SHORELINE STABILITY - SEISMIC RETROFIT OF BART SFTS

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ABSTRACT :
This paper presents the shoreline slope stability analyses performed as part of seismic vulnerability studies for the San Francisco Transition Structure (SFTS) of the Bay Area Rapid Transit system Trans Bay Tube (TBT) system. The SFTS connects the TBT to the east with Bored Tunnels to the west via multiple degree-of-freedom seismic joints. For these analyses factor of safety and yield acceleration were evaluated by state-of-the-art 3-D finite difference model (FLAC^{3D}) that incorporate both the 3-D variations in stratigraphy and the interactions with adjacent structures. Those analyses also provide critical insights into the mechanism of interaction between slope movement and the various buried structures. The project criteria required consideration of extremely high shaking levels (PGA of ~ 0.6 g) associated with an approximately 1,000-year return period event. The slope deformations were calculated by: a) a decoupled approach consisting of a 2-D equivalent linear site response analyses (QUAD4M) and Newmark-type analyses, and b) fully-coupled 2-D nonlinear finite element (PLAXIS) and finite difference (FLAC) analyses. The results of the analyses were used to develop inputs for a project global analyses and helped guide the selection/rejection of retrofit measures for the project.

KEYWORDS: FLAC, FLAC3D, PLAXIS, slope stability, dynamic deformation

1. INTRODUCTION

The Bay Area Rapid Transit District's (BART) Transbay Tube (TBT) is a 5.8-km-long immersed light rail tunnel that connects the cities of San Francisco and Oakland. Two large transition (ventilation) structures are present at either end of the TBT. The San Francisco Transition Structure (SFTS) Area (Figure 1) comprises of: 1) the San Francisco shoreline slope, 2) the SFTS itself, 3) the bored tunnels to the west of SFTS, 4) the TBT to the east, and 5) the seismic joints on each side of the SFTS. As shown in Figure 2, seismic joints, which connect the SFTS to the TBT immersed tunnel on the bay side, and to bored tunnels on its shore side, are specially designed and constructed sliding joints that allow limited movement in longitudinal and transverse directions.

Figure 1: SFTS Site Plan

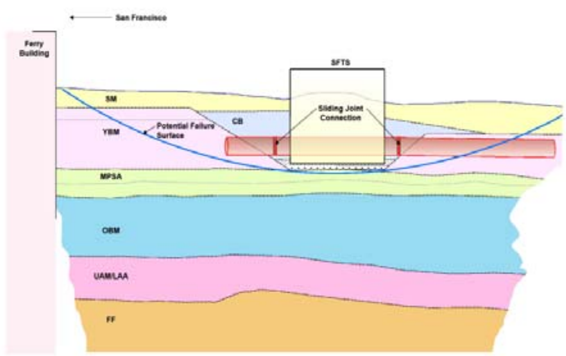


Figure 2: Longitudinal Cross Section

Permanent slope deformation during a major earthquake event may result in: 1) excessive longitudinal displacement demand at the east SFTS seismic joint, and/or 2) failure of the Bored Tunnel sections connecting the west SFTS seismic joint to the Embarcadero Station in San Francisco. This paper presented a detailed evaluation of the displacement demand at the seismic joint caused by the permanent movement of the SFTS due to seismic shoreline instability.

1.1 Methodology

The SFTS shoreline stability studies consisted of the following steps:

1. Using available geotechnical data in the project vicinity to characterize the three-dimensional (3-D) variations in stratigraphy and soil properties underneath and adjacent to the SFTS.
2. Building a 3-D FLAC^{3D} model that incorporates the 3-D subsurface soil geometry to evaluate effects from adjacent structures such as the Ferry Plaza piles, TBT, and the Bored Tunnels on the San Francisco shoreline stability.
3. Using the FLAC^{3D} model, to perform pseudostatic analyses to calculate yield acceleration (K_y).
4. Estimating permanent slope deformations for the existing shoreline conditions using a variety of techniques including: a) a decoupled approach consisting of a two-dimensional (2-D) equivalent linear site response analysis using QUAD4M (Hudson et al. 1994), and Newmark type sliding block displacement analysis, and b) 2-D nonlinear finite element/finite difference analyses using FLAC and PLAXIS.

