SEISMIC SOIL-TUNNEL-STRUCTURE INTERACTION ANALYSIS AND RETROFIT OF THE POSEY-WEBSTER STREET TUNNELS

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ABSTRACT

The purpose of this paper is to present the Soil-Tunnel-Structure Interaction analysis performed for the "innovative" seismic retrofit of the Posey and Webster Street Tunnels located in the San Francisco Bay area. The tunnels are situated between two major faults, the Hayward Fault and the San Andreas Fault (Figure 1). Due to its close proximity to the site, the Hayward Fault is the controlling fault for the Safety Evaluation Earthquake (SEE) event. The Hayward Fault is capable of generating a peak horizontal rock acceleration of 0.76g, and horizontal ground accelerations in excess of 0.5g. The duration of the seismic event may exceed 45 seconds. The high peak rock acceleration of the design earthquake, and the long duration of the event, will allow the build-up of pore pressures. This could lead to extensive liquefaction with the consequent flotation of the tunnels resulting in a structural failure. The 1989 Loma Prieta earthquake (estimated horizontal ground acceleration of about 0.2g at the site) was the most recent major seismic event that occurred near the Posy and Webster Street Tunnels. The Post Earthquake Investigation report indicated evidence of liquefaction on the Island of Alameda and in the vicinity of the tunnels. Consequently, in 1997 ground improvement and retrofit of a number of structural components of tunnel segments became part of the retrofit plan for the tunnels. The ground improvement will prevent flotation of the tunnels, and the retrofit of structural components will prevent major damage to the tunnels themselves. Located at each end of the tunnel is a portal structure. It was necessary to design an opening between the portal structures and tunnels to reduce the tunnel-portal interaction. The retrofit strategy for the structural components is to provide flexibility and significantly reduce the forces in the tunnel segments. Response-spectrum-compatible rock motion time histories were developed along the tunnel alignment. The rock motions were propagated towards the surface to obtain the ground motion at the centerline of the tunnels for the Soil-Tunnel-Structure Interaction analysis. The objectives of the analysis were to estimate the responses of the as-built and retrofit configurations of the tunnels due to seismic excitations. In addition the global model racking analyses complements the global model.

INTRODUCTION

In 1928, the Posey Tunnel, designed by George Posey, Alameda County Engineer was constructed between the Cities of Alameda and Oakland (Figure 2). Precast tunnel segments were placed and aligned in a dredged trench, and connected underwater. For over 34 years the Posey Tunnel, was the only tunnel across the Oakland Estuary and served two-way traffic.

In 1962, the Webster Street Tunnel, was designed, constructed and financed by the California Department of Transportation. The tunnel was built to relieve the Posey Tunnel traffic congestion. The "trench" construction system similar to the Posey Tunnel was the method used in constructing the Webster Street Tunnel.

Each of the reinforced concrete tunnels is more than 4400 feet in length and includes a 3500-foot of buried ventilating section. At its low point the tunnel roadway is about 70 feet below the water surface. At a specially

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constructed drydock in Alameda, the tunnel segments were cast. Each tunnel segment is 200 feet long with a 37 ft outside diameter and 2.5 ft thick circular wall. Temporary watertight bulkheads were placed at each end. The segments were floated in the estuary and barged into position at the tunnel site nearly one mile away. In these trenches four steel pipe piles, 12 inches in diameter, were driven at the location of the leading end of each tunnel segment. Accurately positioned by tugboats, the tunnel sections were filled with sand and water until they slowly sank to a point just above the final position, and were ready to be coupled to the end of previously placed segment. Divers relying on a sense of touch, worked in 90 feet of muddy water to insure proper alignment of the tunnels. Previously erected towers at each end of the segments allowed engineers to check the position from the surface of the water. After all sections were placed, the joints were sealed with steel and concrete. Underneath, the tunnels were filled with loose sand. The Posey trench backfill was barge-dumped with soft clay and loose sand from the Bay. The Webster trench backfill was barge-dumped with clean sand with very little silt. Because of different backfill materials placed at each tunnel; it was necessary to consider seismic retrofit measures for each tunnel's backfill seperatley.

This paper summarizes the development of response-spectrum-compatible rock motion time histories, liquefaction problem and Soil-Tunnel-Structure Interaction using nonlinear finite element analysis of the tunnels.

STRATIGRAPHY OF THE SITE

The geophysical logging and data reduction indicated that, in general, the natural soils underlying the tunnels are competent materials. The entire sequence of strata consists of five major layers, each with its distinctive compositions (Figure 6). At the base, approximately 600 feet below the ground surface is the bedrock. Above the bedrock is a dense medium to coarse gravelly sand with layers of sandy silt to clayey silt, named the Alameda Formation. Above this, under the estuary and the Oakland side is the Posey Formation. The Posey Formation is composed of stiff to very stiff blue-gray sandy-silt to clayey-silt. Above the Posey formation is very dense Merritt sands. Above the Alameda Formation on the Alameda side lies Merritt sands. Above Merritt sands lies the soft Bay Mud.

EARTHQUAKE INDUCED GROUND MOTIONS

The Posey and Webster Street Tunnels are located between two major faults (Figure 1). The site is approximately 3.7 miles west of the Hayward Fault and 14.3 miles east of the San Andreas Fault. Due to its close proximity to the site, the Hayward Fault is the seismic controlling fault for the Safety Evaluation Earthquake (SEE) event. Based on the study for the major toll bridges in the Bay Area a maximum earthquake of magnitude 7.25 could be generated by the Hayward Fault. This event is capable of generating a peak horizontal rock outcrop acceleration of 0.76g and vertical acceleration of 0.80g [4].
A set of three-component-response-spectrum-compatible rock motion time histories was generated [7] to perform seismic Soil-Tunnel-Structure Interaction analysis for the tunnels. TAFT records of the 1952 Kern County earthquake were selected as the initial seed time histories (Figure 3). The Kern County earthquake is used because of its long duration and response over a wide period range. The time histories were developed so that they provided a reasonable match to the corresponding selected target response spectra at station A (Figure 4,5 and 6). The multiple-station, coherency-response-spectrum compatible rock motions were generated from the reference motions at station A along the length of the tunnel at 50 feet intervals to station B.

The rock motions were propagated towards the surface to obtain the ground motion at the centerline of the tunnels using one-dimensional strain-compatible equivalent linear wave propagation formulation [6]. These ground motions are preliminary and are under review. The presence of the Bay Mud at the Alameda side (Figure 6) tends to amplify the ground motion and could induce significant forces and displacements on the tunnel segment adjacent to the Alameda portal building.

LIQUEFACTION CONCERNS

The geotechnical site exploration indicates that the Webster Street Tunnel backfill is loose to medium sand and the Posey Tunnel backfill is a mixture of clay with zones of loose sand. Figure 7 presents a plot of field N-values versus depth for both tunnels. It can be seen that the N-values vary from 4 to 20, except for erratic higher values. In assessing the liquefaction potential, shear stress time histories corresponding to the boring being analyzed were developed. The simplified Seed & Idriss procedures were used to assess the liquefaction potential [11]. The high peak rock acceleration of the design earthquake, and in particular the long duration in excess of 45 seconds, will allow the build-up of high pore pressures. This could cause extensive liquefaction with the consequent flotation and potential failure of the tunnels. The 1989 Loma Prieta earthquake with an estimated horizontal ground acceleration of about 0.2g at the site was the most recent major seismic event that occurred near Posy and Webster Street Tunnels. Post earthquake reconnaissance report indicated evidence of liquefaction on the Island of Alameda and in the vicinity of the tunnels

A three-dimensional global seismic soil-tunnel-structure interaction model was developed to represent the tunnel and the surrounding soil. The tunnel's inertial effects were assumed to be negligible compared to that of surrounding soil. Therefore, a quasi-static-interaction of the tunnel with the surrounding soil in response to the free-field-input-motion, with no mass in the tunnel, was analyzed. Elasto-Dynamic soil-structure-interaction analysis (SASSI) [10] was used to generate soil springs at the centerlines of the tunnels. The desired soil springs were those of low frequencies for the analysis. The vertical and horizontal soil springs are linear elastic, however, the soil-tunnel-slippage-interaction was modeled using non-linear elastic springs in the longitudinal direction of the tunnels. The ultimate values for the longitudinal springs are a function of the soil-tunnel-interface-friction and/or adhesion. These are preliminary soil springs and may be revised.

The tunnels were modeled with ADINA [1] using three-dimensional Hermitian beam elements along the tunnel centerline. The beam element properties were calculated based on the gross cross section of the tunnels. The connections between segments of the tunnels were modeled with "spoke joints" and compression-only springs. A typical spoke model is illustrated in Figure 8. Each spoke joint consists of eight rigid legs ("spokes") at the ends of each segment. There are non-linear compression-only springs between ends of the corresponding spokes.

A Multiple support displacement-time-history analysis that considered tunnel slippage in the longitudinal direction was implemented. Displacement time histories representing seismic excitations were imposed as boundary conditions (Figure 9) at the end of the soil spring elements. The segment connections were continuous for the as-built conditions and are capable of transmitting moments, compression, tension and shear. The segment connections for the retrofit model are discontinuous, Therefore, tensile forces will not be transmitted to the adjacent segments. The purpose of model was to evaluate the proposed retrofit strategy. The results of this analysis indicated that it would be necessary to disconnect the joints between the tunnel segments.

