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A numerical study of the composite behavior of permanent and temporary shaft liners subjected to strong ground shaking

K.C. Ellison, P. Kumar & A. Richmond
Arup, San Francisco, California, USA

S. Chen & L. Maher
San Francisco Public Works, San Francisco, California, USA

Y.M.A. Hashash
University of Illinois, Urbana-Champaign, Illinois, USA

M. Walker
Mott MacDonald, San Francisco, California, USA

S. Toloui
San Francisco Public Utilities Commission, San Francisco, California, USA

ABSTRACT: The seismic performance of a proposed 37-meter-diameter by 52-meter-deep permanent shaft installed within a "temporary" diaphragm wall-supported excavation for a soft-over-stiff soil siteis evaluated via 3D finite element analyses. To this end, results are presented from two sensitivity analyses performed as part of the Central Bayside System Improvement Project in San Francisco, California, USA. The variables considered include the stiffness of the diaphragm wall (to account for possible degradation over time) and the effect of tying (or not tying) the permanent structure to the diaphragm wall (e.g. to resist hydrostatic uplift). The findings indicate there can be significant effects from each of these variables on the demands in the permanent shaft, which are typically not apparent from simplified closed-form solutions. It is concluded that consideration of site-specific conditions is important to the evaluation of composite shaft behaviorand that ignoring the presence of the diaphragm wall is not necessarily conservative.

1 INTRODUCTION

Design for permanent shafts often ignore the presence of the "temporary" shoring system which will be left in place after construction. This simplification is common in practice since: 1) it is often assumed that the effect of the temporary shoring would be to reduce demands and thus ignoring it will be conservative; and 2) the permanent and temporary works are often designed by different entities and have vastly different perceived design lives. The present study examines the applicability of this common simplification with reference to a proposed deep shaft from the Central Bayside System Improvement Project (CBSIP) located at a soft-over-stiff soil site in the high seismicity region of San Francisco, California, USA. The presented evaluation uses advanced analysis tools that represent the temporary and permanent shaft structures, the nonlinear soil behavior and the interface behavior between the soil, the temporary shaft and the permanent shaft. The evaluation was performed specifically to assess the seismic performance of the permanent shaft in the permanent condition (as opposed to the static conditions during construction). Analysis of the temporary shaft in the temporary condition has been performed by others.

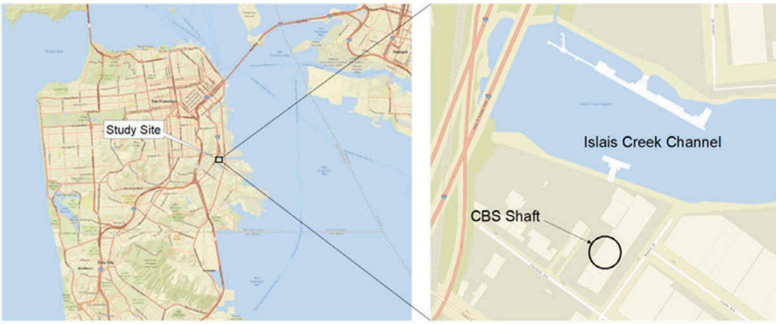

Figure 1. Location of proposed CBS Shaft Site Adjacent to Islais Creek in San Francisco, California, USA.

1.1 *Overview of the Central Bayside System Improvement Project*

The proposed Central Bayside System Improvement Project (CBSIP) includes the 37-meter-diameter by 52-meter-deep Central Bayside Shaft (CBS), the 7-meter-diameter Channel Tunnel (CHTL) and associated hydraulic connections to convey wastewater from the northern and central Bayside areas of San Francisco to the Southeast Water Pollution Control Plant (SEP). CBSIP is part of the Sewer System Improvement Plan (SSIP) implemented by San Francisco Public Utilities Commission (SFPUC). The focus of this paper is the CBS located adjacent to Islais Creek in San Francisco (See Figure 1), which in the temporary condition will be used for launching the tunnel boring machine (TBM) for the CHTL, and in the permanent condition, will house the waste water lift station. A diaphragm wall (i.e. temporary liner) will be installed for the temporary condition for the TBM launch, following which the permanent shaft (i.e. permanent liner) for the lift station will be cast within the diaphragm wall. San Francisco Public Works (SFPW) is the structural engineer for the permanent CBS, with Arup providing seismic soil-structure interaction (SSI) services. Stantec/Jacobs is the geotechnical engineer for the project and the designer for the temporary works.