2. SITE CONDITION

2.1. Soil Stratigraphy

Construction records indicate that the Young Bay Mud (YBM) in the SFTS area was excavated and a layer of weakly cemented sand/concrete mixture and gravels were placed as a bedding material underneath the SFTS. The SFTS was pre-fabricated onshore, floated to the project site and lowered onto the prepared bedding. Following the placement of the SFTS, the area surrounding this structure was backfilled with the excavated materials – primarily clay with a few sand layers. Strength measurements within the backfill materials appeared relatively variable. However, the available pre- and post-construction geotechnical data and available As-Built drawings were used with a few simplifying assumptions to develop an idealized 3D soil model. The idealized soil stratigraphy along the longitudinal central cross section of the SFTS is shown on Figure 2.

Natural materials outside the excavated trench include 1) Young Bay Mud, 2) Merritt-Posey-San Antonio Formation (MPSA), 3) Old Bay Mud (OBM), 4) Alameda Formation (UAM, LAA), and 5) Franciscan Complex Bedrock. In the SFTS area, YBM is present beneath the fill units down to the MPSA. The seismically-induced slope movements that impact the SFTS are largely governed by failure surfaces within the YBM. As shown on Figure 1, approximately 25 borings and 10 CPT soundings penetrated the base of the YBM in the SFTS area. Two primary subunits were identified: 1) typical YBM-type soils with shear strength increasing with depth to strengths on the order of 1,200 psf, and 2) a stiffer clay layer with shear strength on the order of 1,500 to 1,700 psf, with an average value of roughly 1,600 psf. This layer will be referred to as YBM-1600 in the following sections. This layer is on the order of 3 feet thick to the west of the SFTS and approximately 15 feet thick to the east of the SFTS. Beneath the bedding layer of the SFTS, the layer is interpolated to be between 0 to 10 feet thick.

Directly underlying the YBM is a layer of sandy clay/clayey sand (MPSA_Clay) and very dense sand (MPSA_Sand) with shear strengths on the order of 2.5 to 3.5 ksf. Underlying the MPSA are 1) OBM, stiff to very stiff clay, 2) Alameda Formation, sedimentary deposit accumulated over the weathered and erosion-directed Franciscan Complex bedrock and 3) Franciscan Complex, deep-sea sediments and related oceanic crust rocks. As described in Travasarou et al (2008), the Alameda Formation in this area is characterized by shear wave velocities that are compatible with the attenuation relationships used to develop generic rock design ground motions, and therefore the top of the Alameda formation was selected as the ground motion input depth.

In addition to the YBM layer, another potential weak plane was identified as the interface between the SFTS and its underlying bedding. A relatively low friction angle of about 30 degrees was estimated and used in the stability analyses. The soil properties adopted for this study are summarized in Table 1.

Table 1:. Soil Properties Used in Slope Stability Evaluations

Soil Type	Total Unit Weight, (pcf)	Undrained Shear Strength, Su (psf)	Shear Wave Velocity, Vs (fps)	Note
Surficial Mud	90	50 + 6D	150 + 1.6D	D = depth below Mudline
Backfill (stiff)	100 + 0.097D	25D	18*(Su)0.475	For backfills above El. -75.
Backfill (soft)	100 + 0.097D	500	18*(Su)0.475	For backfills below El. -75.
Typical YBM	100 + 0.097D	100 + 12D	300 + 1.6D	
YBM-1600	110	1600	600	Transition zone between ordinary YBM and MPSA
MPSA_Clay	119	2500	1050	Equivalent friction angle of approximately 35 degrees
MPSA_Sand	126	3500	1350	Equivalent friction angle of approximately 38 degrees
OBM	104 + 0.05D	2500 + 12.5d'	536 + 1.83D	d' = depth below the top elevation of OBM

