Seismic Analyses of the Bay Tunnel

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ABSTRACT: The Bay Tunnel is a major water supply facility to be constructed between the San Andreas Fault and the Hayward Fault in the San Francisco Bay Area. The design earthquake for the Bay Tunnel has a return period of 1,000 years and a horizontal peak ground acceleration of approximately 0.6g. A full dynamic analysis of the Bay Tunnel, considering the soil-structure interactions, was performed to evaluate the performance of the tunnel final lining system during the design earthquake. This paper discusses the methods employed to perform the dynamic analysis and the results obtained. A brief comparison of the results from the dynamic analysis with those from the closed-form solutions and numerical racking analysis is also presented.

INTRODUCTION

The San Francisco Public Utilities Commission (SFPUC) is in the process of a major upgrade of their water conveyance system. One key element of this program is the Bay Division Pipeline Reliability Upgrade Project, which consists of constructing a new 21-mile Bay Division Pipeline No. 5. This pipeline extends from the Irvington Tunnel Portal in Fremont, California to the Pulgas Tunnel Portal near Redwood City, California. It includes a tunnel under the San Francisco Bay (the Bay Tunnel) and adjacent marshlands. The Bay Tunnel is approximately 5 miles long and crosses under the Bay in soil conditions (i.e., soft ground). Excavation and construction of the Bay Tunnel will be staged from two shafts constructed at either end of the tunnel: the Ravenswood Shaft at the west end, and the Newark Shaft at the east end.

The Bay Tunnel will have a two-pass lining system with a watertight precast concrete segmental lining as the initial lining. The precast concrete segmental lining thickness will be about 10 inches with an inside diameter of approximately 12 feet. The final lining for the tunnel will be a Grade 40 steel pipe with an inside diameter of 9 feet and a wall thickness of 9/16 inch. The annular space between the steel pipe and the segmental lining will be backfilled with low-density cellular concrete.

The Bay Tunnel is located between two of the most seismically active faults in the Bay Area, the San Andreas Fault to the west and the Hayward Fault to the east. An essential aspect of the Bay Tunnel design was evaluation of the response of the tunnel facilities to the seismic activities of these faults. To support the design of the shafts, connecting tunnel, and pipelines, and to ensure that the seismic performance objectives defined by the SFPUC can be met (SFPUC, 2006), detailed seismic analyses were completed for the Bay Tunnel. This paper discusses the methods employed to perform a dynamic analysis and the results obtained. A brief comparison of the results from the dynamic analyses with those from the closed-form solutions and numerical racking analysis is also presented.

TUNNEL PROFILE AND GEOLOGICAL CONDITIONS

The proposed Bay Tunnel vertical alignment is shown in Figure 1, along with the anticipated geologic conditions along the alignment. The tunnel has a slope of about 0.13%. The high point of the tunnel is at the east end (at the Newark Shaft), where the tunnel invert is at at Elevation -71 feet. The low point is at the west end (at the Ravenswood Shaft), where the tunnel invert is at Elevation -104 feet. The ground cover above the tunnel varies along the alignment. The lowest cover (measured from the ground surface to the tunnel springline) is about 50 feet at the deepest point under the Bay. The greatest cover is about 110 feet at the low point of the tunnel alignment.

