
Philip ESPER, BSc MSc PhD DrEng PEng MBGS MSECED
Technical Director, T A Millard Ltd.

Abstract

This paper discusses the major aspects that should be considered in the evaluation of seismic response of offshore structures through a case study of a concrete gravity substructure supporting a conventional steel Topside structure. It highlights the importance of selecting the most appropriate arrangement for the connection between the Topside and the Substructure and its effect on the seismic performance of the platform. The advantages of performing a detailed global 3-D non-linear analysis of the whole structure in order to predict its dynamic performance during a seismic event are discussed. The seismic analysis showed that the seismic performance of the platform is satisfactory, with plastic hinges developing in a small number of elements in the Topside.

Introduction

It is recommended by most offshore structures design codes, such as API-RP2A and DNV that an offshore structure operating in an area of significant seismicity must be designed to resist a “Strength Level Earthquake” (SLE) without yielding, and a “Ductility Level Earthquake” (DLE) without collapse. Typically the SLE has a return period of 200 years (according to API), and a DLE has a return period of 10,000 years (according to DNV). Except for certain jacket structures with eight or more legs, design codes such as API-RP2A do not provide specific guidelines on performance criteria for platform structures under the DLE loading. In general, it requires non-linear time history analysis to be performed using three or more appropriate seismic input motions to demonstrate that the platform does not collapse in a DLE event. However, In fact, using a proper finite element model which can take into account the 3-D distribution of the structural stiffness and mass of the platform, and if necessary too, the soil supporting it, can give more confidence in the seismic design of these structures.

The offshore platform discussed in this paper, consists of a steel Topside structure and a concrete gravity substructure (CGS), as illustrated in Figs.1 and 2. One of the most challenging problems encountered in the design of this platform is selecting the most appropriate scheme for the CGS-Topside connection in order to achieve a satisfactory seismic performance of both the CGS and the Topside of the platform, with regard to various design requirements and restrictions. For this purpose, the performance of the platform under a DLE event was investigated using four different deck connection schemes. These were, isolated connection, semi-pinned, semi-pinned with braces, and rigid connection. The seismic performance of the platform during a DLE event was studied by carrying out a global 3-D non-linear seismic response analysis. The global 3-D seismic analysis showed that the seismic performance of the platform is satisfactory, with plastic hinges developing in a small number of elements in the Topside.

The Platform Geometry

The concrete caisson of the CGS was approximately 110 m long (N-S), 80 m wide (E-W), and 16 m high. The caisson, which has both structural and storage functions, was divided by internal vertical walls into (9 X 6) as shown in Fig.2. The distance between the centres of the four prestressed concrete shafts was approximately 60 m in the N-S direction and 35 m in the E-W direction. Each shaft has an outer diameter of 11 m, a minimum thickness of 400 mm, and a total height of 40 m above the caisson roof. The entire Topside was supported by four tubular steel legs, which in turn were supported by the four concrete shafts of the CGS (see Fig.3). The four supporting steel legs of the Topside were 9 m high and had hollow circular cross-section. Their outer diameter was 2.0 m and their wall thickness was 65 mm. At the top end, each leg was stiffened by eight stiffeners, which extended over a length of 3.5 m (see Fig. 3). At the bottom end of the steel leg, each leg was stiffened by 12 stiffeners, which extended over a length of 2.0 m.
(see Fig.3). A number of risers, caissons and J-Tubes run between the CGS and the Topside. The total operating weight of the Topside was approximately 13,500 tonnes.

Figure 1. The Seismic Response Analysis Model of the Platform (LS-DYNA Model)

Figure 2. The Design Verification FE Model of the Platform (ASAS Model)
The CGS-Topside Connection Schemes

From the investigation carried out in the Front End Engineering Study (FEED), it was clear that the performance of both the CGS and the Topside under a DLE loading depends strongly on the scheme adopted for the Topside-to-CGS connection. A performance-based seismic engineering philosophy was adopted to carry out this study, where the maximum strength, plastic deformations and relative displacement demands of the primary structural members had to be quantified. The maximum acceleration demand of the Topside was needed also so that a comparison could be made between all these parameters in order to select the most suitable scheme for the deck connection. The results and discussion are presented in the next section, but a brief description of the four connection schemes, between the steel legs and the concrete shafts, which are illustrated in Table (1), is given below.