2.2 Underground Structures

Ferry Plaza Piles: As shown on Figure 3, a reinforced-concrete pile supported pier (referred to as the Ferry Plaza platform) was constructed surrounding the SFTS. The reinforced concrete piles, which were constructed on a 16-foot square grid, are typically 138 feet long, extend through the fill, and YBM, and are tipped 10 to 30 feet within the MPSA Formation. This array of piles supporting the Ferry Plaza also reinforce potential failure surfaces within the YBM The stabilizing effect of the Ferry Plaza piles was estimated as an equivalent increase in soil strength in the footprint of the Ferry Plaza structure. The equivalent increase in soil shear strength was estimated at about 115 pounds per square foot (psf). That shear strength was estimated based on the additional shearing resistance that the piles can provide before failing in shear or bending.

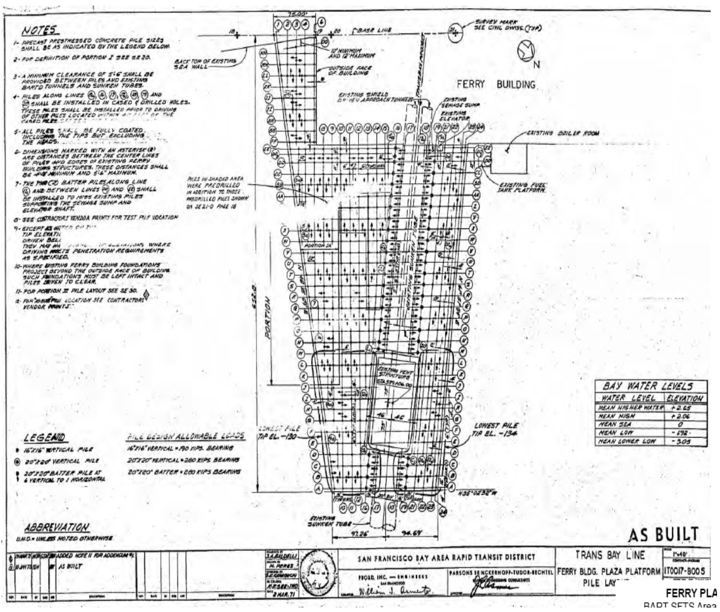


Figure 3: Ferry Plaza Piles Layout

TBT/Bored Tunnel: The TBT and bored tunnels are relatively large structures when considering the dimensions of the problem. Those structures, also tend to mitigate against shoreline instability by forcing the soil to flow around them. Additional details of this effect are presented subsequently with the $FLAC^{3D}$ pseudo-static slope stability analyses.

3. PSEUDO-STATIC SLOPE STABILITY

The assessment of shoreline instability and the interaction of the slope with the various structures in the SFTS area involve significant three-dimensional effects The bowl shape geometry of the SFTS pit and TBT trench excavation, and the stabilizing effects of the TBT structure, Bored Tunnels, and the Ferry Plaza piles all need to be considered in the stability evaluations. Because the soil could flow around the SFTS, arching effects could induce higher pressures acting on the west face of the SFTS than would be suggested by 2-D analyses. Also soil movement around and adjacent to the SFTS would result in drag forces acting along the sides of the structure. Conversely, the natural slope failures on either sides of the structure would likely be shallower. To better incorporate these 3-D effects, pseudostatic stability analyses were conducted using the 3-D finite difference computer code, $FLAC^{3D}$ (Itasca, 2006a).

The soil profile and properties used in the FLAC^{3D} model were based on field exploratory borings and CPT data, the trench excavation construction record, and marine geophysical survey data (to the east of the Ferry Plaza). The FLAC^{3D} model and the incorporation of the 3-D shape of the excavated trench are shown on Figures 4 and 5, respectively.

