Seismic design of New Westminster rapid transit tunnel

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Abstract: Seismic design of underground tunnels is often based on simplified pseudo-static procedures. These pseudo-static procedures can be grouped into two approaches. The first approach involves imposing lateral earth pressures (e.g. Wood, 1973 and Seed and Whitman, 1970) on tunnels buried at shallow depth. These lateral earth pressures, which were originally developed for the design of retaining walls, are not appropriate when applied to tunnel design. The second approach, referred to as the shear strain approach (Monsees and Merritt, 1991, Wang, 1993 etc.), is based on ground deformation. It has been used primarily to evaluate the amount of "racking" that will be imposed on a tunnel during earthquake shaking. While the above simplified procedures are sometimes adequate for design of tunnels, they are not applicable in all situations, such as a site with sloping ground conditions. In this paper, results of dynamic soil-structure interaction analyses are presented for the design of the New Westminster Rapid Transit Tunnel.

Introduction

Underground structures have generally performed better than surface structures in past earthquakes. In many old design specifications for tunnels, aseismic design was not usually considered in the transverse direction. The reason for this is that the underground structures were assumed to follow the deformation of the ground during an earthquake. However, some severe damage, including collapse, has been reported for tunnel structures (Wang,1993). During the Kobe earthquake of January 17, 1995, the Daikai station, which belongs to the Kobe Rapid Transit Line, was completely destroyed. More than 30 columns collapsed. Ceiling slabs deformed extensively, resulting in about a 3 m maximum subsidence of the national road running above the subway (Iida et al. 1996). The Daikai station is the first reported subway structure that was completely destroyed during an earthquake.

The effects of earthquakes on tunnels may be grouped into two categories – faulting and ground shaking. In general, it is not feasible to design tunnels to restrain major ground displacements associated with faulting. If it is not possible to avoid areas with faulting, it may be necessary to design the tunnel to accept the displacements, localize and minimize damage, and provide the means to facilitate repairs.

Seismic effects of ground shaking on a linear running tunnel can induce axial and curvature deformations on the tunnel when seismic waves propagate either parallel or obliquely to the tunnel. These types of deformations are basically in the longitudinal direction along the tunnel axis.

Shallow transportation tunnels are often rectangular in shape built using the cut and cover method. For moment frame box structures, the most critical mode of potential damage to tunnels is due to racking effects. During earthquakes, a rectangular box structure will experience transverse racking deformations (sideways sway motion) due to the shear distortion of the ground. Maximum bending moments are induced at the top and bottom corners of the box structures, as illustrated in Fig. 1.

The dynamic soil-structure interaction analyses conducted for the seismic design of the New Westminster Rapid Transit Tunnel, British Columbia, are described in this paper. The results are also compared to the simple dynamic earth pressure approach and the shear strain approach.

Methods of analyses

Dynamic earth pressure method

In this method, the dynamic earth pressure, which is assumed to be caused by the inertial force of the surrounding soils, acts as an unbalanced load on the tunnel walls. The dynamic earth pressure is related to a seismic coefficient, the soil properties and the amount of relative tunnel movement. The commonly used dynamic earth pressure methods include the Mononobe-Okabe method (Okabe, 1926; Mononobe and Matsuo, 1929) and the Wood solution (1973), both originally developed for above ground earth retaining walls. The Mononobe-Okabe method assumes that the retaining wall structure would move or tilt sufficiently so that an active failure wedge would form in the soil backfill behind the wall. The Wood solution was derived assuming linear elastic soil and rigid retaining wall. As both methods were originally developed for above ground earth retaining structures resisting the inertia load from the backfill, they are inappropriate for use in the design of underground tunnel structures.

Shear strain approach

Designers recognize that it is more realistic to use a deformation-based rather than a load-based approach for the design of tunnels. The amount of racking imposed on the tunnel liner is often assumed to be equal to the free field shear distortion of the surrounding soil medium (Monsees and Merritt, 1988). Figure 2 shows a typical free field shear deformation profile computed using a ground response analysis program such as SHAKE and the resulting differential shear to be used to evaluate the racking distortion, γ_r, on a tunnel structure.

A long rectangular cavity in the soil, with no lining present, will experience a racking distortion, γ_r, much greater than the free field soil racking distortion, γ_{ff}. For this case, it can be shown that, $\gamma_r \approx 4(1-\nu)\gamma_{ff}$, in which ν is the soil Poisson's ratio (Penzien, personal comm., 1999). For a ν of 0.3, γ_r is 2.8 times γ_{ff}. Therefore, for a very flexible tunnel liner, the racking effects will be grossly underestimated based on the free field ground motions.

Wang (1993) has proposed an improved shear strain method by introducing a flexibility ratio, which is defined as the ratio between the soil stiffness in simple shear condition and the stiffness of the tunnel liner against racking. Once the flexibility ratio is determined, the racking distortion for the tunnel can be estimated from the free field distortion using the relationship shown in Fig. 3.

Dynamic soil-structure interaction analyses

A better approach to evaluate the racking effects is to conduct dynamic soil-structure interaction analyses. Commercially available computer programs, such as FLUSH, which uses an equivalent linear elastic finite element approach, or FLAC, which uses an elastic or nonlinear finite difference approach, are suitable for 2D dynamic analyses of tunnels. Effects of tunnel liner stiffness, embedment depth, soil stiffness, variation in subsoil stratigraphy and surface topography can be readily evaluated in the analyses.

Project description

Proposed development

The New Westminster Tunnel is part of the Rapid Transit Project 2000, which involves a new 21 km SkyTrain extension to link the existing New Westminster Street Station to a new station at Commercial Drive in Vancouver. The tunnel is approximately 800 m in length. It extends north-east from the existing SkyTrain spur at Elliot Street, passes beneath the north approach to Pattullo Bridge and exits the ground part way down the slope overlooking the Fraser River at the intersection of McBride Boulevard and Columbia Street. The horizontal and vertical tunnel alignments are shown on Figure 4.

The local transportation authority, Translink, has set up a Rapid Transit Project Office (RTPO) to administer and execute this project. Except for a few special structures, the majority of the SkyTrain extension project was awarded in two design-build contracts. The design-build contract for the New Westminster Tunnel was awarded to Walter Construction (Canada) Ltd./BFC Joint Venture (WBFC). Klohn-Crippen/Simons (KCS) are the design engineers for WBFC.

The New Westminster Tunnel is a twin-tunnel structure of approximately 5 m high by 10 m wide. The tunnel walls vary from 325 mm (Type 1 wall) to 675 mm (Type 5 wall) in thickness, depending on the thickness of the soil cover. The thickness of soil cover varies from zero to as much as 6 m. The tunnel is being constructed by the cut and cover method.

Seismic design criteria for the New Westminster Tunnel were provided by RTPO. The structure is designed to resist a major seismic event with repairable damage and a moderate seismic event with no significant damage. The specified seismic events and corresponding ground motions are summarized in Table 1.

