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Seismic Design of Below-Ground Chambers Underlain by Liquefiable Soils

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ABSTRACT

Designing buried structures such as tunnels and large chambers in soils that are prone to earthquake-induced liquefaction is a challenging task. The design needs to incorporate seismic loads on the walls of chamber prior to, during, and following the onset of soil liquefaction and the associated horizontal and vertical displacements. Assessing these effects require coupled soil-structure interaction models that can simulate the non-linear and strain softening response of soils, development of seismic loading-induced excess pore water pressures, slippage and separations at the soil-structure interfaces. Performing full-blown 3D soil-structure interaction (SSI) analyses to capture the full 3D responses of the buried chambers is time consuming and impractical for most projects. Instead, simplified soil-structure interaction models need to be developed to capture the key input required for structural design. This paper presents a summary of the methodology followed for the design or a 15 m wide, 50 m long and about 25 m deep coal dumper that will be constructed in liquefiable granular soils together with a connecting conveyor tunnel. The response of the underground chamber has been analyzed using a series of 2D FLAC [1] SSI models both parallel and transverse to the longitudinal directions of the structure. The floatation-induced displacements will be controlled by a series of long permanent soil anchors with bond zones located in non-liquefiable soils at depth. Calculated soil responses in both transverse and longitudinal directions are provided as input to a comprehensive 3D structural model, which doesn't model the surrounding soils. The paper presents the solutions adopted for design using structural and geotechnical simulations along with selected results of the 2D FLAC SSI models completed for the project for 2475-yr ground shaking considering the impact of interface earthquakes.

Keywords: Seismic Design, Soil Structure Analysis, Underground Structures, Liquefaction, Numerical Simulation.

1.0 INTRODUCTION

As part of a coal terminal expansion in BC, a new deep coal dumper of 15 m wide by 50 m length to 25m depth will be built to increase the terminal's exporting capacity of steel making coal. Foundation soils comprise compact sand and gravelly sand to about 30m depth, followed by dense deposits of gravelly sand. The granular soils are underlain by a thick deposit of sandy silt/clay, which in turn is underlain by very dense Vashon deposits (till-like soils).

The saturated compact sand and gravelly sand underlying the site are assessed as having a high liquefaction potential when subjected to the design seismic shaking corresponding to a 2,475-yr return period. Ground shaking and the liquefied soils both surrounding and below the dumper make the assessment of the dynamic soil-structure response a complex task.

Being a hollow structure, the dumper is expected to have a tendency to float and move upwards when subjected to soil liquefaction. Upward movement of buried structures, such as manholes and pipelines, is a common phenomenon observed during past damaging earthquakes and documented in the literature when the surrounding soils have liquefied.

A comprehensive dynamic 3D analysis that can capture the structural and geotechnical complications of the system may appear to be the best approach but the difficulties in completing such a comprehensive modeling and the time required make it impossible from an engineering practice point of view. A combination of a detailed 3D static /quasi-static structural model (developed by the project structural consultant, without the surrounding soil) capturing the key structural characteristics and detailed dynamic geotechnical 2D SSI models along a number of representative sections of structure was adopted for this project. In the 3D structural model, the key structural mechanisms were modelled without considering the ground motions or surrounding soils. In 2D FLAC SSI analyses, geotechnical complications like soil-structure interactions, time dependent response of soil including excess pore pressure generation during earthquakes and associated strength/stiffness reduction of the soils were captured but the structural elements were simplified. The key challenge for geotechnical engineers in this project is to provide the necessary geotechnical input to the 3D structural models such that all key aspects of the structural design requirements can be met, including peak demands during shaking, dynamic effects, 3D effects, and localized vs global structural displacements, etc.

2.0 METHOLOGY

The geotechnical inputs to design of the complicated 3D structure were developed by dissecting the structural design challenges into two categories: the input required to determine the structural demands in the transverse direction at the various locations and that required to determine the structural demands due to the global soil and structural displacements induced by the earthquake shaking.

The structural demands in the transverse direction (perpendicular to the local axes of structure) were established from 2D FLAC SSI models, that include the lateral soil and water pressures, the anchor loads and elongations, translational and rotational displacements of the structure during shaking, at the peak moments, and post-liquefaction.

The structural demands due to the global soil and structural displacements were estimated based on the generalized equivalent soil "springs" developed by one of the authors of this paper and the structural displacements estimated from the 2D FLAC SSI models at the selected cross-sections along the dumper and the tunnel.