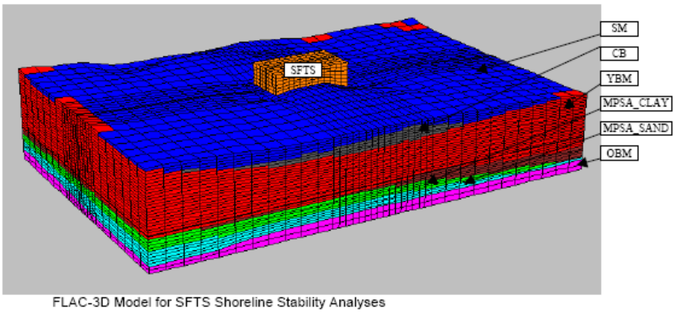


Figure 4: FLAC^{3D} Model

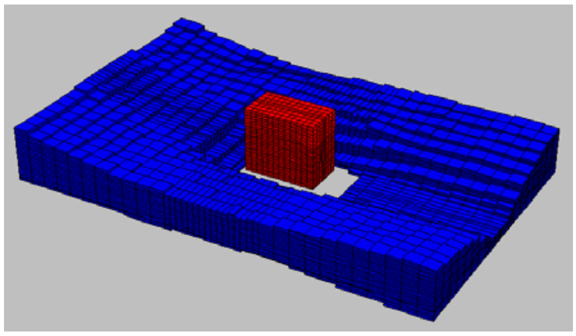


Figure 5: Base of Trench Excavation

Two sets of FLAC^{3D} pseudostatic slope stability analyses were conducted and the yield acceleration was estimated in the following steps:

- Apply gravity until the model reaches force equilibrium;
- Zero out the nodal displacements due to gravity and apply a constant horizontal acceleration in the longitudinal direction throughout the 3-D FLAC model as a body force. The model is then allowed to reach force-equilibrium and the displacement and velocity of the SFTS are monitored during the calculation; and
- Increase the horizontal acceleration incrementally until the SFTS structure is no longer stable.

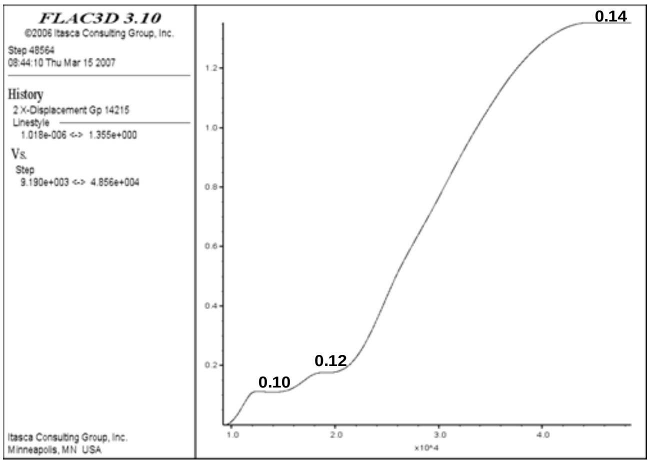


Figure 6: Yield Acceleration Calculation

Figure 6 illustrates the above procedure. The X-axis shows the number of calculation steps in 3-D FLAC to reach equilibrium. The Y-axis shows SFTS downslope displacement to the east. For the problem illustrated on that figure, the initial horizontal acceleration was 0.10g and was gradually increased to 0.14g. The SFTS moved about 1.3 feet under 0.14g. When 0.15g was applied, the SFTS could no longer reach a force-equilibrium state and therefore the yield acceleration was estimated to be 0.14g. It should be noted that the displacement magnitudes observed in these analyses are not representative of the dynamic displacements under earthquake loading conditions. The displacements merely provide an indication as to whether the analysis is numerically stable. The deformation patterns resulting from the analyses do however provide insight into the failure mechanisms and illustrate the relative stabilizing effects of the various structural elements.
The yield accelerations calculated for two conditions are discussed below:

- Base Condition – In this analysis, the only structural inclusion is the SFTS. The FLAC^{3D} model and the deformation patterns associated with the application of the horizontal force are shown on Figure 6. As shown, the SFTS structure moves significantly less than the surrounding soils, and soils in the upper part of the profile appear to flow around the SFTS. As shown on Figure 8, a circular-type failure surface crossed through the bottom of SFTS. A yield acceleration of 0.14g was calculated for this case.

