SEISMIC DESIGN OF SR 99 TUNNEL IN SEATTLE

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Abstract

Recently completed SR 99 Tunnel in Seattle is largest in North America and one of the world's largest TBM bored tunnels. With an excavation diameter of 57'-4" and 9300 ft long, the bored tunnel reaches depths of up to 215 ft under downtown Seattle. Along its alignment, the tunnel traverses through variable glacially over-consolidated soil deposits with high groundwater pressures of up to 7 bars. The tunnel conveys traffic underneath downtown Seattle and replaced the 1950's SR 99 Alaskan Way Viaduct that ran along Seattle's waterfront. The double-deck structure had been deteriorating and was further damaged in the 2001 Nisqually earthquake. This paper discusses the design consideration and the seismic analysis and design of the precast concrete segmental liners of this record-setting tunnel. The focus is on the efficient approach for conducting nonlinear dynamic time history analyses for the seismic design, and the technique to demonstrate that the tunnel satisfies the stringent seismic performance objectives for two levels of design earthquakes with 108-year and 2500-year return events. The selected procedure enhanced the liner design process by significantly reducing computational efforts using efficient beam/spring models instead of commonly used beam/continuum finite element models, while achieving the same level of accuracy in terms of forces and deformations required for the seismic design of the liner.

Keywords: bored tunnel, seismic design, tunnel liner, TBM, non-liner time history

1. Introduction

The SR 99 Tunnel replaced the aging Alaskan Way Viaduct in the city of Seattle. Constructed in the 1950s, the double-deck viaduct was more than two miles long and carried about 110,000 vehicles each day. Studies had shown that the viaduct was nearing the end of its useful life, evidenced by its exposed rebar and deteriorating columns and beams. The viaduct was further damaged by the 2001 Nisqually earthquake, forcing the Washington Department of Transportation (WSDOT) to temporarily close it for inspection and limited repairs. The viaduct remained vulnerable in a future earthquake and continued to show signs of age and deterioration. In 2009, the Washington State Legislature voted to fund the plan to replace the viaduct with a tunnel large enough to build a double-deck high way inside. As a critical local and regional transportation link, the viaduct was required to remain fully operational during the construction of the tunnel

2. Project Description

The SR 99 Tunnel (Fig. 1) begins at the south end with a 260 ft depressed roadway section that contains the mainline and southbound-off and northbound-on ramps; the tunnel is then followed by a 1250 ft of cut-and-cover structure that serves as transition from a side-by-side roadway to a stacked configuration; subsequently continues with a 9300 ft of bored tunnel and ends with a 450 ft cut-and-cover structure at the north end.

The roadway structure inside the bored tunnel has a stacked configuration with two southbound lanes on the upper level and two northbound lanes on the lower level. At the interface between the cut-and-cover and the bored tunnel, the base of the cut–and-cover structure is approximately 67 ft below the ground surface, while the top of the tunnel is about 11 ft below the ground surface. The bored tunnel declines at a 4% grade passing under Alaskan Way, crosses under the existing Alaskan Way Viaduct and reaches a depth of 215 ft from the crown of the tunnel to the ground surface, and then rises at a 2% grade to the north and transition back to a cut-and-cover section. The cut-and-cover section of the tunnel switches the stacked northbound and southbound roadways into a side-by-side configuration that matches the existing grade. Where the bored tunnel emerges, the cut-and-cover excavation is about 76 ft deep.

Fig. 1 – SR 99 Tunnel alignment

3. Regional Setting and Seismicity

Seattle is located within the central portion of the Puget Lowland, an elongated topographic and structural depression bordered by the Cascade Mountains on the east and the Olympic Mountains on the west. This lowland is characterized by low, rolling relief with some deeply cut ravines and broad valleys. The geology along the SR 99 Tunnel consists of complex sequences of glacial and nonglacial deposits that include fine and course-grained sediments that overlie bedrock at great depths.

The complex glacial stratigraphy in the Seattle area has a strong influence on the hydrogeologic regime and the nature of the groundwater flow. The groundwater regime in the area is highly variable.

Groundwater movement is then, in principle, predominantly downward to the discharge areas, eventually draining to the major surface water bodies such as Lake Union, Portage Bay, Lake Washington, and Puget Sound.

