*Malaysian Journal of Civil Engineering 25(2):128-153(2013)*

# ULTIMATE STRENGTH ASSESSMENT FOR FIXED STEEL OFFSHORE PLATFORM

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**Abstract:** Currently, more than 80% of Malaysia's offshore platforms are aged 30-40 years which is beyond the design life of 25 years. Structural assessments are needed to gauge the platforms for the extended use. The two common methods widely used are the simplified ultimate strength analysis and static pushover analysis. Simplified ultimate strength is attained when any of member, joint, pile steel strength and pile soil bearing capacity reaches its ultimate capacity. This is the platform's ultimate strength. Static pushover analysis generally concentrates on RSR (Reserve Strength Ratio) and RRF (Reserve Resistance Factor) for the ultimate strength. This report summarizes a study of the ultimate strength of jacket platforms designed using API RP2A-WSD 21st Edition (2000) using SACS software with the module for Full Plastic Collapse Analysis. Two types of analyses have been carried out. First the ultimate strength of jacket platform with different number of legs is determined and in the second part the collapse load of platforms for different bracing configuration is studied. The non-linear pushover analysis is done by programming the software to analyze the structure with a set of incremental load until the structure collapses. The non-linear analysis module will distribute the load to alternative load paths available within the jacket framework until the structure collapse or have excessive deformation. In order to cater to uncertainties and distribution of data, several criteria of platform site location, age of service, type of platform, number of legs and other critical-related characteristics were considered. From the first phase of study, it is seen that \ a platform with more legs has higher ultimate strength compared to less number of legs. Hence a bigger jacket platform with eight legs is stiffer than smaller platform. The bracing configuration study shows that the X-bracing contributes highest rigidity to the whole platform by retaining the platform until the highest load compared to other configurations.

**Keywords:** *Aged platform, Collapse analysis, Reserve strength, Load factors*

## 1.0 Introduction

Offshore structures are used for oil and gas extraction from under the seabed. It provides a safe, dry working environment for the equipment and personnel who operate the platform. Offshore structures are of two categories namely fixed platform and floating platform. Examples of fixed platforms are steel-jacket platform, jack-up and compliant tower while examples of floating platform are spar, semi-submersible and FPSO. Jacket platforms have a design life in the range of 25-30 years. But many platforms in Malaysia are about 30-40years old. Some of the very early platforms are still in service. Over the last 10 years or so, various structural integrity assessments have been carried out on the platforms to gauge its safety and usability beyond the design life. Assessment of structural integrity can be done qualitatively, semi-qualitatively or quantitatively. The first step for all these is knowledge on basic information (as-built) on platforms, which was reported for Malaysian platforms by Akram and Potty (2011a). Inspections including underwater inspections are also carried out (Akram and Potty, 2011b). Assessment of Malaysian platforms using semi-quantitative method has been reported in Potty *et al.* (2012). Not much work on quantitative assesment of Malaysian platforms have been reported. Data collection for such methods include information on resistance parameters as reported in Idrus *et al.* (2011a; 2011b) and environmental load as reported in Idrus *et al.* (2011c). Quantitative structural assessments can be carried using reliability methods as reported in Cossa *et al.* (2011a, 2011b, 2012a, 2012b). Alternately platforms can be analyzed using pushover analysis. The latest metocean data and SACS (EDI, 2006) input file (model) for the jacket platform are required for the analysis.

### *1.1 Simplified Ultimate Strength Analysis*

Assessment for an aged structure involves documentation of design basis, design level analysis and ultimate strength analysis. The design basis includes key parameters including design life, methodology, standards and codes, design parameters for wind, wave, current, seismic, boat impact etc. An analysis is done mainly to determine the total strength of a structure. Checking for the ultimate strength is done with respect to API RP2A-WSD (2000). Excessive deformation or resistances to total collapse are measures to judge the structural integrity. The structure strength is determined from static pushover analysis and cyclic loading for severe storm condition. API RP2A-LRFD (1993) developed based on reliability based calibration, checks the platform for combined action of extreme wave (storm condition), current and wind that consider the joint probability off-occurrence. The wave forces were computed using the drag and inertia coefficients (API RP2A-LRFD, 1993): For smooth surface $C_d = 0.65$, $C_m = 1.60$ and for rough surface $C_d = 1.05$, $C_m = 1.20$. The code gives equations for checking the cylindrical members under tension, compression, bending, shear and combined loads. Members under combined axial tension and bending should be designed to satisfy equation (1).

$$1-\cos\left[\frac{\{\Pi(f_t)\}}{\{2\phi_t F_y\}}\right]+\frac{\left[(f_{by})^2+(f_{bz})^2\right]^{1/2}}{(\phi_b F_{bn})} \leq 1.0 \tag{1}$$

Where,
$f_{by}$ = bending stress about member y-axis (in-plane)
$f_{bz}$ = bending stress about member z-axis (out-plane)
$F_{bn}$ = nominal bending
$F_y$ = yield strengths
$F_t$ = axial tensile stress
$\Phi_t$ = resistance factor for axial tensile strength (= 0.95)
$\Phi_b$ = resistance factor for bending strength (= 0.95)

Members under combined axial compression and bending should be designed to satisfy equation (2).

$$\frac{(f_c)}{(\phi_c F_{cn})} + \left\{\frac{1}{(\phi_b F_{bn})}\right\}\left[\left\{\frac{(C_{my} f_{by})}{\left(1-\frac{f_e}{(\phi_c F_{ey})}\right)}\right\}^2 + \left\{\frac{(C_{mz} f_{bz})}{\left(1-\frac{f_e}{(\phi_c F_{ez})}\right)}\right\}^2\right]^{\frac{1}{2}} \leq 1.0 \tag{2}$$

And

$$1-\cos\left[\frac{\{\Pi(f_c)\}}{\{2\phi_c F_{xe}\}}\right] + \frac{\left[(f_{by})^2 + (f_{bz})^2\right]^{\frac{1}{2}}}{(\phi_b F_{bn})} \leq 1.0 \tag{3}$$

$$F_c < \phi_c F_{xc} \tag{4}$$

Where,
$C_{my}$ = reduction factor corresponding to the member y-axis
$C_{mz}$ = reduction factor corresponding to the member z-axis
$F_{ey}$ = euler buckling strength corresponding to the member y-axis
$F_{ez}$ = euler buckling strength corresponding to the member z-axis

$$F_{ey} = \frac{F_y}{(\lambda_y)^2} \tag{5}$$

$$F_{ez} = \frac{F_z}{(\lambda_z)^2} \tag{6}$$

$\lambda$ = column slenderness parameter for member about the respective axes
$F_{cn}$ = nominal axial compressive strength
$F_c$ = axial compressive stress due to factored load
$\Phi_c$ = resistance factor for axial compressive strength, 0.85

The equations for strength checks of tubular joints are also given in API RP2A-LRFD (1993).

For assessment of existing platforms, the criteria are dependent on the category of the platform, which considers the life safety, and the consequences of failure. Krieger, *et al. (1994)* has recommended two factors for ultimate strength checks for existing platforms namely:

- Ultimate to Linear Ratio (ULR)
- Reserve Strength Ratio (RSR)

ULR is the ratio of the ultimate resistance load to that causing a unity check of 1.0 in the original design and RSR is the ratio of the ultimate strength load to the storm condition (100-year) design load. For manned platforms with or without significant environmental impact, a ULR of 1.8 and RSR of 1.6 are recommended, while for platforms of minimum consequence a ULR of 1.6 and RSR of 0.8 are recommended.