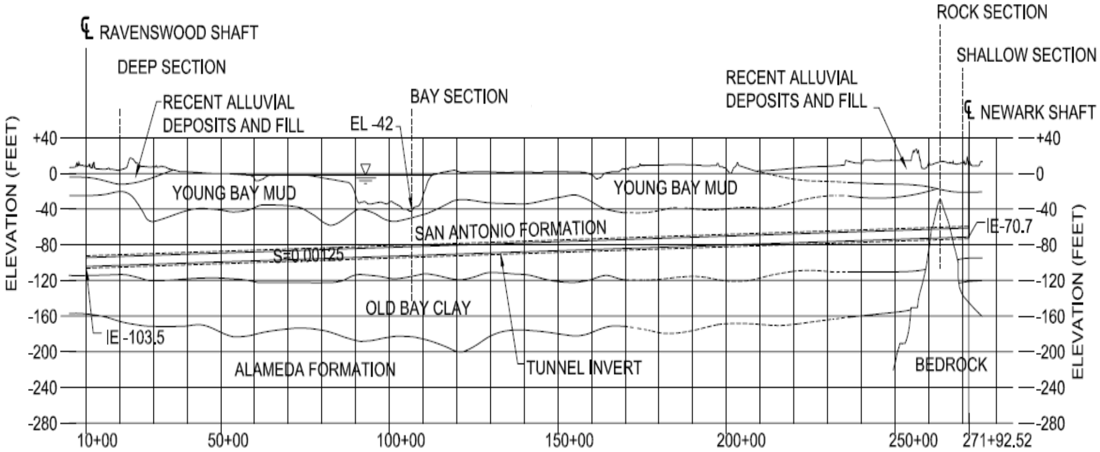

Figure 1. Geologic Profile and Vertical Alignment of the Bay Tunnel

Soil deposits present along the tunnel alignment can be grouped into six principal strata: artificial fill, Young Bay Mud (YBM), San Antonio (SA) Formation, Old Bay Clay (OBC), Alameda Formation, and Franciscan Complex bedrock. The entire Bay Tunnel will be constructed within the SA Formation, except at a reach between Station 260+00 and Station 270+00, where the tunnel will be excavated within the Franciscan Complex rock.

Four tunnel sections were selected for two-dimensional seismic analyses. They are called the Shallow, Bay, Deep, and Rock Sections. The locations of these sections are shown in Figure 1. These sections were selected to represent the range of overburden depths and anticipated ground conditions. Evaluation of the seismic performance of the tunnel at these sections is considered to encompass the range of seismic responses expected along the alignment. A typical cross-section of the tunnel is shown in Figure 2.

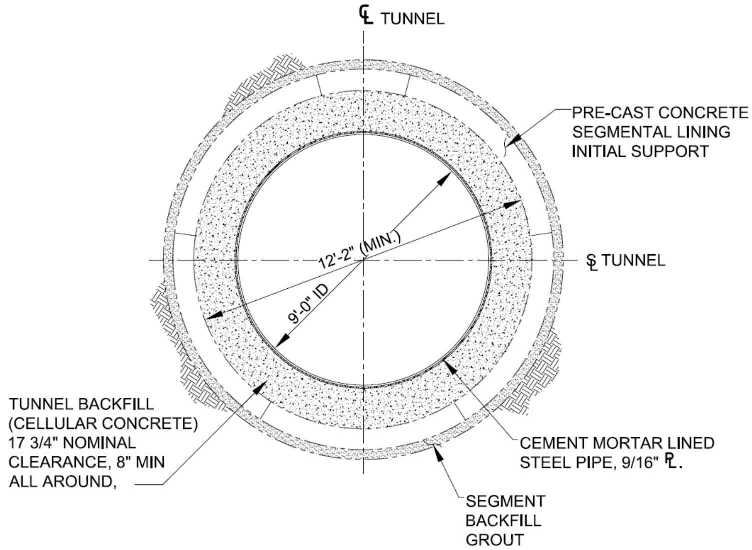

Figure 2. Typical Cross-section of the Bay Tunnel for Seismic Analyses

The key parameters associated with the four tunnel sections and used for seismic analyses are summarized in Table 1. The peak ground velocities were calculated based on the probabilistic seismic hazard analysis (PSHA) (Jacobs Associates, 2007a). The average material property values of various soil and rock formations are presented in Table 2. Soil formation properties were estimated based on laboratory tests and field downhole suspension loggings. Franciscan rock properties were assumed based on relevant data collected for other tunnel projects in the Bay Area. The maximum dynamic shear moduli were calculated from the shear wave velocities and unit weights with a 30% reduction for cyclic degradation at the expected seismic strain levels.

Table 1. Parameters Associated with Tunnel Sections for Seismic Analysis

Parameter	Shallow Section	Bay Section	Deep Section	Rock Section
Overburden Depth to Tunnel Springline (ft.)	76	50	110	81
Groundwater Head (ft.)	76	95	110	81
Head of Internal Water Pressure (ft.)	473	494	505	475
Peak Ground Velocity (PGV) at Tunnel Invert (ft./s)	2.37	2.99	3.42	2.37