WEBSTER TUNNEL

The three-dimensional finite element models of the Webster Street Tunnel, both the as-built and retrofitted cases, were analyzed. The comparison of the axial force envelopes of the as-built and retrofit models are shown in Figure 10. A significant reduction in the axial force demand was observed for the retrofit case. The large compressive and tensile forces in the as-built model demonstrate the necessity of retrofitting the tunnel. As expected, the relative displacements of the segments in the retrofit model were large. The maximum joint separation, of about 8 inches, was observed at joint number 2 (Figure 11). Joint number 2 is adjacent to the Alameda Portal. The reason for such a large displacement was soil plasticity under differential longitudinal displacements of the tunnel with respect to the surrounding soil during the design earthquake. The mobilized longitudinal soil-tunnel-interface-resistance along the length of the tunnel is shown in Figure 12. If the soil behavior were linear elastic, the longitudinal soil spring force at the Alameda Portal would be greater by a factor of 4.5 (Figure 13). This could provide a significant reduction of the segment separation at joint number 2 (Figure 14). The results of seismic soil-tunnel-structure interaction analysis indicate the significance of non-linear elastic behavior of the surrounding soil in the response of the tunnels.

POSEY TUNNEL

The as-built and retrofitted finite element models of the Posey tunnel were analyzed similar to the Webster Street Tunnel. Nearly identical behavior was observed in the two as-built models. However, the retrofit model still indicated a sharp peak shear force at about 1200 feet from the Oakland Portal (Figure 15). This occurred at 9.60 seconds from the beginning of the ground motion. At this time the transverse shear force reached its maximum (Figure 16) and bending moment about the vertical axis had a high gradient (Figure 17). This is due to a sharp change of transverse ground motion (Figure 18). The results of the analysis indicated that ground motion at two adjacent segments were significantly different. The ground motion input at the centerline of the tunnel is currently being reviewed.

TWO-DIMENSIONAL LOCAL MODEL

Assessing the seismic performance of the tunnels requires evaluating the seismic force and/or displacement capacities of the tunnels. The capacity evaluations of the tunnels were based on the local soil-structure-interaction constitutive material modeling behaviors up to the level of failure. A two-dimensional racking analysis (quasi-static push-over) was performed in order to investigate this behavior. The soil properties were characterized using the ADINA Drucker-Prager material model. The concrete (tunnel cross section) and the jet grout were modeled with the ADINA Kupfer's concrete material model. Buoyancy was not considered in this analysis. In order to study the tunnel under 'racking' displacements, the finite element mesh (Figure 19) was subjected to various displacements and the associated force field (Figures 20). The lateral displacement varied linearly up to its maximum (at the surface). The horizontal distributed forces determined in the preliminary analysis were applied at the top surface. The hydrostatic pressure was also applied to the soil as shown in Figure 20. This scheme of loading follows from the generally accepted opinion that under earthquake loading, the mass-force distribution in the vertical plane provides a pure shear stress of soil. However, in order to improve convergence of the non-linear analysis, a combined loading scheme was chosen because the imposed displacements usually provide much better convergence and less expenditure of the computer resources.

The sequence of construction will also affect the nonlinear response of the tunnel-soil system to two-dimensional racking. During construction, the soil was excavated first. Next, the tunnel with its built-in concrete floor slab was positioned in the open trench. The tunnel was then backfilled with soil. Finally, the ceiling slab was constructed in-place, resting on the seats provided in the tunnel walls and supported at its center by a hanger rod attached to the top of the tunnel. If this sequence were not considered in the analysis, high stresses would have resulted in the ceiling slab and its hanger due to dead load. In order to model this construction sequence, the "birth" option in ADINA was employed to activate the ceiling slab after the backfill soil was placed. Therefore, the deformations due to dead load were properly calculated and distributed to the tunnel and surrounding soil before the ceiling slab was placed. Then, the ceiling slab was properly stressed only by its own weight, and not by the soil and water pressure.

The two-dimensional racking model consisted of nonlinear non-homogenous soil and nonlinear concrete. Therefore, the lateral deformations of the soil media were not proportional to the applied displacements at all the sections. A two-step analysis was required to provide the correct boundary conditions at the surface of the soil media. The first analysis investigated the racking of the soil without the contribution of the tunnel structure. In this analysis, a displacement field was applied to the sides and to the surface of the soil. The horizontal reactions at the surface of the soil were extracted. In the second analysis, the tunnel structure was modeled with the soil media, and the reactions at the surface of the soil, obtained from the first analysis, were applied along the surface with the displacement profiles applied at the sides of the soil. The advantage of this analytic procedure was that while the surface of the soil was subjected to a reasonable racking force, it did not restrain the surface in the horizontal direction.

The deformed shape of the soil media and the tunnel is shown in Figure 20. The tunnel crack pattern at four inches of racking displacement is shown in Figure 21. From the crack pattern, it can be seen that the detailing of the ceiling slab resulted in undesirable cracking in the slab and the tunnel walls in the vicinity of the ceiling slab seats. The proposed retrofit scheme included the removal of concrete at the acute corners between the ceiling slab and tunnel.

The two-dimensional racking analysis was then repeated with the retrofit scheme implemented. The crack pattern, shown in Figure 22, indicated an improvement to the tunnel response as a result of this retrofit strategy.

The maximum shear stress distribution in the soil media is shown in Figure 23. The locations of critical stresses on the boundaries of fragile backfill soil and robust in-situ soil as well as concrete tunnel are shown. This figure also demonstrates the effect of the jet grout upgrade on the soil behavior.

RETROFIT MEASURES

Detailed global and local Seismic-Soil-Structure Interaction models provided optimum solutions to the vulnerabilities of the tunnels resulting from a Safety Evaluation Earthquake. In addition to ground improvement to the backfill, many of the existing full-perimeter joints between the tunnel segments were retrofitted to allow joint expansion. Special joints were added at the portal buildings/tunnel connections. A typical joint retrofit is shown in Figure 24.

Because of the different backfill materials at each tunnel; different seismic retrofit were considered. Different types of ground improvement or flotation prevention methods were investigated for their technical and construction viability.

At the Webster Street Tunnel three rows of 2-foot diameter stone columns are to be installed at each side of the tunnel as shown in Figure 8. The purpose of the stone column installation is to densify existing backfill material and provide drainage of the pore water pressure that could develop during a major seismic event.

At the Posey Street Tunnel A 6-foot diameter “Overlapped Jet Grout” (“soilcrete”) wall will be installed as shown in Figure 25. The purpose of the jet grouting is to install overlapping jet grout columns to form a continuous wall of soilcrete in order to laterally isolate the tunnel from the backfill. The overlap between adjacent columns at the points of intersection is 18 inches. Since the sand backfill is confined beneath the tunnels, small settlements causing only relatively minor cracking are anticipated.

CONCLUSIONS

Assessing the seismic performance of the tunnels was performed in two parts. Part one was the evaluation of the effects of the seismic soil-tunnel-interaction on the seismic-response demands of the tunnels. Part two was the evaluation of the seismic force and/or displacement capacities of the tunnels. The first step in part one defined the seismic input motion to the coupled soil-tunnel-structure system. The seismic input motion was defined as multiple support, free-field motions in the soil region surrounding the tunnels. The well-established Elasto-Dynamic three-dimensional finite element analysis (SASSI) was used to generate linear elastic soil-tunnel-springs at the specified discrete elevations along the tunnel alignment. The soil-tunnel-slippage-interaction was modeled using non-linear elastic springs in the longitudinal direction of the tunnels. A two-dimensional non-linear raking analysis was performed on a typical cross-section of the tunnel in order to predict the force/displacement capacities of the local structural members of the tunnels. The complex global and local soil-tunnel-structure analyses of the tunnels enabled the designers to identify specific features of the structures that were vulnerable, and to evaluate various retrofit strategies.

The geotechnical seismic retrofit consists of installing jet grout columns and stone columns along both sides of the Posey and Webster Street Tunnels. The objective of the geotechnical seismic retrofit is to prevent flotation of the tunnels due to liquefaction of the sandy soils beneath the tunnels. The structural retrofit consists of providing expansion joints between many of the tunnel segments and between the tunnel and the portal buildings. This will provide flexibility and a significant reduction in forces applied to the tunnel segments.

ACKNOWLEDGEMENTS

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The contents of the paper reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the State of California.

