Seismic Soil-Structure Interaction Analysis of Steel Gravity Structure Considering Nonlinear Foundation Response

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ABSTRACT
Metocean loads typically govern the design of steel-jacket fixed offshore structures in zones of moderate seismicity. In contrast, the steel gravity structure (SGS) presented in this paper is heavy and stiff. The large mass results in foundation forces from seismic events that may exceed those created by extreme cyclonic storm events. When computing the earthquake response of such structures, it is essential to account for soil-structure interaction (SSI) effects incorporating nonlinear soil behavior.

Seismic SSI analysis of the SGS platform was performed using an advanced version of the SASSI program. A detailed three-dimensional model of the SGS supported on horizontally layered soil was developed. Primary soil nonlinearity in the free field was accounted for through one-dimensional site response analysis. To account for soil consolidation under the self-weight of the structure and its secondary nonlinear behavior under an Abnormal Level Event (ALE) in the SSI analysis, a portion of the soil under the foundation pads was modeled as part of the structure. An iterative scheme using the equivalent linear method was used to iterate on the soil properties in the soil block until the soil shear modulus and damping ratio were compatible with the level of effective shear strain in each soil element.

This paper presents the analytical procedure to develop the initial properties of the consolidated soil block and perform nonlinear SSI analysis with simultaneous application of three orthogonal components of free-field input motions to develop strain-compatible dynamic soil properties under ALE. The results of the SSI analysis in terms of the global foundation demand (base normal and shear forces as well as overturning moment) are compared against those that do not directly account for the secondary nonlinearities in the SSI analysis.

1 INTRODUCTION

Realistic seismic SSI response analysis of large and heavy offshore gravity-based structures requires proper modeling of the nonlinear behavior of the foundation soils subjected to cyclic shear strains due to strong ground shaking. Consideration of soil nonlinearity in SSI analysis primarily depends on the type of analysis, which falls into two principal categories: the direct method and impedance method. In the direct method, the structure(s) and the soil are included in one large coupled finite element (FE) model and the dynamic solution is obtained in the time domain; thus, allowing explicit consideration of soil or structural nonlinearity [Lubkowaski et.el. 2004]. The disadvantage of this method is that a very large FE model is required which will mostly consist of soil elements to place the model boundaries sufficiently far from the structure to ensure that spurious wave reflections at the boundaries do not affect the response of the structure. Thus, the structural part of the model, which is the main reason for the analysis must be represented by a simplified model. The impedance model is a sub-structuring method whereby the structure(s) and the soil are partitioned from the total SSI model and solved in three steps: solving kinematics effects by determining the foundation response, the impedance model by establishing the dynamic stiffness of the foundation and the coupled structural/foundation model [Kausel and Rosset 1975]. In the sub-structuring approach, the soil nonlinearity is considered in two parts: the primary nonlinearity due to free field site response and secondary nonlinearity due to SSI effects. This provides a numerically efficient method for analysis of large and detailed soil/structure systems.

In the conventional substructuring methods, the secondary soil nonlinearity is often ignored because it would require separate nonlinear analysis of the kinematic and impedance cavity models. In the current study, an innovative substructuring method referred to as the Flexible Volume Method (FVM) [Tabatabaie 2014] is used which enables the secondary nonlinearity to be considered in the SSI analysis by including soil blocks as part of the structure. The properties of soil blocks are adjusted for soil consolidation and then monitored through an iterative scheme using the equivalent linear method. The use of soil blocks in the FVM provides a numerically powerful and efficient method for considering secondary soil nonlinearity in the SSI analysis by eliminating the need for solving nonlinear kinematic and impedance cavity model.

The SGS was analyzed for the Abnormal Level Earthquake (ALE) with and without soil blocks to evaluate the effects of secondary soil nonlinearity on the foundation demands (forces and moments). The SSI response analyses were performed for 3 soil cases covering the lower bound, mean and upper bound properties and 4 selected controlling ground motions out of 9 considered for design. This paper presents the methodology, modeling and analysis details and summary of the results and conclusions.

2 METHODOLOGY

The SSI analysis was performed using the Flexible Volume method as implemented in the SASSI program [Lysmer, et.al. 1981]. Per this methodology, the complete SSI system [see Fig. 1(a)] is partitioned into two substructures, called the Foundation and Structure [see Figs. 1(b) and 1(c), respectively]. The Structure consists of the original structure minus the excavated soil (i.e., the soil to be excavated is retained within the foundation, leaving the soil media as a horizontally layered system). The interaction between the soil and foundation occurs at all excavated soil nodes. The Foundation is analyzed first to establish the foundation dynamic impedance at all interaction nodes. The scattering problem is reduced to a site response problem due to the way the Substructuring formulation is carried out. This greatly simplifies the general substructuring procedure whereby the foundation impedance and site response solutions are used as boundary condition to analyze the dynamic response of the Structure.