Table 2. Material Properties of Soil and Rock Formations

Material Constant	Fill	YBM	SA Formation	OBC	Alameda Formation	Franciscan Rock
Unit Weight (pcf)	125	125 (90[a])	130	115	130	140
Poisson's Ratio	0.35	0.35	0.35	0.35	0.35	0.3
Static Shear Modulus (ksf)	50	130	280	230	850	6,700
Static Undrained Shear Strength (psf)	280	100-900	1,500	1,500	2,500	6,000
Maximum Dynamic Shear Modulus (ksf)	287	679	2,052	1,580	3,235	17,280
Dynamic Undrained Shear Strength (psf)	560	1,000	2,000	2,900	3,000	7,000
Shear Wave Velocity (ft./s)	325	500	850	800	1,330	2,000

Note: [a] Unit weight for YBM under the San Francisco Bay (for Bay Section).

GROUND MOTIONS OF DESIGN EARTHQUAKE

The design earthquake for the Bay Tunnel was developed using a probabilistic seismic hazard analysis with two major controlling earthquakes: a M 7.9 earthquake on the San Andreas Fault and a M 7.1 earthquake on the Hayward Fault (Jacobs Associates, 2007a). The San Andreas and Hayward Faults are as close as 8 and 5 miles, respectively, to the Bay Tunnel. The design earthquake has a 5% probability of exceedance in 50 years (a return period of 1,000 years) and a horizontal peak ground acceleration (PGA) of approximately 0.6g. The ground motion time histories of the design earthquake were generated for a hard rock site condition with V_{s30} (shear wave velocity in the top 30 m) equal to 5,000 ft./sec based on the following three seed earthquake records:

- 1999 M 7.4 Kocaeli, Turkey – Arcelik station
- 1992 M 7.5 Landers, California – Joshua Tree station
- 1999 M 7.6 ChiChi, Taiwan – TCU089 station

With these three seed earthquake records, a total of nine sets of spectrally matched site-specific ground motion time histories were calculated for three locations along the tunnel alignment: the Ravenswood Shaft, the Bay Tunnel, and the Newark Shaft (Jacobs Associates, 2007a). Each set of time histories contains two horizontal (fault-normal and fault-parallel) components and one vertical component. A typical time history of accelerations for the horizontal component 2 (ChiChi H2) (fault-parallel) at the Ravenswood Shaft (for the Deep Section) is shown in Figure 3.

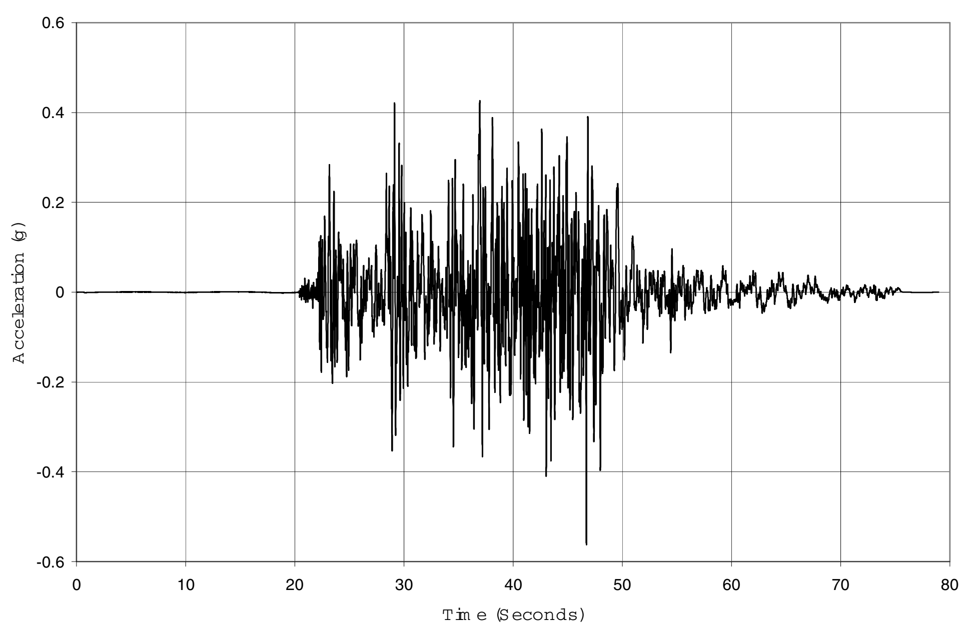

Figure 3. Typical Time History of Accelerations for the Bay Tunnel (Deep Section)

