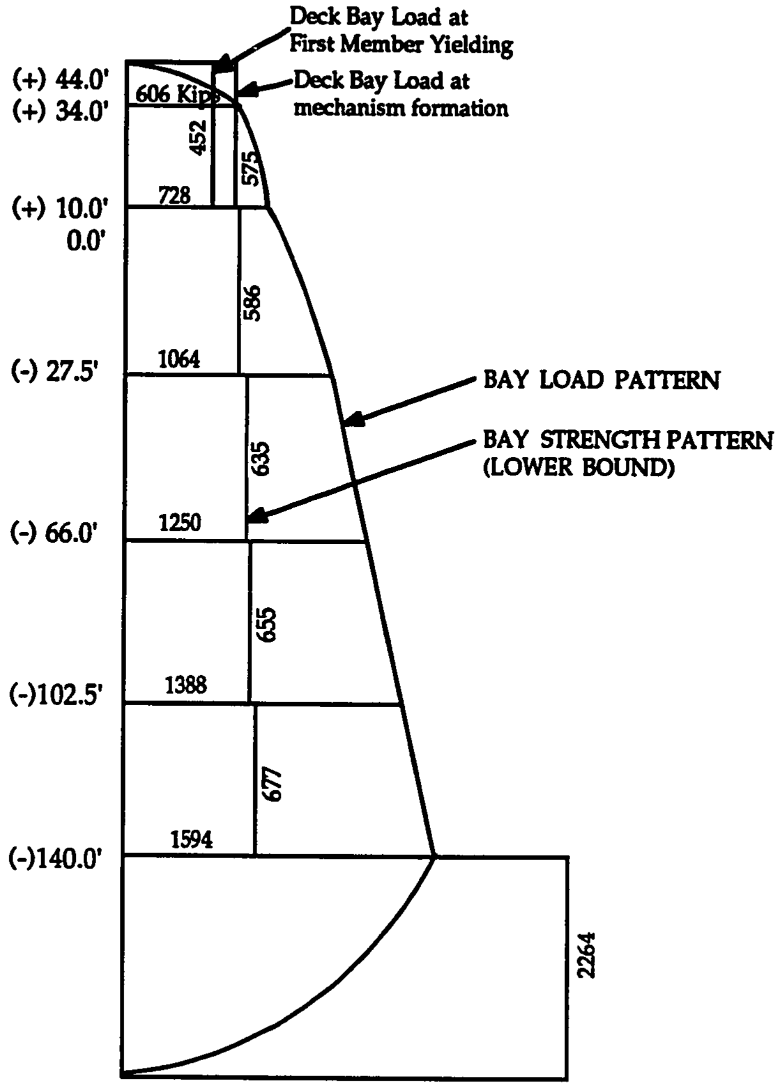


Figure F.3: Load and Strength Patterns - Platform B

F.2 Development of Strength Pattern:

The strength pattern of the platform is developed by considering the various failure modes, which are likely in the different bays of the platform. For all of these failure modes and mechanisms, the lower bound of structural strength or the lateral load level at which the first member in a bay would fail is determined. Then based on this, RSR of the platform at failure of the first component (lower bound) is evaluated.

The major types of failure modes and failure mechanisms for the dominant lateral loads considered in this example platform are as follows:

1. Deck leg mechanism
2. Jacket leg-brace mechanisms (for each bay)
3. Pile yield mechanism

The other important failure modes are as given below:

4. Piie capacity in axial tension
5. Pile capacity in axial compression
6. Strength of deck leg-pile top connection
7. Punching strength of major joints in splash zone and at seabed

I) Deck Bay: It is assumed that a plastic mechanism would form in the deck bay by yielding of the sections at the top and bottom of the four main legs. The yielding of the central leg is not considered as in the deck bay, the central leg would not exist beyond El. (+) 17' and only the smaller diameter conductor would pass.

The size of main legs has been assumed as 36" Φ x 0.5" for which: Area, A = 55.76 in^2; Moment of Inertia, I = 8,786 in^4; Radius of gyration, r = 12.55 in.

The unsupported length of the deck leg, l = 23'.

$Kl/r = 22$ and $\lambda = (1/\pi)\ (\sigma_y / E)^{0.5}\ (Kl/r) = 0.271$

For $\lambda = 0.271$, $P_u / P_y = 0.96$, where $P_y = \sigma_y A = 2{,}509$ Kips

So, $P_u = 2{,}409$ Kips

$Mp = 4\ \sigma_y\ t\ R^2 = 2{,}430$ Kips-ft. and the moment at first yield of a section would be, $My = \pi\ \sigma_y\ t\ R^2 = 1{,}909$ Kips-ft.

The allowable momemt (M) would depend upon the ratio of (P/P_y) for the deck legs. In this example, the actual distribution of the vertical loads on the legs is not known. An approximation estimate has been made here by considering that all the legs are equally loaded.

Total topside D.L. and L.L.	= 3,500 Kips (say)	
Wind load	= 50 Kips @ El. (+) 65'	
Wave load on lower deck	= 412 Kips @ El (+) 41	
Wave load on deck legs	= 122 Kips @ El (+) 26'	
Overturning moment (OTM)	= 50 x 65 + 412 x 41 + 122 x 23.4	= 23,000 Kips
Compressive load in leg due to OTM	= 23,000 / (2 x 45)	= 255 Kips
Total compressive load in leg	= 3,500 / 4 + 255	= 1130 Kips

The ratio of (P/P_y) is equal to **0.47** (1130 / 2409) and the corresponding ratio of (M/M_P) would be (0.74), by using the following equation .