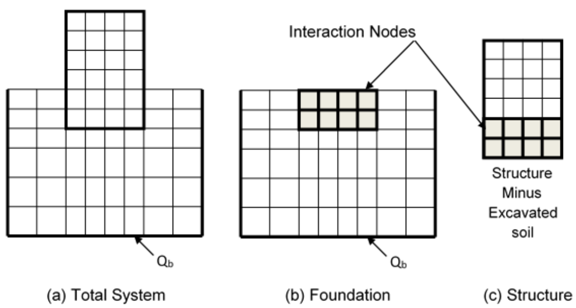

Figure 1 – Substructuring of Interaction Model

FVM is formulated in the frequency domain using complex frequency response method and finite element technique. This provides a convenient method to account for soil nonlinearity in the analysis through an iterative procedure using the equivalent linear method. The nonlinear soil properties consist of strain-dependent shear modulus and material damping. These properties are established for each soil type based on the soil effective shear strength and are assigned to each soil layer/element in the analysis model. A series of analysis iterations are then performed whereby the soil properties (shear modulus and material damping) are updated based on the calculated effective shear strain in each soil layer/element until convergence. This procedure is used in the SHAKE program for 1-D site response analysis [SHAKE] and implemented in an advanced version of the SASSI program for 2- and 3-D SSI analysis [MTR/SASSI].

To account for the soil nonlinear behavior in the analysis, a two-step approach is used. The primary soil nonlinearity in the free field is first analyzed from 1-D site response model using the SHAKE program. The resulting strain-compatible soil shear modulus and damping are then inputted to the layered soil system used in the SSI model. To account for the secondary soil nonlinearity due to the SSI effects, a portion of the foundation soil, called soil blocks (SB) is modeled with finite element and included as part of the structure (see Figure 2). The nonlinear properties of soil elements within the soil block are calculated by scaling the strain-dependent shear modulus and damping for the respective soil type based on the increase in soil confining pressure in each element due to consolidation under the self-weight of the structure. The size of the soil block is thus controlled by the zone of the structure influence as well as the significance of increase in soil strains due to the inertial feedback of the structure from seismic shaking. The shear strains from the primary and secondary nonlinear effects are assumed to be additive. This is a valid assumption when using the equivalent linear method since the goal is not to predict permanent foundation deformations. It is noted that the soil shear strains outside the soil blocks (i.e. strain-compatible soil properties from SHAKE analysis assigned to the soil layer system) are not expected to be affected significantly due to the presence of the structure and thus, are kept constant. It is only the shear strains within the soil blocks that are iterated on from the coupled soil/structure analysis and should converge to those of the layered system at the boundaries of the soil block. Furthermore, because the soil blocks are part of the structure, any reanalysis with new updated soil block properties does not affect the results of impedance analysis; i.e. it will not be complete analysis but rather a restart analysis with new structure model. Therefore, iterating on the soil block properties are performed at significantly less computational effort as compared to complete analysis.

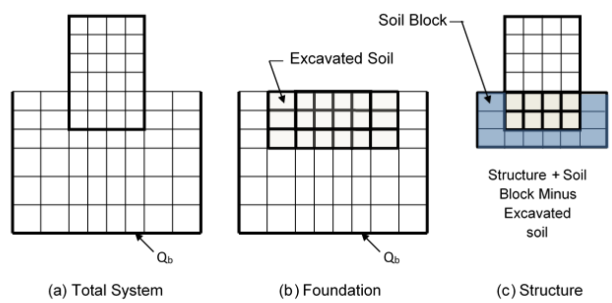

Figure 2 – Substructuring of Interaction Model with SB

2.1 Structure Description

The SGS platform consists of a single integrated topside facility of approximately 35,000 tons, and a steel gravity substructure weighing approximately 22,000 tons installed in 70m water depth. The platform is constructed offshore, and floated to its final position where it is installed and ballasted in on a rock blanket 0.5 to 1m thick. The SGS base footprint is about 75 by 103 meters, with four square foundations (24 x 24 meters) resting on the rock blanket overlying in-situ soils. The foundation mats are connected by four horizontal pontoons. Only the four foundation mats contact the seabed, with each foundation located centrally beneath each of the corner columns. The four SGS columns provide support for the topsides, which incorporate a flare boom, production equipment and the living quarters. The foundation stability during seismic event is a major concern for this construction and is the focus of this paper. To achieve adequate foundation stability, approximately 120,000 metric ton of solid ballast are used to prevent foundation movement during seismic and cyclonic storm events. Additional details of the SGS can be found in [Tajirian, et.al. 2014].

2.2 Model Development

A detailed finite element model of the SGS platform comprising the steel substructure and topsides was provided in the ANSYS program [ANSYS] (see Figure 3). The first step was to simplify the steel substructure and convert the new model "as is" to MTR/SASSI. The model translation was validated by performing gravity load and dynamic response analysis of the fixed-base structure and comparing the results. The next step was to add the 24m x 24m foundation pads and soil layers to complete the SSI model, as shown in Figure 4. To model the secondary soil nonlinearity, a second SSI model was then prepared by adding the rock blanket and soil blocks below the foundation pads, as shown in Figure 5.