SHEAR-MODULUS DEGRADATION AND DAMPING

The amplitudes of seismic waves attenuate with distance when traveling through soil and rock. Factors affecting this attenuation include energy dissipation, volume changes, and stiffness degradation of soil and rock subjected to seismically-induced cyclic ground motions. Stiffness (shear-modulus) degradation and resulting damping of soil and rock increase with the increase of cyclic shear strains. This nonlinear, hysteretic characteristic of soil formations present along the Bay Tunnel was investigated using cyclic triaxial tests. Results of these cyclic tests using the soil samples taken from the SA Formation indicate that the shear-modulus reduction and damping curves for this soil formation can be represented by the empirical curves proposed by Vucetic and Dobry for stiff clay with a plasticity index (PI) of 50, as shown in Figure 4 (Vucetic and Dobry, 1991). For the Franciscan rock, the site-specific data are not available, and assumed shear-modulus reduction and damping curves, also shown in Figure 4, were used. These curves were selected from the seismic analyses for another Bay Area tunnel project: the Caldecott Fourth Bore Tunnel Project (Earth Mechanics, Inc., 2007).

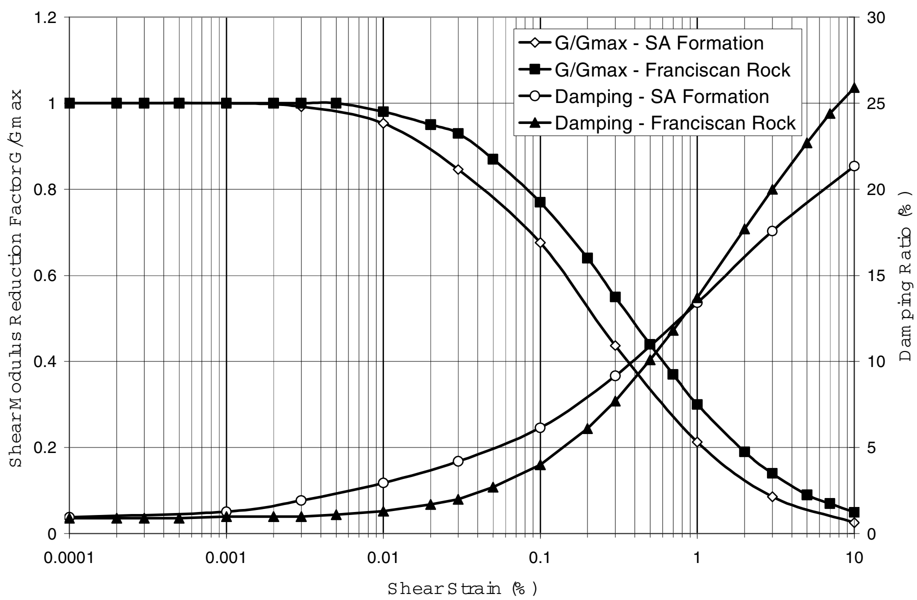

Figure 4. Shear-Modulus Reduction and Damping Curves for San Antonio Formation and Franciscan Rock

METHODS OF ANALYSIS

Seismic analyses for the Bay Tunnel were performed using three methods (Jacobs Associats, 2007b): closed-form solutions based on Hashash et al. (2001), numerical racking analysis based on an equivalent linear elastic approach, and full nonlinear dynamic analysis using a hysteretic damping model. Closed-form solutions are simplified methods based on theory of elasticity. In these solutions, the free-field deformation and ground-lining interaction approaches are used to examine the longitudinal and transverse effects caused by seismic waves. Racking analysis is used to quantify the racking deformations (shear distortions) and their effects on the tunnel final lining due to seismically-induced free-field shear strains in the ground. The racking and dynamic analyses were performed using the FLAC computer program (Itasca, 2005). This paper focuses on the methods employed for the dynamic analyses and the results obtained. Some of the results from the other two methods will be presented for comparison.

A dynamic analysis is usually considered more realistic than closed-form solutions or racking analyses for projects with complex features, such as tunnels, because dynamic analysis can account for more complex soil-structure interactions and can more realistically simulate the motions of a tunnel when subjected to earthquake shaking. Ground motion time histories and a wide range of frequency spectra can be directly input into a dynamic analysis. The behavior and effect of more complex tunnel deformations than only racking can be evaluated. Shear-modulus reduction and damping of soil and rock during earthquake shaking can be incorporated into the dynamic analysis in a realistic manner.