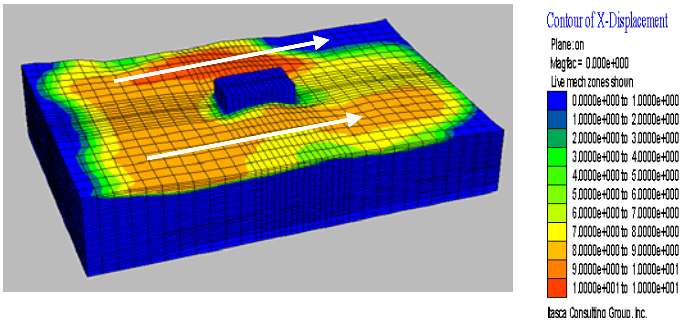


Figure 7: 3-D Slope Failure in SFTS Area

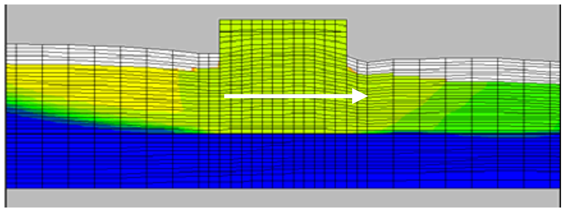


Figure 8: Failure Surface of Base Condition

- Existing Condition with Ferry Plaza Piles, TBT, and Bored Tunnels – In this analysis the TBT and Bored Tunnels were added to the base-case FLAC^{3D} model (see upper illustration on Figure 8). Additionally, the shear strengths of soils within the Ferry Plaza footprint was increased by 115 psf, as mentioned in Section 2.2. Because the longitudinal resistance offered by the tube is only mobilized after the seismic joints are closed, the Bored Tunnels and the TBT were not connected to the SFTS in the numerical model. A simplistic analysis was conducted with the assumption of a small gap between the SFTS and the TBT/Bored Tunnels. Nevertheless, the TBT and Bored Tunnels do provide a stiffening effect by forcing the soil to flow around them, and a yield acceleration of 0.17g was calculated for this case.

The deformation patterns from these analyses are shown on Figures 8 and 9. As shown on the middle illustration of Figure 9, the failure pattern along the longitudinal central cross-section, soils generally fail above and around the underground structures. Unlike the failure pattern shown on Figure 8 where SFTS generally moved with the soil, the displacement of the SFTS is much less than surrounding soils due to constraint provided by the TBT. Similar trend was also observed in the transverse cross-section on Figure 8, which illustrate the movement of soil the SFTS.

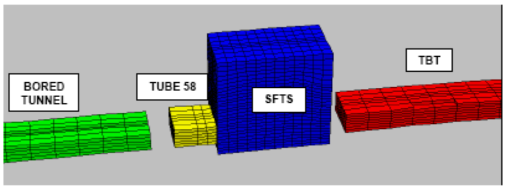
The stiffening effect of the TBT tunnel is more clearly illustrated on Figure 10 which shows the failure surface being pushed above the TBT. This irregular, non-circular type of failure surface increases the yield acceleration from 0.14g to 0.17g. By capturing the 3-D stiffening effects from the underground structures, an approximately 20% increase in the pseudostatic yield acceleration was estimated. Later, soil strengths in the 2-D dynamic analyses were calibrated so that a similar yield acceleration was modeled. Additionally, these analyses point to the need for internal structural retrofits that allow for a controlled closing of the seismic joint such that the forces imposed by the SFTS can be transferred to the TBT without significant structural damage.