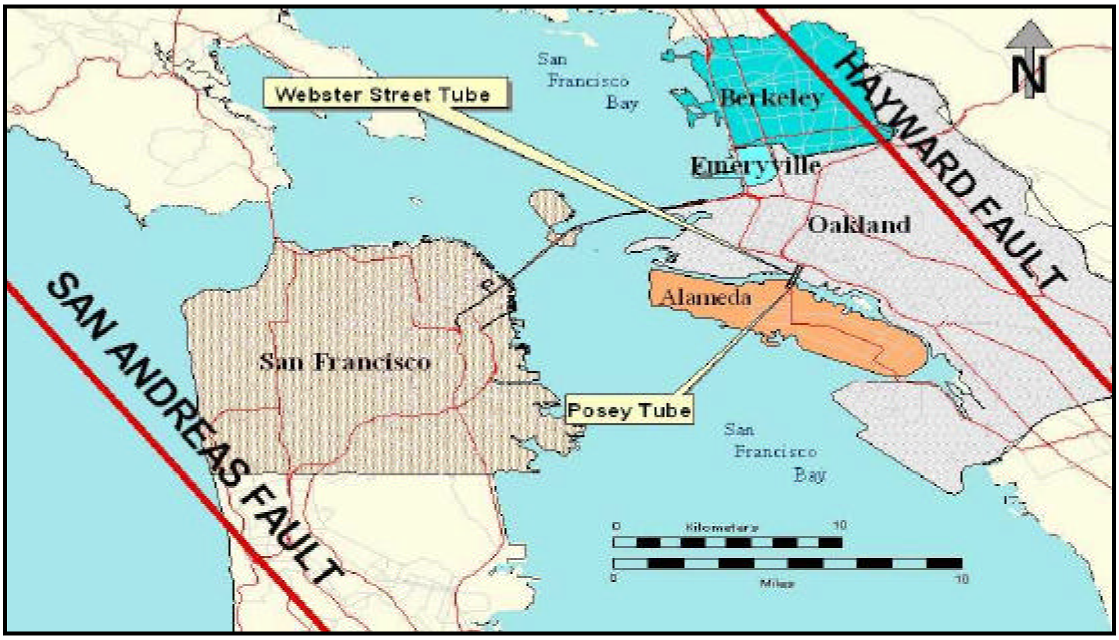

Figure 1 Site Location Relative to Various Faults

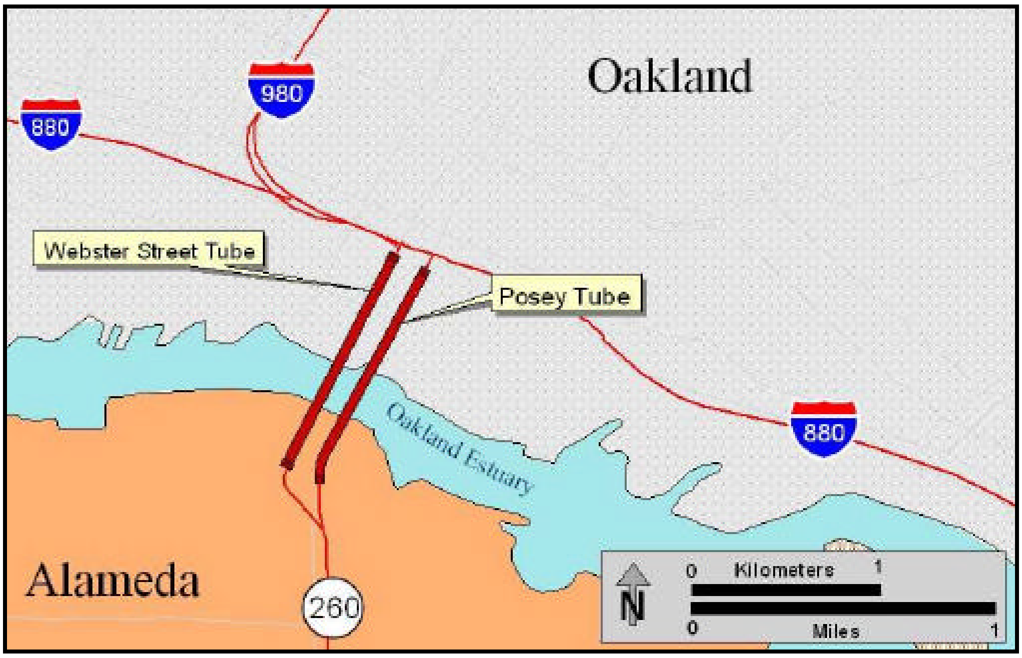

Figure 2 Location of subaqueous Posey and Webster Street Tunnels

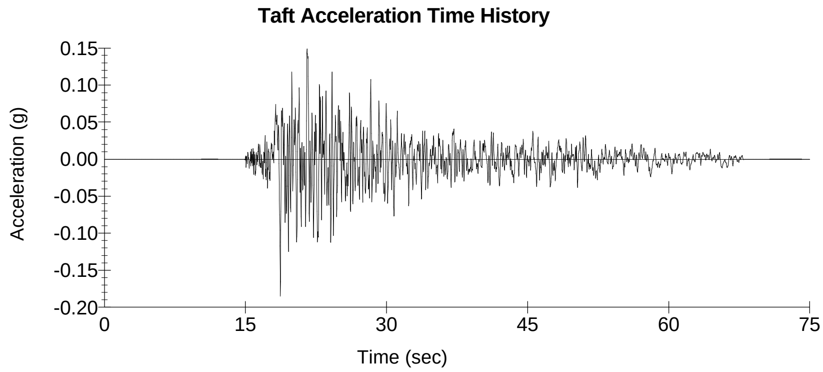

Figure 3 Seed Motion

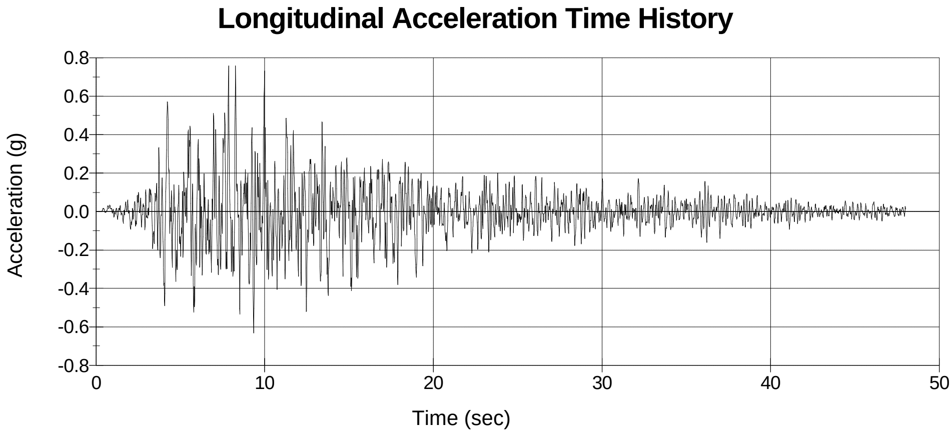

Figure 4 Generated Longitudinal Time Histories.

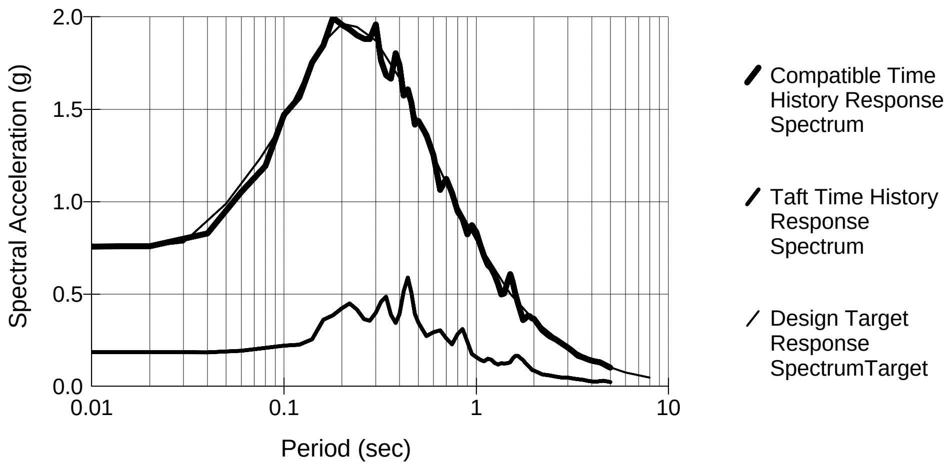

Figure 5 Various Response Spectrum

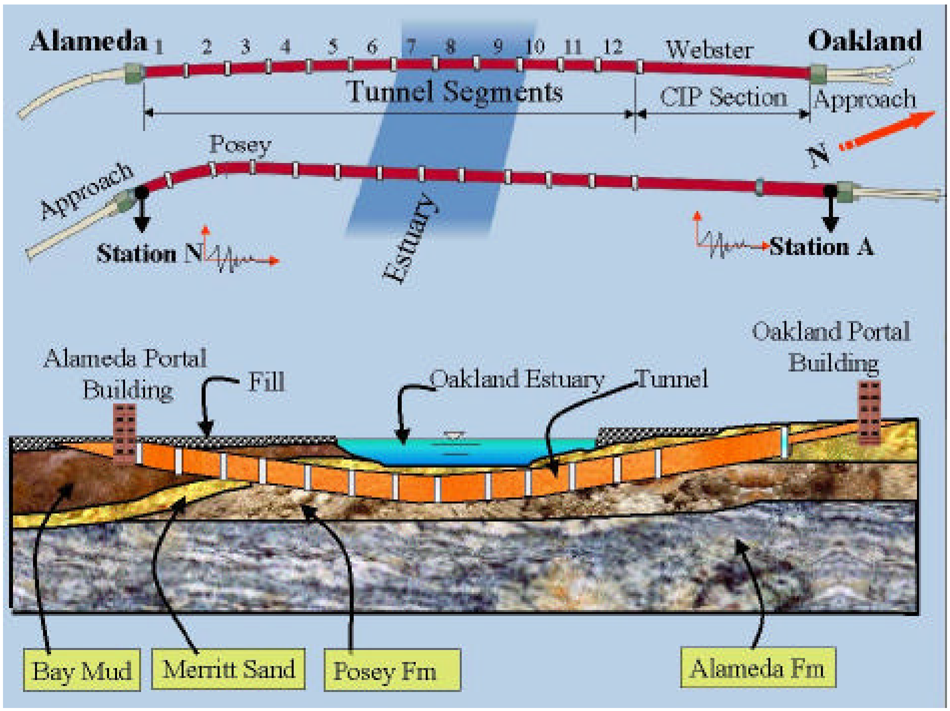

Figure 6 Tunnels Geologic Profile and Plan.