$M/ Mcr = \text{Cos}\ [0.5\ \pi P/ Pcr] \qquad = \text{Cos}\ [0.5\ \pi\ (0.47)] \qquad = 0.74$

For $D/t = 72$, the ratio of $Mcr/ Mp = 0.92$.

Therefore, $M = (0.92)\ (0.74)\ Mp \qquad = 0.68\ Mp$

$\qquad = 0.68\ (2{,}430) \qquad = 1{,}654$ Kips.

or $M = 0.68\ (1{,}909) = 1{,}300$ Kips-ft. at the first yielding of any section. The ultimate moment carrying capability of these sections would vary for different legs, and an average value could be taken to obtain approximate (nominal) estimate.

Rigid Plastic Mechanism:

The ultimate lateral load capacity upto yielding of first section in any deck leg, $P_u = 8\,M_P / L = 8\,(1{,}300) / 23 = 452$ Kips at first yield, or

$= 8\,(1{,}654) / 23 = 575$ Kips at formation of a mechanism.

At large deflections at upper end of the deck leg bay, the lateral stiffness of the bay would reduce due to the additional moment produced by the vertical loads on the legs. The reduction in stiffness of the bay due to this additional moment can be approximately determined as the ratio of the vertical load to the bay height.

The stiffness reduction would be $= 1{,}130 / 33' \approx 34$ Kips/ft of deformation. This value is very less compared to the ultimate load capacity determined earlier.

II) Jacket Bays:

a) Jacket Bay-1: Elev (+) 10' to Elev. (-) 27.5':

The size of horizontal and vertical braces in this bay is 14" Φ x 0.375", for which the Area (A) = 16.052 in^2, Moment of Inertia (I) = 372.76 in^4, Radius of gyration (r) = 4.82 in.

Horizontal braces 1&2:

Unsupported length of horizontal braces, $l = 22.92'$

$Kl/r = 0.8\,(22.92 \times 12) / 4.82 = 45.81$

$\lambda = (1/\pi)\,(\sigma_y / E)^{0.5}\,(Kl/r) = 0.565$

$P_u / P_y = 1.0 - 0.091\,\lambda - 0.22\,\lambda^2 = 0.878$

$P_y = \sigma_y\,A = 722$ Kips (Brace 2 in Tension)

$P_u = 0.878\,P_y = 634$ Kips (Brace 1 in Compression)

Vertical diagonal braces 3&4:

Unsupported length of brace, $l = 46.8'$

$Kl/r = 0.8\,(46.8 \times 12) / 4.82 = 93.6$

$\lambda = (1/\pi)\ (\sigma_y / E)^{0.5}\ (Kl/r) = 1.154$

$P_u / P_y = 1.0 - 0.091\,\lambda - 0.22\,\lambda^2 = 0.602$

$P_y = \sigma_y A = 722.34$ Kips (Brace 3 in Tension)

$P_u = 0.602\,P_y = 435$ Kips (Brace 1 in Compression)

Horizontal component of brace capacity:

Brace 3: $P_y = 722\,\text{Cos}^2\,37^o = \mathbf{461}$ Kips

Brace 4: $P_u = 435\,\text{Cos}^2\,37^o = \mathbf{277}$ Kips

Ultimate Strength of Leg-Brace Joint:

Ultimate load in brace, $P_u = Q_u\ Q_f\ Fy\ T^2 / \text{Sin}\,\theta$

$Q_u = 3.4 + 19\,\beta = 10.22$; where $\beta = d/D = 14/39 = 0.359$

$Q_f = 1 - \lambda\ \gamma\ A^2 = 1$ (say); where $\lambda = 0.03$

$P_u = (10.22)\,(1)\,(45)\,(0.5)^2 / \text{Sin}\,82.4^o = \mathbf{116}$ Kips

The capacity of the leg-brace joint is the lowest and it would be the first to fail against the increasing level of the lateral load. Therefore, the maximum load level in the vertical and horizontal braces would be limited due to the lower joint strength. The loads in the vertical braces and legs just before failure of the joint would represent the component strength (CS). This would represent the lower bound value of RSR, which is the reserve strength ratio at failure of the first component.

Case-1: Before failure of the leg-brace joints.

The horizontal braces which are connected to the jacket legs would transfer loads from the legs to the vertical braces. The vertical components of the loads at the joint at middle of the outer horizontal should balance for equillibrium. Due to symmetrical configuration and loading, the axial loads in the vertical braces within a bay are likely to be equal.

The lateral load capacity of the vertical braces just before failure of a joint could be approximated as $\approx 4\,(116) = \mathbf{464}$ **Kips.**

In addition, the battered legs would also contribute to the lateral load carrying capacity in addition to the capacity of the vertical braces. The leg-pile annulus is ungrouted, thus we can neglect the effect of pile in evaluation of the contribution of legs to the lateral load carrying capacity of the bay.

The size of jacket leg is 39" Φ x 0.50", and it is 38.2' long. Its Area (A) = 60.48 in^2; Moment of Inertia (I) = 11,207 in^4; Radius of gyration (r) = 13.61 in.