Other key aspects of this approach are that the static effects of in situ ground stresses, groundwater pressures, and tunnel internal water pressure can also be taken into account. These effects are assessed by simulating the tunnel excavation, initial lining installation, and final lining installation prior to applying the dynamic loads to the FLAC model.

The static effect was modeled using the static properties of soil or rock and a Mohr-Coulomb failure criterion to govern the behavior of the soil or rock during excavation and normal operational conditions. Under earthquake shaking, the response of the ground and tunnel were simulated using the dynamic properties of soil or rock. In addition, the ground was assumed to follow a hysteretic damping model during earthquake shaking, based on the results of laboratory cyclic tests (as noted above).

<u>Boundary Conditions</u>

In a FLAC dynamic analysis, two boundary conditions are usually used (Itasca, 2005). One is the quiet boundary, and the other is the free-field boundary. A quiet boundary involves dashpots attached independently to the boundary in the normal and shear directions, which provide viscous normal and shear tractions. The quiet boundary prevents the reflection of outward-propagating waves back into the model and allows the necessary energy radiation. A free-field boundary enforces free-field motion at the lateral boundaries of the model such that these boundaries retain their non-reflecting properties (i.e., outward waves are properly absorbed). In FLAC, the free-field boundaries are coupled with viscous dashpots to simulate a quiet boundary.

In a dynamic analysis, seismic ground motions (accelerations or velocities) as a function of time are applied to the model base. For a model with a quiet boundary on the base, the acceleration or velocity boundary condition cannot be applied directly because it is not compatible with the quiet boundary in FLAC. Instead, a shear or normal stress time history is applied. The shear or normal stress time history is converted from the shear or normal component of seismic velocities using the following relations (Itasca, 2005):

$$\sigma_s(t) = 2\rho C_s v_s(t) \tag{1}$$

$$\sigma_n(t) = 2\rho C_p v_n(t) \tag{2}$$

Where σ_s is applied shear stress as a function of time; σ_n is applied normal stress as a function of time; ρ is mass density; C_s is speed of s-wave propagation through ground; C_p is speed of p-wave propagation through ground; v_s is input horizontal component of velocity as a function of time; v_n is input vertical component of velocity as a function of time.

In this study, only the ground motions associated with the shear waves were considered; those associated with the compressional waves were neglected because their effects on the tunnel performance are not very significant compared to those caused by the shear waves. In the Tunnel Analyses, the ground motion time histories determined for the three locations (Ravenswood Shaft, tunnel segment under the Bay, and Newark Shaft) were applied to the Deep, Bay, and Shallow Sections, respectively. The time histories applied to the Rock Section were the same as those for the Shallow Section. The dynamic properties presented in Table 2 were used in the dynamic analyses. Figure 5 shows a typical FLAC model with its dimensions (140 feet × 140 feet) and boundary conditions.