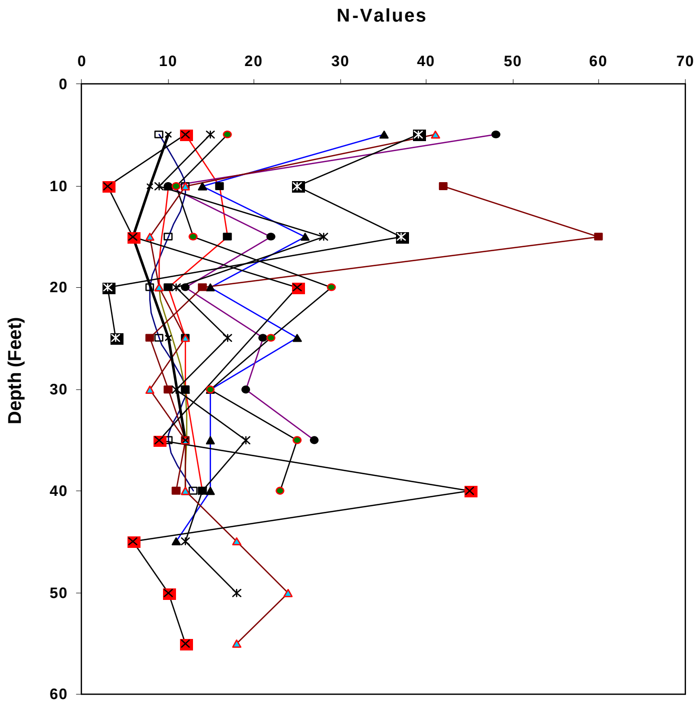

Figure 7 Variation of N-Values for Both Tunnels.

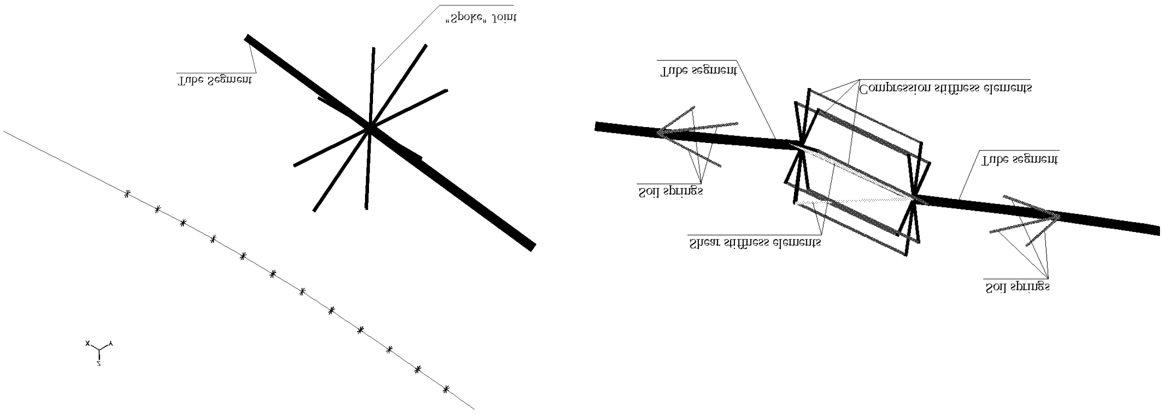

Figure 8 "Spoke" model of the joint.

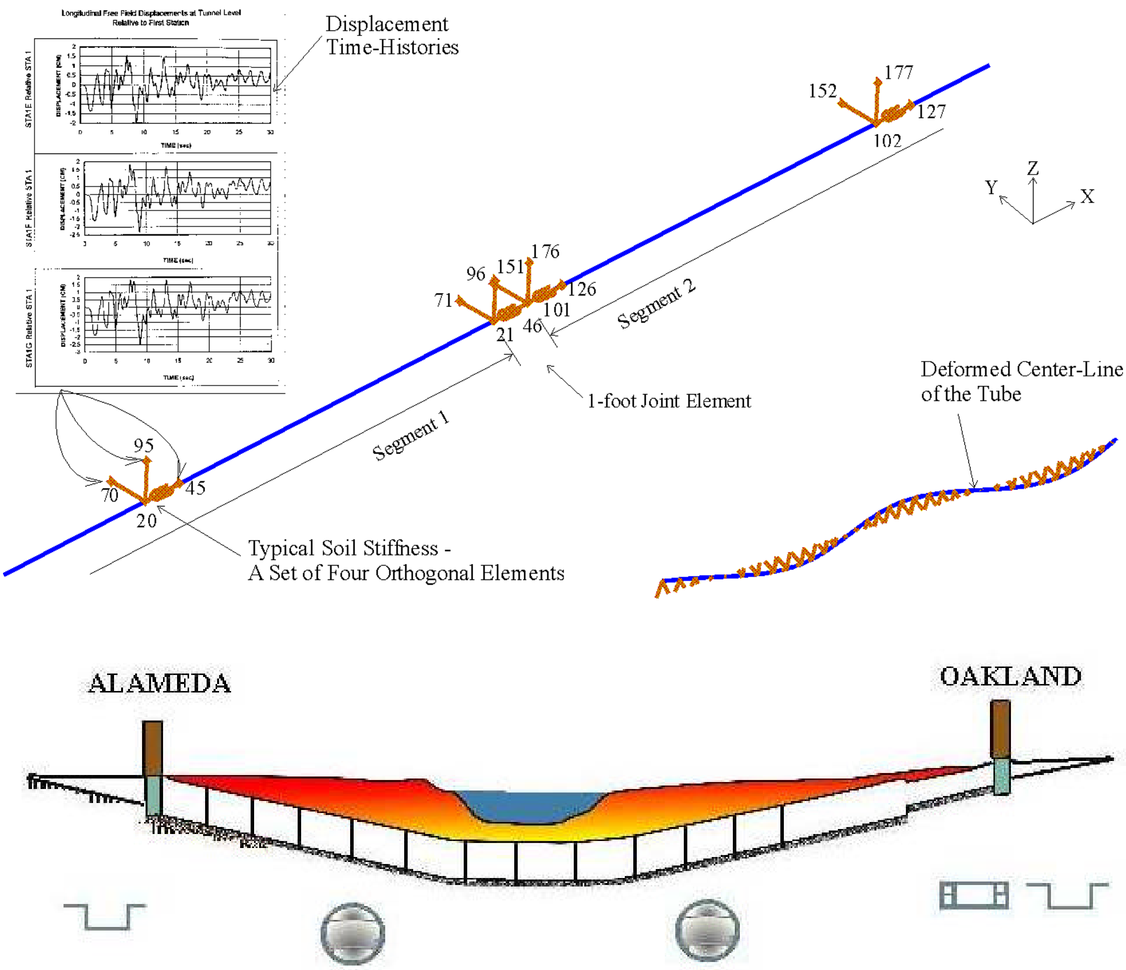

Figure 9 Modeling of soil - structure interaction.

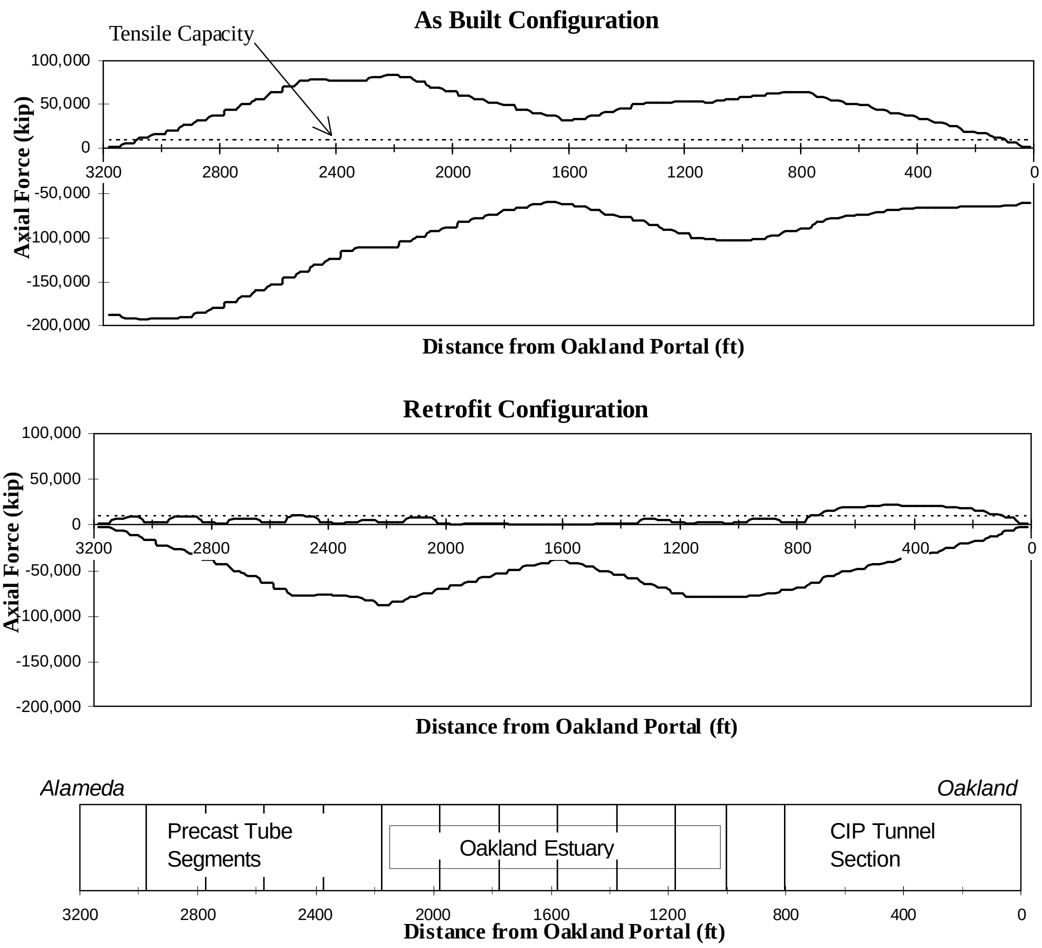

Figure 10 Webster Street Tunnel - Minimum and maximum axial forces.

