Feasibility Study for Earthquake Mitigation Effects by a Spherical, Concrete-Shelled Structure

Report

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1. Introduction

The objective of this study is checking the feasibility of a spherical, concrete-shelled structure in a seismic prone zone. The idea of this type of structure is that if the underlying soil liquefies during the earthquake, “the building would float in the liquefied material in the event of an earthquake and would actually be protected from the main earthquake forces by the liquefied buffer” (J. Woodcock – personal communication). The specific objectives of this preliminary study are:

1. Does pore water pressure build-up under the structure, to what extent, and which is its pattern
2. What are the effects of soil softening (liquefaction) on: (1) seismic waves transmitted to the structure and further on the soil pressures at the interface, and (2) rotations and displacements of the structure

Three-dimensional nonlinear dynamic finite element analyses of the spherical structure and surrounding soil are performed for three seismic motions. For the soil conditions assumed in the study (loose silty sand, with groundwater level at 3’ below ground level) the study results show that, owing to complete soil liquefaction around the structure, the spherical structure is protected from large displacements / rotations and large soil pressures that may be induced by such events to regular structures.

2. Finite Element Model

2.1 Mathematical Model

Nonlinear dynamic finite element analyses are performed using the code DYNAFLOW (Prevost 1999). DYNAFLOW is a finite element analysis program for the static and transient response of linear and nonlinear two- and three-dimensional systems. The solid and fluid coupled field equations are based on an extension of Biot's formulation (Biot 1962) in the nonlinear regime, and are applicable to multidimensional situations. A multi-yield constitutive model is used for simulating the behavior of soil materials. It is a kinematic hardening model based on a simple plasticity theory (Prevost, 1985), and is applicable to both cohesive and cohesionless soils. The yield function is described in the principal stress space by a set of nested conical yield surfaces. A non-associative plastic flow rule is used for the dilatational component of the plastic deformation. The model has been tailored (1) to retain the extreme versatility and accuracy of the simple multisurface J_2 theory in describing observed shear nonlinear hysteretic behavior and shear stress induced anisotropic effects, and (2) to reflect the strong dependency of the shear dilatancy on the effective stress ratio in both cohesionless and cohesive soils. Accurate simulation of shear-induced plastic dilation and of hysteretic effects under cyclic loading, together with full coupling between solid and fluid equations, allow capturing the build-up and dissipation of pore water pressures and modeling the gradual softening and hardening of soil materials.

The required constitutive model parameters can be derived from the results of conventional laboratory (e.g. triaxial, simple shear) and in-situ (e.g. standard penetration, cone penetration, wave velocity) soil tests. Liquefaction strength analysis (e.g. Popescu, 1995, Popescu et al. 1997) is also needed for saturated materials subjected to cyclic loads. The multi-yield plasticity soil constitutive model, its implementation algorithm, and the methodology for estimating the constitutive model parameters have been repeatedly validated in the past for soil liquefaction computations, based on both centrifuge experimental results (e.g. Popescu and Prevost 1993, 1995) and full scale measurements (e.g. Keane and Prevost 1989, Popescu et al. 1992, 1998).

The finite element analysis is performed in one run consisting of two steps. First, gravity loads are applied and the soil is allowed to fully consolidate. The consolidation phase is calculated dynamically, by setting the Newmark algorithm parameters in the integration scheme as $\gamma = 1.5$ and $\beta = 1$. After consolidation is completed, the nodal displacements, velocities and accelerations are zeroed, the time is reset to zero and the input acceleration is applied at the base. The Newmark parameters are chosen as $\gamma = 0.65$ and $\beta = (\gamma+0.5)^2/4 = 0.33$. This choice for γ introduces a slight numerical damping ($\gamma = 0.5$ corresponds to no numerical damping), and the selected value for β maximizes high frequency numerical dissipation. No additional viscous damping is introduced.