Simplified Ultimate Strength (SUS) is generally estimated based on the smallest of the four base shear values obtained when the first of the following component classes reach its ultimate capacity namely joints, members, pile steel strength, and pile soil bearing capacity, The platform base shear values that satisfy each of these conditions are determined from a linear analysis by using respective API RP2A-LRFD equations with the load and resistance.

In simplified approach, a linear static global analysis of the structure is performed for forces due to the combined action of gravity loads and extreme wave loads (100-year return period) and associated current and wind effects. The structure is loaded with series of monotonically increasing environmental load conditions from all directions of interest. Member and joint forces are obtained from the analysis and for each load condition the strength checks are made for the members, joint and etc using API RP2A-LRFD. The load is increased after each stage until any component of the structure fails or reaches its ultimate strength. The platform attains ultimate strength when any member or joints reach its ultimate capacity. The first member/joint failure is obtained and the load factor corresponding to this is calculated as ratio of the base shears corresponding to the first member failure and the 100-year environmental load. The analysis is further performed by removing the failed member from the model, if alternative load paths are available to bypass a failed member. The analysis is terminated when there is no alternative load path or deformation of the structure exceeds the limit from functional considerations. The reserve strength ratio is then calculated as the ratio of the base shears corresponding to collapse load and first member failure. Full ultimate strength analysis using non-linearity can be resorted to if the simplified ultimate strength analysis does not meet the requirements for requalification.

API RP2A-LRFD recommends using linear wave theory and Morrison equation to calculate the wave and current loads on the structure. Yield stress of steel is taken as per design basis requirement. The base shear for first member failure is obtained from each attack angle to get the factor of first member failure, the factor for collapse load and the reserve strength ratio. The output from analysis is categorized as (1) Lateral load for 100-year storm condition, (2) First member failure load, $P_{mf}$, (3) Factor for first member failure, (4) Collapse load, $P_u$, (5) Factor for collapse load, (6) Deformation corresponding to $P_{mf}$, (7) Deformation corresponding to $P_u$, and (8) Reserve strength ratio. The factors are calculated as follows:

$$\text{Factor for first member failure} = \frac{\text{First member failure load}, Pmf}{\text{Lateral load for } 100-\text{year storm condition}} \tag{7}$$

$$\text{Factor for collapse load} = \frac{\text{Collapse load}, Pu}{\text{Lateral load for } 100-\text{year storm condition}} \tag{8}$$

$$\text{Reserve strength ratio} = \frac{\text{Factor for collapse load}}{\text{Factor for first member failure}} \tag{9}$$

Another approach proposed by Vannan *et al.* (1994) where a linear static in place analysis is done by increasing environmental loading until first member or other component failure occurs. Unity check reported above 1.0 is allocated as the ultimate strength of the structure. Other simplified methods introduced by Bea and Mortazavi (1995) provide reasonable estimates of platform load capacity relative to the results obtained from the detailed static pushover analysis.

### *1.2 Static Pushover Analysis*

Research on the response of jacket structure to extreme condition (100-year return period storm wave) requires the estimation of the ultimate strength of the framed structure as well as its reserve capacity . An elastic frame analysis is performed, typically with the elements assumed to be rigidly connected. API RP 2A-WSD 21st Edition, Section 17.0 recommends for the assessment of existing platforms a sequence of analysis from screening, through design level to ultimate strength assessment to demonstrate the structural adequacy. At the ultimate strength level, a platform may be assessed using inelastic, static pushover analysis.

Lloyd and Clawson (1984) discusses the sources of reserve and residual strength of ‘frame behaviour’. Marshall (1979) studied behaviour of elastic element and ultimate strength of the system. Marshall and Bea (1976) demonstrated the reserve safety factor and Kallaby and Millman (1975) studied the inelastic energy absorption capacity of the Maui A platform under earthquake loading. Recent investigation shows that static pushover analysis generally suffices to demonstrate a structure’s resistance to the cyclic loading of the full storm.

Trends for lighter, liftable jackets and new concepts for deepwater have provided additional impetus for such studies. Fewer members in the splash zone may increase the risk to topsides safety in the event of impact, and the deletion of members with the low elastic utilizations to save weight reduces the capacity for redistribution along the alternative load paths. Comparative calculation of reserve capacity for different structural configurations can help ensure that levels of reserve strength and safety embodied within the older designs are maintained. Therefore there is a requirement to develop an understanding and the corresponding analytical tools to predict system reserves beyond individual component failure capacities, in order to demonstrate integrity in the event of such extreme loading scenarios.

Reserve strength is defined as the ability of the structure to sustain loads in excess of the design value. RSR (Reserve Strength Ratio) introduced by Titus and Banon (1988) and RRF (Reserve Resistance Factor) introduced by Lyod and Clawson (1984) are defined below:

$$RSR = \frac{UltimatePlatform\,\mathrm{Re}\,sis\tan ce}{DesignLoad} \tag{10}$$

$$RRF = \frac{EnvironmentalLoadatCollapse}{DesignEnvironmentalLoad} \tag{11}$$

Fixed offshore structure spread the load through a network of paths. As a result the failure of a single member does not necessarily lead to catastrophic structural collapse. The redundancy in the structure is measured in two ways, namely (1) redundancy factor (RF) and (2) the damaged strength rating (DSR). These measurements are load case dependent and any structure may exhibit very different redundancy properties for different loading directions.

Reserve strength is evaluated by applying the maximum loading from the extreme event and then performing the 'pushover' analysis. For an extreme storm, the environmental loading is cyclic, imposed in an underlying dominant direction. The maximum wave is unlikely to be an isolated event, but will be a peak in series of extreme loads. The possibility of cyclic degradation of components which have failed, or approaching failure even though overall structure resistance may remain adequate, therefore needs to be considered.

Static pushover analysis is the application of a single load, applied to any specific location which is incremented in steps until collapse while cyclic analysis is a 'storm load' sequence of particular amplitude applied to the structure. Shakedown effects were studied using non-linear FE analysis at SINTEF (Hellan *et al.*, 1991, 1993, 1994) for the provision of low cycle-high stress fatigue. These studies on North Sea Jackets, recommend that an extreme event static analysis generally suffices to demonstrate structure's resistance to the cyclic loading of a full storm. Research was also carried out supported by Shell (Stewart *et al.*, 1993, 1998; Stewart and Tromans, 1993; Eberg *et al.*, 1993; and Hellan *et al.*, 1991, 1993, 1994). Under the increased loading, the structure converts into elasto-plastic range, yielding occurs thereby reducing the stiffness and introducing permanent plastic deformations. Under cyclic load, the yield repeats and result in three different forms of response namely Low cycle fatigue, Incremental collapse and Shakedown.

## 2.0 Methodology

The scope of work involves the following studies on aged platforms:

(1) Full Plastic Collapse Analysis of 3, 4, and 8 Legged Platforms (A, B and C respectively) and evaluation of the factor for first member failure, collapse load factor and RSR
(2) Bracing configuration study for platform A and determination of collapse load and RSR.

The methodology is described below.