1.2 *Seismic analysis for the Central Bayside Shaft*

The preliminary seismic analyses of the CBS ignored the presence of the temporary system shoring consisting of the diaphragm wall, which will be left in place after construction. These analyses showed that the CBS, which was to be partially socketed in rock, would be subjected to very high shear demands due to the soft over stiff nature of the high seismicity site. Therefore, sensitivity analyses at this early stage looked at the effect of reducing shaft depth as a possible way to mitigate the high shear demands. The results of this sensitivity analysis are presented in Section 3.

Subsequent analysis of the shaft considered the composite behavior of the final shaft structure and shoring diaphragm wall during shaking. The model details and sensitivity analyses for this “composite”model are presented in Section 4.

2 GEO-SEISMIC ASSUMPTIONS FOR DESIGN

2.1 *Idealized ground conditions*

Figure 2 summarizes the soil parameters corresponding to the western and eastern portions of the site (“west” and “east”, respectively), including the unit weight, shear wave velocity (V_S) and shear strength. The shear strength of coarse-grained soils was derived as a function of mean effective stress and effective friction angle, and that for fine-grained soils was defined as the undrained shear strength. A bedrock (half-space) V_S of 1,000 m/s was adopted, consistent

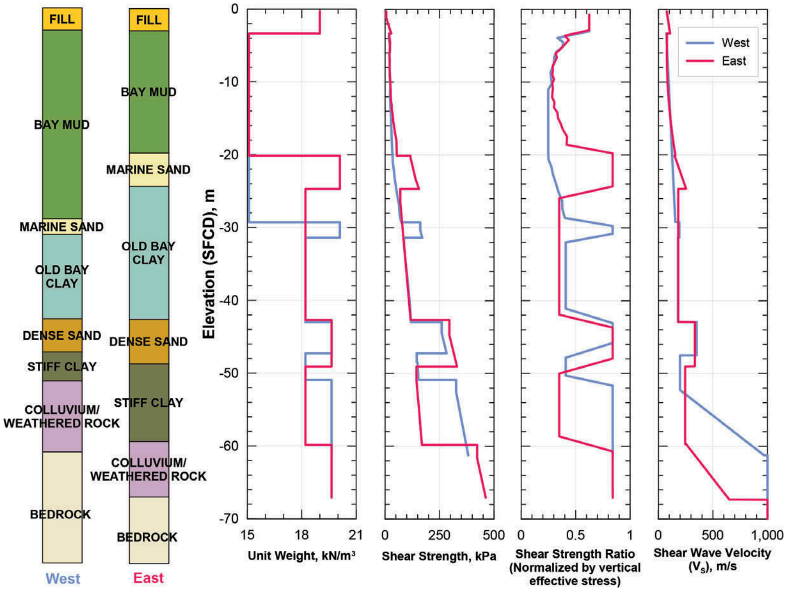

Figure 2. Idealized Soil Parameters for the West and East Soil Profiles.

with input ground motions developed for the site (See Section 2.2). The principal differences between the two profiles are the shallower Franciscan bedrock and thicker Bay Mud unit in the west. Bay Mud is a soft clay that generally governs site response at such sites in the region. The soil shear modulus reduction curves were derived using Darendeli (2001), adjusted to match the shear strengths shown in Figure 2 at large strains using the hyperbolic fitting procedure provided in Groholski et al (2016). Groundwater is assumed to be at the ground surface, with a hydrostatic porewater pressure profile. Note that the studies described herein were carried out before the geotechnical investigation was finalized and are not the final analyses that will be carried out for the project.

2.2 *Bedrock ground motions*

A suite of eleven sets of 975-year input bedrock motions was developed for the project by the geotechnical engineer. The original seed records were obtained from the PEER (2016) NGA-West2 database. Note that the two horizontal components for each earthquake record correspond to "Maximum" (Fault Normal) and "Minimum" (Fault Parallel) orientations. Table 1 below summarizes the details for three of the motions which are used for the analyses presented in this paper.