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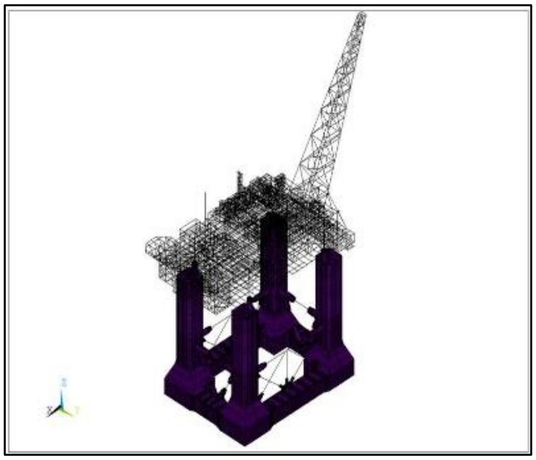

Figure 3 – Detailed SGS Structural Model

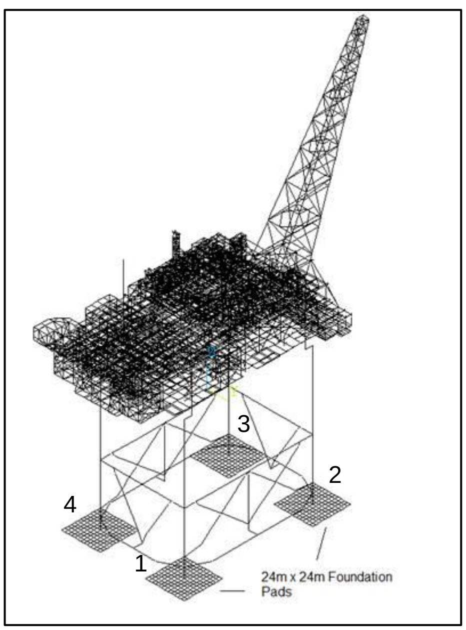

Figure 4 – Simplified SGS SSI Model

The hydrodynamic mass, which is the mass assumed to move in unison with the submerged structural members, is the full mass of water displaced by the structure. This mass is applied as element added mass.

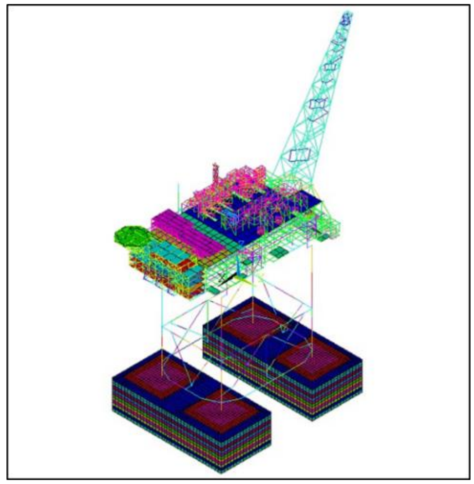

Figure 5 – Simplified SGS / Soil Block SSI Model

2.3 Seismic Design Ground Motions

A probabilistic seismic hazard assessment (PSHA) was performed for the SGS site. The structure is analyzed for two design level earthquakes, the ELE (Extreme Level Earthquake) with a return period of 500 years; and the ALE (Abnormal Level Earthquake) with a return period of 3000 years. The 5%-damped design spectra were developed at the ground surface for site class C conditions (see Figure 6). The peak ground accelerations for the ELE and ALE are 0.1g and 0.28g, respectively. The vertical design spectra were assumed to be half the horizontal spectra. Two separate nine sets of seed acceleration time histories were selected and spectrally matched to the 5 percent damped ELE and ALE design spectra. In the SSI analysis, the input time histories are specified at the mudline. This paper presents the results of the ALE analyses.

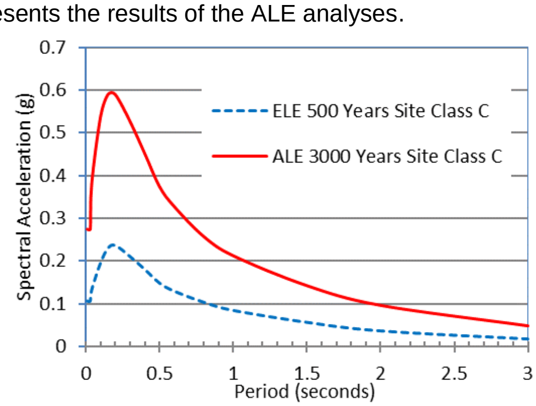

Figure 6 – Horizontal Design Spectra (5% Damping)

3 SSI ANALYSIS STEPS

The SSI analysis consisted of developing the dynamic soil properties, which included the low-strain shear modulus and nonlinear strain-dependent shear modulus and damping curves. These properties were developed for the free-field soil layers and soil block finite elements. Then iterating on the soil properties to develop strain-compatible soil shear modulus and damping, first in the free field and then in the SSI analyses. The development of dynamic soil properties and performance of free-field site response and 3-D SSI analyses for the SGS are detailed below.