The Puget Lowland is located east of and close to the Cascadia Plate Subduction Zone within the Pacific Ocean. Intraplate zone earthquakes occur in the subducting Juan de Fuca plate at depths of 15-60 miles. Studies have shown that the subduction zone can produce earthquakes as strong as magnitude 9.0. Since 1870, there have been six earthquakes in the Puget Sound basin with measured or estimated magnitudes of 6.0 or larger, making the quiescence from 1965 to 2001 one of the longest in the region's history. The largest of these recorded were the magnitude M7.1 Olympia earthquake in 1949, the M6.5 Seattle-Tacoma earthquake in 1965, the M5.1 Satsop earthquake in 1999, and most recently the M6.8 Nisqually earthquake in 2001. Strong shaking during the 1949 Olympia earthquake lasted about 20 seconds; about 40 seconds for the 2001 Nisqually earthquake.

4. Tunnel Liner

Precast segmented lining was chosen for the bored tunnel. The lining segments were installed using an earth pressure balance (EPB) tunnel boring machine (TBM). The Design Life of the bored tunnel is required to be 100 years. Design Life is the time for which structures are expected to function at their design capacity before replacement or major rehabilitation.

4.1 Geometry and Cross Section

The bored tunnel has an interior diameter of 52 ft, which made it the largest TBM bored tunnel in soft ground in the world at the time of design. The tunnel provides a 32 ft curb-to-curb roadway width and 15.5 ft vertical clearance within the bored tunnel (Fig. 2). The bored tunnel accommodates two 11 ft wide travel lanes and 8 ft wide west and 2 ft wide east shoulders in each direction, providing a uniform shoulder travel way and accommodating larger vehicles to transit and move goods and services through the tunnel. The tunnel clearance envelope features a consistent vertical and horizontal cross section from the cut-and-cover sections through the bored tunnel.

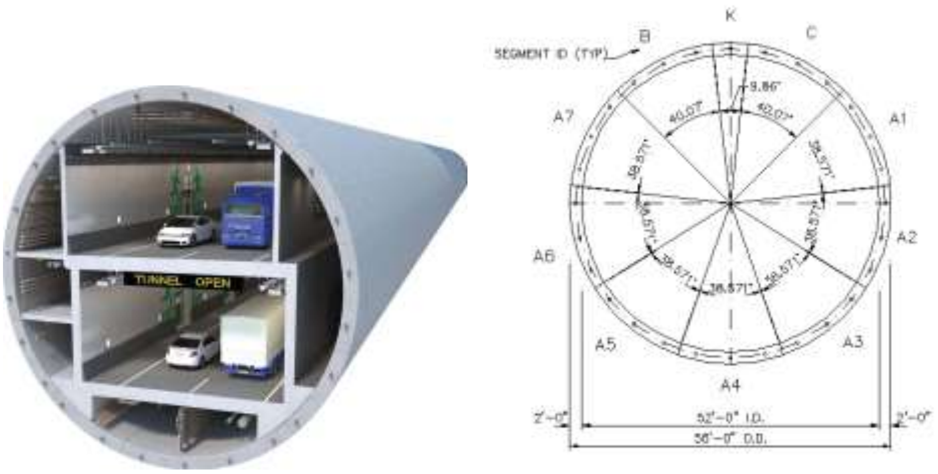

Fig. 2 – Liner cross section

The precast segmented lining is made of universal rings that are 2 ft thick and approximately 6.5 ft wide, consisting of 7 typical segments, 2 counter segments, and one key segment. The universal rings were placed to ensure that there are no continuous joints between ring segments. This is an advantage when considering water tightness and structural strength. In addition, it allowed the TBM to negotiate sharp vertical curves if needed to respond to an unexpected change of face conditions.

4.2 Design Criteria and Design Considerations

The concrete for the liner is a special mix with a specified compressive strength of 7,000 psi at 56 days. The steel reinforcements in the liner are deformed wire rods conforming to ASTM A496, Grade 75 ksi. The strength design of the liner is in accordance with AASHTO Load and Resistance Factor Design (LRFD) method [1], which considers the statistical variability of member strength and of the magnitude of the applied loads. The load factors in AASHTO have been modified according to the FHWA Manual [2].

The design of the tunnel liner considered the required 100-year Design Life, the scale and diameter of the tunnel, the intended use, ground conditions, seismic conditions, ground water conditions, depth of cover, buoyancy, ground and ground water chemistry, fire resistance, and construction sequence and schedule. The design also considered existing buildings, structures, and utilities that are along the tunnel alignment that fall within the tunnel's zone of ground movement influence. An additional uniform gravity loading along the tunnel alignment was included to account for future building loads. The uniform loading is 7000 psf, projected across a width of 80 ft and centered about the tunnel at a height of 55 ft above the crown of the tunnel. Two major construction conditions were considered in the design of the liner: one, at the end of tunneling for the tunnel ring only, and two, the completed tunnel consisting of tunnel lining and interior structures and systems. Multiple analytical methods were used in the design and parametric studies were conducted to validate the results.