Soil conditions

The surficial geology of New Westminster is described in Geological Survey of Canada, Paper 57-5, by J.E. Armstrong (1957). The surficial geology map presented in that report has more recently been updated and re-issued as Map 1484A. An inferred geologic profile along the tunnel centreline is shown in Figure 4. Key geological units, from surface down, consist of the following:

- Unit 1 (FILL) - The fill is a mixture of silt, sand and gravel, with some layers of cobbles and boulders, and is generally medium dense to dense but has loose layers.
- Unit 2 (GLACIO-MARINE) - The glacio-marine deposits appear as a thin layer mantling much of the surface of the area, but absent in some areas. Where present, it is generally less than 3 m thick, consisting of low to medium plastic silt with traces of sand and gravel.
- Unit 3 (VASHON TILL) - The till is a broadly graded mixture of silt, sand, gravel, cobbles and boulders. It is generally very dense. Within the till there are layers and lenses of substratified clean sand and sand and gravel, of unknown and variable extent, that are saturated and more permeable than the till, such that they will yield water when exposed in the sides of excavations or when pumped.
- Unit 4 (PRE-VASHON DEPOSIT) - This unit is mostly an interbedded sequence of sand and low to medium plastic clayey silt. It was likely originally laid down in a marine deltaic or estuarine environment, and was subsequently overconsolidated by glacial ice.

The proposed tunnel invert is founded in either Unit 3 or 4 material. The proposed tunnel alignment is subparallel to and for the most part, 40 m to 80 m north of the bluffs overlooking the Fraser River. The topography continues to rise to an elevation of about 100 m one kilometre to the west. This topographic setting significantly influences the local groundwater conditions.

Dynamic soil-structure interaction analyses

Methodology

The racking effects on the New Westminster Tunnel were assessed by conducting two-dimensional dynamic finite element analyses using the computer program FLUSH (Lysmer et al. 1975). Nonlinear stress-strain behavior of the soil is modeled by an equivalent linear elastic method. The tunnel lining is modeled with structural elements.

Two cross sections, A and B were analyzed, where the tunnel roof is under 6 m and 3 m soil cover, respectively. Properties of the tunnel walls, Types 1 and 5, are summarized in Table 2. In addition, sensitivity analyses were carried out for different combination of tunnel wall thickness and soil cover to assess their effects on dynamic soil-tunnel response.

Finite element meshes and input soil properties for Sections A and B are shown on Figure 5. The published dynamic modulus and damping vs. shear strain relationships by Murphy et al. (1978) and Seed et al. (1986) were used for Till, and Sand & Gravel units, respectively.

Four earthquake input motions were used in the FLUSH analyses. These records are listed in Table 3. These records were first fed through one-dimensional site response columns using the computer program SHAKE to convert the recorded rock outcrop motions to base motions as input to the FLUSH models to account for the compliance of the material underlying the FLUSH model. As a result, the input peak horizontal ground accelerations (PGA), which range from 0.15 g to 0.17 g, are slightly smaller than those shown in Table 1. In this paper, only the results from Caltech record for the 475 year return period event are presented. Similar results were obtained with the other earthquake records.

Results and discussions

Ground Amplification - Some of the PGA values as computed from FLUSH analyses for Sections A and B are shown on Figure 6. Due to the sloping ground condition, some topographic amplification of PGA at ground surface is observed. Near the tunnel walls, the computed PGA is also higher than the input PGA at the model base. It is also noted that the computed PGA adjacent to the tunnel liner does not change significantly with different liner thickness, although a slightly higher PGA is computed for the more flexible tunnel.

Tunnel Wall Bending Moment - Figure 7 shows typical computed bending moment time histories of beam elements at the uphill tunnel walls at Sections A and B. It is seen that the thinner wall in Section B has much smaller bending moments than the thicker wall in Section A, and the peak bending moments of each beam element occur almost at the same time, about 6 sec. Figure 8 shows the distribution of bending moments within both uphill and downhill tunnel walls at about 6 sec for Section A and a Type 5 wall. It is seen that the tunnel essentially follows the ground motion and undergoes a simple shear or "racking" type of deformation. However, the bending moment diagrams are unsymmetrical with higher bending moments occurring in the uphill side of the tunnel wall.

Comparison with simplified methods

Dynamic earth thrust

The maximum dynamic earth thrust on the tunnel walls can be estimated using the bending moment diagrams computed from FLUSH. These dynamic earth thrust can be compared with those derived using conventional retaining wall theories as given by Mononobe-Okabe for the yielding wall and by Wood for the rigid wall, as illustrated on Figure 9. It is noted that there are many uncertainties associated with the calculations of the dynamic earth thrust (e.g. location of the resultant load, effects of the soil cover etc.). Table 4 summarizes the dynamic earth thrust on tunnel walls for different wall stiffnesses and soil covers, as well as those derived using yielding and non-yielding wall theories. FLUSH results indicate that wall stiffness is an important factor with regards to dynamic earth pressures on tunnel walls; more rigid tunnel walls will attract higher dynamic earth pressures. For the same type of wall, influence of the depth of soil cover on the earth pressures is not significant. Comparison with retaining wall theories shows that both Mononobe-Okabe and Wood solutions would significantly underestimate dynamic earth thrusts on the more rigid Type 5 wall regardless of the depth of soil cover.

Bending moment

A simple structural frame model was set up in PFRAME to estimate bending moments in the tunnel wall for both simplified methods. Dynamic earth thrusts described above for yielding and rigid walls were applied on the uphill wall at a distance of 0.6 times the wall height above the base. In the shear strain method, a point load was applied at the top corner of the uphill wall until the estimated racking angle was attained. Figure 10 shows the structural frame models, typical deformed shapes of the tunnel and bending moment diagrams from both simplified methods. Table 5 summarizes the computed maximum bending moments, as well as those from the FLUSH analyses for Types 1 and 5 walls at Section A with 6 m of soil cover.

As compared to the FLUSH results, the dynamic earth pressure method is not sensitive to the wall stiffness; it significantly overestimates the maximum bending moment for a Type 1 wall and underestimates that for a Type 5 wall. In addition, the maximum bending moment occurs at the location of the applied load (i.e. 0.6 times the wall height above the base) instead of at the expected top and bottom corners of the tunnel.

As shown in Table 5, the shear strain approach is sensitive to the wall stiffness. However, the original shear strain method grossly underestimates the maximum bending moments for both Types 1 and 5 walls. The differences are smaller for the improved shear strain method; however, the computed bending moments are still only 44% and 61%, respectively of the corresponding FLUSH results. This is due to the effects of sloping ground, which cannot be readily accounted for in the simplified methods.