3.0 FLAC 2D SSI ANALYSIS

In the FLAC 2D SSI analyses, the non-linear, stress level-dependent responses of the soils surrounding and underlying the structures were modelled in a relatively realistic manner with the mechanisms of excess pore water pressure generation and dissipation and the associated soil strength/stiffness reductions incorporated in the model. The numerical models were developed using the public domain finite difference computer code FLAC and incorporated the user-defined constitutive model UBCSAND904aR [2] that can capture the effects of generation and dissipation of excess pore water pressures induced by earthquake shaking.

The numerical models were extended horizontally to some 300 m on either side of the dumper to minimize the effects of model boundaries and vertically to firm-ground (glacial till). The models were provided with a quiet boundary at the base to simulate seismic energy propagating both upwards and downwards at the base. Figure 1 shows the configuration of the 2D FLAC model.

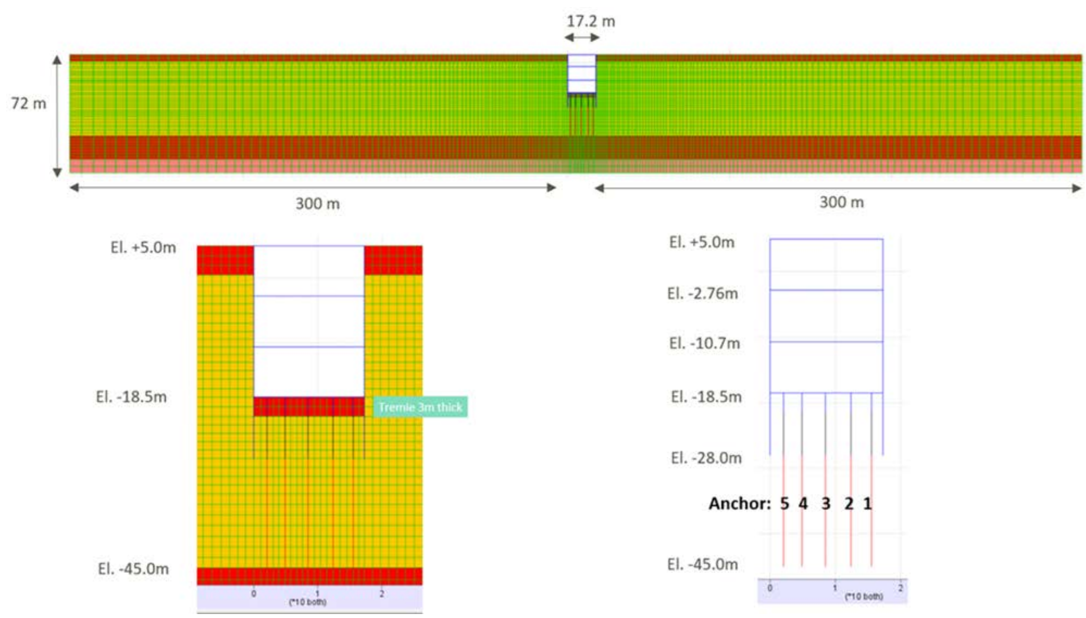


Figure 1: 2D FLAC Model Configuration

In the 2D FLAC models, each anchor was modelled using a combination of a *beam, a cable, and a pile element.* The segment of the anchor embedded in the tremie slab was modelled using a *beam element*, that below the tremie slab and extending to bottom of anticipated liquefiable soils was modelled using a *cable element*, and that within bond zone of the anchor below liquefiable soil was modelled using a *pile element.* The *beam* and *pile elements* were connected to the soil zones via bi-linear normal and shear springs. The approach of modelling the 3 different segments of a single anchor with 3 different types of structural elements was developed for capturing the key responses/characteristics of the anchors during the period of earthquake shaking when the soils surrounding the anchors are subjected to major strength/stiffness changes. This method was developed based on our in-depth understanding on the capabilities and limitations of the FLAC program developed over the past many years in various projects.

The 2D FLAC models were developed with the structural and soil responses tracked/recorded at selected locations/zones so that the values of the following design input can be provided:

a) The lateral earth pressures on the vertical face of the Vault and Tunnel walls under the static, seismic, and post-seismic loading conditions;

b) The axial forces and elongations along the anchors induced by the earthquake shaking. This input is required by the structural designer to evaluate the structural demands and ensure that sufficient capacity will be available to prevent floatation of the dumper during and immediately following the seismic shaking / soil liquefaction; and

c) Total and differential displacements imposed on the structures by the earthquake-induced ground movements during shaking at the various selected locations and in the various directions (i.e. lateral, vertical, and rotational) can be estimated.