Joint Number	Axial Separation		Max Separation	
	value (in)	time (sec)	value (in)	time (sec)
2	8.23	11.78	8.39	11.78
3	3.33	11.56	3.37	11.56
4	2.03	7.29	2.05	7.29
5	0.96	7.27	1.01	7.27
6	3.27	13.15	3.29	13.15
7	1.46	7.29	1.52	7.22
8	0.31	4.45	0.31	4.45
9	0.09	2.88	0.11	1.44
10	1.24	7.71	1.27	7.71
11	1.23	12.22	1.26	12.24
12	1.31	8.43	1.31	8.43
13	1.96	8.23	1.97	8.22

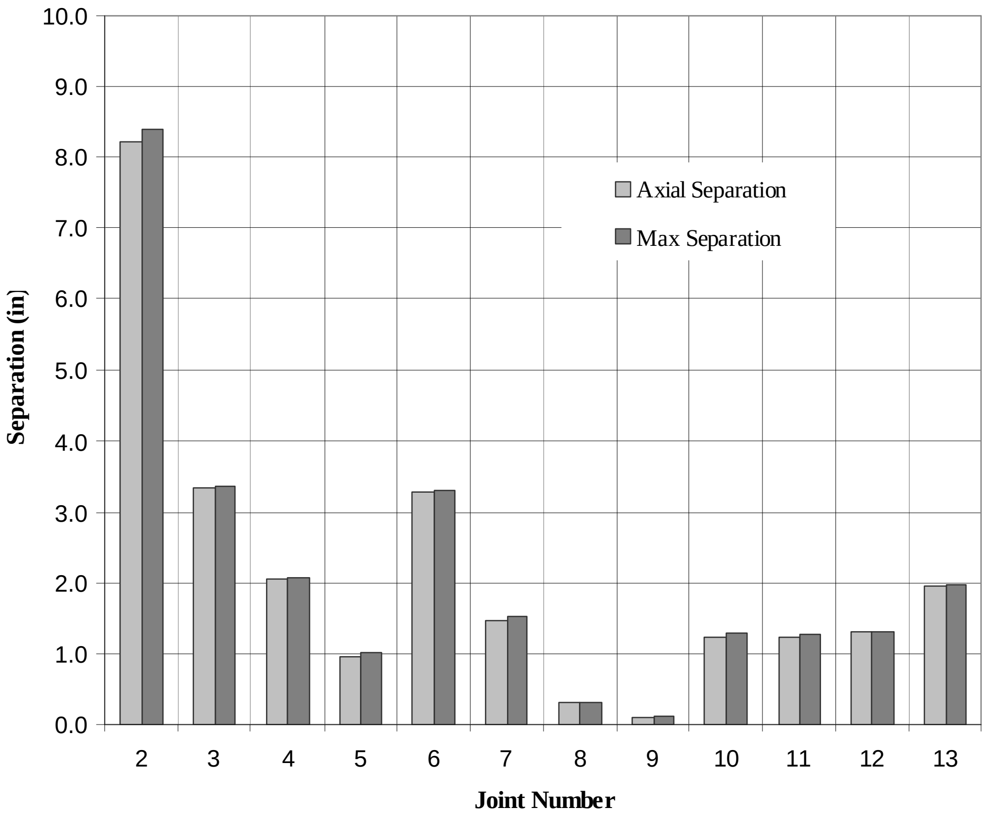

Figure 11 Retrofit configuration of the Webster Street Tunnel: Maximum joint separation.

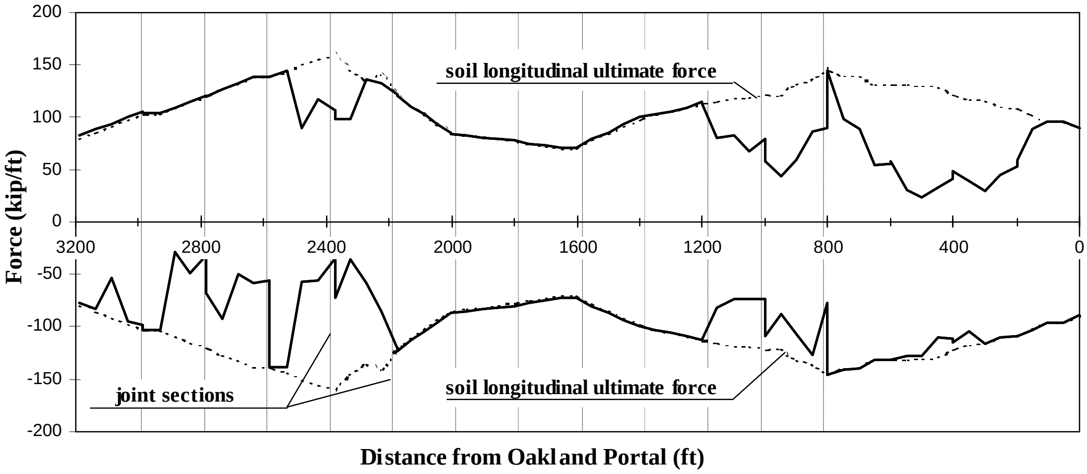

Figure 12 Retrofit configuration of the Webster Street Tunnel: Minimum and maximum longitudinal soil spring forces.

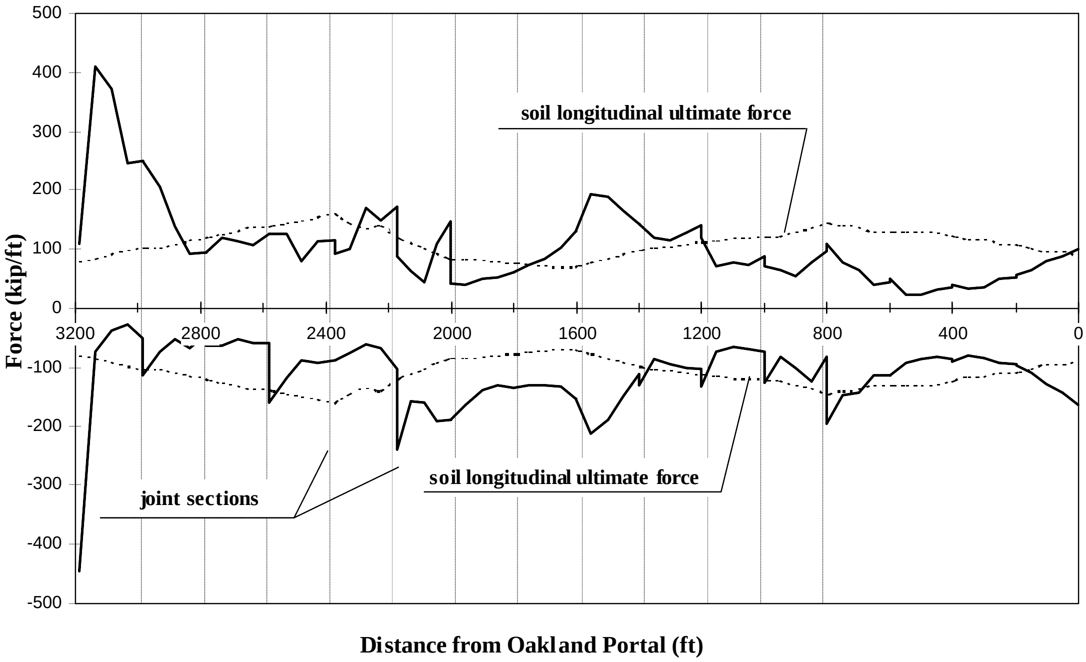

Figure 13 Retrofit configuration of the Webster Street Tunnel: Minimum and maximum longitudinal soil spring forces if soil was linear elastic.