To illustrate the sloping ground effects on maximum tunnel wall bending moment, soil-structure interaction analyses using FLUSH were also carried out for a tunnel (both Types 1 and 5 walls) embedded in level ground with 6 m of soil cover. The results are also summarized in Table 5 for comparison. It is seen from the FLUSH results that sloping ground results in much higher bending moments in tunnel walls, as compared to the level ground case, which is currently not accounted for by the simplified methods. Although it could still underestimate the dynamic maximum bending moment in the tunnel, the improved shear strain method is the best among the simplified pseudo-static methods reviewed in this paper for estimating the maximum bending moment and its distribution for the level ground condition.

Summary and conclusions

Dynamic earth pressure methods are not considered suitable for seismic design of tunnels, as they are not sensitive to the tunnel lining stiffness. Both the magnitude and location of the maximum bending moment determined from the simplified methods are significantly different from the FLUSH results.

The improved shear strain method is better than the dynamic earth pressure method, as it can approximately account for the effect of tunnel lining stiffness. However, in certain circumstances (e.g. sloping ground), the shear strain method could still underestimate the racking effects.

Dynamic soil-structure analyses should be considered in seismic design of tunnels. Various design parameters (e.g. wall stiffness, soil cover, sloping ground etc.) can be readily accounted for in such analyses.

Acknowledgements

The Authors wish to thank Walter/BFC Joint Venture and Rapid Transit Project Office for their permission to publish this paper. The Authors also wish to thank Dr. Joseph Penizen for his input to the project, and Dr. Alex Sy and Mr. Adrian Wightman of Klohn-Crippen for their review and valuable comments.

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Table 1 - Specified Major and Moderate Seismic Events

Seismic Events	Probability Of Exceedance In 50 Years	Mean Return Period	Peak Ground Acceleration (*)
Moderate	40%	100 year	0.09 g
Major	10%	475 year	0.23 g

(*) Firm ground conditions.

Table 2 - Properties of Tunnel Lining

Section	Soil Cover (m)	Type Of Tunnel Wall	Thickness Of Tunnel Lining (mm)	Young's Modulus, E (kPa)	Section Modulus, I (m^4/m)
A	6	5	675	25 E6	0.0255
B	3	1	375	25 E6	0.0045

Note: (1) I = 0.0037 m^4/m for the cross wall at both sections A and B.

Table 3 - Earthquake Input Motions for FLUSH Analyses

Earthquake Event	Station	Epicentral Dist. (km)	PGA (g) Before Scaling	PGA (g) After Scaling
San Fernando, 1971 M6.5	CalTech Lab S90W	35	0.19	0.23 and 0.09
San Fernando, 1971 M6.5	Lake Hughes, S21W	35	0.146	0.09
San Fernando, 1971 M6.5	Griffith Park Obs. S90W	33	0.17	0.23
Northridge, 1994 M6.7	Griffith Park Obs. 360 deg	27	0.167	0.23

Table 4 - Summary of Dynamic Earth Thrust from FLUSH and Retaining Wall Theories

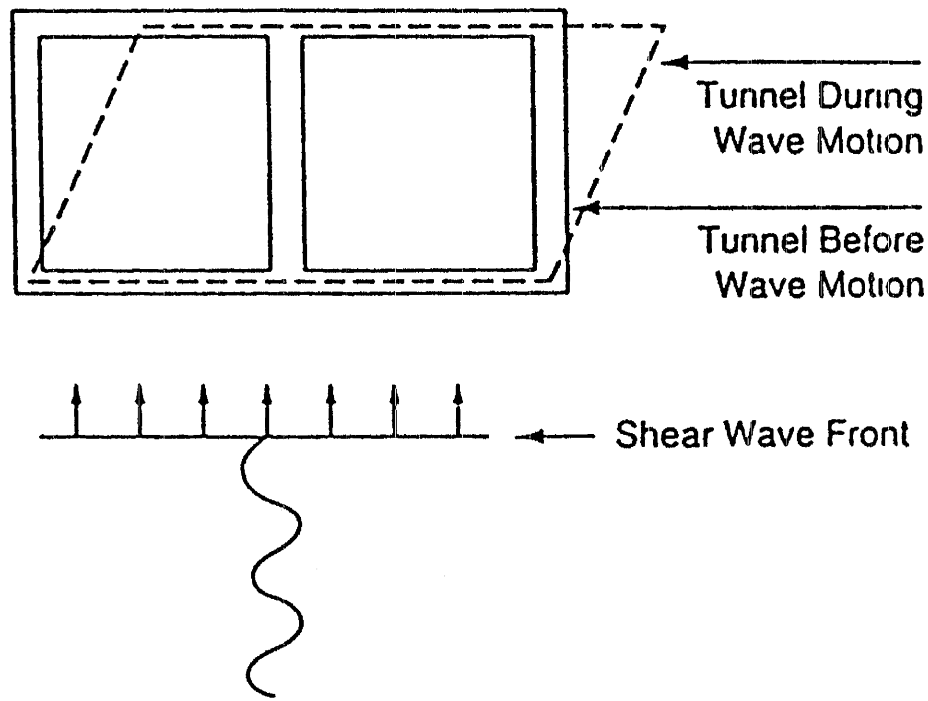
				FLUSH Results(1) (kN/m)		Retaining Wall Theories(2) (kN/m)			
Section	Type of wall	Wall thickness (mm)	Soil cover (m)	Uphill wall	Downhill wall	Seismic Coeff. k_h (g)	Yielding wall with surcharge	Rigid wall without surcharge	Rigid wall with surcharge
A	1	375	6	153	160	0.23	175	139	219
A	5	675	6	340	340	0.23	175	139	219
B	1	375	3	156	142	0.32	159	190	248
B	5	675	3	346	352	0.32	159	190	248

Notes: (1) based on Caltech record.
(2) yielding wall uses Mononobe-Okabe solution and rigid wall uses Wood solution (Ebeling and Morrison 1992) with total soil unit weight to approximately account for the effects of the pore water pressure (Whitman, 1996).
(3) static lateral earth loading is additive.

Table 5 – Summary of Maximum Computed Bending Moment(1)

Wall Types	FLUSH	Shear Strain Method		Dynamic Earth Pressure Methods			FLUSH
	Section A(2) (kN-m/m)	Original (kN-m/m)	Improved (4) (kN-m/m)	Yielding wall with surcharge (kN-m/m)	Rigid wall without Surcharge (kN-m/m)	Rigid wall With Surcharge (kN-m/m)	Level Ground(3) (kN-m/m)
1	88	14	39	156	125	196	59
5	271	65	160	165	129	205	189

Note: (1) based on Section A - Types 1 and 5 walls.
(2) sloping ground conditions.
(3) level ground with 6 m soil cover.
(4) for the improved shear strain method, flexibility ratios of 28 and 7.2 were used with Figure 3 for Types 1 and 5 walls, respectively.