3.1 Development of Dynamic Soil Properties

3.1.1 Low-Strain Soil Properties

3.1.1.1 Free-field

Site-specific CPT data were used to develop median low-strain soil shear modulus based on the correlation between normalized shear modulus and cone resistance proposed by Lunne et. al (1997). To account for the variability and epistemic uncertainty in the soil properties, three soil cases incorporating the mean (ME), lower bound (LB) and upper bound (UB) soil properties were used in the seismic response analysis. Equation 1 provides the low-strain shear modulus, G_{max} for the three soil cases where q_c is cone resistance in MPa, σ'_{vo} is effective vertical stress, P_a is atmospheric pressure 101.324 kPa and α = 900, 1800 and 3,600 for the LB, ME and UB soil cases, respectively.

$$G_{max} = \alpha\, q_c\, [(q_c / P_a)\, \sqrt{(P_a / \sigma'_{vo})}]^{-0.9} \qquad [1]$$

For the one-dimensional site response analysis, σ'_{vo} is calculated at the center of each soil layer using a saturated density of 21.58 kN/m^3. The measured cone penetration resistance varied with depth ranging from 68.5 MPa at 5m to 27.3 MPa at 50 m below the mudline.

3.1.1.2 Soil Block Initial Properties

For each soil element within the soil blocks, the low-strain shear modulus was calculated from consolidated soil stresses obtained from a gravity load analysis using MTR/SASSI. Figure 5 shows the gravity load model, which is the same as that used for seismic response analysis except that the soil stiffness was calculated using low-strain soil shear modulus obtained from Equation 1 and drained Poisson's ratio. The vertical stress in each soil element obtained from the gravity load analysis was then added to the effective overburden stress to calculate σ'_{vo}, which was then substituted in Equation 1 to calculate G_{max}. The low-strain shear modulus for the rock blanket obtained from laboratory testing of reconstituted samples was 167.2, 250.4 and 333.5 MPa for the LB, ME and UB soil cases, respectively.

3.1.2 Nonlinear Strain-Dependent Soil Properties

Nonlinear properties consisting of the soil shear modulus versus effective shear strain at several depths below the mudline (Stewart, et. al, 2008) and are shown in Fig. 7 for the Mean soil case. For the damping versus effective shear strain relationship, a single curve was developed and used for all layers, as shown in Figure 8. Similar properties for the rock blanket developed from laboratory cyclic shear tests performed on reconstituted rock blanket samples are shown in Figures 9 and 10, respectively.