Joint Number	Axial Separation		Max Separation	
	value (in)	time (sec)	value (in)	time (sec)
2	3.67	11.78	3.83	11.77
3	2.35	7.39	2.45	7.39
4	2.01	7.29	2.02	7.29
5	1.02	7.27	1.07	7.27
6	2.57	7.25	2.58	7.25
7	1.14	7.29	1.2	16.01
8	0.58	7.20	0.58	7.20
9	0.38	7.57	0.48	7.56
10	1.04	16.77	1.05	7.71
11	1.57	8.34	1.62	8.32
12	1.33	8.43	1.33	8.41
13	1.93	8.23	1.95	8.22

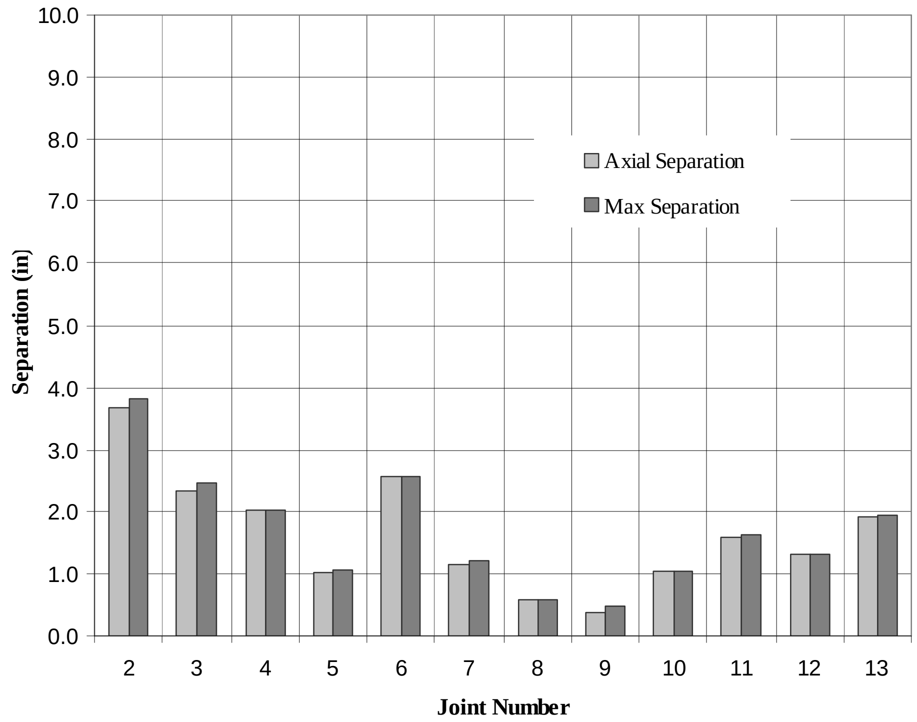

Figure 14 Retrofit configuration of the Webster Street Tunnel: Maximum joint separation if soil was linear elastic.

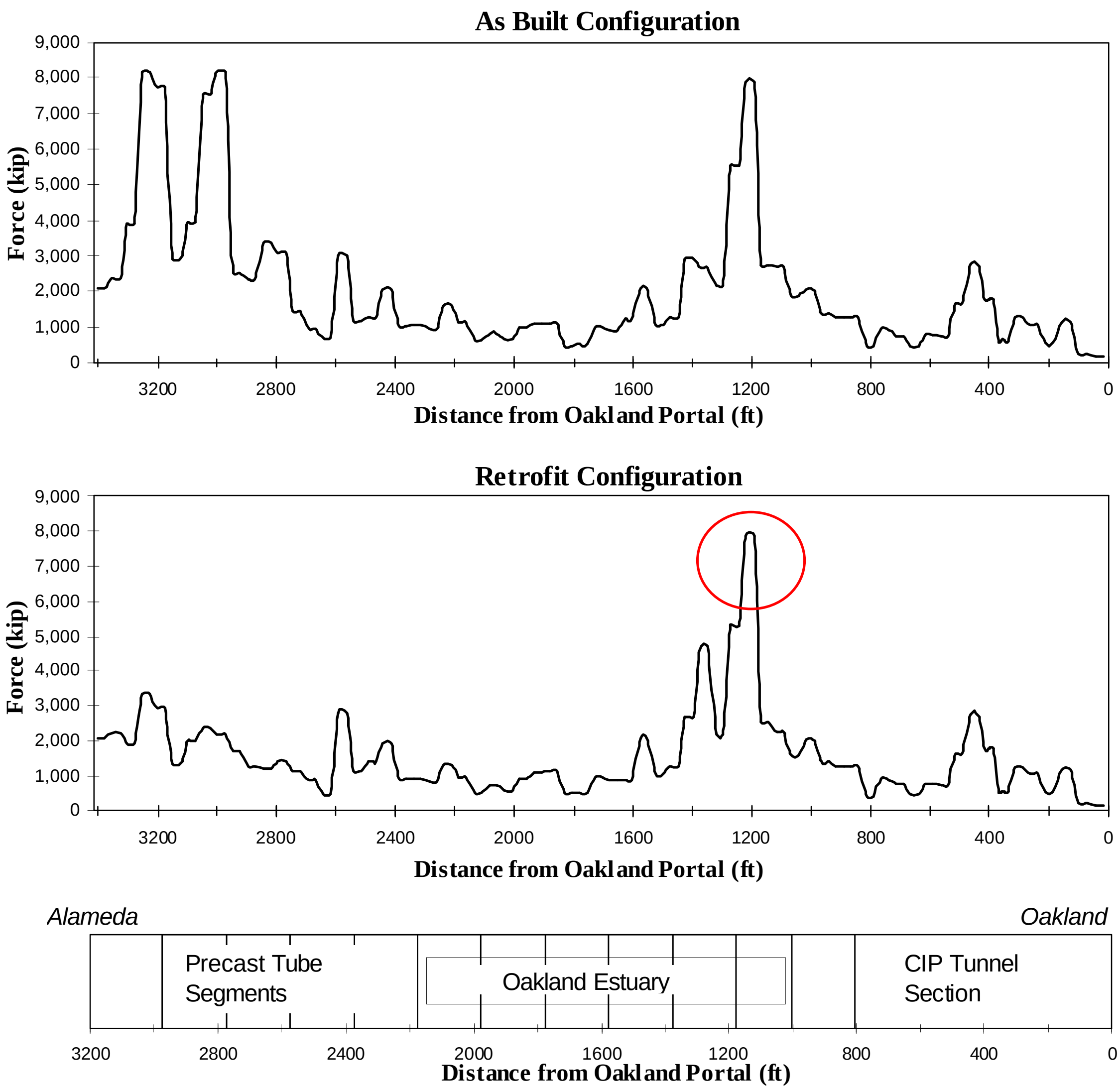

Figure 15 Posey Tunnel Maximum total shear force.

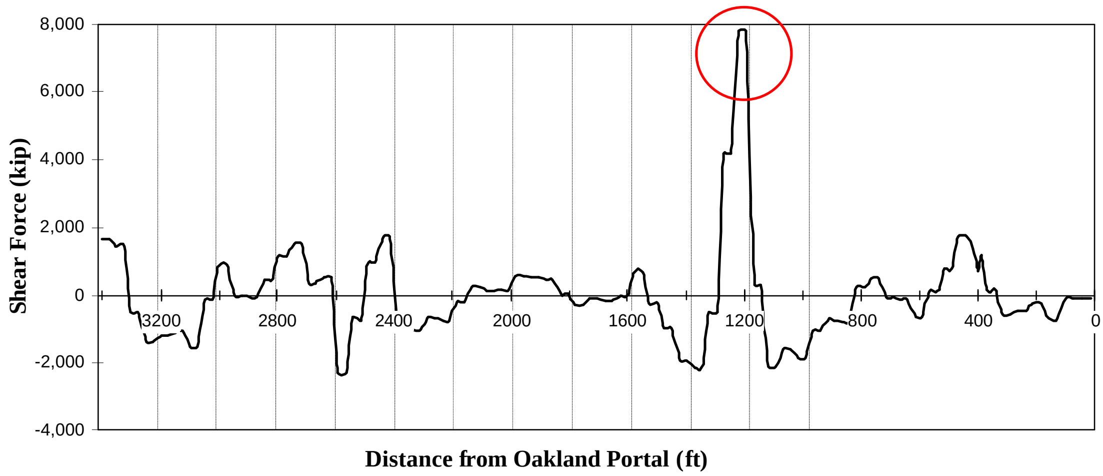

Figure 16 Posey retrofit configuration - Transverse shear force at t = 9.60 s.

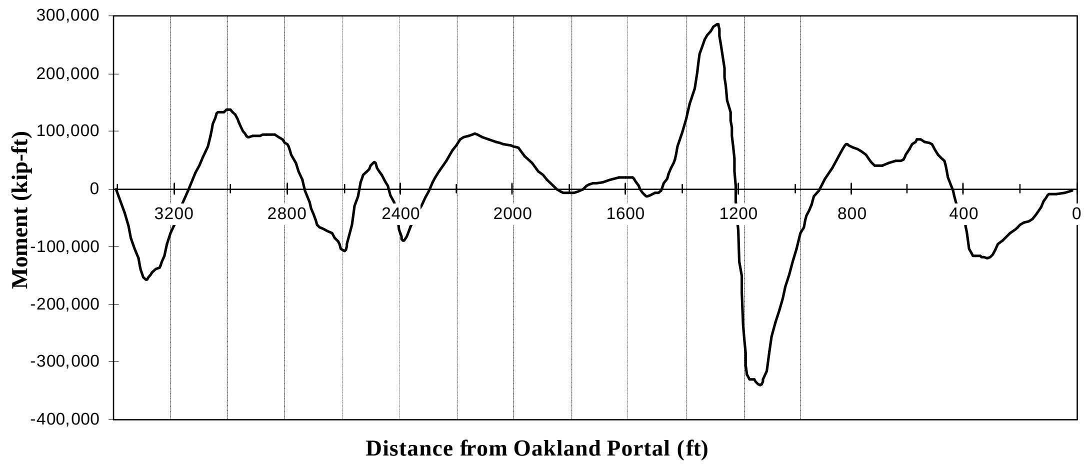

Figure 17 Posey retrofit configuration - Bending moment about vertical axis at t = 9.60 s.