Total and differential post-earthquake displacement demands on the structures were estimated using the published empirical methods.

3.1 GROUND MOTIONS AND CRITICAL TIME STEPS

3.1.1 Ground motions

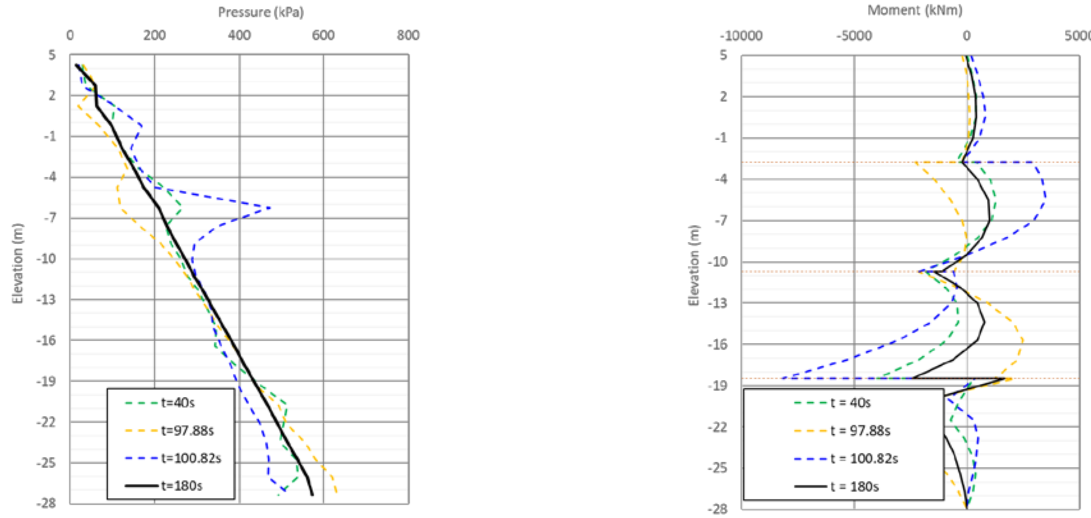
The seismic hazards at the sites in the Pacific Northwest region result main from the thrusting of the Juan de Fuca Plate underneath the North American Plate. This unique plate tectonic arrangement results in three typical types of earthquakes, each with its own characteristics; a) shallow, short duration crustal earthquakes occurring within the North American Plate, b) deep, moderate duration in-slab earthquakes occurring within the subducting plate, and c) large magnitude long duration interface earthquakes occurring as a result of slippage at the plate contacts.

A total of 16 single component horizontal acceleration time-histories were developed for the subject site and used in the 2D FLAC SSI analyses, including 5 crustal ones, 5 in-slab ones, and 6 interface ones. Three input ground motions, one from each earthquake type were selected and more intensively used as the “representative / governing” input time histories, based on evaluation of the Arias Intensity values and results of 1D site response analyses using all 16 input motions. Full-set time-histories records including reversal of ground motion directions for assessment of polarity effects were applied to one typical modeling case for verification purposes. The results from this set of analyses confirm that the three selected “representative / governing” ground motions are appropriate for structural design purposes.

3.1.2 Computed responses at selected critical time steps

Observations from FLAC SSI models suggest that the wall demand changed throughout earthquake shaking duration with shear force / bending moment / and displacement reversal as the direction of inertial loading changed. The peak structural demands, such as the earth pressures and anchor loads were not necessarily happening at the end of shaking. Complication increases with non-uniform soil profile / layering where liquefaction triggering in different layers occurs at different times, and some soil layers were liquefied before arrival of the peak level shaking when the interface motions were applied. Also, the analysis results suggested that the commonly used simplified empirical dynamic earth pressure formulas/correlations may not be able to sufficiently capture the peak 2475-yr earthquake earth pressure demands due to the relatively high degree of approximation simplification embedded within these formulas/correlations. The structural demands at the critical time step were developed based on the responses calculated using 2D FLAC SSI simulations.