M_p = (2,852 Kips-ft.) (Axial Load Factor) = 0.7 (2,852) = **1,996 Kips-ft.**

The jacket legs carry significant axial load from the topside loads, and due to the vertical load component from the overturning moment of the waves.

Before failure of the first joint, the horizontal brace would carry the load effectively equal to the punching capacity of joint. In this example, it will be 116 Kips. The lateral load carried by the battered jacket legs could be determined based on the relative stiffness contribution of the legs and braces at the node A.

$$
\begin{aligned}
Ko &= \Sigma EA/L \, Cos^2 \phi + \Sigma 3EI/L^3 \\
&= 4EA_1/L_1 \, Cos^2 53.28 + 4EA_2/L_2 \, Cos^2 82.4 + 3EI_2/(L_2)^3 \\
&= E\,[4\{16.052/(46.8 \times 12)\}\, Cos^2 53.28 + 4\{60.48/(38.2 \times 12)\}\, Cos^2 82.4 \\
&\quad + 4\,\{3 \times 11{,}207/(12 \times 38.2)^3\} + \{3 \times 6742/(12 \times 38.2)^3\}] \\
&= E\,[4\,(0.01022) + 4\,(0.00231) + 4\,(0.00035) + 0.0002] \\
&= 0.0517\,E
\end{aligned}
$$

DF brace = 0.198 for ungrouted legs and 0.183 for the grouted legs.

Pum ≈ 116 Kips for the ungrouted legs.

Pbay = 586 Kips.

EA/L of leg	= 0.133 E and
EA/L of horizontal brace	= 0.058 E

Horizontal component of axial stiffness of leg = 0.133 E Cos 82.4°

= 0.0176 E

Relative stiffness of leg = 0.0176 / (0.0176 + 0.058) = 0.233

Relative stiffness of brace = 1 - 0.233 = 0.767

Thus lateral load carried by leg = 116 [0.233 / 0.767] = 34 Kips

i.e., Axial load in leg corresponding to this lateral load = 34 / Cos 82.4°

= 257 Kips

The ultimate lateral load capacity of legs could be determined as follows:

$Kl/r = 35.26$ $\lambda = 0.435$

$P_u/P_y = 0.919$ $P_y = \sigma y\ A = 2{,}721$ Kips

$P_u = 2{,}501$ Kips

$P / P_u = 801 / 2{,}501 = 0.32$

Thus before failure of first member, the jacket will be moderately loaded. The lateral load on the jacket could be approximated as follows:

Lateral load on Jacket = 4 (116) + 4 (34) = **600 Kips.**

<u>Case-2: Upon failure of joint(s):</u>

Upon buckling failure of a compression member, it will shed compression load on the member at buckling with increase in deflection. At large deflection, the buckled member could only carry about 0.10 to 0.20 of the buckling capacity of the member.

At the stage of formation of a mechanism, the deflections would be very high. Thus the buckled braces would carry about 35 to 70 Kips of the ultimate lateral load on the bay. Then the adjacent compression members would attract some of the load shed by the compression member depending on the ultimate capacity of the tension members.

Thus, $P / P_u = 1{,}050 / 2{,}501 = 0.42$

The ultimate capacity of the bay would also have some contribution due to the plastic hinge formations in the legs due to large deflections of the bay. At such a stage, the ratio of P / Pu in leg = 0.42.

Thus M/M_p would be limited to 0.70.

Mp = 2,852 Kips-ft.

Therefore, M = 0.70 (2,852) = 1,996 Kips-ft., which would correspond to lateral load of $(2 M_p / L) = 2 (1,996) / 38 = 105$ Kips.

Thus, lateral load capacity provided by legs = 4 (105) = **420 Kips.**

<u>Case-3: Upon failure of intermediate bracings:</u>

Upon failure of the intermediate bracings, the two bays would act as a portal frame, in the extreme case. The leg has an interior pile of 36 "Φ x 1". The hinges would first form in the interior piles just under the deck.

Mp = 3,888 Kips

Thus, the ultimate load level at (-) 27.5' Elevation would be about 506 Kips.

b) <u>Bay 3: Elev (-) 27.5' to (-) 66':</u>

The sizes of the horizontal and vertical braces is 16"Φ x 0.375", for which A = 18.408 sq. in.; I = 562.08 in^4

<u>Joint strength:</u>

Ultimate load in brace, $P_u = Q_u \, Q_f \, Fy \, T^2 / \sin\theta$

Q_u = 3.4 + 19 β = 11.19, where β = d /D = 16/39 = 0.410

$Q_f = 1 - \lambda \gamma A^2 = 1$ (say), where λ = 0.03

$P_u = (11.19) (1) (45) (0.5)^2 / \sin 82.4 = 127$ Kips

Load carried by leg = (0.24 / 0.76) 127 = 40 Kips

Total lateral load just before joint failure = 4 [127 + 40] = **668 Kips.**

c) **<u>Bay 4: Elev (-) 66' to (-) 102.5':</u>**

The sizes of the horizontaland vertical braces is 18"Φ x 0.375", for which area, A = 20.764 sq. in.; I = 806.63 in^4