**Scheme (1): Isolated Connection**

A friction pendulum type of seismic isolators was considered in this study, which combines the capability of providing adequate vertical load support and introducing at the same time lateral flexibility and energy dissipation in the horizontal direction. The top part of this type of isolator starts to slide relative to the
bottom part if the applied shear force reaches the slip force, which is the product of the applied vertical gravity force and the friction coefficient (0.05-0.1 typically). Once sliding occurs, an additional restoring force (as well as the frictional force) is provided by gravity as the upper spherical surface rises up the lower spherical surface. The isolator lateral stiffness depends on the curvature of the spherical surface and quantifies the vibration period of the isolator. By adjusting the friction coefficient and the period, different isolator performances can be achieved. In the FE model, these isolators were represented by non-linear inelastic springs in ANSYS/LS-DYNA [Ref.1, 2 and 11], which are capable of simulating the force-relative displacement relationship.

**Scheme (2): Semi-pinned Connection**

The concept of this type of connection can be demonstrated as follows (see Fig.5); once the Topside is installed on top of the CGS, the gravity load of the Topside results in compressive contact stresses between the bottom end of the steel legs (the base plates) and the top surface of the concrete shafts. These gravity compressive contact stresses enable the joints to resist a certain amount of bending moment without requiring any holding-down bolts. The moment capacity is limited by the lift-off at the tension side of the joint. Once the contact stresses at the tension side reduces to zero, the base-plates start to lift up from this end and the compressive contact stresses increase at the opposite end, as shown in Fig.5. The moment resisting capacity of the joints, which approximately equals to the compressive axial force multiplied by the radius of the steel leg cross-section, remains almost constant after one end of the tube starts to lift up. In the FE model, these joints were represented by spherical joints in ANSYS/LS-DYNA [Ref.1, 2 and 11], which prevent any relative displacement between the two nodes linked by these joints, but can have user defined non-linear rotational stiffness.

![Figure 4. Axial Force-Bending Moment Interaction Envelope Curves](image-url)
Scheme (3): Semi-pinned Connection with Braces

The effects of introducing braces to the semi-pinned connection (described above) were two-fold; first, they make the connection scheme stiffer; and second, they dissipate energy by inelastic deformation of the braces under DLE loading. The braces (see sketch in Table-1) are structural steel members connected to steel legs at one end and to the nearest truss node of the cellar deck. In the FE model, these braces were represented by beam-column elements which were discretised to four elements along their length, thus having the potential to simulate non-linear buckling by the formation of plastic hinges at both ends and at mid-length.

Scheme (4): Rigid Connection

In this scheme, the deck connection was designed to be capable of resisting the maximum bending moments and axial forces delivered to the concrete shafts by the steel legs. The design of these joints followed the 'capacity design' procedure in order to ensure that the joints can resist potential over-strength of the legs without crushing of the concrete, or other failure modes. In the FE model these joints were modelled simply by assuming a full-strength link between the two nodes representing the joint.

The Global 3-D Seismic Response Analysis Model

A complete 3-D finite element (FE) model of the platform and the soil supporting it was built in order to evaluate the performance of the CGS and the Topside under the DLE loading, and to develop a scheme for the CGS-Topside connection. For this purpose Oasys LS-DYNA [Ref.1&2], which is a 3-D non-linear structural dynamic analysis software, was used. The program has been developed further and validated [Ref.5] by Ove Arup and Partners over the last ten years in order to simulate seismic response of soils and structures explicitly in the time domain. The FE mesh was selected and validated for this analysis, as discussed below, so that a sensible run-time was achieved without compromising the accuracy of the analysis results.

The floor plates of the Topside were modelled by shell elements. All the main beams and members of the Topside were modelled as beam elements, but not the risers, caissons and J-tubes, since these are non-structural members. All the primary joints in the Topside were assumed to have a strength capacity at least as high as that of the connecting members, in accordance with the design requirement of API-RP2A [Ref.9]. Axial force/bi-axial bending moment interaction was taken into account in determining the
formation of plastic hinges, and a Py-Mmxx-Mmzz interaction surface is established for each member (see Fig.4). The steel legs and the braces were divided into sufficient beam-column elements (at least four) in order to allow plastic hinges to form at both ends and at the centre of each member.