Table 1. Input Bedrock Ground Motions.

PEER[1] RSN[2]	Event	M_W	Mechanism	Station ID	Distance (km)	V_{S30}[3] (m/s)
28	Parkfield, California (1966)	6.19	Strike Slip	C12	17.6	409
5813	Iwate, Japan (2008)	6.9	Reverse	44B71	7.9	413
6893	Darfield, New Zealand (2010)	7.0	Strike Slip	DFHS	11.9	344

[1] Pacific Earthquake Engineering Research Center
[2] Record Sequence Number
[3] Time-averaged shear wave velocity (V_S) in the upper 30 m

3 PRELIMINARY STUDY ON THE EFFECT OF SHAFT DEPTH

The potential for high seismic shear demands on the permanent liner of the CBS was identified as an early driver for design of the permanent shaft walls. As a check during the preliminary design phase, a simplified half model of the CBS was developed in the finite element program LS-DYNA that excluded the temporary diaphragm wall. This model relied upon material properties (including backbone shear modulus reduction curves) that were available from site response analyses of the east and west profiles that had already been performed using the software DEEPSOIL (Hashash et al. 2017). The constitutive model MAT_HYSTERETIC_SOIL was utilized to model the soil elements, which is a nested surface model with ten superposed "layers" of elasto-perfectly plastic material, each with its own elastic moduli and yield values. The moduli for the surfaces are defined based on the backbone shear modulus reduction curves. The east and west soil profiles were stitched together to create the 3D soil domain and the shaft was idealized as a series of linear elastic beams. The beam elements representing the shaft were rigidly connected to a surface at the "true" diameter of the shaft that interacted with the soil domain via friction, as shown in Figure 3. The "maximum" of the two horizontal components of each of the three bedrock motions identified in Table 1 was applied to the bottom boundary of the simulation via Lysmer Dampers, which represent the application of outcrop bedrock motions at the elastic half-space of the model.

The RSN28 (Parkfield) motion was found to result in the highest demand for the CBS during preliminary design. A sensitivity study was performed using this motion to assess whether a reduction of the shaft depth could significantly reduce demands by raising the bottom of the shaft out of the colluvium/weathered rock layer. Results from this sensitivity study are presented in Figure 4 as envelope shear and moment demands through the shaft with depth.

As anticipated, a reduction in demands is observed with reduction in shaft depth from the "0 m Rise" to the "5 m Rise" and "10 m Rise" cases. However, the computed change in peak demands were found to vary approximately linearly with the reduction in shaft length (rather than a step change with separation from the colluvium/weathered rock layer), with the

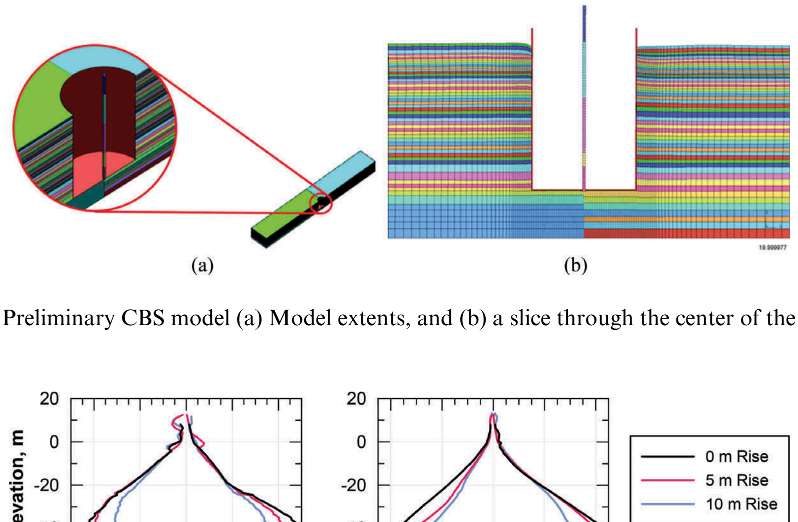

Figure 3. Preliminary CBS model (a) Model extents, and (b) a slice through the center of the shaft.