### *2.1 Full Plastic Collapse Analysis of 3, 4, and 8 Legged Platforms*

SACS modelling for platforms A, B and C commenced by adjusting the original model with the site visit findings and latest drawing. The dead and live load of the SACS model were retained as per the design basis. Minor adjustments were made to the model in terms of the latest metocean data for the area. Latest data of maximum wave height (Hmax), associate period (Tass), wind speed, current speed and tidal height, HAT and

LAT were used. The environmental loading impact on the platform considers eight (8) directions. The storm condition is applied to the platform according to the metocean data as the maximum load acting on the structure. Stokes's 5th order theory defined in API RP2A-WSD is used for wave / current loading computation. For the purpose of analysis, eight (8) models were created for platform A and C while twelve (12) models were prepared for platform B which is a tripod and required 12 directions to be covered.

The SACS Collapse module is a non-linear finite element analysis system for structures. It can solve for the geometric and material non - linearities and determine the ultimate load capacity by using large deflection, iterative direct stiffness solution technique. The members are divided into several sub-segments along the length and sub-areas to define the cross section. The method allows for gradual plasticification along the member length. Tubular connection flexibility, capacity and failure are revised empirically during the analysis. The linear analysis model is modified to be suitable for collapse analysis. The model is designed to cater for the storm condition wave/current in order to get the strength of the structure under maximum loading criteria, and then the model undergoes the SACS COLLAPSE analysis. The load sequence and load increment in collapse input file is prepared based on the design basis. The other properties in the collapse input file are retained as per the default design. The SACS model was modified to apply wave and current loads for different directions. For a tripod jacket model, 12 models are required to cater for the 12 attack angles as defined in the metocean data. The four and eight legged platform models were modified to cater for only eight directions as defined in their metocean data. The main issue is to analyze the jackets for all the directions of loading and to determine the direction having the highest collapse load. A series of incremental load defined in collapse input file will generate collapse load by utilizing the module of FULL PLASTIC COLLAPSE ANALYSIS in the SACS. Upon completing the analysis, the output available are (1) Base shear and overturning moment, (2) Basic load case summary, (3) Load combination summary, (4) First member failure load, (5) Collapse load. The factor for first member failure and reserve strength ratio based on base shear and collapse load are determined. Collapse view module is used to view the platform collapse mode and properties. The data is also used to determine the structure ultimate strength and corresponding wind/ wave direction. Figure 1 shows the flow chart of full plastic collapse analysis.

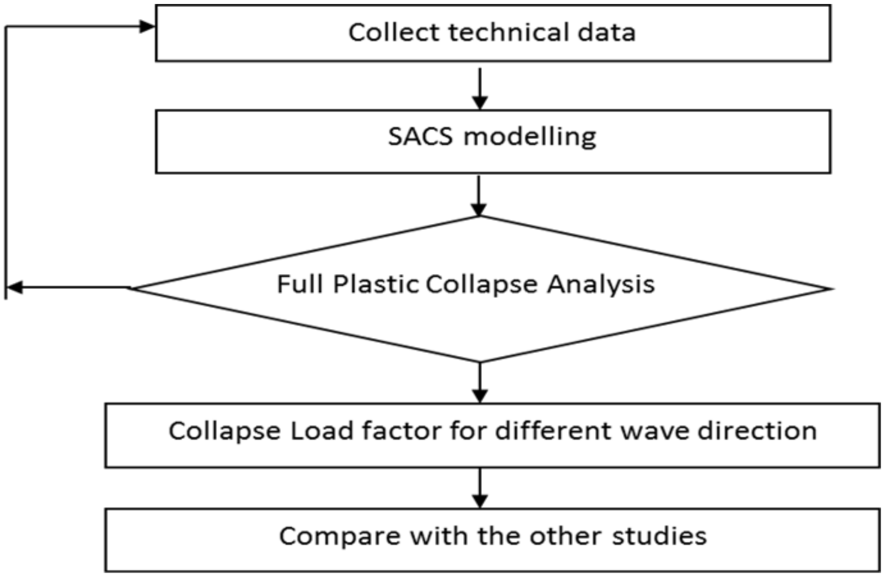


Figure 1: Flow chart of full plastic collapse analysis (Fawaz, 2010)

### *2.2 Bracing Configuration Study*

This involves collapse analysis of a selected jacket with varying of bracing schemes. Some common bracing types are X bracing, Y bracing, Single diagonal bracing, K bracing, Inverted K bracing and Diamond bracing (Figure 2). Figure 3 shows the flow chart for the bracing configuration study.

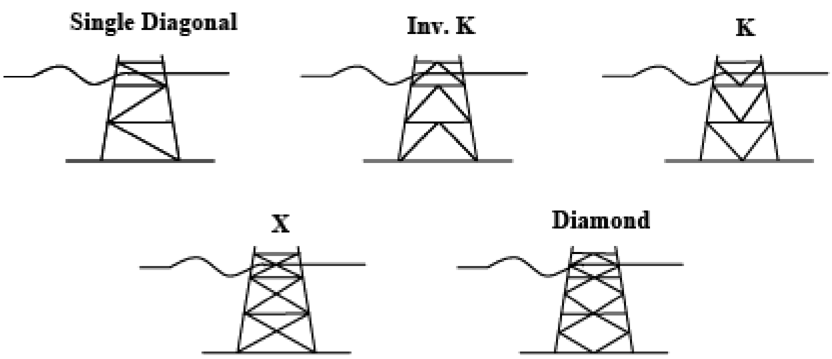


Figure 2: Bracing framework schemes (Nelson, 2003)

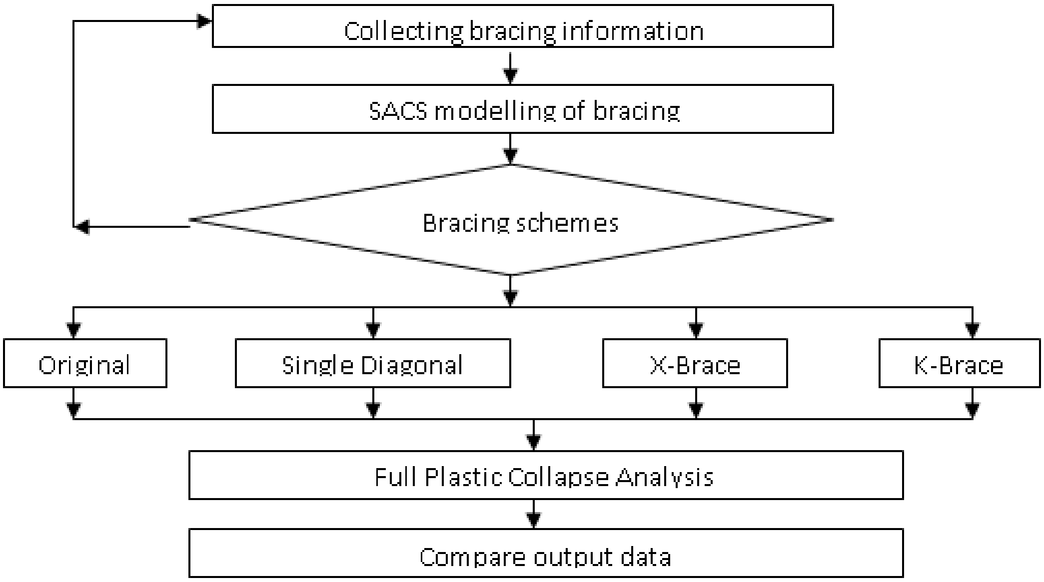


Figure 3: Flow chart of bracing configuration study (Fawaz, 2010)

The share of the load carried by different bracing schemes differs. The platform A was remodeled to cater for all types of bracing. Utilizing linear analysis, each bracing design will undergo stability check in term of UC value of the respective bracing. Allowable value of UC<1.0 indicate that the new designs are acceptable for the collapse analysis. For different bracing schemes of platform A, the identical bracing properties of size, shape and wall thickness were used. New joint was allocated at the critical location when modelling for X-bracing and K-bracing where the member intersects at the middle. All bracing models were analyzed and compared. The collapse load of the jacket for different bracing framework strength was determined. The maximum load for first member failure was noted for each case. The output obtained from the analysis consists of (1) base shear and overturning moment, (2) Collapse Solution Summary, (3) Collapse Load, (4) Maximum deflection and (5) RSR.