2.2 Site and Soil Conditions

The site selected for this study is located in San Francisco area, and the soil conditions are obtained from actual field exploration data (Pease and O'Rourke). Liquefaction occurred at this site during the 1906 San Francisco and 1989 Loma Prieta earthquakes. Available for this study are: (1) one SPT boring with soil types description to a depth of 20m and N_{SPT} blowcounts to a depth of 13m, and (2) two CPT profiles providing cone tip resistance and friction ratio to a depth of 20m. A method consisting of empirical correlation formulas and liquefaction strength analysis (see Popescu 1995, and Prevost and Popescu 1996 for more details) was employed for estimating the soil constitutive model parameters. The results of parameter calibration for saturated loose silty sand are presented in Table 1.

Table 1. Soil constitutive model parameters – saturated silty-sand

Constitutive parameter	**Symbol**	**Value**	**Type**
Mass density – solid	r_s	5.16 slug	State
Porosity	n^w	0.40	parameters
Hydraulic conductivity	k	$1x10^{-4}$(h), $1x10^{-5}$(v) ft/s	
Low strain elastic moduli	B_0, G_0	3250ksf, 1500 ksf	Low strain
Reference effective mean normal stress	p_0'	2.09ksf	elastic
Power exponent	n	0.50	parameters
Friction angle at failure	f	33.5^0	Yield and
Maximum deviatoric strain	e_{dev}^{max}	8%	failure
Coefficient of lateral stress	k_0	0.58	parameters
Stress-strain curve coefficient	a	0.4	
Dilation angle	y	32^0	Dilation
Dilation parameter	X_{pp}	0.14	parameters

The groundwater table at the site is located at a depth of 2.7m (9ft). To increase possible effects of soil liquefaction on the structure, for the purpose of this study, it is assumed that the groundwater table is located much closer to the ground level – at a dept of about 3ft.

2.3 Finite Element mesh

The 3D analysis domain includes the sphere and a volume of soil extending laterally, in both directions, for 98ft from the sphere axis, and down to the dense sand and gravel layer that is assumed rigid for the purpose of this analysis, and is located at 33ft below ground surface. The saturated soil is discretized into 1086 8-node brick coupled porous solid-fluid finite elements, with six degrees of freedom (dof) per node – three for the solid phase and three for the fluid phase displacements. The dry soil close to the surface is discretized into 120 8-node brick finite elements with 3 dof per node. The sphere will is discretized into 162 8-node brick coupled porous solid-fluid elements with very low permeability, to ensure an impervious soil structure interface.

The boundary conditions are as follows:

a. Prescribed input acceleration in one horizontal direction at the base of the analysis domain (solid dof), all other solid dof and vertical fluid dof at the base – fixed

b. Free field motion at the lateral boundaries of the soil domain, prescribed by slaving all dof of pairs of nodes situated at the same levels and on opposite locations.

In this study the structure consists of a rigid sphere, with 32' radius, weighing 5 million lbs., with the center of gravity at 6' below the dead center, which equals the elevation at which the structure "daylights". Perfect stick is assumed at the soil structure interface. The circumferential balcony that is situated at the equator of the structure, well above ground level, was not included in the analysis.

The geometry, boundary conditions, and applied seismic motion with presence of a vertical symmetry plane allow analysis of only one half of the domain. The finite element mesh is shown in Figure 1.

2.4 Seismic ground motion

The ground motion selected for this study consists of synthesized acceleration time histories, compatible with prescribed response spectra, and having prescribed maximum values and strong motion durations (the procedure is described by Popescu et al. 2000). Two types of seismic motions were considered: one compatible with the Type 3 design spectrum prescribed by the Uniform Building Code (1994) and corresponding to soft to medium clays and sands, and one compatible with the Type 1 design spectrum and corresponding to rocks and stiff soils. The maximum seismic accelerations considered in each analysis are listed in Table 2.