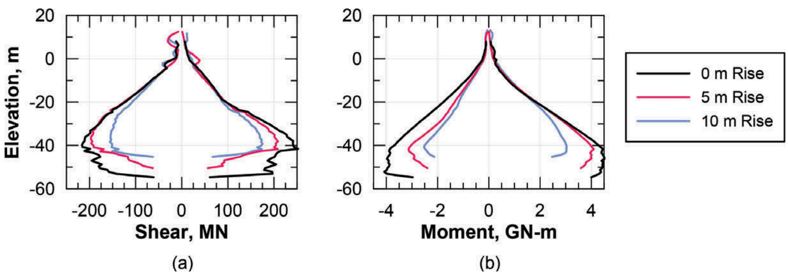

Figure 4. Effect of shaft depth on envelopes of (a) shear, and (b) moment from preliminary study.

elevation of peak demands (approximately -42 m) coinciding in each case with the transition between Old Bay Clay and dense sand.

4 STUDIES ON THE COMPOSITE BEHAVIOR OF THE PERMANENT LINER AND TEMPORARY SHORING WALL

Following the preliminary design, a more detailed engineering study was conducted to understand the composite action of the diaphragm wall used as part of the temporary shoring and the permanent CBS for the “-5 m Rise” scenario, as shown in Figure 5. The resulting model differed from the initial model described in Section 3 in several ways: 1) the soil domain included a gradual transition between the east and west profiles; 2) structural details of the shaft such as the shaft walls, internal walls, columns and diaphragms were included; 3) the temporary diaphragm wall was included; 4) orthogonal bedrock motions were applied simultaneously, and 5) construction stages were included to approximately represent the modified stress conditions but were not sufficiently detailed nor intended for the evaluation of construction conditions of the shaft.

Sensitivity studies examined the impact of “upper range” and “lower range” stiffness assumptions for the diaphragm wall as well as the effect of tying or not tying the CBS to the temporary diaphragm wall for the purpose of resisting hydrostatic uplift in the long term.

4.1 *Structural parameters*

All structural elements of the permanent shaft were modeled as linear elastic with a Young’s Modulus of 30 GPa. The columns, permanent shaft walls (interior and exterior) and the base of the shaft were modeled via beam, shell and solid elements, respectively.

The structural elements for the temporary diaphragm wall consisted of shell elements representing the primary panels, secondary panels and the interfaces between primary and secondary panels. During the construction stages, the primary and secondary panels were modelled with smeared concrete properties (i.e. using a concrete material model) and reinforcing properties (i.e. using a linear elastic material model) across the thickness of the shell. The shell elements representing the interfaces were modeled as plain concrete.

Both the interaction between the permanent shaft wall and the diaphragm wall as well as the interaction between the bottom of the mat and the soil were governed by contact surfaces with a coefficient of friction of 0.2 (approximately representing the anticipated friction across a vapor barrier).

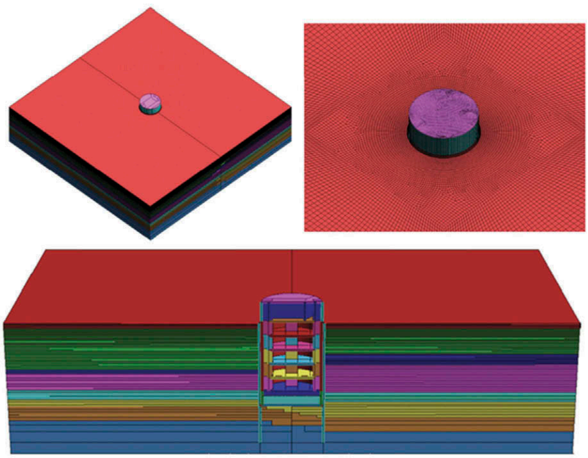

Figure 5. LS-DYNA 3D Finite Element Model Extents and Mesh Refinement.

In the “upper range” shoring scenario, the shoring parameters remain unchanged during the earthquake. For the purposes of the sensitivity study, a “lower range” shoring scenario assumed that the stiffness of the primary and secondary panels was halved and the elements at the interfaces were assumed to have an effective friction angle of approximately 24 degrees.

4.2 *Construction stages*

The construction stages modeled are summarized in Table 2.