## 3.0 Platform Overview

Platform A is a four pile-through-leg drilling platform installed in 1979 and located in PMO (Figure 4). One boat landing is on the Platform South face and other two boat landings are on the Platform West face. The topside comprises of the Upper Deck (EL +19202), Lower Deck (EL+12192). Details of the structure are in Table 1 and environmental data are in Table 2. Platform B in SKO is a three pile-through-leg platform installed in 1977 (Figure 5). The platform supports four numbers of 10.75” Ø

(27.31 cm) risers. Details of the structure are in Table 1 and environmental data are in Table 2. Platform C is a eight pile-through-leg drilling platform installed in 1979 in PMO (Figure 6). The topside comprises of the Upper Deck (EL +21184), Lower Deck (EL+14021). Details of the structure are in Table 1 and environmental data are in Table 2.

Figure 4: Platform A

Figure 5: Platform B

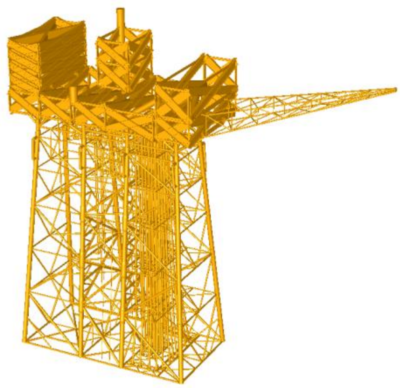

Figure 6: Platform C

Table 1: Platform description

| | Platform A (PMO) | Platform B (SKO) | Platform C (PMO) |
|---|---|---|---|
| Structure Function | Drilling Platform | - | Drilling Platform |
| Installation Date | 1979 | 1977 | 1979 |
| TAD Rig | Jack-Up | - | - |
| Water Depth(MSL) | 70.71 m (209.56 ft) | 70.93 m (236 ft) | 67.21 m (209.56 ft) |
| No. of Piles | 4 54"Ø, 137.16cm | 3 30"Ø, 76.20cm | 8 54"Ø, 137.16cm |
| Pile penetration below mudline | 79.25 m | 68m | 109.73m |
| Number of Conductor | 12 nos | - | 32 |
| Diameter of conductor | 24'Ø, (66.96 cm) | | 24'Ø, (66.96 cm) |
| Number of Anode | 136 | 136 | - |
| Number of Boat landing | 3 | 1 | 1 |
| Number of pipe Caissons | 3 | - | 1 |
| Number of Riser pipes | 2 | 4 | 10 |
| Number of Riser Guard | 1 | - | 2 |

Table 2: Environmental Data for platform design

| Wave Parameter | 100-Year Directional Wave (deg) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Platform A (PMO) | | | | | | | |
| Direction (degress) | 0 | 42.11 | 90 | 137.89 | 180 | 222.11 | 270 | 317.89 |
| Max. Height, Hmax (m) | 6.3 | 6.3 | 11.4 | 7.6 | 7.6 | 7.6 | 5.0 | 6.3 |
| Assoc. Period, Tass (s) | 7.3 | 7.3 | 9.3 | 8.4 | 8.4 | 8.4 | 6.6 | 7.3 |
| | Platform B (SKO) | | | | | | | |
| Directions | N | NE | E | SE | S | SW | W | NW |
| Max. Height, Hmax (m) | 10.0 | 9.0 | 5.1 | 5.1 | 5.1 | 6.9 | 9.0 | 10.0 |
| Assoc. Period, Tass (s) | 9.7 | 9.4 | 8.3 | 8.3 | 8.3 | 8.6 | 9.4 | 9.7 |
| | Platform C (PMO) | | | | | | | |
| Direction (degress) | 0 | 65 | 90 | 115 | 180 | 245 | 270 | 295 |
| Max. Height, Hmax (m) | 5.8 | 7.3 | 10.1 | 8.2 | 5.8 | 5.8 | 5.8 | 5.8 |
| Assoc. Period, Tass (s) | 8.0 | 8.5 | 10.0 | 9.0 | 8.0 | 8.0 | 8.0 | 8.0 |

## 4.0 Computer Modelling

The program SACS (Structural Analysis Computer System) used to perform the analyses described in the study was developed by EDI (Engineering Dynamic Inc). The full plastic collapse module is used for the purpose of determining the collapse load.

The SACS input files were checked for latest metocean data and latest modification to the actual structure. Localized wave direction model introduced to design a model caters only to 1 direction per analysis. In order to determine the platform critical direction, the effects from each wave attack direction were compared. In the analysis, 4 and 8 legged platforms were designed for 8 directions of waves whereas the tripod platform was designed for 12 directions. The collapse input file for the model consisted of series of incremental load by a defined factor until the structure collapses; when no load paths are available or deformation exceeds the allowable value.

Different leg arrangement or complexity of the SACS input file require more period for completion of the analysis. Table 3 gives the duration for modeling and analyzing for each platform. The table shows that a complex and larger platform required more time to complete an analysis.

Table 3: Average duration of analysis

| No of leg | Period of modeling (hr) | Period of analyzing (hr) |
|---|---|---|
| 3 | 1.0 | 0.5 |
| 4 | 2.0 | 4 - 6 |
| 8 | 2.5 | 6 - 8 |

## 5.0 Results of SACS Analysis

Two types of analysis were carried out and the results of each are given below:

### *5.1 Full Plastic Collapse Analysis of Platform with different number of legs*

Final values of each analysis for all three (3) platforms which are located in different block (PMO and SKO) and has different arrangements are compared (Table 4,5, and 6).