<u>Joint strength:</u>

Ultimate load in brace, $P_u = Q_u\ Q_f\ Fy\ T^2 / \sin\theta$

Q_u = 3.4 + 19 β = 12.17, where β = d /D = 18/39 = 0.462

Q_f = $1 - \lambda\ \gamma\ A^2$ = 1 (say), where λ = 0.03

P_u = $(12.17)(1)(45)(0.5)^2 / \sin 82.4$ = 138 Kips

Load carried by leg = (0.24 / 0.76) 138 = 49 Kips

Total lateral load just before joint failure = 4 [138 + 49] = **748 Kips.**

d) **<u>Bay 5: Elev (-) 102.5' to (-) 140':</u>**

The size of the Horizontal and vertical braces is 20"Φ x 0.375", for which A = 23.12 sq. in.; I = 562.08 in^4

<u>Joint strength:</u>

Ultimate load in brace, $P_u = Q_u\ Q_f\ Fy\ T^2 / \sin\theta$

Q_u = 3.4 + 19 β = 13.14, where β = d /D = 20/39 = 0.513

Q_f = $1 - \lambda\ \gamma\ A^2$ = 1 (say), where λ = 0.03

P_u = $(13.14)(1)(45)(0.5)^2 / \sin 82.4$ = 149 Kips

Load carried by leg = (0.24 / 0.76) 149 = 52.3 Kips

Total lateral load just before joint failure = 4 [149 + 52] = **804 Kips.**

3) <u>PILE - YIELD MECHANISM:</u>

The overall resistance against the lateral load or base shear would be provided by the following elements of jacket and its foundation:

i)	Piles-	4 No.	36 "Φ x 1"
ii)	Central conductor	1 No.	30" Φ x 5/8"
iii)	Conductors	8 No.	20" Φ x 5/8"

a) **Evaluation of ultimate resistance of soil, pu (For Piles: 36 "Φ x 1"):**

In soft clays: $P_u = 3c$ to $9c$ for X = o to Xr

$X_r = 6D / [(\gamma D / c) + 0.5]$

$= 6 (3') / [(55 \times 3 / 1{,}600) + 0.5] = \mathbf{29.84'}$

For $X = 0$ to X_r

$P_u = 3c + \gamma X + [c / (2D)] X$

$= 3c + X[\gamma + c / (2D)]$

For the extreme wave load condition, the pilehead displacements would be very large. Thus the soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation. This would not need development of complete p-y curves for a pile.

Thus $P = 0.72 [X / X_r] P_u$

$X_r = 29.84'$, $D = 3$ ft. $P_s = P D$

$P_u = 3c + X [\gamma + c / (2D)]$

X (ft)	c (Ksf)	γ (Ksf)	P_u Kips/ft	$P_u = 9c$ Kips/ft	P Kips/ft	P_s Kips
0	0.4	0.050	1.20	–	0.00	0.00
5	0.4	0.050	1.78	–	0.22	0.66
10	0.4	0.050	2.37	–	0.59	1.77
15	1.6	0.055	–	–	–	–
20	1.6	0.055	11.23	–	5.58	16.74
25	1.6	0.055	12.84	–	8.00	24.00
30	1.4	0.055	–	12.60	9.07	27.00
35	1.4	0.055	14.29	–	9.07	27.00

b) **Evaluation of ultimate resistance of soil, pu**

(For Central Pile 30 "Φ x 5/8"):

Mp = 2,109 Kips-ft.

$X_r = 6D / [(\gamma D / c) + 0.5]$

$= 6 (2.5') / [(55 \times 2.5 / 1{,}600) + 0.5] = \mathbf{25.60'}$

For X = 0 to Xr

$P_u \quad = 3c + \gamma X + [c/(2D)]\,X$

$\quad\quad = 3c + X[\gamma + c/(2D)]$

For the extreme wave load condition, the pilehead displacements would be very large. Thus the soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation. This would not need development of complete p-y curves for a pile.

Thus $p = 0.72\,[X/Xr]\,P_u$

$X_r = 25.60'$, $\quad D = 2.5$ ft. $\quad P_s = p\,D$

$P_u = 3c + X\,[\gamma + c/(2D)]$

X (ft)	c (Ksf)	γ (Ksf)	P_u Kips/ft	$P_u = 9c$ Kips/ft	P Kips/ft	P_s Kips
0						
10						
15						
20						
25	1.6	0.055	14.18	14.40	9.97	24.90

c) **Evaluation of ultimate resistance of soil, pu**

(for Conductors: 20 "Φ x 5/8"):

In soft clays: $P_u = 3c$ to $9c$ $\quad$ for $X = 0$ to Xr

$X_r \quad = 6D / [(\gamma D/c) + 0.5]$

$\quad\quad = 6\,(20/12)\;/\;[\{55 \times (20/12)/1{,}600\} + 0.5] = \mathbf{18.00'}$

For $X = 0$ to X_r

$P_u \quad = 3c + \gamma X + [c/(2D)]\,X = 3c + X[\gamma + c/(2D)]$

For the extreme wave load condition, the pilehead displacements would be very large. Thus the soil resistance developed at very large deflections (as in p-y curves) could be considered as a good approximation. This would not need development of complete p-y curves for a pile.