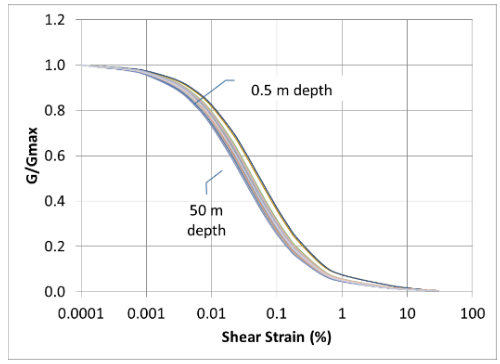

Figure 7 – Strain-Dependent Shear Modulus, Mean Soil

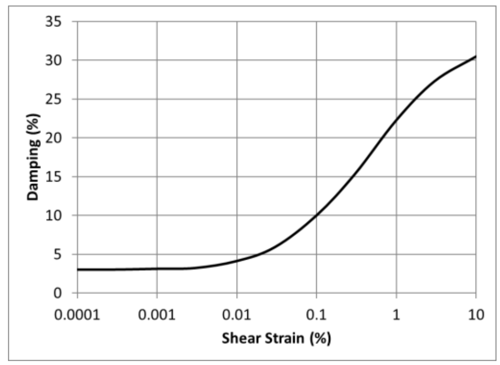

Figure 8 – Strain-Dependent Damping, Mean Soil

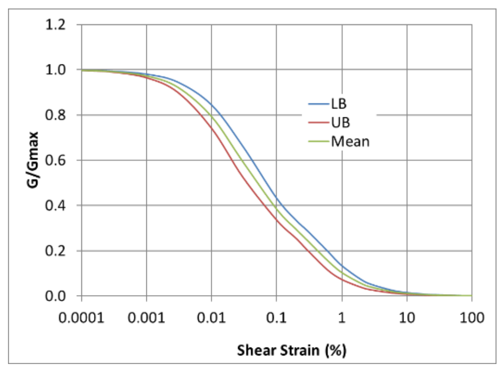

Figure 9 – Strain-Dependent Shear Modulus, Rock Blanket

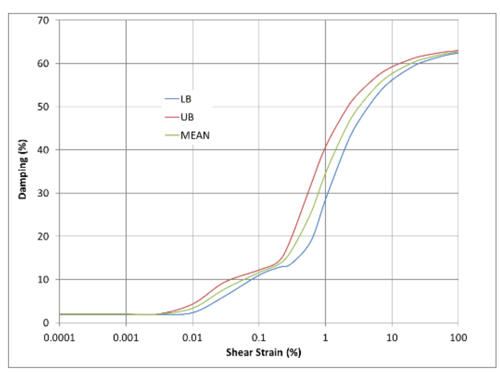

Figure 10 – Strain-Dependent Damping, Rock Blanket

3.2 One-Dimensional Site Response Analysis

Fifty-four one-dimensional SHAKE analyses (3 soil cases x 2 horizontal components x 9 ground motions) were performed for the ALE design case to establish median-centered, strain-compatible soil shear modulus and damping profiles. The input to this analysis consisted of soil density, low-strain shear moduli (G_{max}) and nonlinear G/G_{max} and damping curves, as discussed above. For the vertical response analysis, no further soil stiffness degradation is assumed (i.e. the SHAKE analyses were performed assuming the final strain-compatible properties obtained from the horizontal response analysis). Rock blanket is not part of the free field; therefore, was not included in the SHAKE model.

3.3 Three-Dimensional SSI Analysis

Two models were prepared for the SSI analysis using MTR/SASSI. The first model incorporates detailed topsides and simplified steel substructure with four foundation pads modeled with shell elements supported on a horizontally layered site over uniform halfspace. Strain-compatible soil properties obtained from the SHAKE analyses were used to characterize the site. This model accounts for the primary soil nonlinearity with no further iterations performed in the SSI analysis (i.e. the effects of secondary soil nonlinearity are ignored). The second model is the same as the first model except that the rock blanket and soil blocks were added to model secondary soil nonlinearity, as discussed above. The effective shear strain in each soil element in the rock blanket and soil blocks were monitored and their properties (shear moduli and damping) were adjusted accordingly after each iteration until convergence. In general, 6 to 7 iterations were necessary to obtain convergence within the specified tolerance. Initially, a ratio of uniform shear strain to maximum shear strain (uniform strain ratio) equal to F=0.52 was calculated from Equation 2 (where M=6.2 is the earthquake magnitude), as recommended in SHAKE91 and used for iterating on soil properties. This value was later calibrated to 1.0, which was found to provide better match with input shear stress-shear strain relationship, as discussed later.

$$F = (M - 1) / 10 \qquad [2]$$

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The SSI models were analyzed for four controlling input ground motions identified from ELE design analysis. These were Fremont, Hector, UCLA and Rionero events. The SSI models were subjected to a three-dimensional seismic environment consisting of vertically propagating shear and compression waves with the control motion specified in the free field at the mudline level. The control motion consisted of three components of the input motions applied simultaneously in the global XYZ directions.

4 DISCUSSION OF RESULTS

The results of one-dimensional site response analyses in terms of the calculated strain-dependent effective shear strain, shear modulus, material damping and maximum acceleration profiles for the Mean soil case for 18 horizontal components of input motions are shown in Figs. 11 through 14, respectively. Also, shown in the above figures are the median-centered response values. The low-strain shear modulus profile in Figure 12 is shown for comparison. A comparison of the low-strain versus median-centered strain-compatible shear modulus shows significant degradation of soil shear stiffness due to primary soil nonlinearity.