Table 2. Maximum seismic accelerations

Analysis Name	Response Spectrum	Maximum Acceleration (g)
JSPA3	Type 3	0.30
JSPA1	Type 1	0.30
JA1SM	Type 1	0.15

The total duration of the seismic motion is about 20sec, with a duration of the strong motion of about 10 – 12sec (Figure 2). It is mentioned that Type 3 acceleration was first considered, as a lower frequency content is believed to maximize the seismic effects for such a soil deposit (see e.g. Popescu 2002). As full liquefaction was predicted for this motion (analysis JSPA3), additional seismic motions were considered, having a higher frequency content (JSPA1) and a lower maximum value (JA1SM) to assess structure behavior during less severe earthquakes.

3. Analysis Results

3.1 Stresses and Excess Pore Pressures

Contours of initial effective stresses (vertical and mean) are presented in Figure 3. The maximum pressures at soil structure interface for normal operation are about 600 psf.

An indicator of the liquefaction behavior of a soil deposit is the evolution of the excess pore water pressure (epwp) ratio with respect to the initial effective vertical stress. If this ratio is above about 0.8, then the soil is almost liquefied, and lost most of its shear strength. An epwp ratio of one indicates complete liquefaction. The predicted liquefaction behavior of the spherical structure foundation, when subjected to Type 3 seismic motion (analysis JSPA3) is presented in Figure 4 for several time instants. It can be observed that the soil is almost liquefied after 3 sec, and remains in this state throughout the earthquake.

Note: The epwp values shown in the sphere have no meaning; they resulted as an artifact of the analysis options described in Section 2.3, and do not affect the water pressures and stresses in the soil.

Following build-up of epwp, the effective stresses in soil decrease, and the pressures at soil structure interface become lower than the ones in static conditions. Figure 5 shows predicted effective vertical stresses at T=1sec and T=12sec, with resulting maximum pressures of 300psf and 200psf, respectively. The same contours have been used in Figure 5 as in Figure 3, to facilitate the comparison.

Predicted epwp ratios and effective vertical stresses are presented in Figures 6 and 7 for case JA1SM (smaller seismic acceleration). The liquefaction is not as severe as in case JSPA3, and the resulting interface pressures are slightly higher, but they are still below static pressures. The predicted epwp and effective stresses for case JSPA1 are in between those predicted for cases JSPA3 and JA1SM.

Other stresses are also reduced during the earthquake due to soil liquefaction (Figure 8 presents contours of effective horizontal stresses and shear stresses for case JA1SM in the plane of the seismic motion at time T=8sec – when the peak seismic accelerations occur).

3.2 Structure Displacements and Accelerations

Rapid liquefaction of the soil prevents seismic wave propagation towards the surface. Early after the quake starts, the major displacements are limited to deep soil layers, as shown in Figure 9. This leads to a dramatic reduction of structure accelerations, as shown in Figure 10, for case JSPA3. The plots in Figure 10 are presented at the same scale. Predicted displacements and rotations of the structure are also negligible, as shown in Figure 11 for case JSPA1.

4. Conclusions

For the cases analyzed in this study, comprising a structure founded on a liquefiable, uniform soil deposit, and assuming the water table at 3' below ground level, the seismic response of a spherical structure seems to be strongly improved due to soil liquefaction. Pressures at soil-structure interface resulted consistently lower than the static pressures due to a more uniform distribution after soil liquefaction. Los of shear strength of the soil below the sphere leads to preventing seismic waves to travel towards the surface. In this way, the structure is isolated from the earthquake motion by the liquefied soil layer, and both accelerations and displacements are dramatically reduced.

It can be concluded from the results of this study that the basic idea of using a spherical structure in seismic areas with potentially liquefiable soils may lead to significant improvement in structural response. As a note of caution, it is mentioned, however, that there are a series of aspects that have not been addressed in this study, and that may lead to a different seismic response of the soil-structure system. Those aspects are:

- presence of a denser soil layer, more resistant to liquefaction, in the vicinity of the structure
- deeper groundwater level
- inherent heterogeneity of the soil properties, leading to patches of non-liquefied soil that may lead to higher local pressures on the structure

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