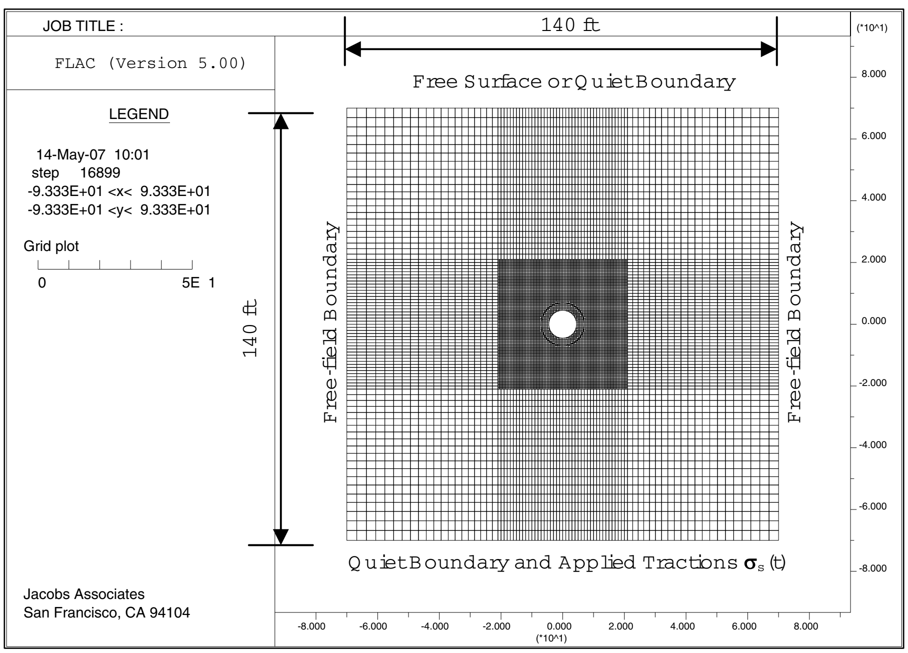

Figure 5. Model Configurations and Boundary Conditions for Dynamic Analysis

Calibration of Shear-Modulus Reduction Curves

Hysteretic damping models used in the dynamic analysis impose the shear-modulus reduction and damping ratios as a function of cyclic shear strains. A built-in three-parameter hysteretic damping model in FLAC was selected for the dynamic analysis. This three-parameter hysteretic damping model is defined in Eq. 3 (Itasca, 2005), and its parameters were calibrated by best-fitting to match the shear-modulus reduction curves proposed for the SA Formation and Franciscan Complex rock (shown in Figure 4). The resulting model parameters are: a=1.01, b=-0.55, and x_o=-0.25 for SA Formation; and a=1.01, b=-0.525, and x_o=-0.0005 for Franciscan rock.

$$M_s = \frac{a}{1+\exp[-(L-x_o)/b]} \quad (3)$$

Where M_s is normalized shear modulus; L is logarithmic shear strain.

To verify that the shear-modulus reduction and damping curves used were appropriate for the analyses, a series of numerical simple shear tests with a single-zone FLAC model were carried out. In these tests, damping ratio curves corresponding to the shear-modulus functions were developed over a range of shear strains of interest. Both the shear-modulus reduction and damping ratios as a function of shear strains were recorded during these test runs, and the three parameters were adjusted if necessary to produce a reasonable fit. Figure 6 shows a comparison of the calibrated curves with the proposed functions for the SA Formation. A similar comparison was made with Franciscan rock. It appears that the calibrated shear-modulus reduction and damping curves are in good agreement with the proposed curves over the range of shear strains considered suggesting that the hysteretic damping model in FLAC is reasonable for use in the dynamic analysis of the Bay Tunnel.

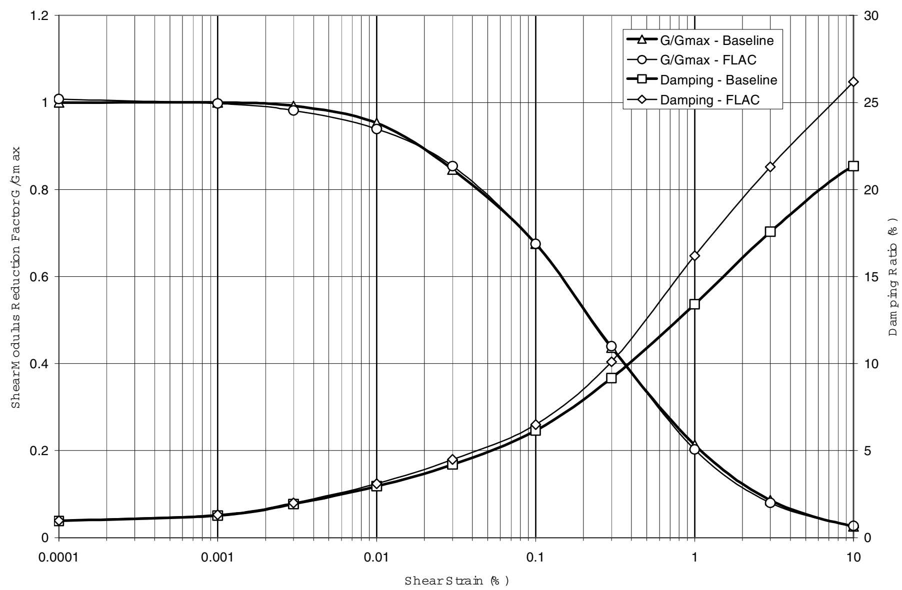

Figure 6. Comparison of Shear-Modulus Reduction and Damping Curves for San Antonio Formation