Thus $P = 0.72\,[X/X_r]\,P_u$

$P_u = 3c + X[\gamma + c/(2D)]$

X (ft)	c (Ksf)	γ (Ksf)	P_u Kips/ft	$P_u = 9c$ Kips/ft	P Kips/ft	P_s Kips
0	0.4	0.050	1.20	–	0.00	0.00
10	0.4	0.050	2.90	–	1.16	1.94
18	1.6	0.055	14.44	–	10.37	17.31
25	1.6	0.055	14.40	–	14.40	24.05

d) Evaluation of Ultimate Resistance:

The equilibrium equation at the level of hinge formation at distance "L" below seabed could be formulated as given below:

$2M_p = R_u L - 0.5\ P_s (L-10)(L-10)/3$ ....1)

$R_u = 0.5\ P_s (L - 100)$ ....(2)

or $L = 2R_u / P_s + 10$

or $L - 10 = 2R_u / P_s$

Substituting in equation (1), we get:

$2M_p = Ru[2R_u / P_s + 10] - (1/6)\ Ps[2R_u / P_s]^2$

$= (2R_u^2 / P_s)(1 - 2/6) + 10R_u$

$M_p = (2/3)\ (R_u^2 / P_s) + 5R_u$

or $2R_u^2 + 15P_s R_u - 3P_s M_p = 0$

or $R_u = 0.25[-15P_s + \{225P_s^2 + 24P_s M_p\}^{0.5}]$

$\boxed{R_u = 0.25[-15P_s + \{225P_s^2 + 24P_s M_p\}^{0.5}]}$

i) Piles: 36 "Φ x 1"

Area, A = $110.0\ in^2$

Section Modulus, Z_p = $1{,}225\ in^3$

Plastic moment, M_p = 4,860 Kips-ft.

Base moment = 106,930 Kips

Thus axial load due to OTM = 106,930 / [2 (86)] = 621 Kips for wave below deck.

Axial load, when wave hit the deck = [106,930 + 230 x 180]/ [2 (86)] = 862 Kips

The pile is laterally supported all along its length. Thus, there will not be any reduction in Mp due to axial load of 2,000 kips.

From last section, P_s = 24 Kips/ft.

R_u = 0.25 [-15 (24) + { 225 (24)^2 + 24 (24) (4,595}$^{0.5}$] = **326.55 Kips**

l = 2 R_u / P_s + 10 = 37.21 ft.

ii) __Central Conductor: 30 "Φ x 5/8"__

Area, A = 57.68 in^2

Section Modulus, Z_p = 414.93 in^3

Plastic moment, M_p = 2,109 Kips-ft.

From last section, P_s = 24 Kips/ft.

R_u = 0.25 [-15 (24) + { 225 (24)^2 + 24 (24) (2,109}$^{0.5}$] = **200 Kips**

l = 2 R_u / P_s + 10 = 26 ft.

iii) __Conductors: 20 "Φ x 5/8"__

Area, A = 110 in^2

Section Modulus, Z_p = 178.7 in^3

Plastic moment, M_p = 844 Kips-ft.

From the last section, P_s = 18 Kips/ft.

R_u = 0.25 [-15 (18) ± { 225 (18)^2 + 24 (18) (844}$^{0.5}$] = **98 Kips**

l = 2 R_u / P_s + 10 = 20.9 ft.

iv) __Ultimate Lateral Load Capacity of the Foundation Bay:__

Ru = 4 (326.55) + 1 (200) + 8 (98)

= 1,306.2 + 200 + 784 = **2,264 Kips**

BIBLIOGRAPHY

AGGARWAL, R.K., BEA, R.G., GERWICK, B.C., IBBS, C.W., REIMER, R.B., and LEE, G.C. 1990. Development of a Methodology for Safety Assessment of Existing Steel Jacket Offshore Platform. *Proceedings, Offshore technology Conference.* OTC No. 6385. 351-362.

AMERICAN PETROLEUM INSTITUTE. 1991. *Recommended Practice for Planning, Designing, and Constructing Fixed Offshore platforms, API-RP-2A, Nineteenth edition,* Washington D.C. : American Petroleum Institute.

AMERICAN PETROLEUM INSTITUTE. 1989. *Draft Recommended Practice for Planning, Designing, and Constructing Fixed Offshore platforms - Load and Resistance Factor Design API-RP-2A-LRFD, First edition,* Washington D.C. : American Petroleum Institute.

ANG, A. H-S., and TANG, W. H. 1984. *Probability Concepts in Engineering Planning and Design Vol II: Decision, Risk, and Reliability.* New York: John Wiley and Sons.

ANTES, R., and SAUNDERS, J. Marathon Oil Company. Interview with R. K. Aggarwal. Houston, 7 October 1988.

ARNOLD, K.E., KOSZELA, P.J., and VILES, J.C. 1989. Improved safety of Production Operations in the U.S. OCS. *Proceedings, 21st Offshore Technology Conference.* OTC 6079. 349-358.

ASTEO. 1987. *Risk Analysis for Offshore Structures and equipment,* London: Graham & Trotman. The Commission of the European Communities.

BEA, R.G. 1990-a. *Design Criteria for Marine Structures,* Lecture Notes Course: NA 290C. Department of Naval Architecture and Ocean Engineering, University of California, Berkeley.

BEA, R.G. 1990-b. Offshore Platform Reliability Acceptance Criteria. *Session-70: Maintenance and Qualification of Offshore Platforms* . Structural Congress'90. Baltimore, Maryland.

BEA, R.G., and AUDIBERT, J.M.E. 1980. Offshore Platforms and Pipelines in Mississippi River Delta. *Journal of the Geotechnical Division, American Society of Civil Engineers.* August 1980: 853-869.