Table 4: Platform B (Tripod) Collapse Analysis Results (SKO)

| Wave Direction (deg) | Lateral load for 100-year storm condition, kN | First member failure load Pmf, kN | Factor for first member failure | Collapse load, Pu, kN | Factor for collapse load | Reserve strength ratio |
|---|---|---|---|---|---|---|
| 0.00 | 1729.45 | 3298.68 | 1.91 | 4136.79 | 2.39 | 1.25 |
| 30.00 | 1664.24 | 4017.31 | 2.41 | 4331.39 | 2.60 | 1.08 |
| 60.00 | 1768.69 | 3207.17 | 1.81 | 4020.10 | 2.27 | 1.25 |
| 90.00 | 1712.93 | 2678.52 | 1.56 | 3956.14 | 2.31 | 1.48 |
| 120.00 | 1769.47 | 3296.41 | 1.86 | 5142.96 | 2.91 | 1.56 |
| 150.00 | 1665.23 | 3522.15 | 2.12 | 4260.86 | 2.56 | 1.21 |
| 180.00 | 1729.94 | 3299.57 | 1.91 | 3880.84 | 2.24 | 1.18 |
| 210.00 | 1664.90 | 2775.55 | 1.67 | 4262.99 | 2.56 | 1.54 |
| 240.00 | 1769.86 | 3208.95 | 1.81 | 3618.19 | 2.04 | 1.13 |
| 270.00 | 1715.55 | 4134.51 | 2.41 | 4491.81 | 2.62 | 1.09 |
| 300.00 | 1771.57 | 3300.51 | 1.86 | 3736.09 | 2.11 | 1.13 |
| 330.00 | 1665.84 | 2694.31 | 1.62 | 4287.16 | 2.57 | 1.59 |

Table 5: Platform A (4 legged) Collapse Analysis Results (PMO)

| Wave Direction (deg) | Lateral load for 100-year storm condition, kN | First member failure load Pmf, kN | Factor for first member failure | Collapse load, Pu, kN | Factor for collapse load | Reserve strength ratio |
|---|---|---|---|---|---|---|
| 0.00 | 2343.89 | 9230.77 | 3.94 | 9488.49 | 4.05 | 1.03 |
| 42.11 | 2737.78 | 7844.51 | 2.87 | 9174.21 | 3.35 | 1.17 |
| 90.00 | 5958.80 | 12226.07 | 2.05 | 12974.80 | 2.18 | 1.06 |
| 137.89 | 3444.07 | 8140.04 | 2.36 | 9489.52 | 2.76 | 1.17 |
| 180.00 | 3616.53 | 10582.18 | 2.93 | 11465.45 | 3.17 | 1.08 |
| 222.11 | 3975.24 | 8396.79 | 2.11 | 10078.17 | 2.54 | 1.20 |
| 270.00 | 2043.84 | 10746.32 | 5.26 | 11690.44 | 5.72 | 1.09 |
| 317.89 | 2200.71 | 7944.94 | 3.61 | 9018.19 | 4.10 | 1.14 |

Table 6: Platform C (8 legged) Collapse Analysis Results (PMO)

| Wave Direction (deg) | Lateral load for 100-year storm condition, kN | First member failure load Pmf, kN | Factor for first member failure | Collapse load, Pu, kN | Factor for collapse load | Reserve strength ratio |
|---|---|---|---|---|---|---|
| 0.00 | 3275.85 | 30781.52 | 9.40 | 31730.06 | 9.69 | 1.03 |
| 65.00 | 6837.27 | 33858.24 | 4.95 | 34839.44 | 5.10 | 1.03 |
| 90.00 | 12459.22 | 32464.96 | 2.61 | 37317.46 | 3.00 | 1.15 |
| 115.00 | 8111.45 | 34632.22 | 4.27 | 38437.97 | 4.74 | 1.11 |
| 180.00 | 3744.34 | 32551.37 | 8.69 | 35644.39 | 9.52 | 1.10 |
| 245.00 | 4352.87 | 6809.40 | 1.56 | 6809.40 | 1.56 | 1.00 |
| 270.00 | 4784.71 | 30910.52 | 6.46 | 31968.74 | 6.68 | 1.03 |
| 295.00 | 4470.12 | 26443.94 | 5.92 | 32003.96 | 7.16 | 1.21 |

Table 4, 5 and 6 shows the analysis output from the tripod, 4 legged and 8-legged platforms subject to series of incremental loads predefined in the Full Plastic Collapse module of SACS. Wave directions in the metocean data (Table 2) for respective platforms in the PMO and SKO are considered. Lateral load for 100-year storm condition correspond to the base shear at the base of the structure. The base shear is computed using the Stokes's 5th order theory for computing the drag and inertia forces. The first member failure is the indication of the first member or other component reaching ultimate capacity within the plasticity zone.

Table 7 Column 4 shows the effect of number of jacket legs on the collapse loads. The existence of alternative load paths through leg member and bracing provide more stiffness to the structure. Table 7 Column 4 clearly indicates a relation between the load needed for structure collapse and the number of legs and direction. Furthermore, platform C with eight legs needed larger loads for collapse compared to platform A with 4 legs and platform B, tripod type. With more legs, the structure configuration is more complex, rigid and strong. This is proved by the behaviour of platform C in retaining larger loads.  Column 5 shows the RSR values, which indicates that platform B with tripod leg had more strength reserve compared to the more redundant structures. The design was made for three leg jacket structure to have more reserve strength before the structure reached the critical point of ultimate load.

Table 7: Full Plastic Collapse Analysis of Different Legged Platform

| Platform | Wave Direction (deg) | First Member Fail Load (kN) | Collapse Load (kN) | RSR | Factor For Collapse |
|---|---|---|---|---|---|
| B | 0.00 | 3298.68 | 4136.79 | 1.25 | 2.39 |
| | 30.00 | 4017.31 | 4331.39 | 1.08 | 2.60 |
| | 60.00 | 3207.17 | 4020.10 | 1.25 | 2.27 |
| | 90.00 | 2678.52 | 3956.14 | 1.48 | 2.31 |
| | 120.00 | 3296.41 | 5142.96 | 1.56 | 2.91 |
| | 150.00 | 3522.15 | 4260.86 | 1.21 | 2.56 |
| | 180.00 | 3299.57 | 3880.84 | 1.18 | 2.24 |
| | 210.00 | 2775.55 | 4262.99 | 1.54 | 2.56 |
| | 240.00 | 3208.95 | 3618.19 | 1.13 | 2.04 |
| | 270.00 | 4134.51 | 4491.81 | 1.09 | 2.62 |
| | 300.00 | 3300.51 | 3736.09 | 1.13 | 2.11 |
| | 330.00 | 2694.31 | 4287.16 | 1.59 | 2.57 |
| A | 0.00 | 9230.77 | 9488.49 | 1.03 | 4.05 |
| | 42.11 | 7844.51 | 9174.21 | 1.17 | 3.35 |
| | 90.00 | 12226.07 | 12974.80 | 1.06 | 2.18 |
| | 137.89 | 8140.04 | 9489.52 | 1.17 | 2.76 |
| | 180.00 | 10582.18 | 11465.45 | 1.08 | 3.17 |
| | 222.11 | 8396.79 | 10078.17 | 1.20 | 2.54 |
| | 270.00 | 10746.32 | 11690.44 | 1.09 | 5.72 |
| | 317.89 | 7944.94 | 9018.19 | 1.14 | 4.10 |
| C | 0.00 | 30781.52 | 31730.06 | 1.03 | 9.69 |
| | 65.00 | 33858.24 | 34839.44 | 1.03 | 5.10 |
| | 90.00 | 32464.96 | 37317.46 | 1.15 | 3.00 |
| | 115.00 | 34632.22 | 38437.97 | 1.11 | 4.74 |
| | 180.00 | 32551.37 | 35644.39 | 1.10 | 9.52 |
| | 245.00 | 6809.40 | 6809.40 | 1.00 | 1.56 |
| | 270.00 | 30910.52 | 31968.74 | 1.03 | 6.68 |
| | 295.00 | 26443.94 | 32003.96 | 1.21 | 7.16 |

Table 8 and Figure 7 enable identification of the critical attack angle of wave on the structure.