Racking Deformation of a Rectangular Cross Section

FIGURE 1. EFFECTS OF EARTHQUAKE GROUND SHAKING ON TUNNELS (AFTER WANG, 1993)

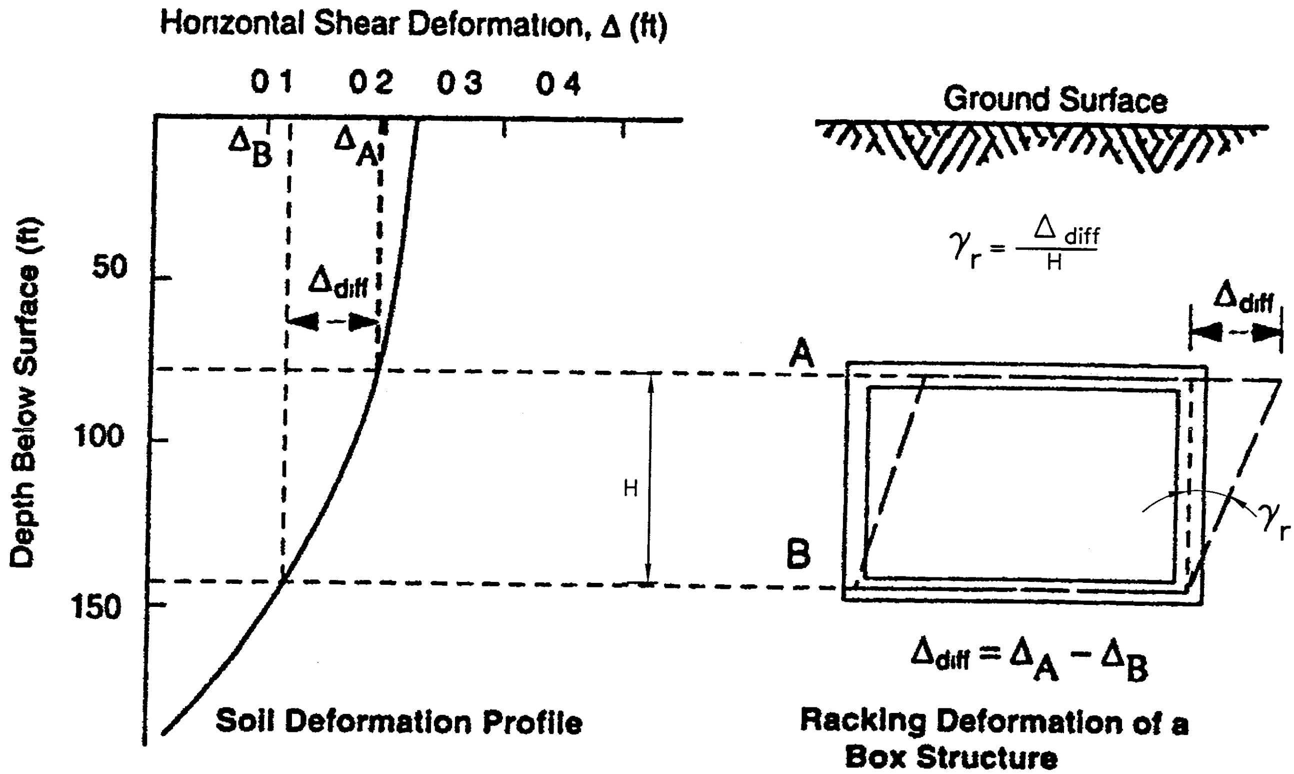
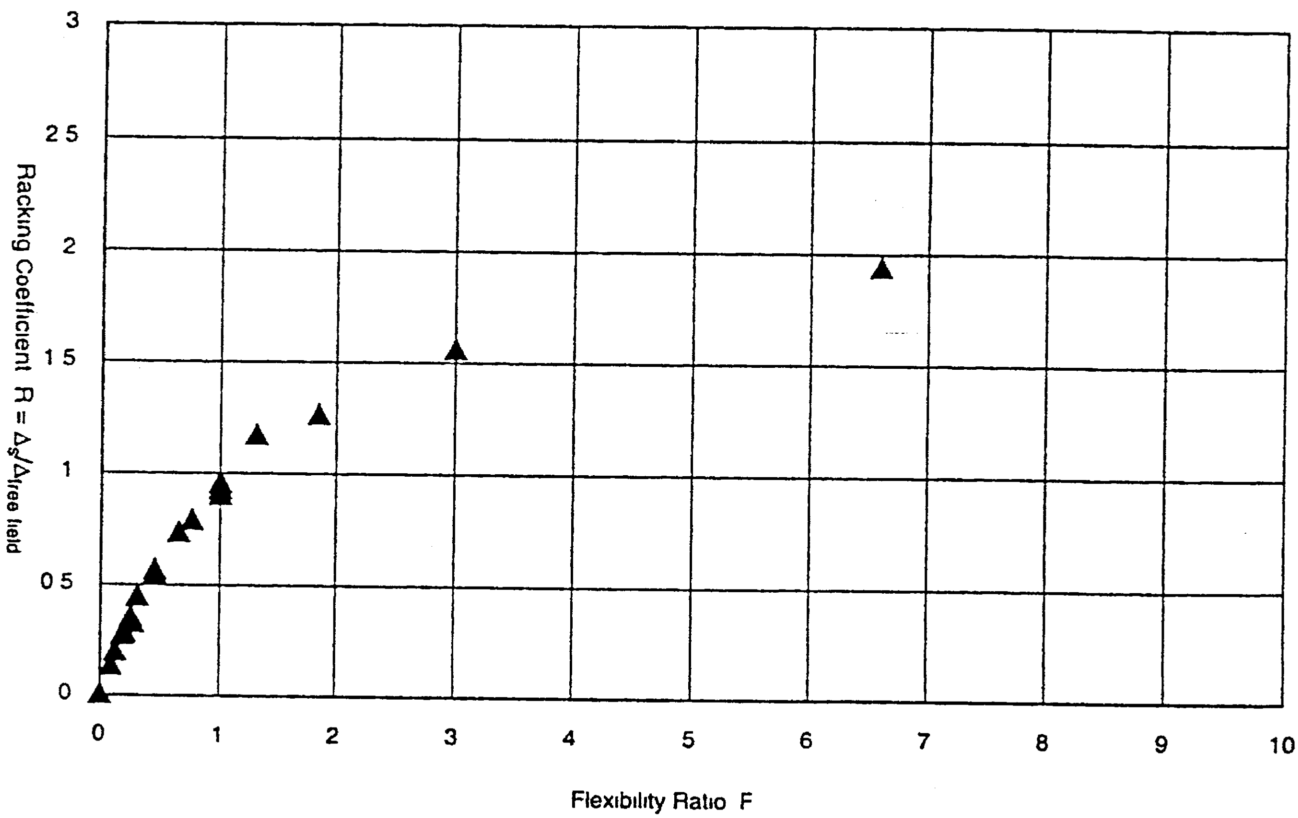


FIGURE 2. TYPICAL FREE–FIELD RACKING DEFORMATION IMPOSED ON A BURIED RECTANGULAR FRAME (AFTER ST. JOHN AND ZAHRAH, 1987)