BEA, R.G., HONG, S.T., and MITCHELL, J.S. 1984. Decision Analysis Approach to Offshore Platform Design. *Journal of Structural Engineering, American Society of Civil Engineers.* January 1984.

BEA, R.G., and LAI, N. W. 1978. Hydrodynamic Loadings on Offshore Platforms. *Proceedings, 10th Annual Offshore Technology Conference.* OTC No. 3064.

BEA, R. G., and NIEDORADO, A. W. 1983. Gulf of Mexico Shallow Water Wave Heights and Forces. *Proceedings, 15th Offshore Technology Conference.* OTC No. 4586.

BEA, R.G., PUSKAR, F.J., SMITH, C., and SPENCER, J. 1988. Development of AIM (Assessment , Inspection, Maintenance) Programs for Fixed and Mobile Platforms. *Proceedings, 20th Offshore Technology Conference.* OTC No. 5703.

BEA, R.G., and SMITH, C.S. 1987. AIM (Assessment, Inspection, Maintenance) and Reliability of Offshore Platforms. *Marine Structural Reliability Symposium, Virginia.* 57-75.

CHAKRABARTI, S.K. 1987. *Hydrodynamics of Offshore Structures.* CML Publications, Great Britain.

CHEN, W.F., and HAN, D.J. 1985. *Tubular Members in Offshore Structures,* Massachusetts: Pitman.

COOPER, G. W. 1967. Hurricane Damage to Structures. *Ocean Industry.* October: 30-34.

DE, R. 1990. *Offshore Structural System Reliability: Wave-Load Modelling, System Behavior, and Analysis.* Ph.D. Dissertation. University of California, Berkeley.

DEAN, R.G. 1974. *Evaluation and Development of Water Wave Theories for Engineering Applications, Volumes I and II.* Special Report No. 1. Fort Belvoir, VA: Coastal Engineering Research Center.

DIGRE, K. Senior Staff Civil Engineer, Shell Oil Company. Interview with R. K. Aggarwal. Houston, 7 October 1988.

DODSON, J.K. 1987. U.S. Gulf Platform Work Reviving in 1988-89. *Offshore.* November: 27-29.

DYHRKOPP, F. 1990. Keynote Address on Offshore Structures *California Sea Grant Symposium on Preservation of Ageing Marine Structures.* Aug. 13-14, 1990. ed. R. G. Bea. UC Berkeley.

DYHRKOPP, F., SCHONEKAS, G., and STEWART, M. 1987. *Investigation of October 27-28, 1985, Structural Failures.* Gulf of Mexico OCS Regional Office: Minerals Management Service, OCS Report MMS 87-0075.

ENGSETH, A.G. 1985. *Finite Element Collapse Analysis of Tubular Steel Offshore Structures.* Trøndheim: Marine Technology Center, The Norwegian Institute of Technology. Report No. UR-85-46.

FRIEZE, P.A. 1990. API RP2A Practice for the 1990s - a Step Forward. *Trans. IMarE.* Vol. 102, pp. 175-182.

FRISBIE, F.R. 1987. Inspecting and Repairing Offshore Platforms Today. *Ocean Industry* .

GERWICK, B.C. 1986. *Construction of Offshore Structures.* John Wiley & Sons.

GRENDA, K.G., CLAWSON, W.C., and SHINNERS, C.D. 1988. Large-Scale Ultimate Strength of Tubular K-Braced Frames. *Proceedings, 20th Offshore Technology Conference.* OTC No. 5832.

HOLNICKI-SZULC, J., and GIERLINSKI, J.T. 1989. Structural Modifications Simulated by Virtual Distortions. *International Journal for Numerical Methods in Engineering.* Vol. 28: 645-666.

JACKSON, P. 1990. *Introduction to Expert Systems.* 2nd ed. Addison Wesley.

JORDAAN, I. J. 1988. *Safety Levels Implied in Offshore Structural Design Codes: Application to CSA Program for Offshore Structures.* Report to SubCommittee on Verification, CSA Program for Fixed Offshore Structures.

KARAMCHANDANI, A., and CORNELL, C.A. 1987. *Structural System Reliability Analysis Methods.* Stanford: The John A. Blume Earthquake Engineering Center, Stanford University. Report No. 83.

KEENEY, R.L., and RAIFFA, H. 1976. *Decisions with Multiple Objectives: Preferences and Value Tradeoffs.* Chichester, England: John Wiley & Sons, Inc.

LALANI, M., and SHUTTLEWORTH, E.P. 1990. The Ultimate Limit State of Offshore Platforms Using Reserve and REsidual Strength Principles. *Proceedings, 22nd Offshore Technology Conference.* OTC No. 6309.

LEE, G.C. 1981. Hurricane Losses. *Safety and Offshore Oil.* Background Papers of the Committee on Assessment of Safety of OCS Activities. Washington, D.C.: Marine Board, National Research Council. 201-209.

LLOYD, J.R., and CLAWSON, W.C. 1983. Reserve and Residual Strength of Pile Founded, Offshore Platforms. *Proceedings of the Symposium on the Role of Design, Inspection, and Redundancy in Marine Structural Reliability.* Washington, D.C. : National Academic Press. 157-195.

MARSHALL, P.W. 1979. Design Strategies for Monitoring, Inspection, and Repair of Fixed Offshore Platforms. *Second WEGMENT Graduate Schhol, Advanced Aspects of Offshore Engineering, The Norwegian Institute of Technology.* Module 2a. 23 pp.