Table 8: Wave attack angle for maximum and minimum values of collapse load for platforms

| Platform | Minimum value (kN) | | Maximum value (kN) | |
|---|---|---|---|---|
| | Wave attack angle (deg) | Load | Wave attack angle (deg) | Load |
| B | 90.00 | 3956.14 | 270.00 | 4491.81 |
| A | 42.11 | 9174.21 | 90.00 | 12974.80 |
| C | 245.00 | 6809.40 | 115.00 | 38437.97 |

The maximum value of the tabulated data in column 4 for each platform indicates the strongest direction of the structure and the highest load (base shear) it can retain before it collapses. The minimum value of the tabulated date in column 4 for each platform indicates the critical angle to the structure having the minimum load (base shear) required for the structure to fail. First member failure load indicates the base shear force at which first member failure is observed.

This study considers the relationship of the wave generated forces in different directions to the corresponding collapse load. The highest value of the base shear indicates the collapse load at the angle. For platform B, considering the different directions, the highest base shear is generated at the angle of 300 degree to the platform. Non-linear analysis indicates that the minimum load for structure to collapse occurs at the angle of 90 deg while at 270 degree, maximum load is needed. For platform A, the highest reported base shear is at 90 degree. The platform C metocean data (Table 2) show that the critical wave force occurs at 90 degree angle but the collapse load analysis (Figure 7) shows that 90 degree direction is not critical (whereas the 250 degree angle is critical).

Platform with different number of legs showed different behaviour, response and rigidity when subjected to collapse analysis. Complexity and rigidity are the main criteria for a platform to have higher ultimate strength.

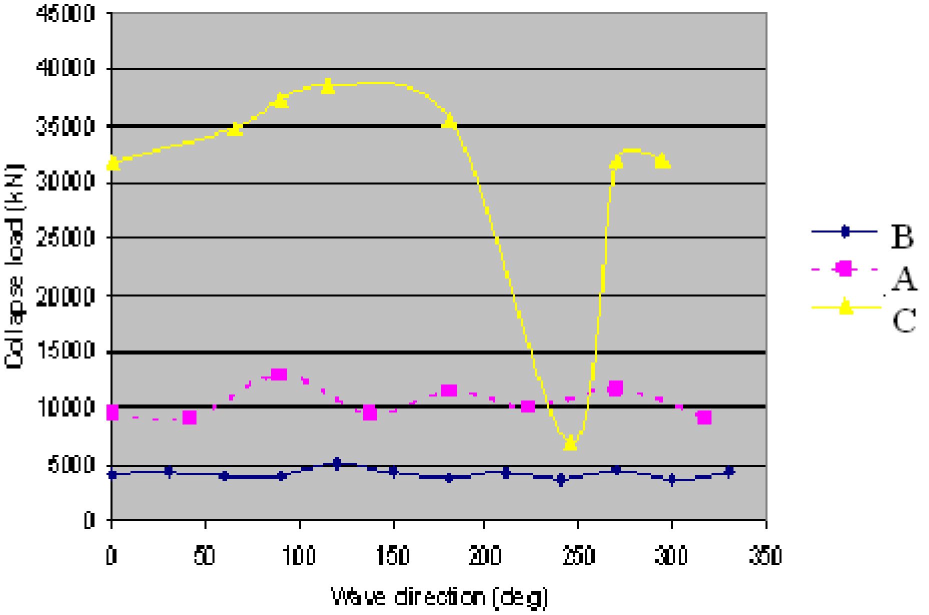


Figure 7: Collapse Load VS Wave Direction

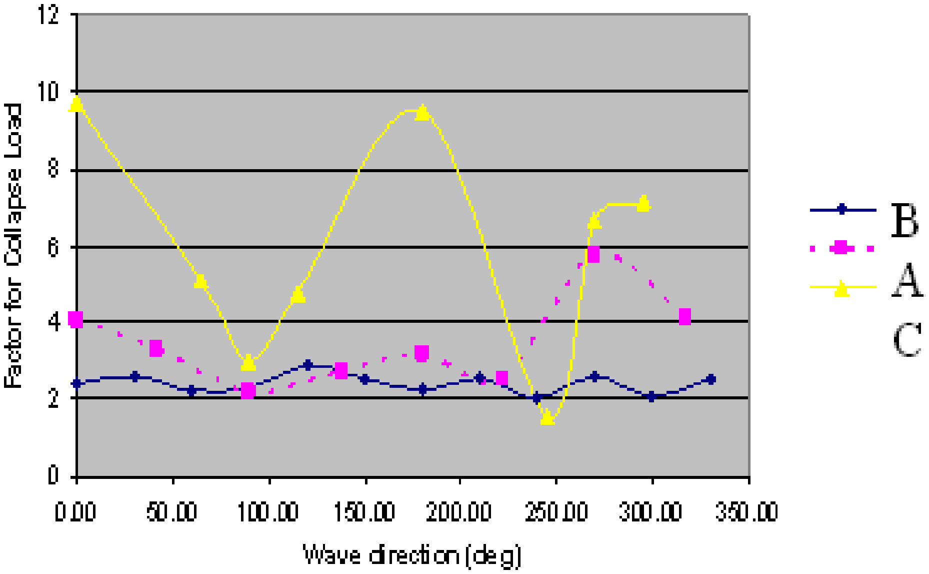


Figure 8: Collapse Load factor VS Wave Direction

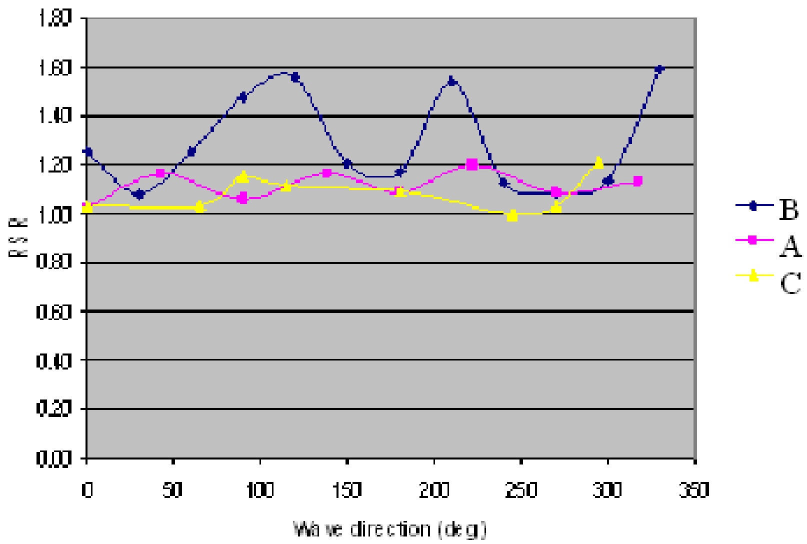


Figure 9: RSR VS Wave Direction

### *5.2 Bracing Configuration Study*

The different bracing configurations for platform A used for analysis are (1) X bracing (Figure 10), (2) Design basis (Figure 11) and (3) Single Diagonal bracing (Figure 12).