Dual levels of design earthquakes, Expected and Rate Earthquakes, were considered for the design of the tunnel liner. The Expected Earthquake has a 108-year return period and is associated with an Operational Performance Objective, while the Rare Earthquake that has a 2500-year return period and is associated with a Life Safety Performance Objective. Under the Expected Earthquake, minimal damage to the liner segments, joints and water tightness is anticipated because the lining is designed to respond in an elastic manner. For each of the two return periods three seismic events were considered (PACDAM, TABAS and TCU052 for Rare Earthquakes and Firesta, GLE and SewPk for Expected Earthquake), for a total of six cases, and the results were subsequently enveloped for respective return events. Seismic ground motions at the tunnel were generated by performing a site response analysis.

Concrete compression strain is limited to 0.003 and tensile strain in reinforcing steel is limited to 0.002. Under the Rare Earthquake, the objective is to prevent collapse of the tunnel liner. Inelastic deformations are allowed under the Rare Earthquake but limited to the acceptable levels. Concrete strain is allowed to exceed 0.003 but limited to 0.005 provided that the strain is predominantly due to flexure. The tensile strains in a mild reinforcing steel is limited to 0.06 for reinforcing bars up to US #10 size and 0.045 for US #11 size and larger.

The liner segments are connected with two bolts at all radial joints, three bolts and three shear bicones at the circumferential joints for the typical and counter segments, one bolt and one shear bicones at the circumferential joints for the key segment. The gaskets are manufactured of EPDM with 1.75 in bottom groove width, that is fused with a strip of hydrophilic material. The gasket was chosen to seal the groundwater under normal service conditions as well as seismic conditions when the joints between liner segments likely to open more than that of static conditions.

4.3 Static Design

Fifteen sections were analyzed along the length of the tunnel alignment for the static design of the segmental liner. The design sections were selected to capture widely varying loads imposed on the tunnel liner and to assess the geologic and hydrogeologic variability along the tunnel alignment. Since the structural design of the tunnel is in accordance with the Load and Resistance Factor Design (LRFD) methodology, the loads from various sources were combined using different load factors. Because the continuum models simulate the sequences of tunnel construction, the loads at each construction stage are dependent on the previous stages; thus, applying load factors explicitly in the continuum model will yield erroneous results. To overcome this inconvenience, both 2D continuum models and beam-on-spring models were created for each of the 15 sections. The continuum model was created using FLAC software and the beam-on-spring models were created with CSiBridge software.

The primary purpose of the FLAC 2D continuum models was to determine the soil loads on the liner; the liner is subjected to various loading conditions including soil overburden, groundwater, existing building loads, and a predefined future building load. In addition, these models were also used to determine the soil springs that consider soil-tunnel interaction. A sample FLAC 2D continuum model is shown in Fig. 3. The loads and springs generated from the continuum models were then used in the 2D beam-on-spring models (Fig. 4), where the loads were factored and combined. Another benefit of using two models was to integrate the work of the geotechnical and structural engineers in a streamlined design process. The geotechnical engineers mainly focused on the continuum models which provide loads and springs on the liner, with an emphasis on the geotechnical aspect of the liner design. The structural engineers primarily focused on the beam-on-spring models as well the structural design of the liner, with an emphasis on the structural aspect of the liner design.