PREDICTED SEISMIC RESPONSE OF BAY TUNNEL

The applied ground motions used in the seismic analyses were shear waves propagated vertically from the bedrock to the ground surface. The critical ground deformations caused by the shear waves were the shear distortions. Shear distortions induce additional shear strains in the ground and the tunnel final lining. Since the dynamic analyses are two-dimensional, evaluation of the tunnel response is limited to the transverse effects. Variations of the tunnel shear distortions and stresses in the final lining along the tunnel alignment can be assessed by comparing the results for different sections, which represent the tunnel at various critical locations.

Seismically-induced Tunnel Shear Distortions

The shear distortions defined here are the relative horizontal displacements between the tunnel crown and invert. Time histories of the calculated shear distortions at the four tunnel sections are presented in Figure 7. It is apparent that the shear distortions are transient, following similar cycles of ground displacements. The magnitude of shear distortions is highly dependent on the amplitude of shear waves and ground conditions. The peak shear distortion occurs when the amplitude of shear waves reaches its peak over the duration of shaking.

It is noted that the input ground motion time histories vary along the tunnel alignment. These variations can be seen from the PGV values presented in Table 1. Among the four tunnel sections analyzed, the highest PGV is associated with the Deep Section, and the lowest with the Shallow and Rock Sections. This is why the calculated shear distortion at the Deep Section is larger than that at the Shallow Section, even though there is greater ground cover at the Deep Section.

For the Shallow and Rock Sections, the same time histories of shear wave velocities were used as input to the analyses. But due to the difference in ground stiffness, the calculated shear distortions are different at these two sections. The higher ground stiffness associated with the Rock Section reduces the shear distortion by about 0.2% over that of the Shallow Section. These results suggest increasing the stiffness of the backfill materials between the initial segmental lining and final lining may be a viable approach to limit shear distortions, if required.

Near the soil and rock transition zone, a differential shear stain of about 0.2% was estimated. This differential shear strain is expected to occur over a distance of more than 50 feet, so the transition between rock and soil will be gradual. In addition, the presence of concrete segments and backfill grout will serve as a cushion to make the transition less abrupt. Therefore, this differential shear strain is not expected to have a significant impact on the tunnel final lining performance.

Seismically-induced Stresses in Final Lining

Time histories of the calculated hoop stresses that developed in the final lining are shown in Figure 8 for the four tunnel sections. Along the entire tunnel alignment, the final lining is in tension. Like shear distortions, hoop stresses are dependent on the amplitude of shear waves and ground conditions. The maximum tensile hoop stress is predicted to be about 29 ksi, occurring at the Deep Section. This stress is below the allowable design stress of 30 ksi for the Grade 40 steel specified for the final lining (Jacobs Associates, 2007b). As indicated in Figure 8, stresses are transient, and the maximum stress occurs only once during the design earthquake. Stresses in the second-highest load cycle are approximately 25 ksi, and average dynamic stresses are even lower at only about 18 ksi, which is about 40% lower than the highest stress of 29 ksi and only about 45% of the yield stress of 40 ksi.

Comparison of Results from Different Methods

Table 3 compares the seismically-induced maximum hoop stresses in the final lining as calculated from closed-form solutions, numerical racking analysis, and dynamic analysis. The estimated stress levels calculated using the three different methods are generally consistent, within 2 to 4 ksi of each other. The largest difference is associated with the Deep Section. The stresses from the dynamic analysis are higher than those from the other two methods. One possible reason is that in the dynamic analysis, the ground degradation represented by the shear-modulus reduction curve is accounted for, while in the closed-form solutions and racking analysis, the ground degradation is ignored. This may result in a larger shear distortion, and therefore a higher hoop stress in the final lining.