FLEXIBILITY RATIO, F = $\frac{\text{STIFFNESS OF RECTANGULAR SOIL ELEMENT IN SIMPLE SHEAR}}{\text{RACKING STIFFNESS OF STRUCTURE}}$

FIGURE 3. EFFECT OF RELATIVE STRUCTURAL STIFFNESS ON RACKING DEFLECTIONS (AFTER WANG, 1993)

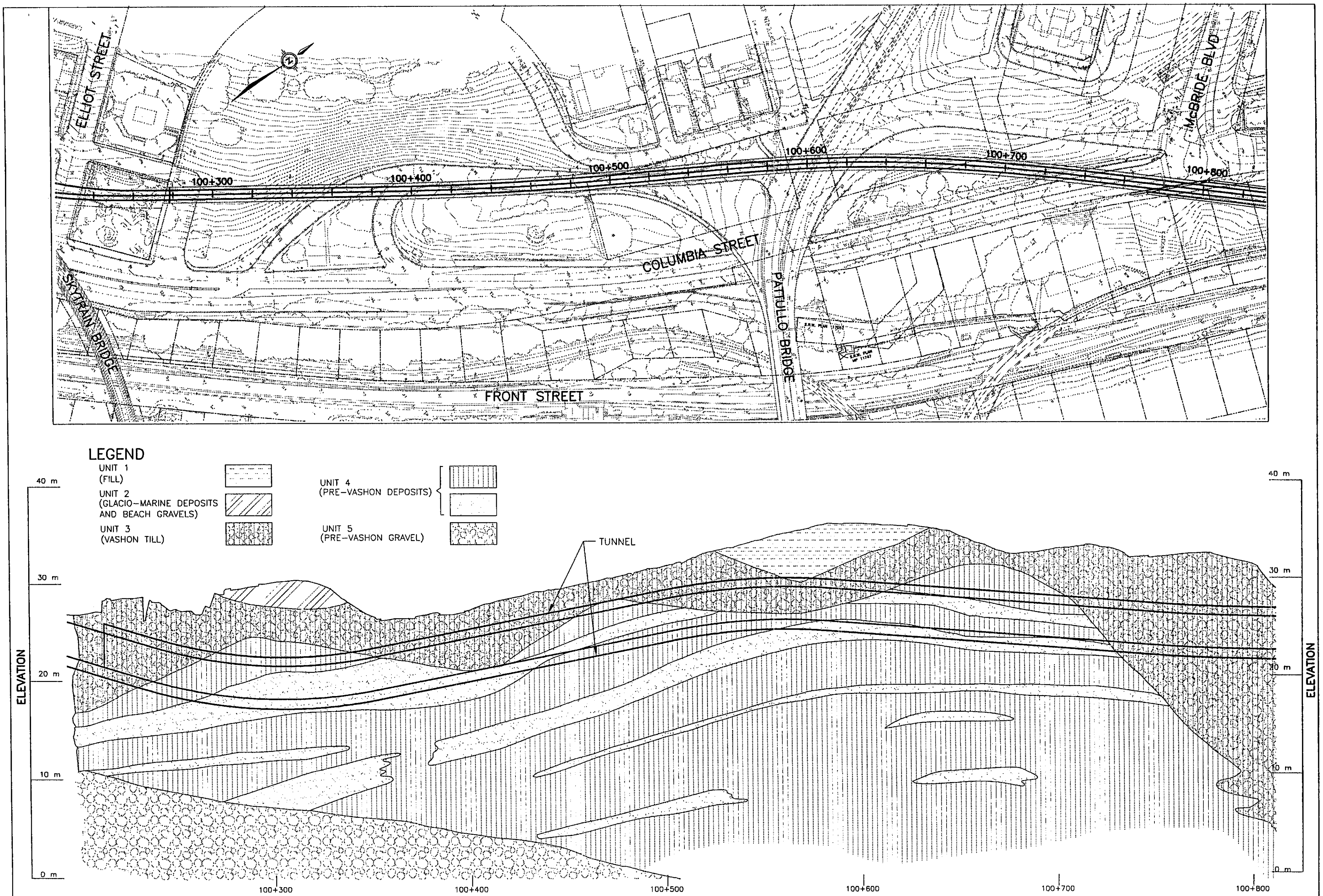