4.3 *Sensitivity studies*

4.3.1 *Variation with ground motions*

Figure 6 presents comparisons of the total shear and moment demands through a ring section versus depth (“cut-section”)arising from each of the three bi-directional motions for a simulation with lower range shoring parameters and with the permanent shaft tied to the diaphragm wall. The overall behavior is generally similar in each case with peak demands in the permanent liner located at the depth where the permanent shaft is tied to the diaphragm wall (i.e. approximately -48 m). The depth of maximum shear and moment in the diaphragm wall is just below the base of the permanent shaft.

Despite the revised model taking advantage of the composite stiffness of the pump station and diaphragm walls, the shear and moment demands in Figure 6 are greater than those observed during the preliminary study in Figure 4. This increase in demands resulted from tying the shaft to the diaphragm wall as discussed in Section 4.3.2.

4.3.2 *Effect of tying permanent liner to temporary liner*

Figure 7 presents comparisons of cut section shear and moment demand versus depth in both the pump station wall and the diaphragm wall for the following cases using the RSN28 motion: 1) CBS tied to the diaphragm wall, 2) CBS not tied to the diaphragm wall, and 3) no diaphragm wall. As in the previous section, these scenarios assume “lower range” shoring parameters during shaking. Demands on the shaft increase marginally for the “LR untied” scenario and more significantly for the “LR tied” scenario. This is because the diaphragm shaft is socketed into rock. Thus, the presence of the diaphragm wall serves to prevent rocking of the permanent

Table 2. Construction Stages

Stage #	Stage Description
1	Wish-in-place diaphragm wall
2-11	Excavate shaft
12	Construct permanent CBS
13	Recharge water table
14	Apply ground motion

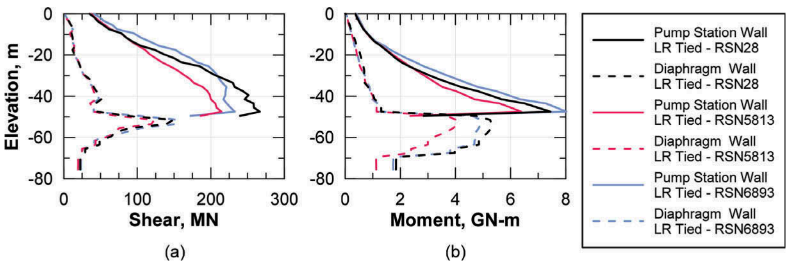

Figure 6. Cut-section shear (a) and moment (b) versus depth for different ground motions.

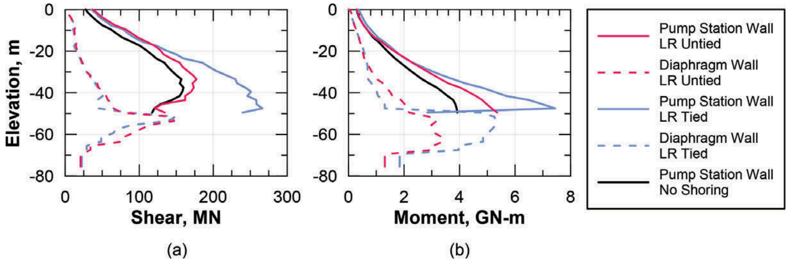

Figure 7. Cut section shear (a) and moment (b) versus depth for analyses assessing the effect of tying or not tying the permanent CBS to the temporary diaphragm wall

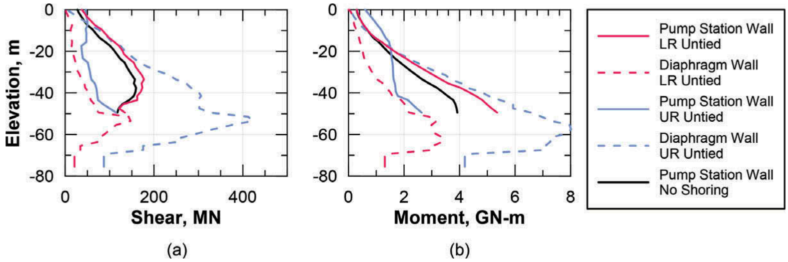

Figure 8. Effect of diaphragm wall stiffness on (a) shear and (b) moment demands on CBS

shaft, which tends to increase shear and moment demands. Demands are higher in the "LR Tied" case than the "LR Untied" case because the rocking is more rigidly restrained.