Figures 2(a) and (b) show typical analysis results: the computed earth pressures and bending moments on the dumper wall at a number of critical time steps during shaking as well as that at the end of shaking (t=180s). The earth pressures at the end of shaking stage is consistent with that estimated assuming the soils becoming a heavy liquid, but the earth pressure values could be higher during the period of intensive shaking, such as that occurred at around 100 seconds in this case of analysis.



(a) (b)

Figure 2: (a) Earth pressures and (b) bending moment profiles on the dumper wall at different time step during shaking

The critical time steps were identified by monitoring time-histories of structural responses (displacement, moment, shear or anchor load). The peak displacement and earth pressures on the structure at these time steps were extracted and used as input to structural model.

3.2 ANCHORS

When a hollow buried structure is subjected to soil liquefaction prior to occurrence of the peak ground acceleration, the maximum load on the tie-down anchors could potentially be quite different from that at the end of shaking. The inertial forces will introduce racking effects, which will induce significant anchor force fluctuations and will likely control the maximum anchor load. The asymmetry of the structure itself and the presence of asymmetric loads near the dumper further increased the level of complication to the design. The racking effects were analyzed using 2D FLAC SSI simulations.

Figures 3(a) and 3(b) present a typical set of anchor force time-histories and displacement patterns computed from one of the 2D FLAC models. The result suggests that the maximum anchor load happened at around 98 to 100s during shaking. The racking of the dumper was indicated clearly by the variations in the computed anchor load histories and deformed shape of the structure at a selected time step. The racking effects can be evaluated/determined by the variations in anchor loads.

(a) (b)

Figure 3: (a) Anchor Load Time Histories, and (b) Structure Displacement Pattern at t =40s

Anchor load and anchor displacement time-histories were monitored in 2D FLAC simulations, for the various cross sections, load cases, and ground motions analyzed. The maximum anchor demand was determined based on comparison of the anchor load values computed during shaking with that at the end of shaking.

In addition to defining maximum anchor demand, the computed anchor responses (axial loads vs displacements) were used as one of the key inputs to the 3D structural models. The anchor load-anchor head displacement curves computed from 2D FLAC SSI models were used as "springs" at the base of the structural model for selected analysis cases.

4.0 SOIL SPRINGS (P-Y CURVES)

The 3D models developed by the structural engineer were unable to include the soils surrounding the structures. As a result, "springs" or moduli of subgrade reaction values are required as input to the 3D structural models. Simulating the very complicated soil-structure interaction responses induced by earthquake shaking using "springs" or moduli of subgrade reaction values represents a significant approximation, but this is the only type of geotechnical input that can be adapted/implemented practically in the 3D structural models developed for the subject project.

The 3D structural model is critical for estimating the structural demands induced by the global soil and structural displacements, e.g., the kinematic demands.

The recommended "springs" or moduli of subgrade reactions are discussed below.

4.1 Prior to Significant Excess Pore Water Pressure Development

The "springs" or moduli of subgrade reactions for soils prior to any significant/substantial softening or liquefaction were estimated based on closed form/semi-closed form formulae developed for rigid embedded foundations. The following summarizes the approach/methodology followed in developing the soil springs.

The lateral earth pressure on the vertical face of the dumper wall prior to commencement of earthquake shaking was estimated using the at-rest earth pressure coefficient (K_o). The lateral movement of the walls should be taken as zero at the at-rest state, which represents the starting point of the p-y curves (or springs). The maximum earth pressure that can be developed when the wall is moved towards the soil was estimated using the passive earth pressure coefficient (K_p). Where, K_p was calculated using the angle of internal friction of the soils surrounding the structure. For simplicity, Rankine theory was adopted, i.e., the direction of the earth pressure was taken as perpendicular to the wall face by ignoring the contribution from the interface friction between the wall and the soils. The approximate horizontal movement ratio "y/H" was assumed to be 0.01 for full mobilization of the passive earth pressure. Where, H is the total height or embedment depth of the wall (Canadian Foundation Engineering Manual, CFEM [3]), assuming that the wall is subjected to mainly translational movements. As a simplified approach, a linear variation of the lateral earth pressure was assumed from its K_0 state value to the corresponding K_p state value, ignoring the "non-linear" effects.