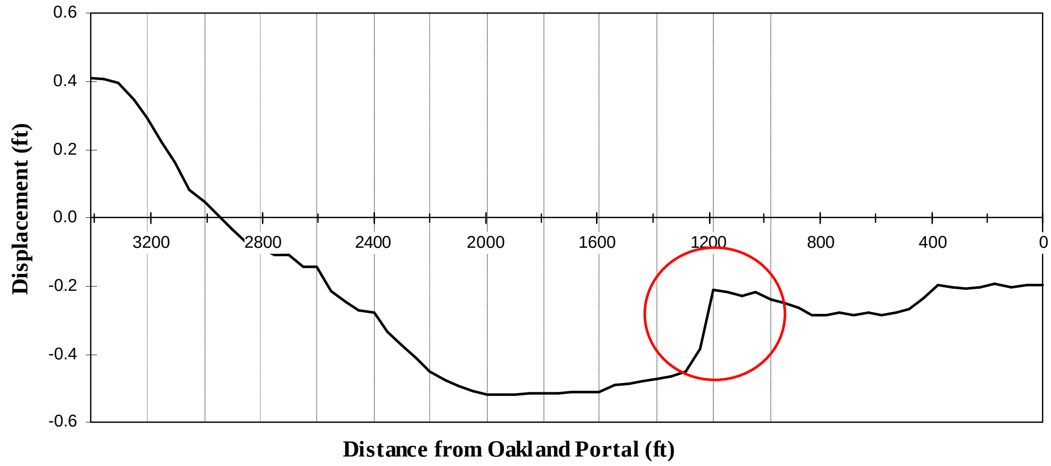

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Figure 18 Retrofit configuration of the Posey -Transverse ground motion at t = 9.60 sec.

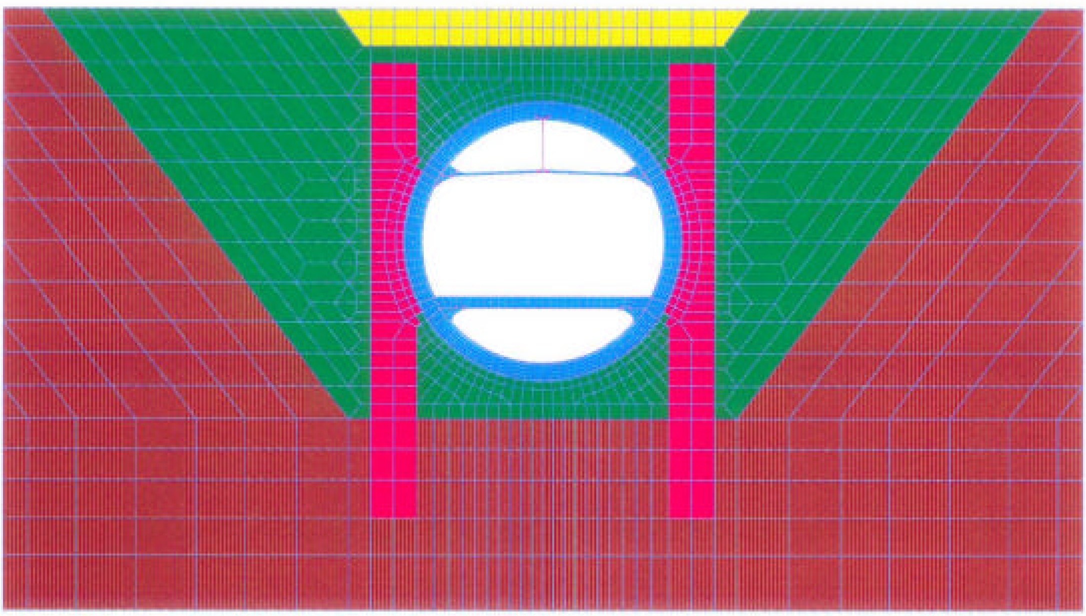

Figure 19 Two-Dimensional Racking Finite Element Mesh.

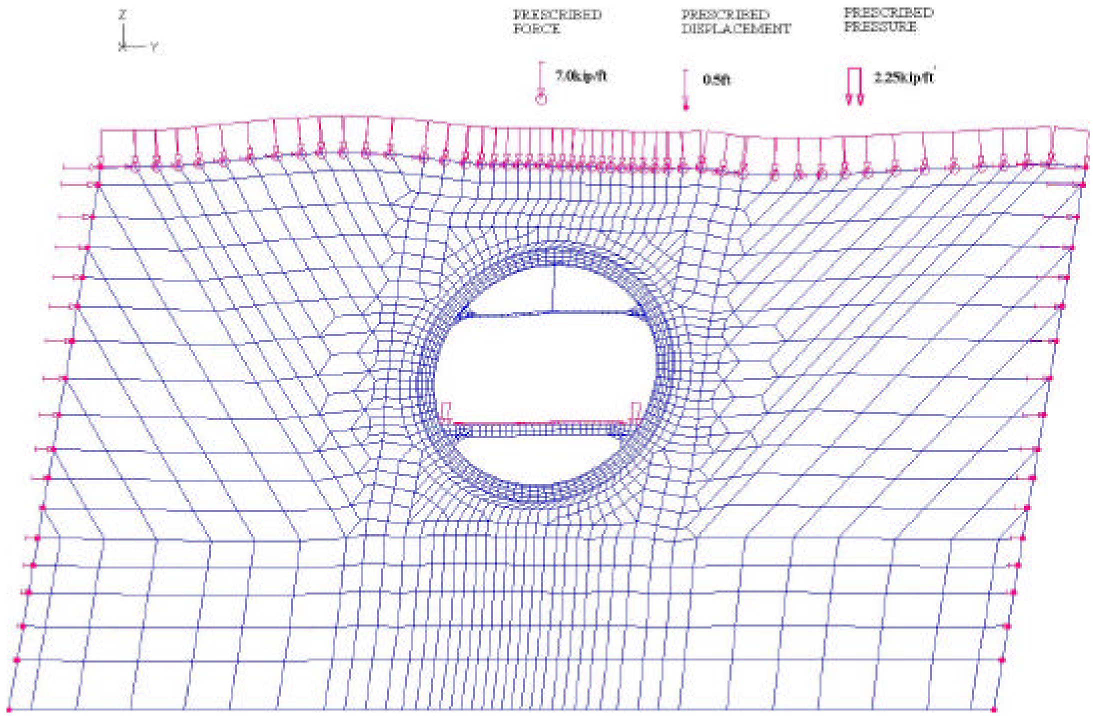

Figure 20 Two Dimensional Racking Finite Element Mesh - Deformed Shape.

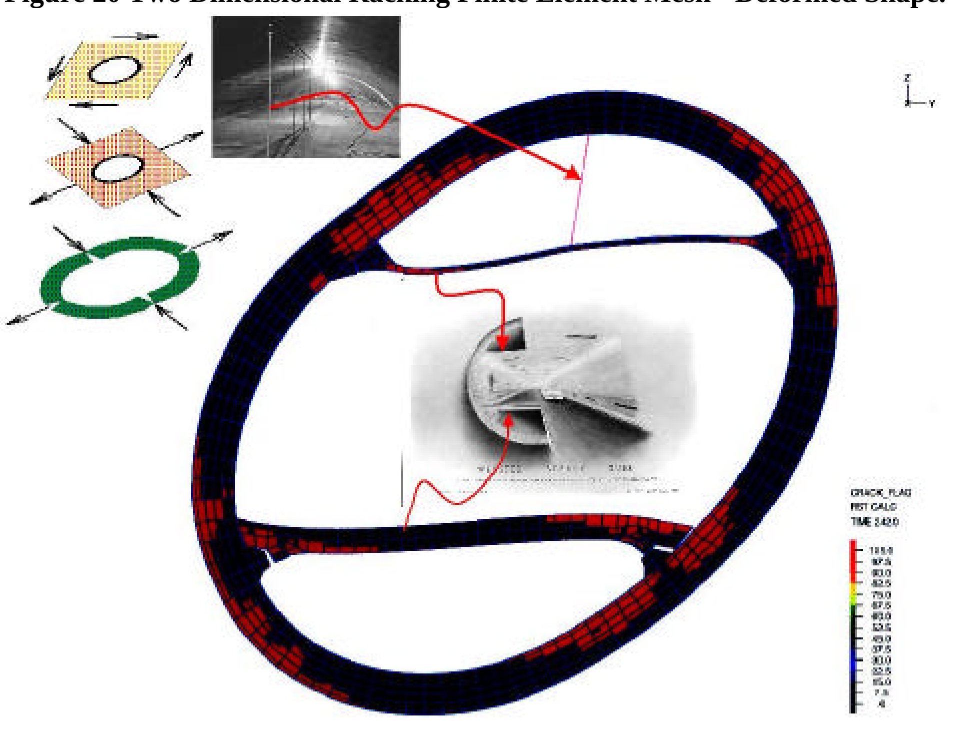

Figure 21 Tunnel deformation and crack pattern.