MARSHALL, P.W., and BEA, R.G. 1976. Failure Modes of Offshore Structures. *Proceedings, Ist Behavior of Offshore Structures (BOSS) Conference.* Trondheim. 579-626.

MARTINDALE, S.G., KRIEGER, W.F., PAULSON, S.K., HONG, S.T., PETRAUSKAS, C., HSU, T-M., and PIEFFER, J.E. 1989. Strength/Risk Assessment and Repair Optimization for Ageing, Low-Consequence, Offshore Fixed Platforms. *Proceedings, 21st Offshore Technology Conference.* OTC No. 5931.

MELCHERS, R.E. 1987. *Structural Reliability Analysis and Prediction.* John Wiley & Sons.

MELCHERS, R.E. 1989. Human Error in Structural Design Tasks. *Journal of Structural Engineering, American Society of Civil Engineers.* Vol. 115, No. 7, July 1989.

McCLELLAND, B. and COX, W.R. 1976. Performance of Pile Foundations for Fixed Offshore Structures. *Proceedings, Ist Behavior of Offshore Structures (BOSS) Conference.* Trondheim. 528-544.

MINERALS MANAGEMENT SERVICE (MMS). 1986. *Accidents Connected with Federal Oil and Gas Operations on the Gulf of Mexico Outer Continental Shelf, Vol.1, 1956-79.* Report No. MMS86-0039.

MINERALS MANAGEMENT SERVICE (MMS). 1988. *Rules and Regulations.* Federal Register. Vol. 53: No. 63 .

MOAN, T. 1983. Safety of Offshore Structures. *Proceedings, 4th International Conference on Applications of Statistics and Probability in Soil and Structural Engineering.* Florence, Italy, Vol.1: 41-85.

MOAN, T. 1986. Advances in the Design of Offshore Structures for Damage-Tolerance. *Advances of Marine Structures.* ed. C.S. Smith, and J.D. Clarke. Proc. of an International Conference held at Admirality Research Establishment, Dunfermline.

MOAN, T., and AMDAHL, J. 1989. Catastrophic Failure Modes of Marine Structures. *Structural Failures.* ed. T. Wierzbichi, and N. Jones. John Wiley. Chapter 14.

MOAN, T., AMDAHL, J., ENGSETH, A.G., and GRANLI, T. 1985. Collapse Behavior of Trusswork Steel Platform. *Proceedings, 4th Behavior of Offshore Structures (BOSS'85) Conference.* Amsterdam: Elsevier Science Pub. BV. 255-268.

MOAN, T., and TABY, J. 1988. Strength Assessment of Damaged Tubular Offshore Structures. *Proceedings, Brazil Offshore, 1987.* London: Pentech Press.

MOAN, T. et al. 1991. Collapse Behavior of Offshore Structural Systems. *Advances in Marine Structures-2.* Edited by Smith, C.S. and Dow, R.S. Elsevier Applied Science.

MORISON, J.R., O'Brian, M.P., Johnson, J.W., and Schaff, S.A. 1950. *The Force Exerted by Surface Waves on Piles.* Journal of Petroleum Technology., Vol. 189: pp. 149-154.

MOSES, F. et al. 1987. *Fatigue Evaluation Procedures for Steel Bridges.* Transportation Research Board Report 299, Washington.

MOSES, F., and STAHL, B. 1978. Reliability Analysis Format for Offshore Structures. *Proceedings, 10th Offshore Technology Conference.* OTC 3046.

MOSES, F. 1989. *Load and Resistance Factor Design Application to Design of Fixed Offshore Platforms.* API RP 2A - LRFD Tutorial, Houston.

MOSES, F. 1990. A Global Approach for Reliability Based Offshore Platform Codes. *Integrity of Offshore Strcutures-4.* Glasgow.

NELSON, K.J., and GRAFF, W.J. 1987. Categorizing Component Failure in Offshore Structures. *Journal of Waterways, Port, Coastal, and Ocean Engineering.* American Society for Civil Engineers. 11: .

NIKOLAIDIS, E., and KAPLAN, P. 1991. Uncertainties in Stress Analyses on Marine Structures. Ship Structures Committee Report SSC-363.

NORDAL, H., CORNELL, C.A., and KARAMCHANDANI, A. 1987. A Structural System Reliability Case Study of an Eight-leg Steel Jacket Offshore Platform. *Proceedings of the Marine Structural Reliability Symposium.* Virginia: SNAME, USA.

OKADA, T., and BRESLER, B. 1976. *Strength and Ductility Evaluation of Existing Low-Rise Reinforced Concrete Buildings - Screening Method.* Earthquake Engineering Research Center, University of California, Berkeley. Report No. EERC 76-1, February, 1976.

OLUFSEN, A., and BEA, R.G. Uncertainties in extreme Wave Loadings on Fixed Offshore Platforms. Proc. Offshore Mechanics and Arctic Engineering Conference, 1989.

PMB SYSTEMS ENGINEERING, INC. 1987. Development of Platform AIM (Assessment, Inspection, Maintenance) Program, Phase-II. Final Report. San Francisco.

PMB SYSTEMS ENGINEERING, INC. 1988. AIM (Assessment, Inspection, Maintenance) Program, Phase-III. Final Report. San Francisco.