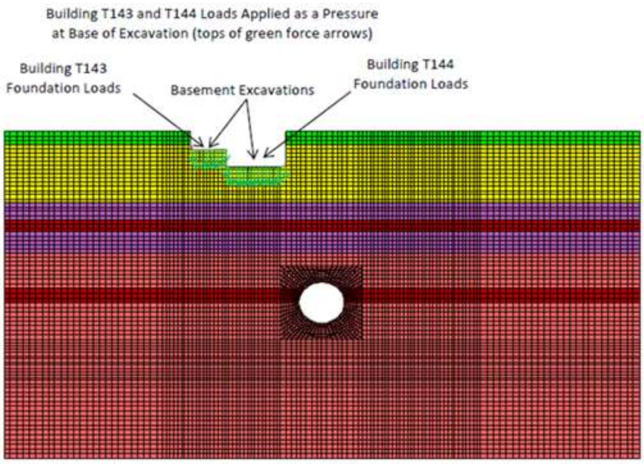

Fig. 3 – FLAC 2D continuum model

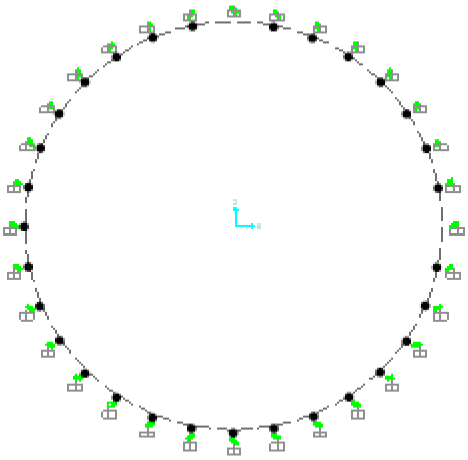

Fig. 4 – Static Beam-on-spring model

Static soil loads on the liner generated from the FLAC continuum models represented the end of tunnel liner construction, but prior to construction of the interior structure. The same loads were applied in the beam-on-spring models for structural design for both the temporary and final design cases with and without the effects of the interior structures and loads.

The liner reinforcement was designed for several strength limit states based on the enveloped axial forces and bending moments from the beam-on-spring models. Because of the relatively large tunnel diameter to line thickness ratio, the liner behaves predominantly as a compression member. The liner design requirements stipulate a minimum volume of reinforcement of one percent of gross volume of concrete distributed as 0.25% longitudinal and 0.25% circumferential on intrados and extrados of the segment. This design requirement does not meet code-specified minimum requirements for compression or bending members. An in-depth analysis of the code-specified minimum reinforcement requirements revealed that the minimum reinforcement was aimed at reducing the effects of creep and shrinkage of concrete under sustained compressive stresses. Unless a limit is placed on the ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. Therefore, if the applied load is limited such that the combined strain in the steel is less than the yielding strain with a reduction factor, the code-specified minimum reinforcement would then not be applicable. The analysis concluded that both primary (circumferential) and longitudinal reinforcements were governed by the minimum reinforcement stipulated in the liner design requirements, that being the fore mentioned 0.25% at each face in each direction.

Since the loads acting on the circumferential joints due to the jack thrust are very large due to the size of the TBM machine, a substantial amount of bursting reinforcement was needed for resisting bursting. The design of segments also included consideration of demolding, shipping, lifting, and stacking with a concrete strength of 2000 psi, representing the strength at the time of release. Concrete spalling was also considered for the gasket groove when the gaskets are being compressed.

4.4 Seismic Design

Tunnel structures are less susceptible to earthquake damages than surface structures because they are constrained by the surrounding soils, so inertia plays a secondary role and the amplitude of ground motion tends to decrease with depth. Nevertheless, some underground structures have been subjected to considerable damages in recent large earthquakes. Analysis of the bored tunnel included loading from seismic deformations and ground accelerations considering three primary modes of deformation during seismic ground movement; 1) ovaling, 2) axial, and 3) curvature deformations.

A two-step analysis procedure was adopted to analyze seismic ovaling. First, deformations of the soil surrounding the liner due to the seismic waves propagating from bed rock through soil media and in absence of the liner were computed with a continuum model (Fig.5).

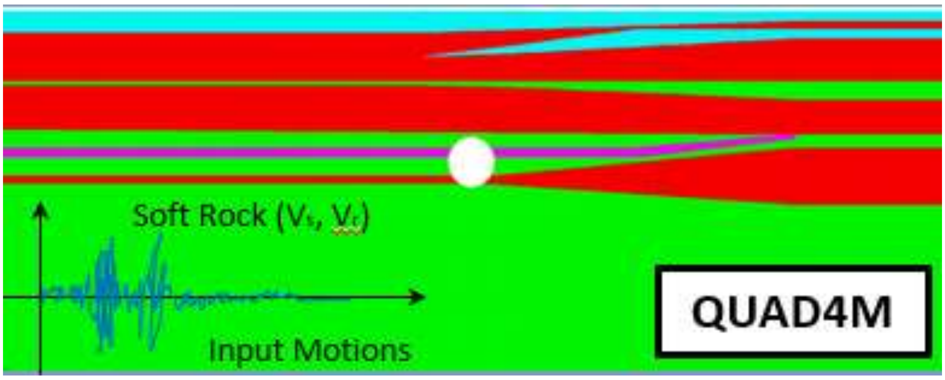

Fig. 5 – QUAD4M continuum model

Second, the ground deformations were imposed to the liner through supporting elements (non-linear springs) using CSiBridge beam-on-spring models by performing non-linear dynamic time history analysis (Fig. 6). The liner was analyzed for three Expected Earthquake events, and three Rare Earthquake events. The results for the Expected and Rare Earthquakes events were then enveloped respectively.