4. DYNAMIC SLOPE DEFORMATION

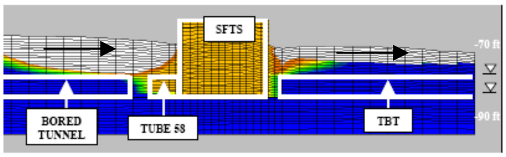
As mentioned earlier, permanent slope displacements were estimated using: 1) nonlinear 2-D dynamic analyses, and 2) decoupled QUAD4M/Newmark approaches. The 2-D nonlinear dynamic analyses were conducted using FLAC and PLAXIS. Each of these analyses was conducted for both polarities of the seven design ground motions, resulting in a total of 14 runs for each method. The estimated deformations were subsequently incorporated (Singh et al, 2008) in the project global analyses that were used to confirm that the seismic joint was capable of accommodating the displacement and force demands induced by the project design earthquake.

4.1 Two Dimensional Finite Difference Analyses – FLAC

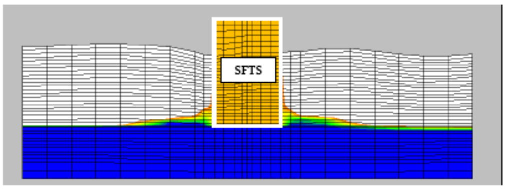
Slope deformation along the longitudinal cross section at the centerline of SFTS was analyzed. The soils in the FLAC analyses were simulated using the hysteresis model developed by Itasca (2006b). The hysteresis model uses a nonlinear backbone curve with unloading and reloading defined by Masing criteria. The resulting stress-strain loops produce hysteretic damping. A small amount of Rayleigh damping was added to provide numerical stability at low strain levels. The Itasca hysteresis model also allows for the definition of a Mohr-Coulomb type peak strength for each soil unit.

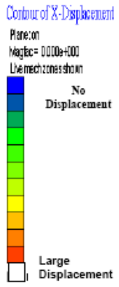


FLAC-3D Model including TBT, Tube58, and Bored Tunne



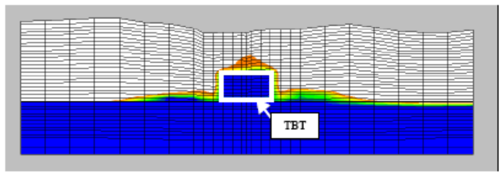
Downslope Failure Pattern along the Longitudinal Central Cross-Section



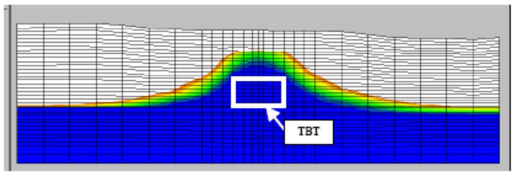


Downslope Failure Pattern along the Transverse Central Cross-Section

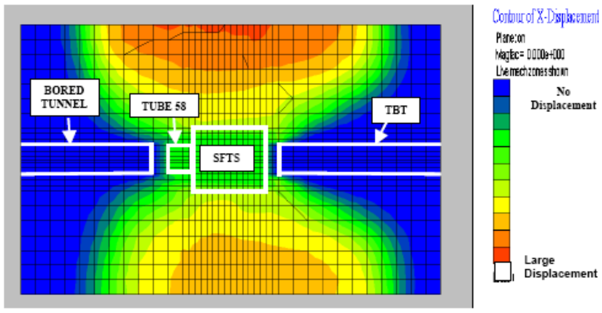
Figure 9: Slope Deformation Pattern – Existing Condition



Downslope Failure Pattern along the Transverse Cross-Section (20 ft east of SFTS)



Downslope Failure Pattern along the Transverse Cross-Section (90 ft east of SFTS)



Downslope Failure Pattern at Tube-Level Elevation (Elevation –80ft)