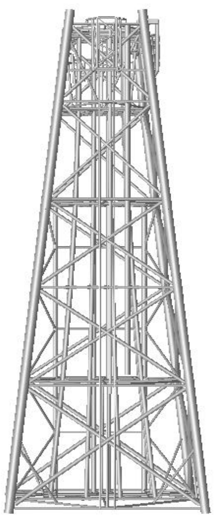

Figure 10: X Bracing

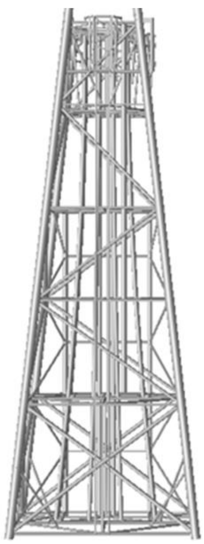

Figure 11: Design basis

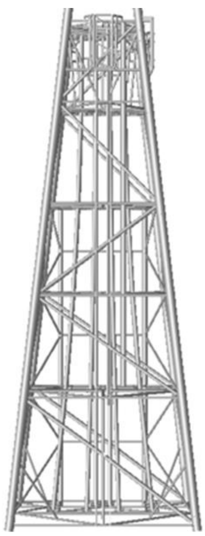

Figure 12: Single Diagonal Bracing

The design basis and the single diagonal bracing differ only in the top and bottom panel where the design basis platform has extra member as shown in Figure 9. The results of the analysis are presented in Table 9. The performance of the bracing schemes is assessed by the following plots which compares and differentiates them.

1. Collapse loads Vs Direction (Figure 13)
2. Collapse Load Factor Vs Direction (Figure 14)
3. RSR Vs Direction (Figure 15)

Table 9: Bracing Configuration Study for platform A (4 legged)

| Configuration | Wave Direction (deg) | First Member Fail Load (kN) | Collapse Load (kN) | RSR | Factor for Collapse |
|---|---|---|---|---|---|
| Design Basis | 0.00 | 9230.77 | 9488.49 | 1.03 | 4.05 |
| | 42.11 | 7844.51 | 9174.21 | 1.17 | 3.35 |
| | 90.00 | 12226.07 | 12974.80 | 1.06 | 2.18 |
| | 137.89 | 8140.04 | 9489.52 | 1.17 | 2.76 |
| | 180.00 | 10582.18 | 11465.45 | 1.08 | 3.17 |
| | 222.11 | 8396.79 | 10078.17 | 1.20 | 2.54 |
| | 270.00 | 10746.32 | 11690.44 | 1.09 | 5.72 |
| | 317.89 | 7944.94 | 9018.19 | 1.14 | 4.10 |
| **X-Bracing** | 0.00 | 10410.32 | 10630.12 | 1.02 | 4.27 |
| | 42.11 | 8311.26 | 9144.72 | 1.10 | 3.15 |
| | 90.00 | 11604.26 | 15280.47 | 1.32 | 2.44 |
| | 137.89 | 8449.42 | 10042.01 | 1.19 | 2.75 |
| | 180.00 | 11397.21 | 11584.19 | 1.02 | 3.02 |
| | 222.11 | 8049.95 | 10862.32 | 1.35 | 2.58 |
| | 270.00 | 10816.69 | 12445.34 | 1.15 | 5.76 |
| | 317.89 | 7960.63 | 9885.51 | 1.24 | 4.24 |
| **Single Diagonal** | 0.00 | 9215.76 | 9493.51 | 1.03 | 4.06 |
| | 42.11 | 7826.34 | 9029.29 | 1.15 | 3.31 |
| | 90.00 | 10424.99 | 11221.38 | 1.08 | 1.89 |
| | 137.89 | 8296.00 | 9125.70 | 1.10 | 2.66 |
| | 180.00 | 10557.71 | 11097.71 | 1.05 | 3.08 |
| | 222.11 | 8568.37 | 10600.58 | 1.24 | 2.68 |
| | 270.00 | 9683.77 | 9751.93 | 1.01 | 4.79 |
| | 317.89 | 7929.77 | 9527.65 | 1.20 | 4.34 |