The dumper (and tunnel) structure is sufficiently rigid in comparison with the surrounding soils, in both the transverse and the longitudinal directions. As a result, if one side of the structure moves towards the soil, the other side of the structure will move away from the soil. In this case, the lateral earth pressures on the walls that move away from the soil will vary from its K_0 state value towards the corresponding K_a (active earth pressure) state value. Similar to the K_p, the K_a value was calculated using the angle of internal friction of the surrounding soils also. The lateral wall displacements required to reach the K_a state was approximately estimated using a "y/H" value of 0.001.

Theoretically, the earth pressures will decrease further after reaching the K_a state, should the lateral wall movements continue, and reach zero eventually when the soil is detached from the wall face. In light of the displacements predicted by the 2D FLAC SSI models for the current study, the lateral earth pressure is considered remaining constant at its K_a value for larger displacements after passing the "threshold" of "y/H" = 0.001. The variations in lateral earth pressure, p, with the lateral wall movement, y, developed based on the assumptions described above are illustrated in Figure 4 for a select depth as a typical example.

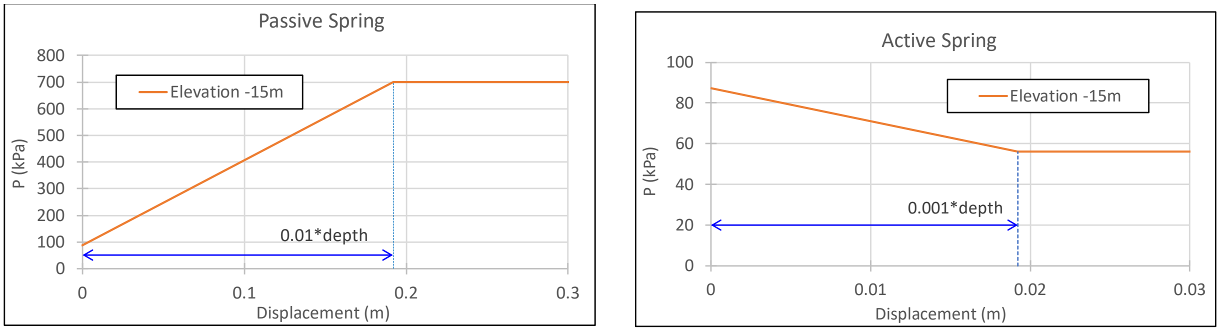


Figure 4: Lateral Earth Pressure – Displacement Variations for Static and 100-yr return period Demands

The springs required for input to the structural models, in the form of lateral force (in kN) versus the lateral displacements (in mm) was developed based on the earth pressure distributions estimated using this method and the tributary area on the wall face represented by the given spring.

4.2 With Significant Excess Pore Water Pressure Development

A set of simplified lateral soil springs were developed to estimate the response of the dumper under the condition of significant excess pore water pressure generation with soils liquefied or nearly liquefied, to suit the needs of the structural design(carried out by others).

Following the same approach and with the similar assumptions as for developing soil springs as discussed in Section 4.1, the soil springs applicable for liquefied soils were estimated based on the following additional assumptions:

It is assumed (1) the liquefied and/or nearly liquefied soils will still have some residual shear strength that is equivalent to having an equivalent friction angle of approximately 9 degrees (equivalent to a Sur/σ'_v ratio of 0.16) at the subject project site, (2) the conventional formulae for calculating the coefficients of earth pressure, K_0, K_a and K_p remain applicable, and (3) the lateral wall movement required to mobilize the passive (or active) earth pressure for the liquefied soils will be similar to that for very soft cohesive soils.

Based on the assumptions made above, the computed variation in lateral earth pressure, p, with the lateral wall movement, y, for the liquefied soils or soils with excess pore pressure ratio greater than about 80% is illustrated in Figure 5, as a typical example.