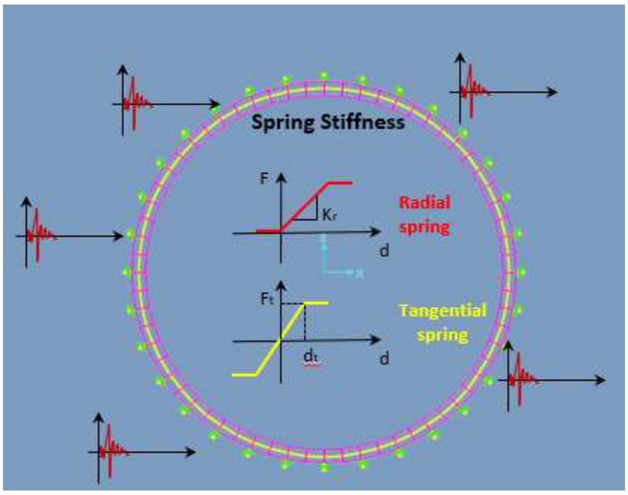

Fig. 6 – Seismic Beam-on-spring model

The results from the time history analysis showed that the maximum ovaling is about 1.5 in or 0.2% of the ring diameter for the Rare Earthquakes. Figure 7 is a sample snapshot of the ring deformation for a Rare

Earthquake event. It is observed that maximum ovaling is generally in a diagonal direction, which is consist with the open round cavity deformation caused by a free-filed ground shear distortion [6].

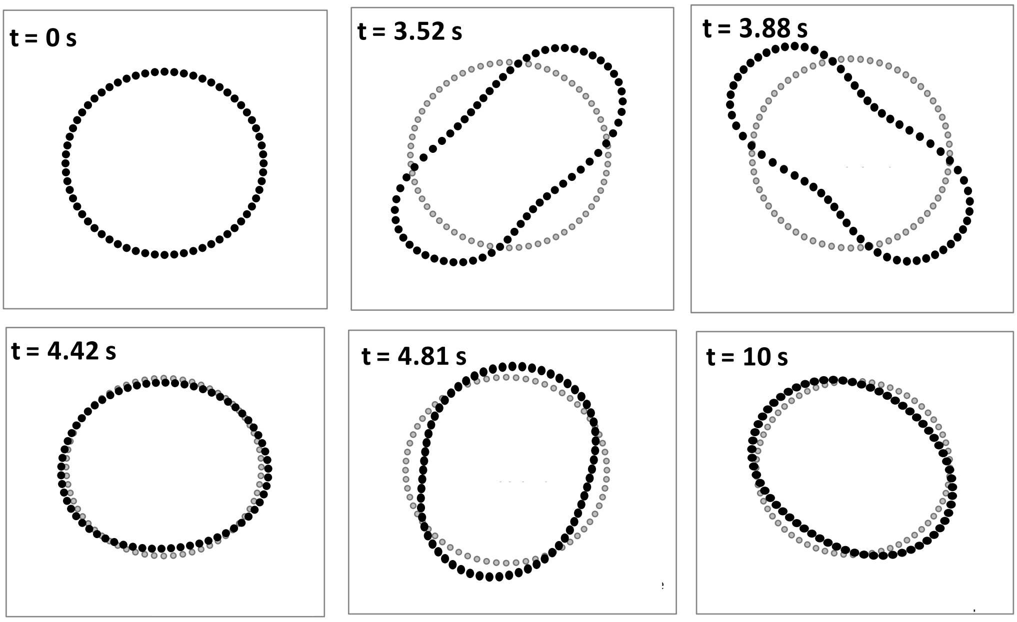

Fig. 7 – Sample seismic ring deformation

In addition to the 2D models for studying ovaling effect, a 3D global spine model of entire tunnel was created to determine forces and deformations along the longitudinal axis of the liner for axial and curvature deformations, as well as the seismic movements at each end of the tunnel (Fig. 8). Displacement time series in three principal directions were applied to the tunnel structure considering seismic site response, stiffness of the liner and the surrounding soil, stiffness of adjacent cut-and-cover structure, to capture the longitudinal response of the tunnel. The results from the global 3D seismic analysis controlled the design of shear bicones and circumferential joint bolts to meet the seismic shear demand.