FIGURE 4. SITE PLAN AND TUNNEL ALIGNMENT

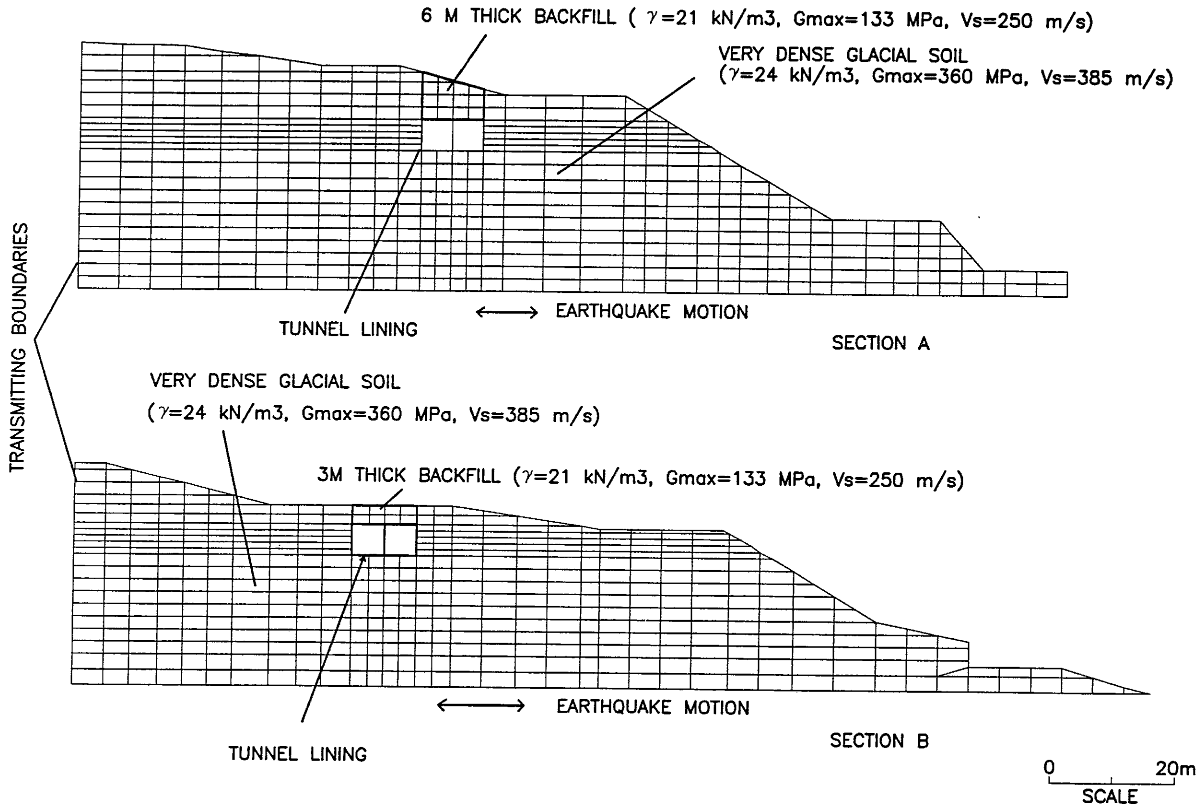


FIGURE 5. FLUSH MODELS FOR SECTIONS A AND B

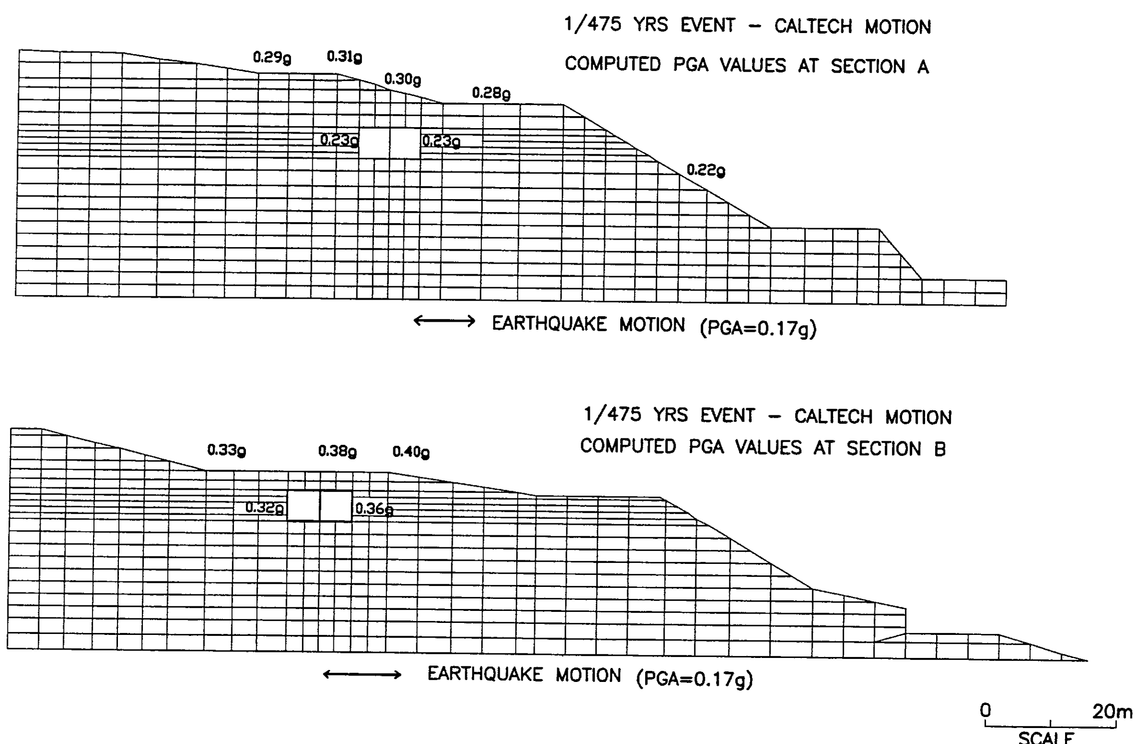
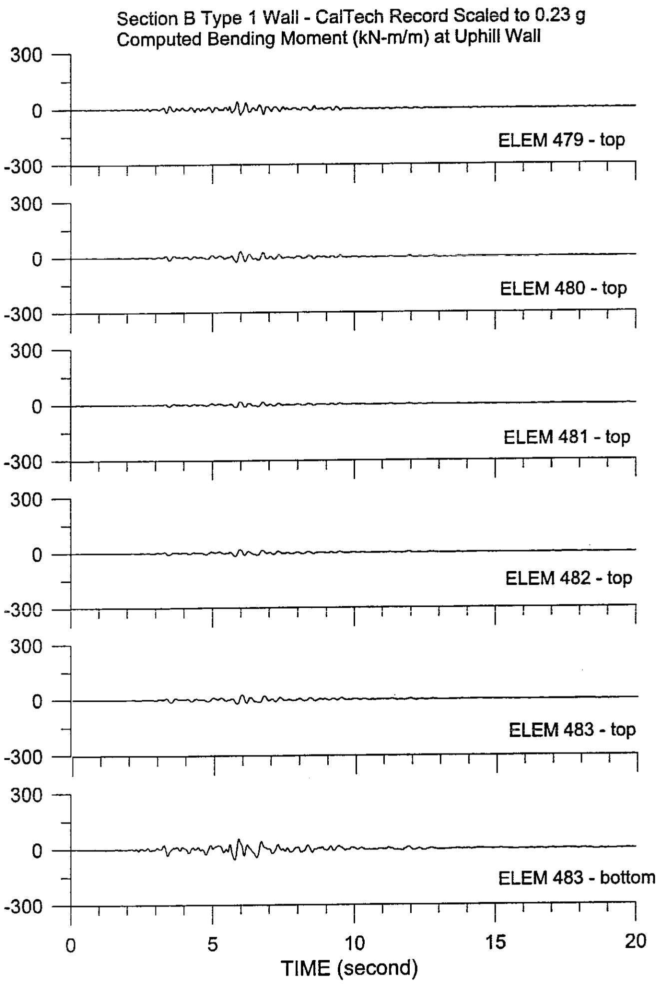
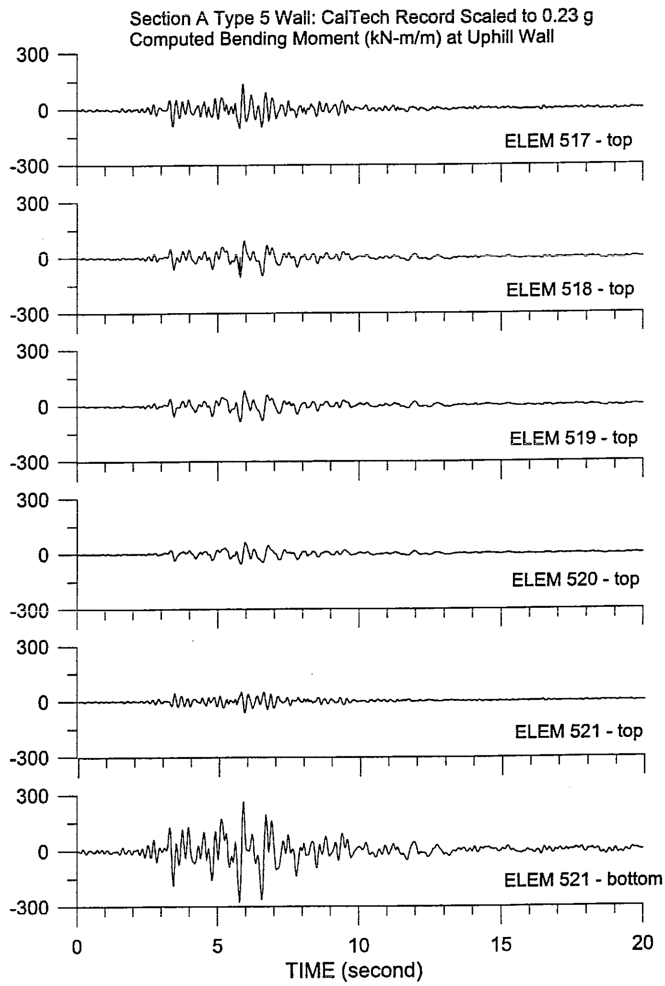
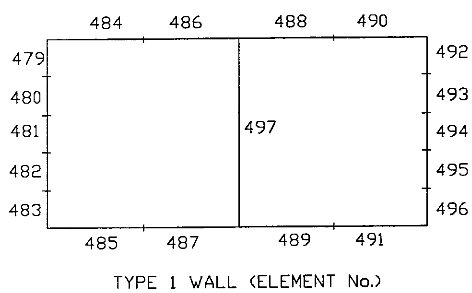