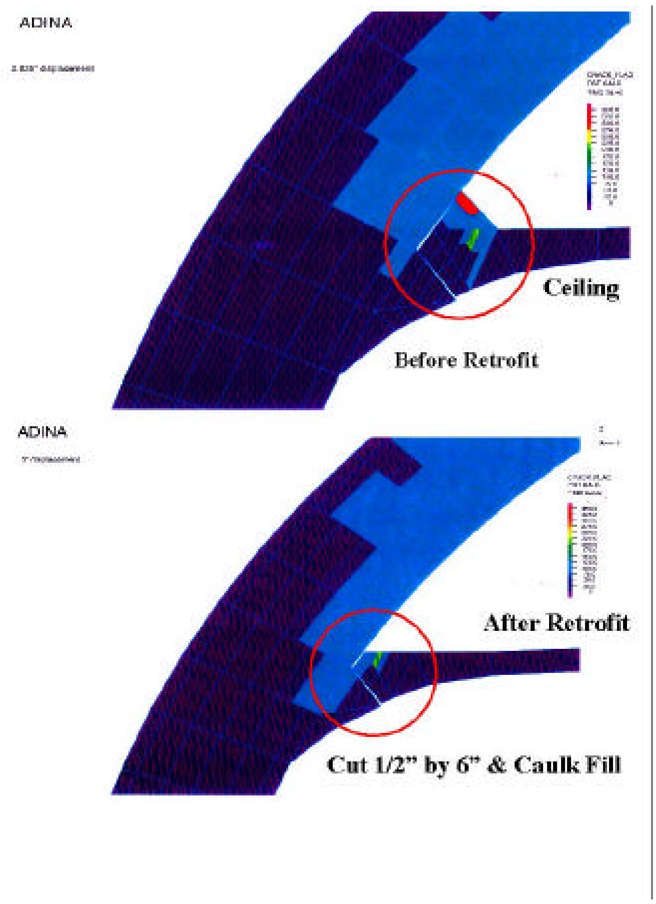

Figure 22 Crack Pattern Around the Ceiling.

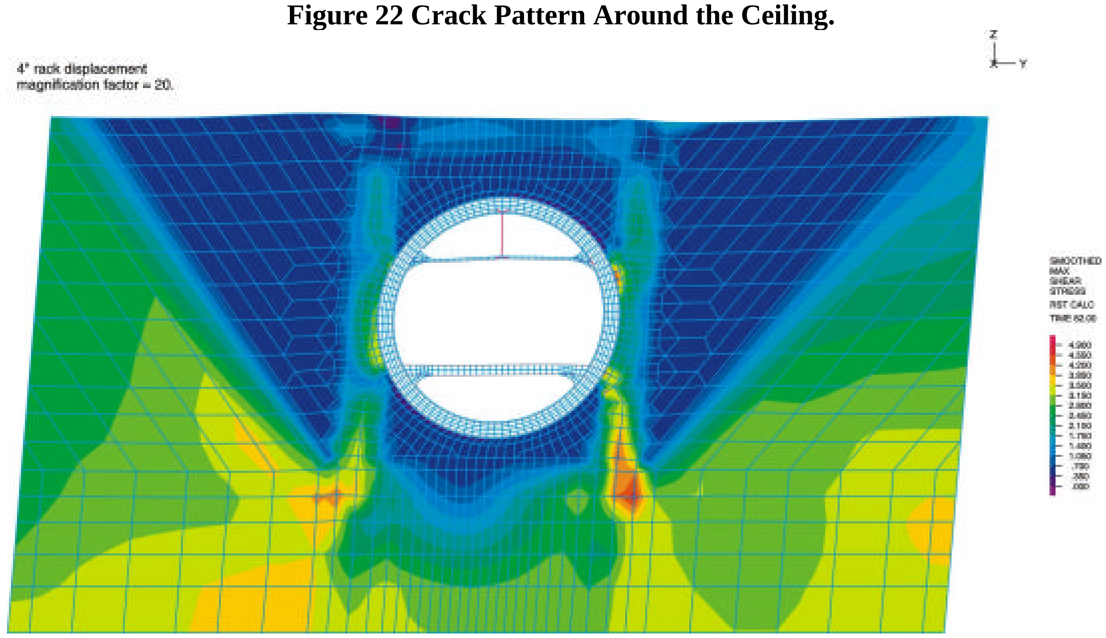

Figure 23 Maximum Shear Stress in Soil and Enclosure.

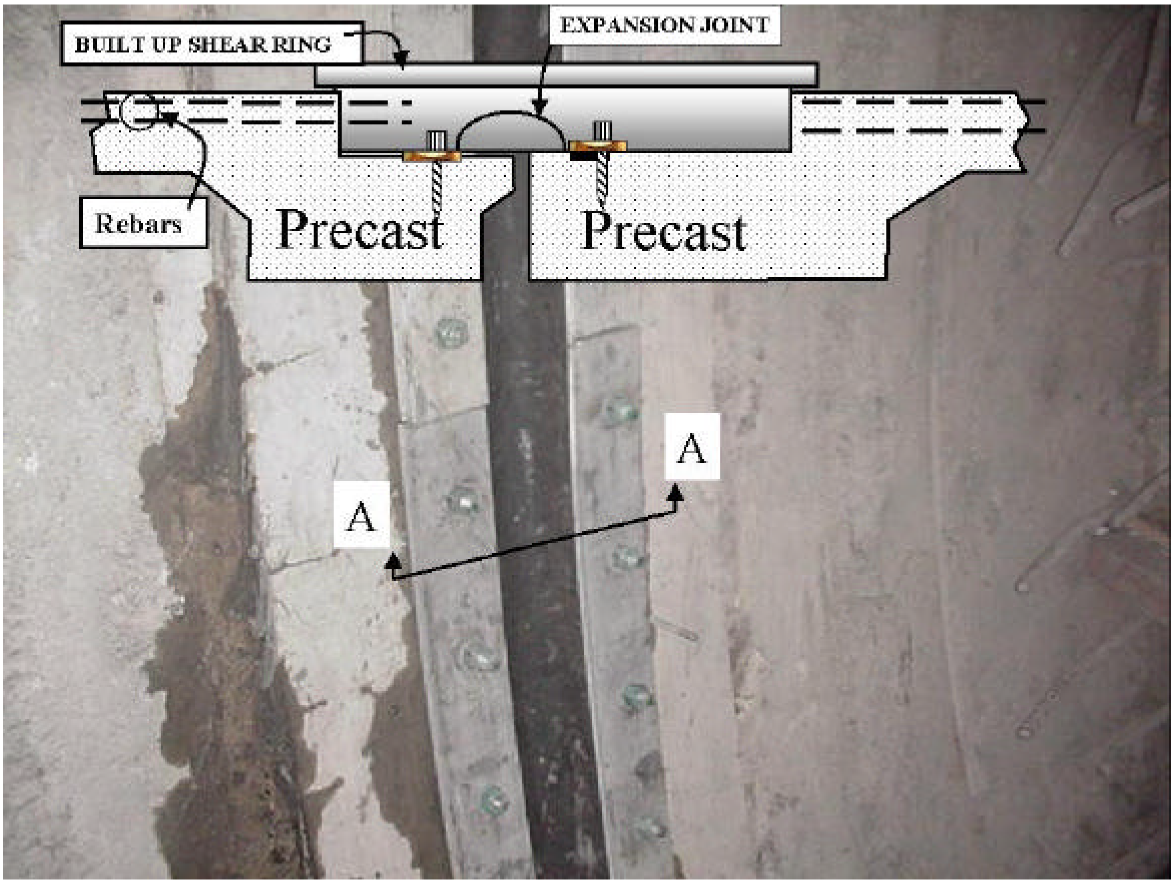

Figure 24 Typical Joint Modification.

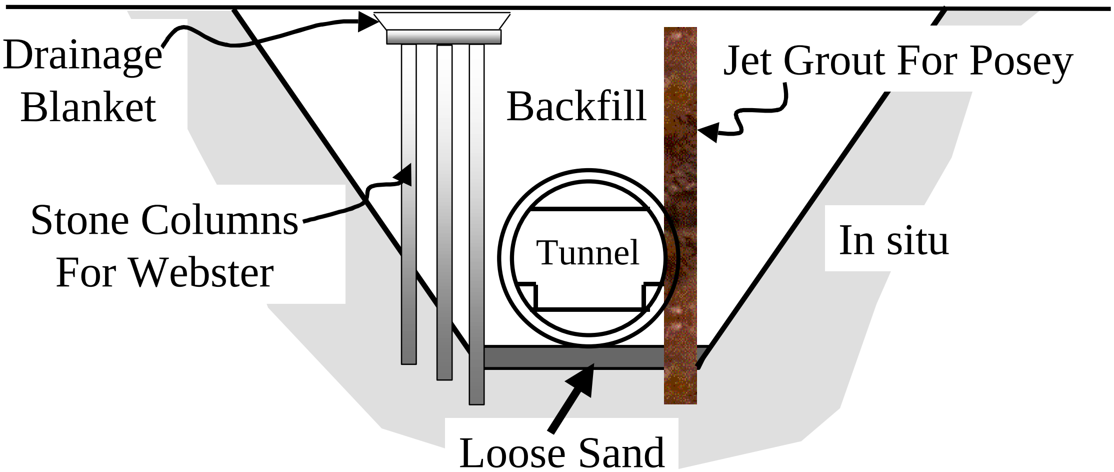

Figure 25 Stone Columns for Webster Street Tunnel, Soilcrete Walls for Posey Tunnel.

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