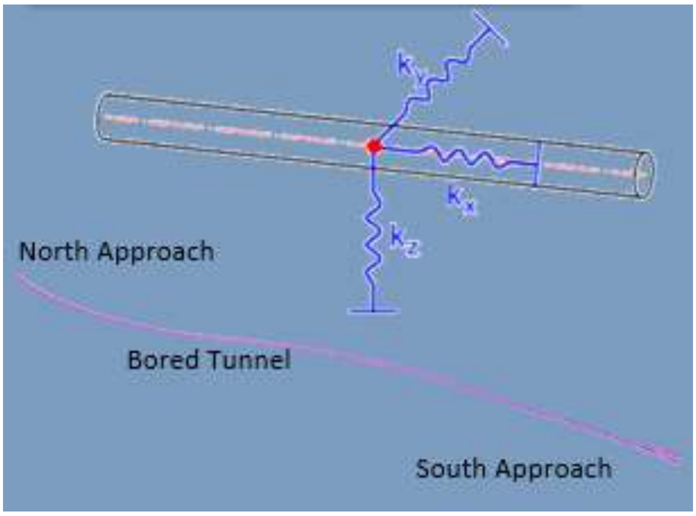

Fig. 8 – Global spine model

Table 1 shows the calculated joint opening and closing from global spine model at the south end of the tunnel. The values formed the base for the flexible joint design. Note that there is a significant difference between the movements from the Rare Earthquake and those from the Expected Earthquake. The difference can be attributed to slippage between the liner and the surrounding soil that takes place during a Rate Earthquake.

Table 1 – Tunnel movement at south end

Joint Gap	Rare Earthquake [in]	Expected Earthquake [in]
Opening	6.6	0.1
Closing	8.6	0.2

To predict the local behavior of the radial and circumferential joints and determine the required gasket sizes, a 3D finite element model of four rings was created (Fig. 9). The maximum seismic ovaling deformation from the 2D model and maximum curvature from the 3D spine model were applied to the finite element model and the openings of the circumferential and radial joints were then determined.

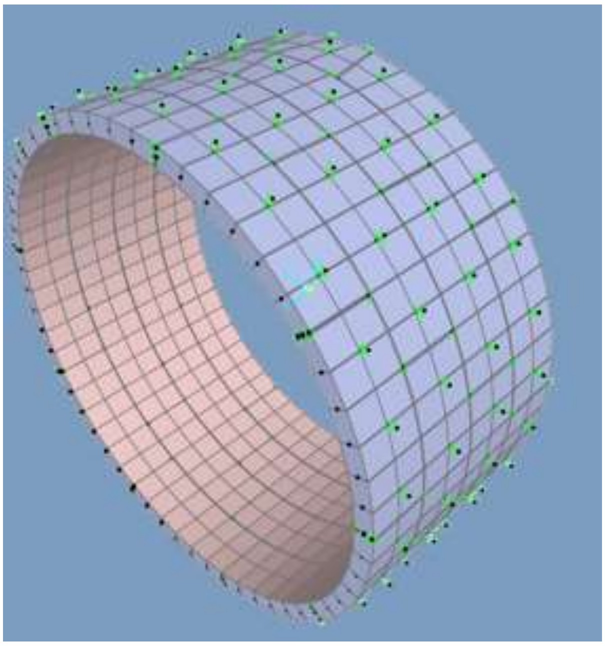

Fig. 9 – 3D finite element model

The maximum joint opening was found to be about 0.11 in. Gasket size was then determined for a combined value of opening from static and seismic loads as well as construction tolerances, in addition to the consideration of gasket offset and water pressure.

5. Conclusions

SR 99 Tunnel is a significant undertaking due to its size, geological location, and the site conditions under which the tunnel is built. The facts that the tunnel is in a highly active seismic region and was the largest diameter soft ground TBM bored tunnel in the world at the time of design, and the deteriorating Alaskan Way Viaduct needed to remain fully operational during the construction of the tunnel made the design a challenge. Among others, the design of SR 99 Tunnel led to conclude that 1) dynamic time history analysis is an invaluable tool to quantify liner seismic deformations and forces; 2) temporary stress conditions such as resulted from TBM jacking become even more critical for large diameter liner segments, and 3) the two-step procedure enhanced the liner design process by significantly reducing computational efforts while achieving the same level of accuracy for the seismic design of the liner.

6. Acknowledgments

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7. References

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