FIGURE 6. COMPUTED PGA VALUES FROM FLUSH







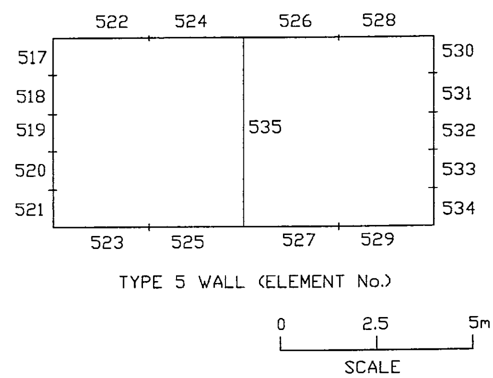


FIGURE 7. COMPUTED BENDING MOMENT TIME HISTORIES FROM FLUSH

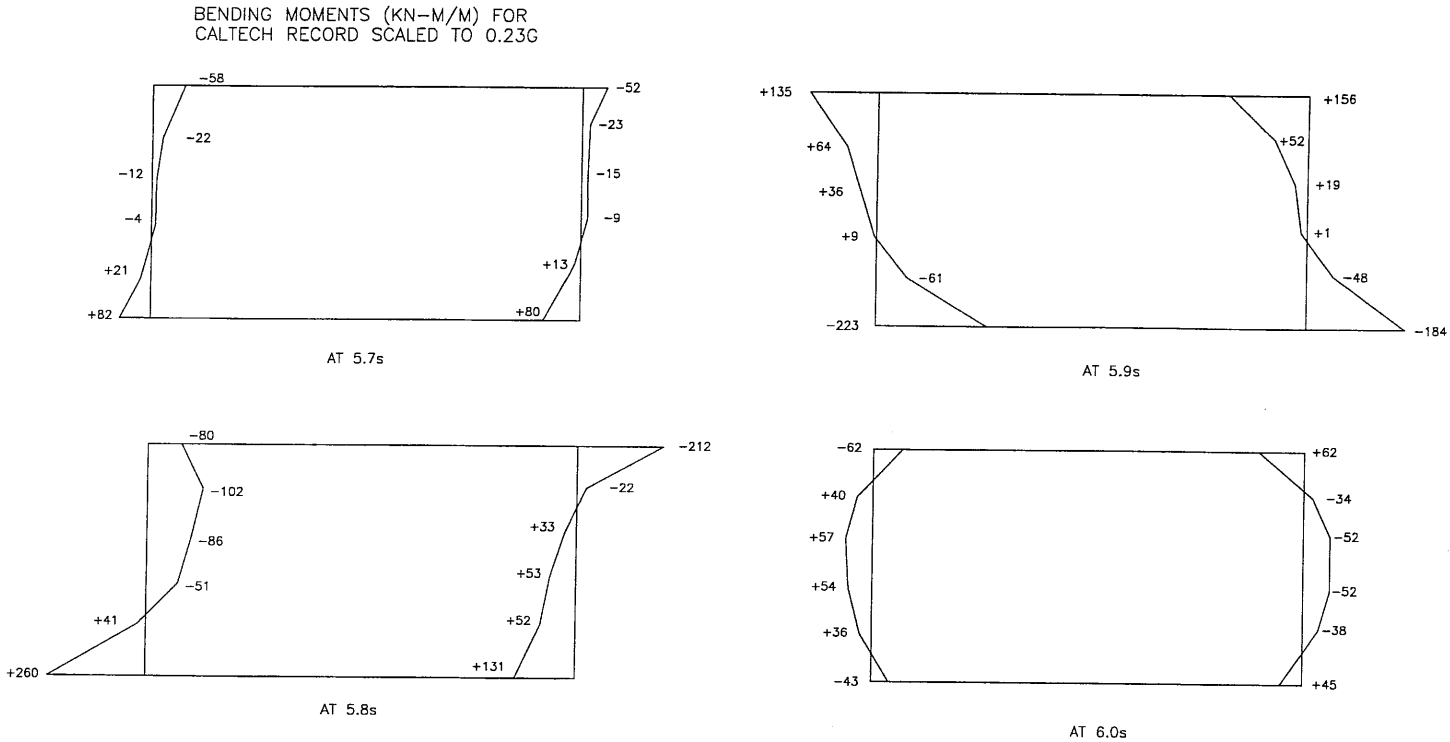


FIGURE 8. COMPUTED BENDING MOMENT DIAGRAMS AT UPHILL AND DOWNHILL WALLS FOR SECTION A AND TYPE 5 WALL FROM FLUSH

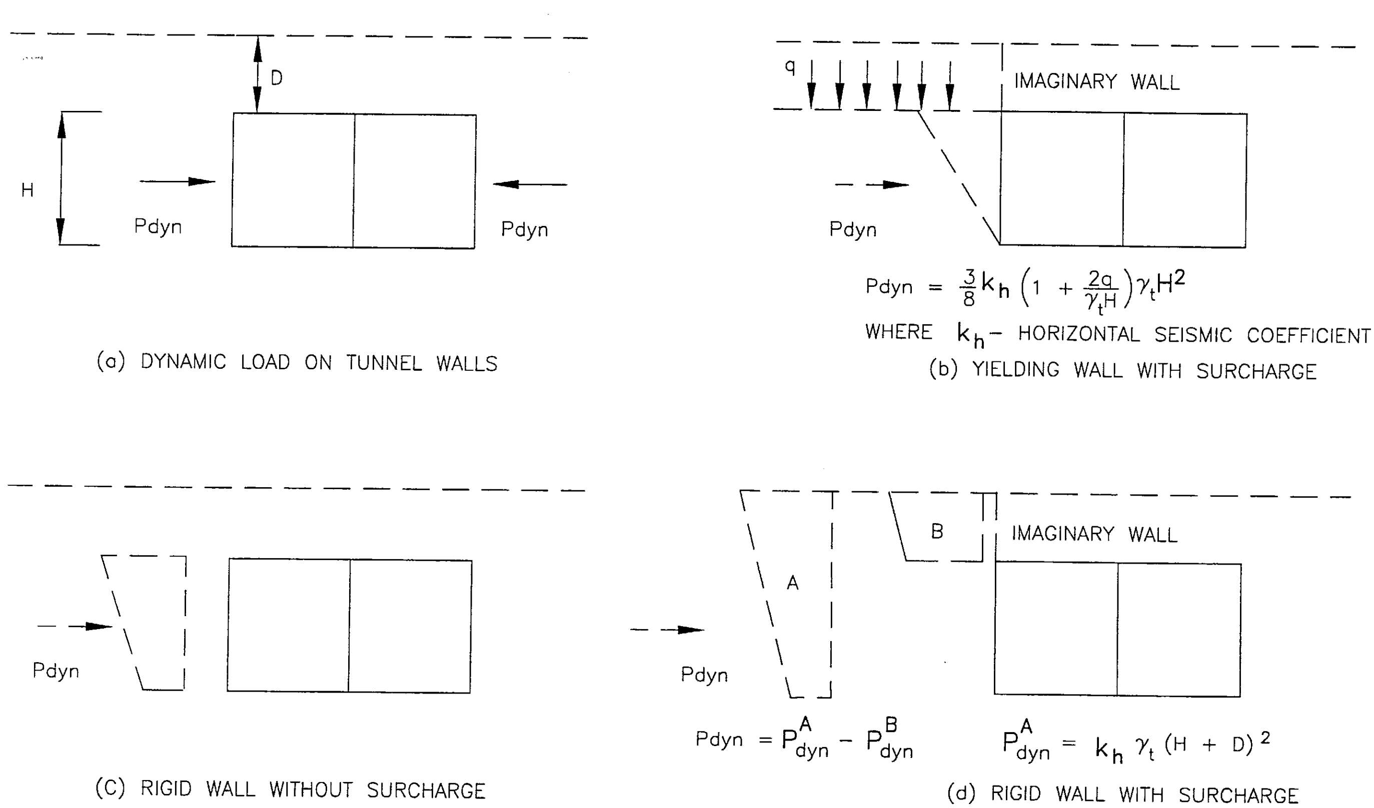
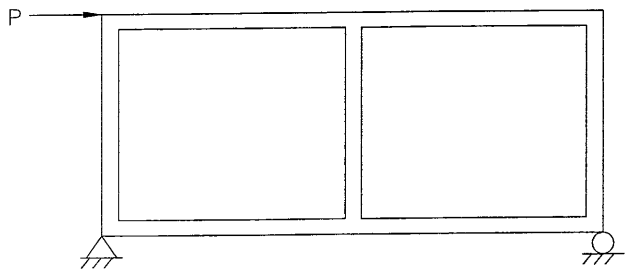
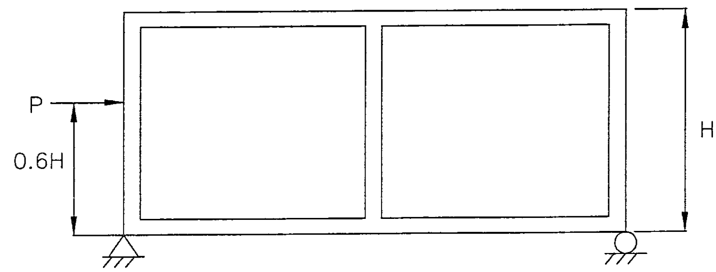


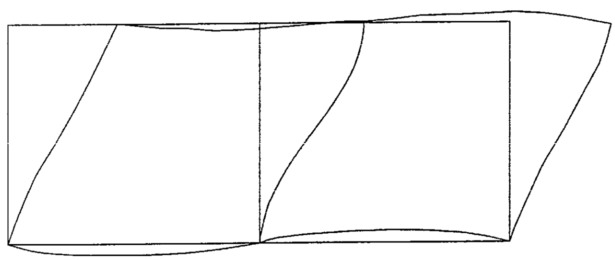