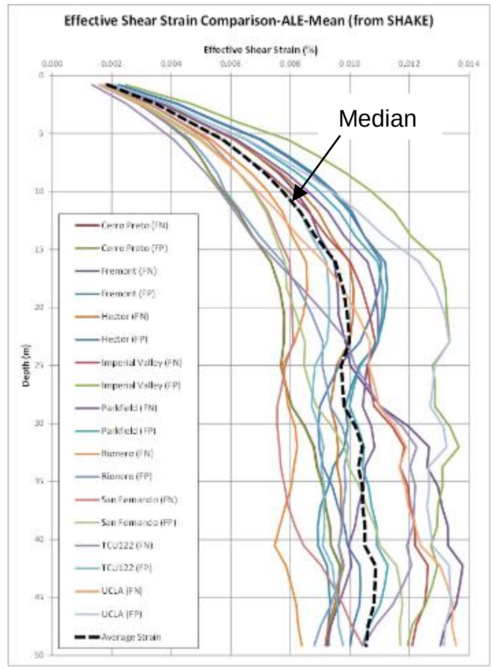

Figure 11 – Strain-Compatible Effective Shear Strain Profile, ALE, Mean Soil

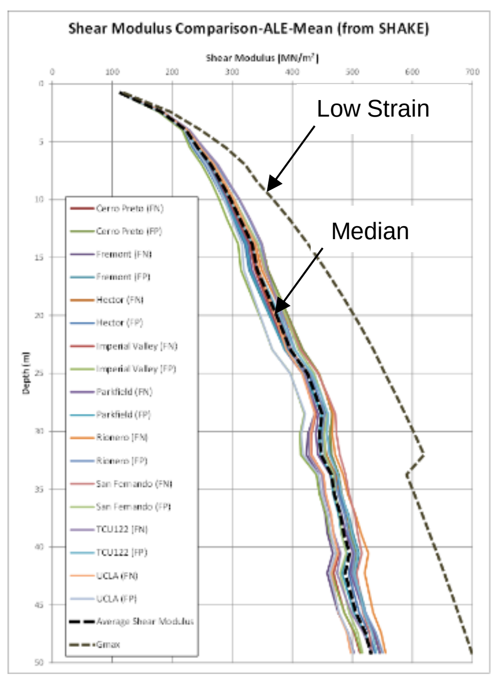

Figure 12 – Comparison of Strain-Compatible vs. Low-Strain Shear Modulus, ALE, Mean Soil

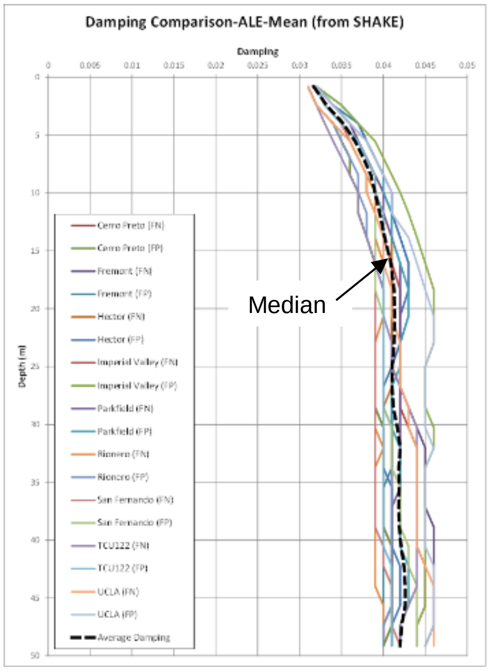

Figure 13 – Strain-Compatible Material Damping Profile, ALE, Mean Soil

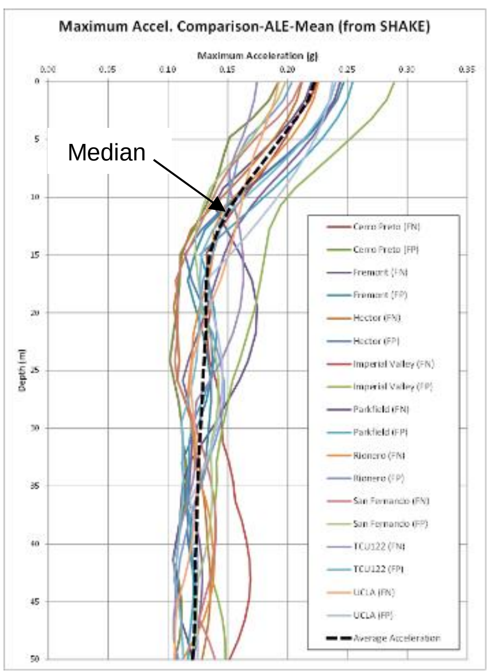

Figure 14 – Maximum Acceleration Profile, ALE, Mean Soil

The median strain-compatible soil properties calculated above were assigned to the layered soil system in the simplified SGS SSI model (Figure 4). This model was then analyzed without further adjusting the soil properties. For the simplified SGS/Soil Block SSI model (see Figure 5), the soil properties were assigned as follows:

1. The median strain-compatible soil properties (see Figures 12 and 13) were assigned to the layered soil system. These properties were also assigned as initial estimated soil properties for the soil block elements in the respective layer.
2. The effective vertical stress for each soil element was then obtained from gravity load analyses and added to the corresponding effective overburden stress and used to calculate the low-strain shear modulus, G_{max} from Equation 1 for that soil in the soil block.
3. Nonlinear G/G_{max} and damping curves for each soil element were adopted from the respective soil layer, as shown in Figures 12 and 13 for the soil and Figures 14 and 15 for the rock blanket, respectively.

Using the above properties, the SSI analysis of the SGS/Soil Block model was performed. In this analysis, the soil properties within the soil blocks were iterated on until the calculated shear modulus and damping for each soil element was compatible with the effective shear strain induced from seismic shaking. No iterations are performed on the layered soil system that represents the soil media outside the soil blocks.

In evaluating the SGS/Soil Block SSI model results, the following observations are made:

1. Soil consolidation due to self-weight of the structure increases the soil shear stiffness under the foundation pads. Foundation soils then undergo further softening due to secondary nonlinearity during seismic shaking. The net effect is higher soil stiffness under the pads as compared to the free-field conditions that do not see the same increase in stiffness due to low confining pressure (see Figures 15 through 18 for typical distribution of shear moduli within soil blocks).