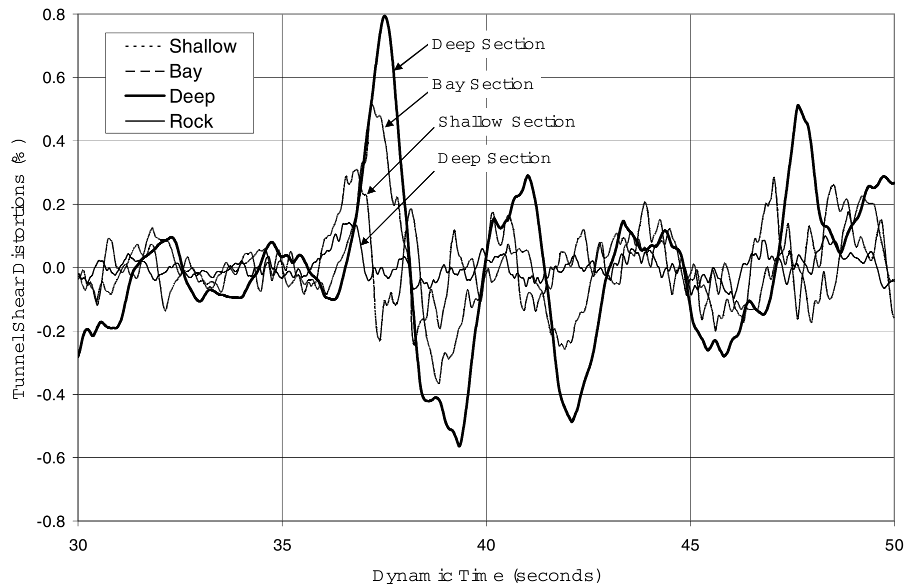

Figure 7. Time Histories of Final Lining Shear Distortions at Various Sections

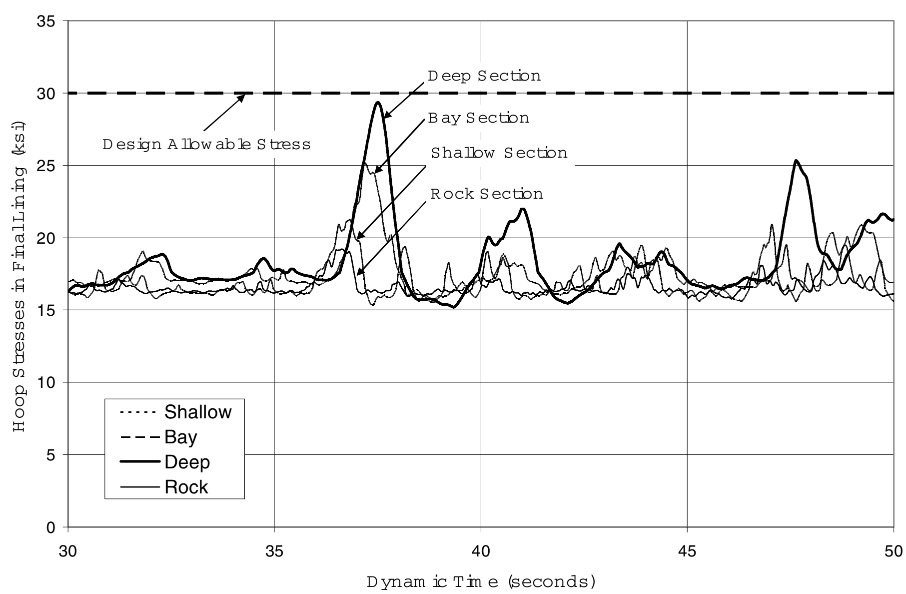

Figure 8. Time Histories of Hoop Stresses in Final Lining at Various Sections

Table 3. Maximum Hoop Stresses in Final Lining Due to Seismic Effects

Tunnel Section	Closed-form Solutions (ksi)	Racking Analysis (ksi)	Dynamic Analysis (ksi)
Shallow	±5.1	±5.1	±5.0
Bay	±6.4	±6.4	±4.8
Deep	±7.4	±7.4	±12.4
Rock	±1.6	±3.3	±3.0

Note: Maximum stresses can be tension (+) or compression (-).

CONCLUSIONS

The following conclusions may be drawn from the seismic analyses completed for the Bay Tunnel:

- Use of a hysteretic damping model to represent the ground degradation during seismic ground motions is reasonable for the dynamic analysis of the Bay Tunnel.
- Dynamic analysis can produce more realistic results since it can account for various load combinations considered in the tunnel design.
- The predicted response of the proposed Bay Tunnel final lining during the design earthquake meets the SFPUC's seismic performance objectives.
- Closed-form solutions, numerical racking analysis and dynamic analysis appear to produce generally consistent results.

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