The stiffened segments of the legs were modelled by a series of short beam elements of 0.5 m long each. The tube section of the legs was discretised into 24 ‘fibres’ located at 15 degrees centres at the mid-surface of the tube wall and each fibre was assigned an area of 1/24 of the cross-section of that of the tube. Each stiffener was discretised into 2 ‘fibres’ located at the geometric centres of the two halves of each stiffener.

The concrete shafts were modelled as series of vertical elastic beam elements. Under the DLE loading, the shafts will crack, but will not yield. As a result, a cracked shaft cross-section stiffness, $E_{lc}$, of 50% of that of the gross un-cracked cross-section stiffness value, $E_{lg}$, was adopted. All four shafts were assumed to be flooded with seawater to the same elevation level as the seawater surface. The hydrodynamic effects of the external seawater on the shafts have been taken into account by adopting the “added mass” method proposed by Westergaard [Ref.4]. The internal water mass and the hydrodynamic “added mass” were included as part of the mass of the shafts by appropriately increasing the mass density of the shaft concrete material. The slabs and walls of the caisson have been modelled using shell elements.

The seabed soils consisted of a levelling course of gravel, over a layer of sand over soft rock. The thickness of the levelling course together with the layer of sand was approximately 4 m. Solid (brick) elements have been used to model the levelling course and the sand. Upper bound non-liquefied soil properties were assumed, since the maximum responses in the Topsider structure and in the shafts will occur with the stiffest and strongest foundation material. The upper bound value of the small strain shear modulus, $G_0$, was calculated to be 200 MPa. The shear stress-strain relationship for both the gravel and the sand is illustrated in Fig.6. In the FE model, the soil was represented by the following parameters, low strain shear modulus ($G_0$), strain degradation curve ($G/G_0$), Poisson’s ratio ($\nu$) and unit weight ($\gamma$). The soil model exhibited hysteretic behaviour in accordance with the Masing principles. Further analyses were performed with lower bound soil properties to estimate the maximum sliding movement of the caisson.

![Image of Sand Shear Stress-Shear Strain Relationship](image_url)

**Figure 6. Shear Stress-Strain Relationship of the Surficial Sand**
Damping And Earthquake Strong Motion Input

Structural damping depends on the stress amplitudes in the Topside steel members and the extent of cracking in the concrete shafts. Under the DLE loading, the structural damping was to be 4% of critical for all significant modes. In this analysis, damping was mathematically modelled by introducing damping forces acting on the nodes in ANSYS / LS-DYNA based on the modal damping matrix. Additional hysteretic damping in the structure to represent the energy dissipated due to plastic deformation of structural members and the underlying soil was considered. Hysteretic damping was simulated in the element formulation of structural steel members as they undergo plastic deformation. Hysteretic damping of the soil was automatically represented by the non-linear stress-strain relationship and loading / unloading hysteresis loops of the soil material model. Radiation damping has been ignored due to the large difference in stiffness between the sand and the underlying rock.

Three sets of artificial earthquake records were generated for the purpose of this study to simulate a DLE event from a site-specific seismic hazard study, each set consisting of three components (tri-axial) of velocity time-histories (see Fig.7). The three ground motion components were along the E-W, N-S, and Vertical directions, respectively. The peak ground acceleration of the horizontal components of these records was approximately 0.4g and that of the vertical component was 0.27g.

Analysis Procedure And Results

For each connection scheme, a modal analysis was carried out to obtain the free vibration modes and frequencies of the platform. A two-phase analysis procedure was applied in carrying out the non-linear seismic response analyses. Firstly, an analysis was carried out to obtain the stress state for the platform structure under its self-weight or gravity load. The second phase, which is namely the non-linear transient seismic response analysis, is then performed by applying the ground motion excitations. The maximum strength, plastic deformations and relative displacements demands of the primary structural members and the maximum acceleration demands of the Topside decks were quantified for all four Deck Connection schemes and are summarised in Table (1). The analysis results and the reasons behind selecting a semi-pinned joint for the platform are discussed in the next Section of this paper. Fig.8 illustrates the first four mode shapes and frequencies of the platform, for the semi-pinned joint case. In the first mode, which has a frequency of 0.73 Hz, the Topside and the four shafts vibrate horizontally in the E-W direction coupled with plan rotation. The torsional coupling was due to the eccentricity between the centre of stiffness of the four identical shafts and the centre of mass of the Topside. The fundamental period of the platform was therefore, approximately 1.4 seconds. In the second mode with a frequency of 0.75 Hz, the Topside and the four shafts vibrated horizontally in the N-S direction without torsional coupling. The third mode, which has a frequency of 0.95 Hz, shows the Topside and the four shafts vibrating rotationally coupled with translation. Finally, in the fourth mode, the flare boom vibrates vertically at a frequency of 1.2 Hz. The maximum horizontal resultant accelerations at various locations were obtained based on the time histories of the horizontal acceleration components in the X- and Y- directions.
Discussion