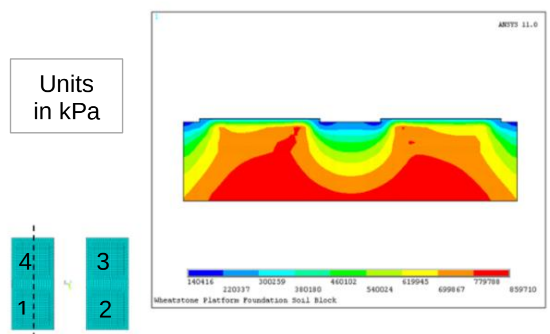

Figure 15 – Strain-Compatible Shear Modulus, Soil Block

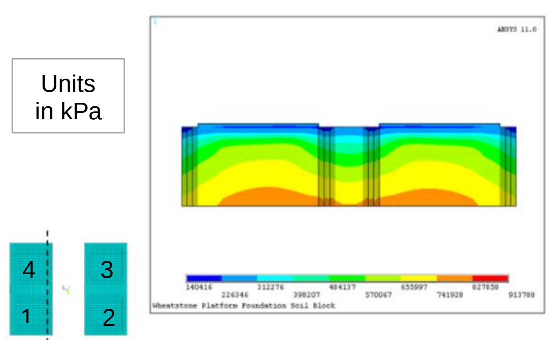

Figure 16 – Strain-Compatible Shear Modulus, Soil Block

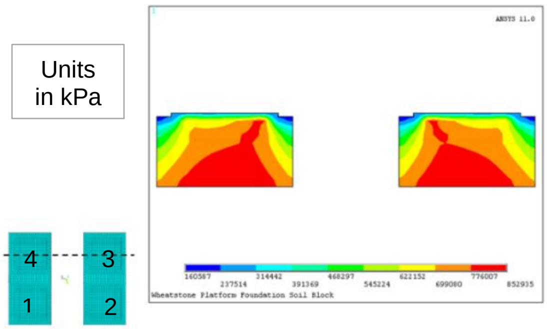

Figure 17 – Strain-Compatible Shear Modulus, Soil Block

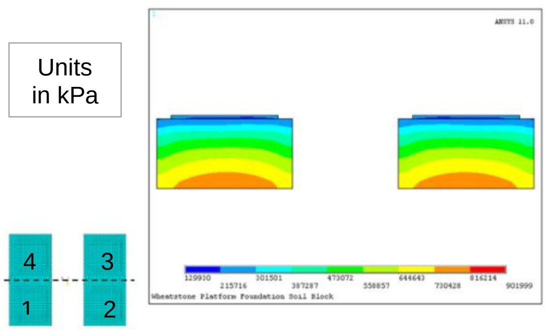

Figure 18 – Strain-Compatible Shear Modulus, Soil Block

2. The median maximum shear stress profile calculated in the soil block below the center, edge and halfway between the center and edge of foundation pad 1 (see Figure 4) are shown for two uniform cyclic strain ratios of F=0.52 and F=1.0 in Figure 19, and 20 respectively. Comparison of the results in Figure 19 and 20 show similar distribution of shear stress using F=0.52 and 1.0. Nonetheless, when pairing the stress and strain values at the time of maximum stress, the results using F=1.0 in general come closer to the input stress-strain curves as compared to those using F=0.52 (see Fig. 21). Therefore, for calculating the foundation demands, the result using uniform strain ratio of 1.0 were adopted.