PATE-CORNELL, M.E., and BEA, R.G. 1989. Organizational Aspects of Reliability Management: Design, Construction, and Operation of Offshore Platforms. *Research Report No. 89-1, Department of Industrial Engineering and Engineering Management, Stanford University.*

PIKE, P.J., and GRENDA, K.G. 1987. A Load-Shedding Algorithm for Buckled Compression Braces. *Proceedings, Offshore Technology Conference.* OTC 5441.

SCOLA, P.T. 1989. *U.S. Navy Underwater Inspection of Waterfront Facilities.* Proceedings, PORT' 89. 72- 83.

SHINNERS, C.D., EDWARDES, R.J., LLOYD, J.R., and GRILL, J.M. 1988. Structural Upgrading of Original Bass Strait Platforms. *Proceedings, 20th Offshore Technology Conference.* OTC 5701.

SPM. 1984. *Shore Protection Manual.* Coastal Engineering Research Center, Department of the Army. Vol. 2.

SMITH, A.G., SMITH, T.E., and MONSHAUGEN, T. 1988. Efofisk Subsidence: Conceptual and Design Considerations Along the Road to Jacking. *Proceedings, 20th Offshore Technology Conference.* OTC 5652.

SØREIDE, T.H., AMDAHL, J., and ASTRUD, O.C. 1986. Collapse Analysis of Framed Offshore Structures. *Proceedings, 18th Offshore Technology Conference.* OTC 5302.

STAHL, B. 1986. Reliability Engineering and Risk Analysis. Chapter 5 of *Planning and Design of Fixed Offshore Platforms.* ed. B. McClelland and M. D. Reifel. Van Nostrand Reinhold Co., NY.

STERLING, G.H., and STROHBECK, G.E. 1975. The Failure of the South Pass 70 Platform B in Hurricane Camille. *Journal of Petroleum Technology.* March: 263-268.

STEWART, G., and VAN DE GRAFF, J.W. 1990. A Methodology for Platform Collapse Analysis Based on Linear Superposition. *Proceedings, 22nd Offshore Technology Conference.* OTC No. 6311. 381-391.

STEWART, G., EFTHMIOU, M., and VUGTS, J.H. 1988. Ultimate Strength and Integrity Assessment of Fixed Offshore Platforms. *Proceedings of 5th Behavior of Offshore Structures Conference (BOSS'88),* Netherlands. 1205-1221.

TABY, J. 1986. Ultimate and post-Ultimate Strength of Dented Tubular Members. *Marine Technology Center. The Norwegian Institute of Technology.* Report No. UR-86-50

TABY, J., and MOAN T. 1985. Collapse and Residual Strength of Damaged Tubular Members. *Proceedings, 4th International Conference on Behavior of Offshore Structures (BOSS'85).* Amsterdam: Elsevier Science Publishers B.V. 395-408.

THE MARINE TECHNOLOGY DIRECTORATE LIMITED. 1989. *Underwater Inspection of Steel Offshore Installations: Implementation of a New Approach.* MTD Ltd. Publication 89/104, U.K.

TITUS, P.G., and BANNON, H. 1988. Reserve Strength Analysis of Offshore Platforms. *Proceedings, 7th Offshore South East Asia Conference.* Singapore.

UEDA, Y. and RASHED, S.M.H. 1985. Behavior of Damaged Tubular Stuctural Members. *Proceedings 4th Offshore Mechanics and Arctic Engineering Conference.* Vol. 2.

U.S. GEOLOGICAL SURVEY. 1979. *Requirements for Verifying the Structural Integrity of OCS Platforms.* Washington D.C.: Department of the Interior.

UNDERWATER ENGINEERING GROUP. 1985. *Repairs to North Sea Offshore Structures - a Review.* London. UEG Report No. UR 33.

UNDERWATER ENGINEERING GROUP. 1983. *Design of Tubular Joints for Offshore Structures.* London. UEG Report No. UR 21.

VAN DE GRAFF, J.W., and TROMANS, P.S. 1991. Statistical Verification of Predicted Loading and Ultimate Strength Against Observed Storm Damage for an Offshore Strcuture. *Proceedings 23rd Offshore Technology Conference,* Houston.

WHITMAN, R. 1984. *Evaluating Calculated Risk in Geotechnical Engineering.* Journal of Geotechnical Engineering. American Society of Civil Engineers, Vol. 110, No. 2.

YAO, J.T.P. 1985. *Safety and Reliability of Existing Structures.* Pitman.

ZAYAS, V.A., MAHIN, S.A., POPOV, E.P., and Shing, P.-S. B. 1981. Inelastic Structural Analysis of Braced Platforms for Seismic Loading. *Proceedings of Offshore Technology Conference.*

ZAYAS, V.A., MAHIN, S.A., and POPOV, E.P. 1982. Ultimate Strength of Steel Offshore Structures. *Proceedings of Behavior of Offshore Structures Conference.* Vol.2.

ZHANG, X.J., and YAO, J.T.P. 1986. *Methodologies for Safety Evaluation of Existing Structures - Literature Review.* Purdue University: School of Civil Engineering. Report No. CE-STR-86-28.

ZHANG, X.J., and YAO, J.T.P. 1986. *The Development of SPERIL Expert Systems for Damage Assessment.* Purdue University: School of Civil Engineering. Report No. CE-STR-86-29.