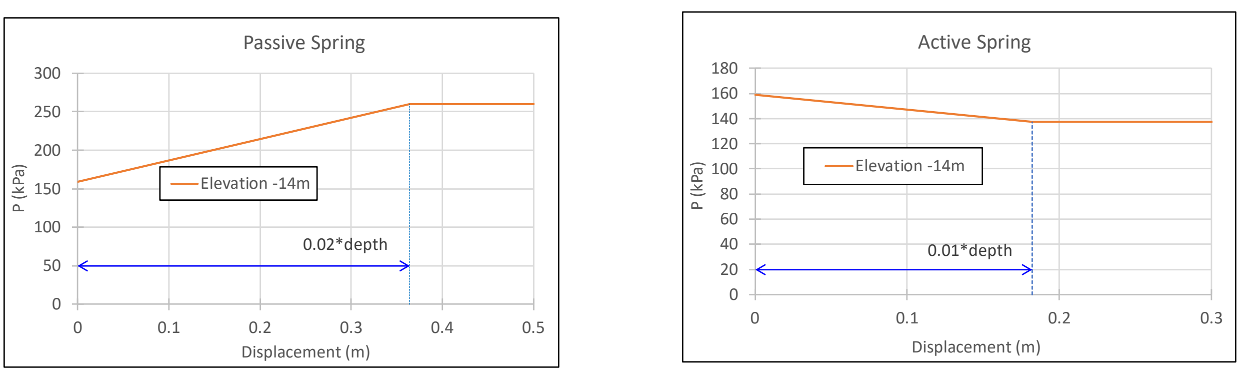


Figure 5: Lateral Earth Pressure – Displacement Variations for Liquefied Soil

5.0 DUMPER VAULT RESPONSE WITHOUT ANCHOR SUPPORT

A number of publications document the field observations on floatation of smaller hollow structures such as manholes and pipelines due to soil liquefaction during past earthquakes. No directly applicable case-histories were found with documented evidence on the floatation of large buried chambers during past earthquakes where extensive soil liquefaction has been reported. Of relevance to the current project are the floatation studies and designs completed for the following two high profile submarine tunnel crossings on the west coast of Canada and USA, consisting of analytical predictions supplemented with results of centrifuge testing.

a) Numerical predictions and centrifuge tests carried out for the George Massey Tunnel located in Delta, BC, Canada [24 m wide, 7 m high]. The results of centrifuge tests [4] and numerical models indicate that the tunnel would uplift 0.2 to 0.3 m due to soil liquefaction. Reverse bearing capacity failure is cited as the mechanism leading to floatation. The tunnel response was sensitive to the overall geometry and slopes assigned to the soil strata.

b) Numerical predictions and centrifuge tests carried out for the Bay Area Rapid Transit (BART) Tunnel in California, USA [15 m wide, 8 m high]. The centrifuge tests indicated that the tunnel would experience upward movements of about 0.25 m. The floatation mechanism was identified as that due to ratcheting plus flow of sand to the area below the base of the tunnel due to high porewater pressure gradients that exist as a result of soil liquefaction. Centrifuge testing carried out by Chou et al. [5] illustrate the inferred mechanisms of uplifting of the tunnel and evidence supporting sand moving below the base slab of the structure.

Both the above tunnels are located in liquefiable soils, and the design shaking levels correspond to PGAs in the order of 0.15 to about 0.65 g.

The dynamic response of the dumper without any soil anchors was also studied to estimate the likelihood of uplift movement of the Dumper Vault when the anchors don't provide any resistance against floatation. The 2D FLAC models predicted up to 500 mm upward movement. The two major mechanisms contribute to the uplift movements – the ratcheting movements and flow of sand below the base slab were observed in the 2D FLAC models. Each positive and negative horizontal displacement pulse appears to induce a marked increase in vertical movements in the structure of the two walls.

6.0 CONCLUSIONS

Using a complex 3D model which can capture all geotechnical complications and structural details in a short time period is not practical within the current state of geotechnical engineering practice. A practical solution was adopted in this project (as described in this paper) to resolve the challenging structural design issues for such large structures subjected to complex loading conditions: detailed 3D structural models with all important structural aspects in combination with a series of detailed 2D FLAC SSI analyses including all important geotechnical aspects and ground motions. Impact from each of the load cases / factors / mechanisms can be identified and studied separately. The total demand can be quantified by supper imposing/comparing the impacts from individual load cases / factors / mechanisms.

Response at the end of shaking is not always necessarily most critical for structural design when the soil is prone to liquefaction prior to the peak level shaking. However, the critical time steps can be determined, and the peak structural response can be quantified using 2D FLAC SSI simulations. Peak earthquake demands can be captured in 2D FLAC SSI models and used as input to structural models.

Soil springs or moduli of subgrade reaction recommended in this paper represent a practical solution to a very complicated problem. However, it should be noted that this "solution" involves a large number of assumptions and simplifications, and should be used by geotechnical / structural engineers with a thorough understanding of the implications of those assumptions and simplifications.

2D FLAC models can capture the mechanisms of ratcheting movements and flow of liquefied sands below the base slab of the large buried chambers effectively and provide reasonable prediction when compared to the published literature.

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