FIGURE 9. CALCULATION OF DYNAMIC LOADS ON TUNNEL WALL



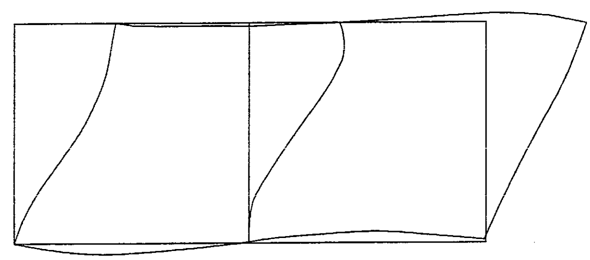
SHEAR STRAIN METHOD



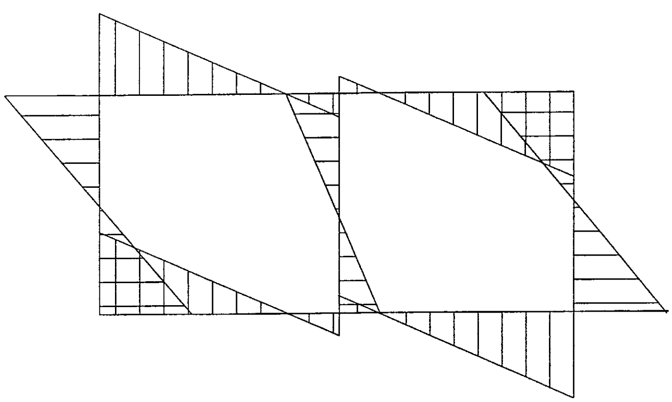
FORCE METHOD



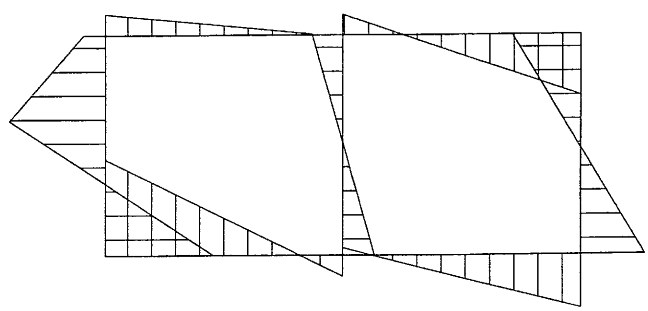
SHEAR STRAIN METHOD – DEFLECTION



FORCE METHOD – DEFLECTION



SHEAR STRAIN METHOD – BENDING MOMENT



FORCE METHOD – BENDING MOMENT

FIGURE 10. STRUCTURAL FRAME MODEL FOR CALCULATION OF TYPE 5 WALL BENDING MOMENTS USING SIMPLIFIED METHODS