The Deck Connection Scheme

The deck connection schemes in Table (1) are arranged such that the stiffness of the connection scheme increases from left to right, with the isolation scheme, being the most flexible and the rigid connection scheme, being the most stiff, as the two extremes. The trade-off between the maximum deck acceleration and the maximum deck-to-shaft relative displacement can clearly be observed:

1) The isolation scheme led to the lowest maximum deck acceleration of approximately 0.3g in the E-W direction and 0.25g in the N-S direction. However, it resulted in the highest deck-to-top of shaft relative displacement (approximately 400 mm) in both horizontal directions. This large displacement was mainly due to the large relative displacement across the isolators, which was approximately 330 mm in total.

2) In the rigid joint scheme the maximum deck acceleration was approximately 0.7g in both horizontal directions, and the maximum deck-to-top of shaft displacement was approximately 150 mm in the E-W direction and 90 mm in the N-S direction.

3) The ‘semi-pinned’ and the ‘semi-pinned with braces’ connection schemes lead to similar maximum deck acceleration values. However, the maximum deck-to-top of shaft displacement corresponding to the scheme with braces is significantly lower. The effects of the diagonal braces were two fold; firstly, they stiffen the connection by acting as braces to the legs; and secondly, they act as
dampers by dissipating energy due to plastic hinge rotations. Plastic hinges developed at both ends, as well as at the middle of the braces due to buckling.

4) Both the maximum deck acceleration and the maximum Deck to top of Shaft displacement were important response parameters affecting the performance of equipment, vessels, and pipelines. On one hand, a low maximum deck acceleration was desirable for the vessels and equipment, but on the other hand, a small deck-to-top of shaft displacement was desirable for the risers and caissons. A balanced evaluation of the effects of the above response parameters under DLE loading, together with constructional preferences, resulted in selecting the ‘semi-pinned’ scheme for the deck connection.

**The Performance of the ‘Semi-Pinned’ Connection Platform**

In terms of the adequacy of the design of the platform with the ‘semi-pinned’ connection, in general, the following points can be made:

i) The maximum resultant horizontal acceleration was below the required maximum horizontal acceleration of 0.65g. Therefore, in terms of the maximum deck horizontal acceleration, the performance of this platform was considered satisfactory under the DLE loading.

ii) The Topside structure maintained stability without collapse throughout the entire duration of the earthquake. The plastic hinge rotations at the top of the legs and in the other yielding Topside members were not excessive. Hence, in terms of collapse prevention and maintaining stability, the performance of the platform was also satisfactory under the DLE loading.

iii) Local buckling of the steel legs was prevented by the provision of suitable stiffeners. This is discussed in Section (5) below.

iv) The performance of the concrete shafts was found to be satisfactory. The maximum resultant bending moment of approximately 1164 MNm at the bottom of the shafts is well below the yielding moment of the shafts at their bottom cross-sections, but significantly higher than the cracking moment.

**Conclusions**

In summary, the global FE seismic non-linear analysis was the tool that predicted the performance of the whole platform, including the CGS, the Topside and the deck connection, under a DLE event. A balanced evaluation of the effects of the response parameters under DLE loading, together with constructional preferences, resulted in selecting the ‘semi-pinned’ scheme for the deck connection.

**References**


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<th>Response Parameter</th>
<th>Topside-CGS Connection Scheme</th>
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<td>(1) Deck Acceleration</td>
<td>0.30g (X) 0.25g (Y)</td>
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<td>(2) Cellar Deck-top of Shaft Displacement</td>
<td>400 mm (X) 400 mm (Y)</td>
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<td>(3) Bottom of leg shear force</td>
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<td>(4) Bottom of leg bending moment</td>
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<td>(5) Bending moment in the legs at the Leg-Cellar Deck joints</td>
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<td>(6) Locations of plastic hinges developed</td>
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<td>(9) Max plastic hinge rotation</td>
<td>0.0</td>
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