4.3.3 *Effect of diaphragm wall stiffness*

Figure 8 presents comparisons of cut section shear and moment demand versus depth in both the pump station wall and the diaphragm wall for the following cases: 1) lower range shoring stiffness, 2) upper range shoring stiffness, and 3) no diaphragm wall. These scenarios each assume the pump station wall is not tied to the diaphragm wall. As evidenced by the results from the upper range shoring scenario, there can be a significant benefit to the seismic demands for the permanent shaft from composite action with the diaphragm wall if the shoring wall is sufficiently stiff and the interface between primary and secondary panels is sufficiently strong (e. g. if the diaphragm wall were designed for the same lifespan as the permanent shaft).

4.4 *Normal contact stresses and out-of-plane shear and bending moment on pump station wall*

The sensitivity analyses presented in Section 4.3 only considered the cut-section shear and overturning moment, whereas there are several other parameters that could govern design, such as the out-of-plane shear and bending moments, which are a function of the normal contact stresses transferred to the pump station wall. Figure 9 compares the average normal contact stress on a circumferential slice of the pump station wall between the composite option ("LR Untied" case) and "no shoring wall" case. The "end of construction" contact stresses on the pump station wall are significantly lower for the composite case, as most of the loads from the surrounding soil domain are already taken by the diaphragm wall due to being installed earlier in the construction sequence. Compared to the composite case, the "no shoring wall" case exhibited higher computed peak contact stresses during the ground motion stage as well. The greater contact stresses for the "no shoring wall" case result in higher out-of-plane shear and bending moment demands on the pump station wall, demonstrating the potential benefits of considering composite liner action.

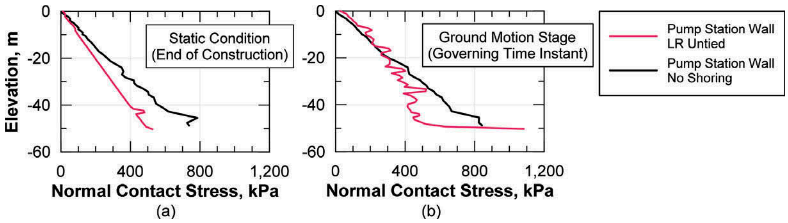

Figure 9. Comparison of average normal contact stresses on the pump station wall between the composite case and the "no shoring wall" case: (a) static condition (end of construction stages in simulation); (b) ground motion stage (governing time instant).

4.5 *Practical considerations for future studies with composite liner system*

As evident from the sensitivity analyses presented in Section 4.3, there are various aspects of the composite action between the temporary and permanent liners that could impact the structural demands on the permanent shaft, and it is not necessarily conservative to ignore the presence of the diaphragm wall. For instance, tying the permanent shaft wall to the diaphragm wall may restrain the rocking of the permanent wall, which could lead to increased demands. Composite action analyses can also help demonstrate reduction in normal contact stresses on the shaft walls (as illustrated in Section 4.4), which is associated with reductions of out-of-plane bending moment and shear demands, which can govern the design in some cases. Note that the cost and time associated with achieving greater strength/stiffness of a diaphragm wall for reliance in the permanent condition should be considered.

5 CONCLUSIONS

A number of sensitivity studies were performed to assess the potential shear and moment demands in the permanent liner for a deep shaft from the CBSIP project in San Francisco. The analyses exemplify how large shear and moment demands may be generated for deep shafts in soft over stiff soil sites, particularly at the interface of the soft and stiff soils. They also exemplify how those demands can potentially be amplified if the shaft rocking is restrained, for example, by tying the shaft to a temporary liner socketed in bedrock. Thus, it is not necessarily conservative to ignore the presence of the temporary liner in the permanent condition.

Once the governing design combinations are adequately assessed, these analyses can be used to help demonstrate the benefits to taking advantage of the temporary liner for permanent conditions, provided the strength and stiffness of the panels and panel interfaces are sufficiently high. In particular, the out-of-plane bending moment and shear demands on the permanent liner can be demonstrated to be lower through composite liner action. Without performing numerical analyses, the influence of these variables would have been difficult to evaluate, which highlights the value of performing such analyses for complex geological settings in seismically active regions.

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