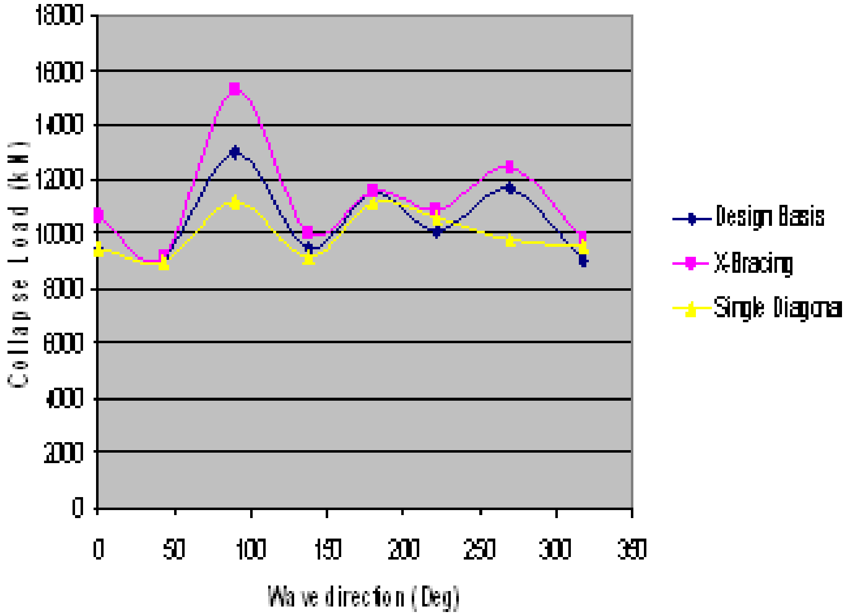


Figure 13: Collapse Load Vs Wave direction

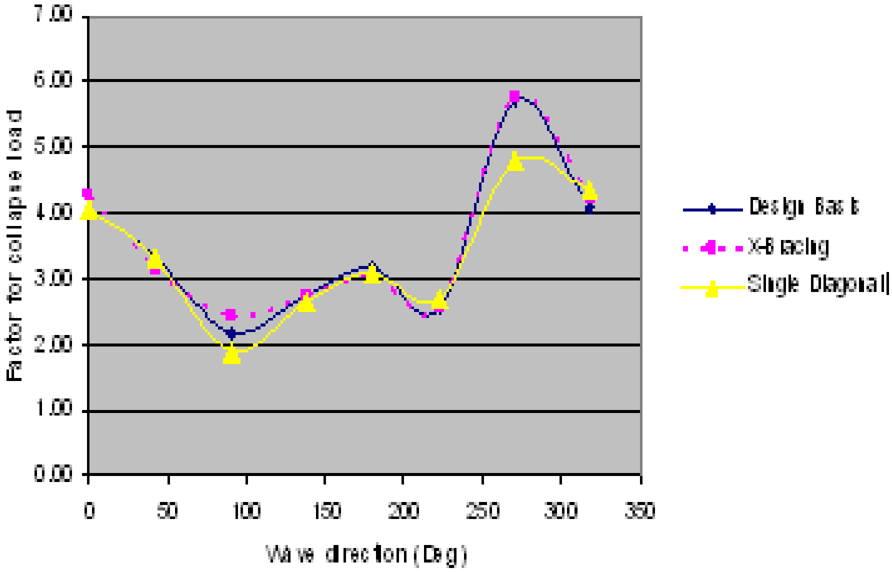


Figure 14: Collapse Load factor Vs Wave direction

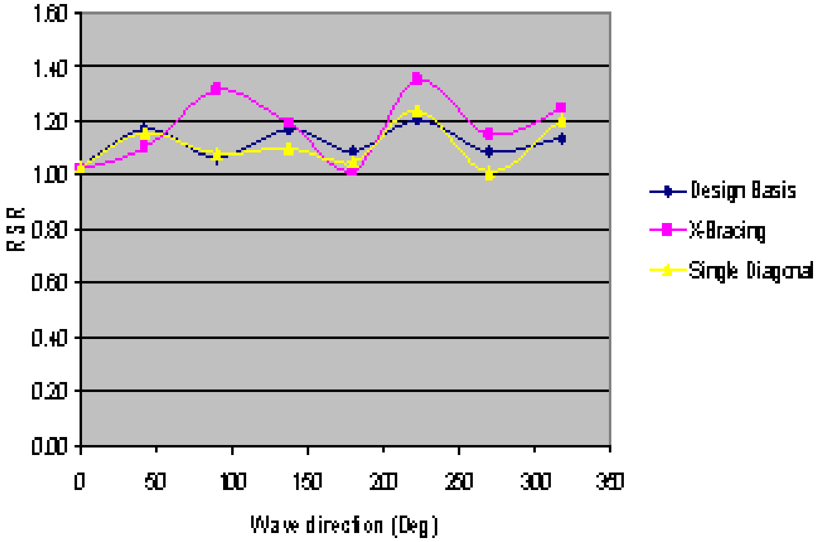


Figure 15: RSR Vs Wave direction

From Table 9 and Figure 13, it is evident that the X-bracing provide more stiffness for the platform A compared to other bracing types. X-bracing scheme provides additional load paths and redundancy to the substructure. At the angle of 90 degree, the collapse load for all bracing types had similar peak but differed in magnitude. Lesser ultimate loads are shown by single diagonal bracing where the framework created by a single cross member provide less efficient load paths for load to be shared. The single diagonal bracing had collapse loads close to the design basis but lower. This is due to the presence of additional members in the top and bottom panel in the design basis. RSR diagram (Figure 15) indicates that the X-bracing resulted in more strength reserve compared to the other bracing schemes. The bracing provided more reserve strength before the structure reached the critical point of ultimate load. The design basis had performance in between the X-braced platform and the Single diagonal braced platform. From Table 9 and Figures 14-15 the maximum and minimum performance of the bracing schemes studied and corresponding wave directions are summarized for platform A in Table 10.

From Table 10, it is seen that the platform has more stiffness for incoming wave at 90 degree. Higher loads are required to make the structure collapse from the 90 direction than the other directions. All bracing configurations provide with the highest load at wave direction of 90 degree which are parallel to individual base shear generated. So it makes sense to orient the platform’s strongest direction with the strongest wave load direction.

Interpreting the tabulated data above, the single diagonal is the weakest, X-bracing provide highest rigidity while the original or design basis is in between the X bracing and the diagonal bracing. The circumstances indicate that the design for platform A is adequate and effective to the environmental area of the site location. As for conclusion, the original design of platform A is cost effective and suitable with the surrounding area.

Table 10: Summary of bracing configuration study results

| Bracing Configuration | Collapse Load (kN) | | RSR | |
|---|---|---|---|---|
| | Minimum (direction) | Maximum (direction) | Minimum (direction) | Maximum (direction) |
| Design Basis | 9018.19 (317.89º) | 12974.80 (90.00º) | 1.03 (0.00º) | 1.20 (222.11º) |
| X-bracing | 9144.72 (42.11º) | 15280.47 (90.00º) | 1.02 (180.00º) | 1.35 (222.11º) |
| Single Diagonal Bracing | 9029.29 (42.11º) | 11221.38 (90.00º) | 1.01 (270.00º) | 1.20 (317.89º) |

Due to leg arrangement of platform A at ROW B, the other types of bracing like the K-bracing, the Inverted K-bracing and the Diamond bracing are inappropriate. The problem occurs where the Launch Cradle is used for sliding the jacket onto the barge used for installing the structure. With the launch cradle attached to the substructure, there was no horizontal member framing at designed elevation. The horizontal was offset by several dimensions to cater the launch cradle framing. With one face of the jacket structure not suitable for the other types of bracing, they were not chosen. The strength of the original jacket would be affected by adding horizontal member for the K member to intersect.

## 6.0 Conclusions and Recommendations

Platforms beyond design life need an assessment for the ultimate capacity for further service. A study was carried out to check the platform reliability for extended service. The first part of analysis clearly concluded that larger number of legs affect the overall strength. Platform C an eight-legged platform gave the highest ultimate load compared to the 4-legged platform and tripod. The Reserve Strength Ratio (RSR) of the platforms A, B and C ranged from 1.0 to 1.6 while the collapse load factor range from 2.0 to 9.69. The highest ultimate load among the platforms studied was 38437.97 kN reported for platform C at wave angle of 115.00º. The highest RSR among the platforms was for the platform B was computed as 1.56 at 120.00º.

Platforms A, B and C represent the major and common platforms installed in the country. The location of each platform at different oil blocks in the region was to mainly check the environment factor effect on the structures. The location of platform A and C in PMO and platform B in SKO are sufficient to cater to the differences between the two environment conditions.

The platform A was modified with different bracing schemes. The highest collapse load was achieved with X-bracing model at wave attack angle of 90 degree as 15280.47 kN. RSR computed at the load was 1.32. For platform A bracing schemes, the RSR ranged from 1.01 to 1.32 while the collapse load factor ranged from 2.18 to 5.76. The bracing type K-bracing, Inverted K-bracing and Diamond bracing were excluded from the study due to no horizontal member at specified elevation due to the installed launch cradle for transportation and lifting procedure

Recommendations for the future studies include collapse analysis for platform designed for API RP2A-LRFD and comparison with platform designed as per API RP2A WSD. With different code, the results are expected to be similar but slightly different in behaviour of the load paths and method of computation. In the API RP2A-LRFD approach to solution load factors are used in computing and there are also differences in the value of constants such as drag, inertia and others. Comparison of the results obtained with the LRFD and WSD code can help us to differentiate one code from the other. Furthermore, the trend of higher utilization in the LRFD code can be in analyzed using the collapse load.  The comparison provides better accuracy and also enables cost effective decisions for maintenance.

Alternate method of analysis of the models for Linear Static Analysis is the Simplified Ultimate Strength (SUS) Analysis. This analysis comprises of the same procedure by incrementing the load combination of storm wave until one the component fail or meets capacity namely joints, members, pile steel strength , and pile soil bearing capacity. The analysis is further done by removing the failed component to allow the alternative load paths which exist in the framework. The analysis is completed when the software cannot find solution meaning that no more load paths are available or exceed deformation. The corresponding load is the collapse load and the base shear generated by the directional waves.

In order to have better comparison, all types of conventional bracing should be evaluated including the K-bracing, Inverted K-bracing and Diamond bracing which were excluded in this analysis. For this, platforms which are appropriate for the bracings mentioned should be chosen. A study of three or eight legged platform can provide data on how bracing configuration affect such a configuration. Platform B and C can be taken for such a bracing configuration studies. The overall data can provide a comprehensive idea on the performance of different bracing schemes.

## 7.0 Acknowledgements

The authors acknowledge the use of the SACS program available at UTP as also the data on the three platforms provided for which the source is not revealed.

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