Figure 10: Slope Deformation Pattern – Existing Condition

Pseudostatic 2-D FLAC analysis was conducted to calibrate the 2-D model soil properties. Calibration involved increasing, the strength and stiffness of soils above the MPSA layer until yield accelerations estimated in 2-D FLAC were the same as in the pseudostatic FLAC^{3D} analyses. The required yield acceleration of 0.17g (FLAC^{3D} estimate for the Existing Condition) required increasing the strength and stiffness of the soils above the MPSA layer by 9%. The calibrated strengths were then used in dynamic 2-D FLAC analyses to predict the slope deformations that incorporate the stabilizing effects of the tube, bored tunnels, and Ferry Plaza Piles.

Figure 11 shows an example horizontal deformation profile form the dynamic 2-D FLAC analyses. The deformation patterns suggest a failure surface that is generally similar to the critical failure surface predicted in the pseudostatic analyses. The failure surface in this case goes through the gravel layer underneath the SFTS. Figure 12 shows the permanent horizontal displacement profiles along the centerline of the SFTS for all 14 input motions. As shown on that figure:

- The estimated displacements for the Base Condition range from 2 to 21 inches, with an average displacement of 9 inches; and
- The estimated displacements for the Existing Condition (including additional resistance from surrounding structures) are slightly lower and range from 0 to 15 inches with an average of 7 inches.

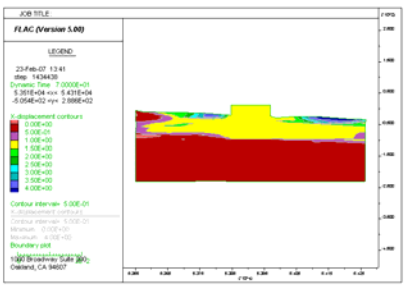


Figure 11: Horizontal Deformation Pattern

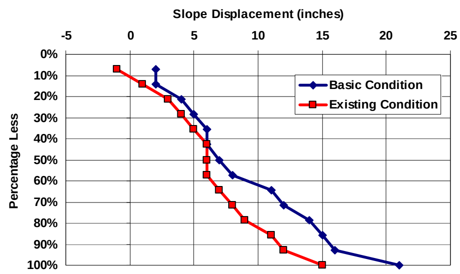


Figure 12: Lateral SFTS Displacements per FLAC2D

4.2 Two-Dimensional Finite Element Analyses - PLAXIS

Figure 13 shows the mesh used in the PLAXIS model. PLAXIS analyses were conducted using a nonlinear hyperbolic model in which the reference shear strain was modified to best match the modulus reduction curve for each of the soil units. Although the hysteretic damping develops at all strain levels, a small amount of Rayleigh damping (1 percent) was included to supplement damping levels at low strains.

The 2-D PLAXIS dynamic analyses were only conducted for the Base Condition. The estimated displacements from the 14 different ground motions are summarized on Figure 14. As shown on that figure:

- The estimated displacements range from less than 1 inch to 9 inches, with a median displacement of 4 inches, and
- The horizontal displacements predicted using PLAXIS are typically 50 to 80 percents less than predicted in the FLAC analyses.