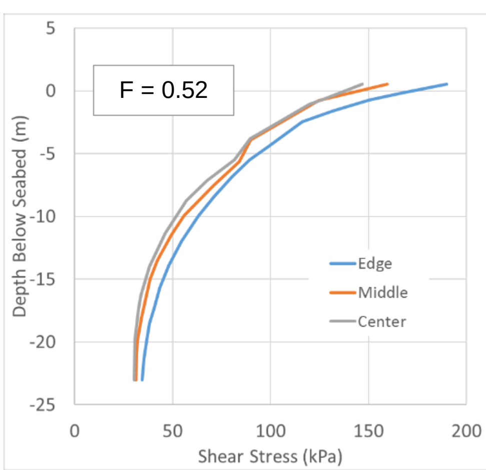

Figure 19 – Maximum Shear Stress-Shear Strain Curve, Mean Soil Case, F=0.52

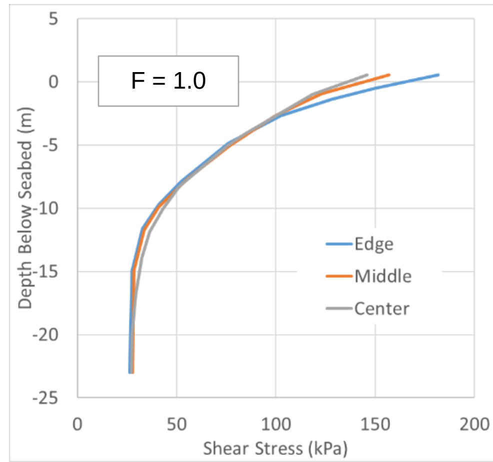

Figure 20 – Maximum Shear Stress-Shear Strain Curve, Mean Soil case, F=1.0

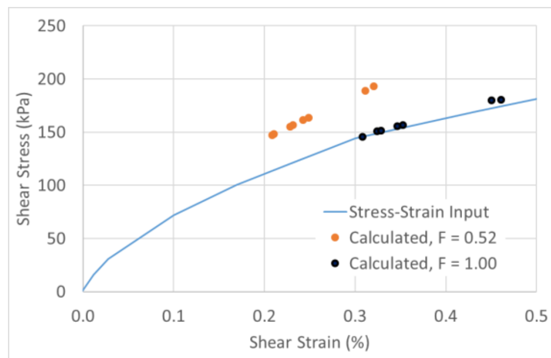

Figure 21 – Maximum Shear Stress-Shear Strain Curve

The results of SSI analysis of the simplified SGS model without and with soil blocks in terms of the total global foundation demand (normal and shear forces and overturning moments) for the four controlling ALE ground motions are presented in Table 1 and 2, respectively. As shown in Table1 and 2, the calculated foundation demand for SGS with soil blocks (i.e. including the effects of increase in soil confining pressure due to self-weight of the structure and secondary soil nonlinearity) is higher than those without soil blocks (i.e. ignoring those factors) for all 4 earthquake scenarios. The maximum increase in foundation demand is about 30, 25 and 60 percent for the normal force, shear force, and overturning moment, respectively. The largest increase in the normal force and overturning moment is due to the UCLA motion while for the shear force is due to the Rionero motion.

Table 3 and 4 breaks down the results of foundation demand for the UCLA motion for each individual pad for the SGS without and with soil blocks, respectively. Again, the foundation demand from SGS with soil blocks is higher than those without soil blocks except for normal force on Pad 1. The increase in foundation demand per pad somewhat varies from those of the total global demand, as shown in Table 1 and 2. The maximum increase in foundation demand per pad is about 90, 25 and 120 percent for the normal force, shear force, and overturning moment, respectively.

Table 1 – Total Global Foundation Demand, Mean Soil Simplified SGS Model (without soil Blocks)

Input Motion	Normal Force (MN)	Shear Force (MN)	Overturning Moment (MN-m)
FREMONT	280	499	10,652
HECTOR	319	529	13,102
UCLA	237	550	11,538
RIONERO	276	474	13,696

Table 2 – Total Global Foundation Demand, Mean Soil Simplified SGS Model (with Soil Blocks)

Input Motion	Normal Force (MN)	Shear Force (MN)	Overturning Moment (MN-m)
FREMONT	358	568	12,045
HECTOR	319	611	14,794
UCLA	311	646	18,436
RIONERO	339	593	15,313

Table 3 – Maximum Foundation Demand per Pad, UCLA Input, Mean Soil, Simplified SGS (without soil Blocks)

Foundation Pad	Normal Force (MN)	Shear Force (MN)	Overturning Moment (MN-m)
Pad 1	131	143	1,009
Pad 2	118	134	881
Pad 3	91	139	952
Pad 4	101	136	1,014

Table 4 – Maximum Foundation Demand per Pad, UCLA Input, Mean Soil, Simplified SGS (with soil Blocks)

Foundation Pad	Normal Force (MN)	Shear Force (MN)	Overturning Moment (MN-m)
Pad 1	122	156	1,797
Pad 2	153	158	1,948
Pad 3	149	172	1,926
Pad 4	193	162	2,015

5 SUMMARY AND CONCLUSIONS

Seismic SSI response analysis was performed for a Steel Gravity Structure. The analyses employed an advanced version of the SASSI program [MTR/SASSI] that allows the foundation soil nonlinearity due to SSI effects to be considered using soil blocks and the equivalent linear method. This allows for:

a) Accounting for increase in soil shear stiffness below foundation pads caused by soil consolidation due to self-weight of the structure, and

b) Subsequent softening of soil shear stiffness below foundation pads caused by increase in soil shear strains due to SSI effects during seismic excitation.

Two analysis cases with and without soil blocks were performed using 3 soil cases (UB, ME and LB) and 9 sets of design input motions. The foundation demand (normal and shear forces and overturning moments) were calculated from both analyses and comparison of the results were presented for the mean soil and 4 controlling out of 9 design motions. Conclusions drawn from the analyses include:

- Primary soil nonlinearity considered in the 1-D free-field site response analysis accounts for majority of soil nonlinearity due to cyclic strains.
- Secondary foundation soil nonlinearity including soil consolidation effects increases global foundation demand forces and moments.
- The use of uniform strain ratio of 1.0 instead of the recommended value of 0.52 is found to provide a better match with input nonlinear stress-strain curves.
- The use of soil blocks provides a numerically efficient and practical method for incorporating secondary soil nonlinearity due to SSI effects in 3-D seismic response analysis of large offshore gravity-based structures.

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