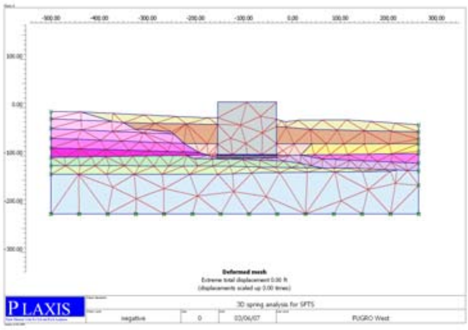


Figure 13: PLAXIS 2D Mesh

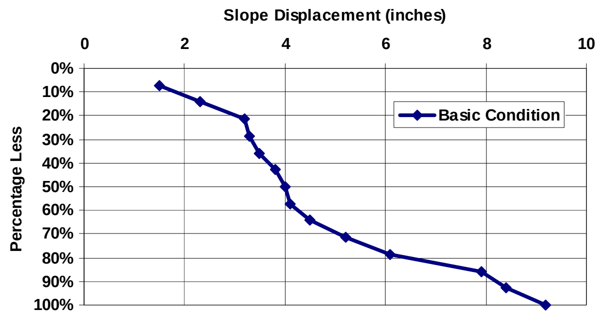


Figure 14: Lateral SFTS Displacements per PLAXIS

4.3 Two-Dimensional Equivalent Linear Slope Stability Analyses

Seismic slope deformations were also estimated using a decoupled approach. That approach combined estimates of yield acceleration (K_y) obtained from a pseudostatic slope stability analysis with horizontal equivalent acceleration (HEA) time history calculated using the equivalent linear computer program QUAD4M. Figure 15 presents the lateral deformation estimates versus K_y using the estimated HEA values. As shown on that Figure:

- Using the HEA time histories, the estimated displacements for ky of 0.14g (Base Condition) range from less than 1 inch to about 12 inches, with an average value of about 7 inches;
- The estimated displacements for ky of 0.17g (the Existing Condition) range from less than 1 inch to about 11 inches, with an average value of about 7 inches; and
- The k_y values corresponding to an average displacement of 1 inch is about 0.2g.

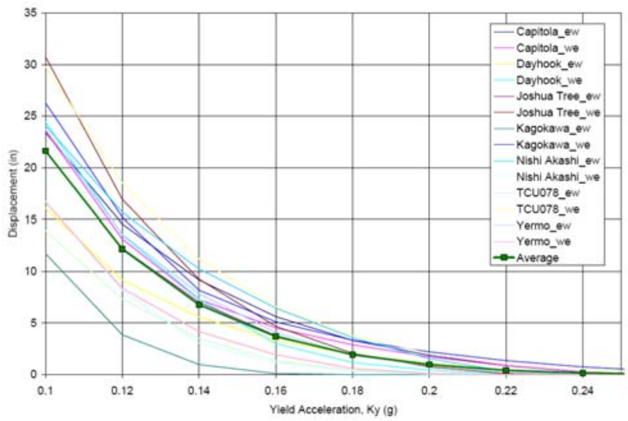


Figure 15: Yield Acceleration vs. Displacement of SFTS

5. SUMMARY AND CONCLUSION

The evaluation of the impact of seismically-induced slope instability on a relatively large, buried structure involved consideration of complex 3D mechanisms and nonlinear soil behavior. Extensive site characterization was required to properly define these variations, and a 3-D numerical model was developed to evaluate the problem. The use of the 3D-model was extremely useful in understanding the behavior of the slope, and the interaction of the slope movement with the various existing structures. In particular, the analyses:1) helped illustrate the flow of soft soils around the structures which would not be captured in 2D or linear 3D analyses; 2) helped quantify the additional resistance to slope movement offered by the presence of the various structures in particular the Tube tunnel downslope of the SFTS; and 3) helped illustrate the need and benefit of incorporating an internal retrofit to facilitate a controlled closure of the joint.

The results of the 3D effects were incorporated in a 2D model and dynamic analyses conducted to quantify the potential magnitude of downslope movement for the project design earthquakes. A number of different techniques were used and all of them helped confirm that the magnitude of movement, although significant, was relatively low with the average permanent displacement estimates on the order of 8 inches. Those displacements were subsequently incorporated in a global analyses model of the entire TBT system to confirm that the joint was able to meet the project criteria for the design earthquake event.

The relatively small magnitudes of predicted slope movement, the observed mechanisms where slope movement occurs primarily with soil moving around the structure, and the significant resistance to downslope movement offered by the Tube structure once the seismic joint is closed allowed BART to decide that specific external retrofits to reduce slope movement were not required. That decision resulted in tens of millions of dollars of costs savings and allowed for the elimination of significant construction related risks for this critical transportation infrastructure facility.

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