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Seismic design of tunnels

J-N. Wang[1], G.A. Munfakh[2]
[1]*Senior Professional Associate, Parsons Brinckerhoff Quade and Douglas, Inc. USA*
[2]*Senior Vice President, Parsons Brinckerhoff Quade and Douglas, Inc. USA*

Abstract

An analytical approach is presented for dealing with the seismic design and analysis of both bored (circular) and cut-and-cover (rectangular) tunnels. This approach considers the soil-structure interaction effect and focuses on the ovaling/racking deformation aspect of the tunnel structures. The procedure presented for the bored tunnels is developed from a theory that is familiar to most mining/underground engineers (Peck et al., 1972). Simple and easy-to-use seismic design charts are presented. The design charts are expressed primarily as a function of relative stiffness between the structure and the ground. The results are validated through a series of finite element/difference soil-structure interaction analyses.

For the cut-and-cover tunnels, the design solutions are derived from an extensive study using dynamic finite-element soil-structure interaction analyses. A wide range of structural, geotechnical, and ground motion parameters are considered in this study. Specifically, five different types of cut-and-cover tunnel geometry are studied, including one-barrel, one-over-one two-barrel, and one-by-one twin-barrel configurations. To quantify the effect of relative stiffness on tunnel lining response, varying ground profiles and soil properties are used in the parametric analyses. Based on the results of the parametric analyses, a deformation-based design chart is developed for cut-and-cover tunnels.

1 Introduction

For underground structures such as tunnels, the seismic design approach differs from that of the surface structures such as bridges and buildings. Surface structures are not only directly subjected to the excitations of the ground, but also

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experience amplification of the shaking motions depending on their own vibratory characteristics. If the predominant vibratory frequency of the structures is similar to the natural frequency of the ground motions, the structures are excited by resonant effects.

In contrast, underground structures are constrained by the surrounding medium (soil or rock). It is unlikely that they could move to any significant extent independently of the medium or be subjected to vibration amplification. Compared to surface structures, which are generally unsupported above their foundations, the underground structures can be considered to display significantly greater degrees of redundancy thanks to the support from the ground. These are the main factors contributing to the better earthquake performance data for underground structures than their aboveground counterparts.

The different response characteristics of aboveground and underground structures suggest different design and analysis approaches. For aboveground structures, the seismic loads are largely expressed in terms of inertial forces. The traditional methods generally involve the application of equivalent or pseudo-static forces in the analysis. The design and analysis for underground structures should be based, however, on an approach that focuses on the displacement/deformation aspects of the ground and the structures, because the seismic response of underground structures is more sensitive to such earthquake induced deformations. The deformation method is the focus of this paper.

2 Ovaling of circular tunnels

Ovaling of a circular tunnel lining is primarily caused by seismic waves propagating in planes perpendicular to the tunnel axis. Usually, it is the vertically propagating shear waves that produce the most critical ovaling distortion of the lining. These shear distortions produced by the ground can also cause a rectangular tunnel to rack (sideways motion), as shown in Figure 1. The results are cycles of additional stresses/strains with alternating additional compression and tension in the tunnel lining. These dynamic stresses/strains should be superimposed on the existing static state of stress/strain in the lining (Owen and Scholl, 1981) for design and evaluation.

The seismic ovaling effect on the lining of bored/mined circular tunnels is best defined in terms of change of tunnel diameter, ΔD_{EQ}. ΔD_{EQ} can be considered as seismic ovaling deformation demand for the lining. The procedure for determining ΔD_{EQ} and the corresponding lining strains is outlined as follows (Wang, 1993).

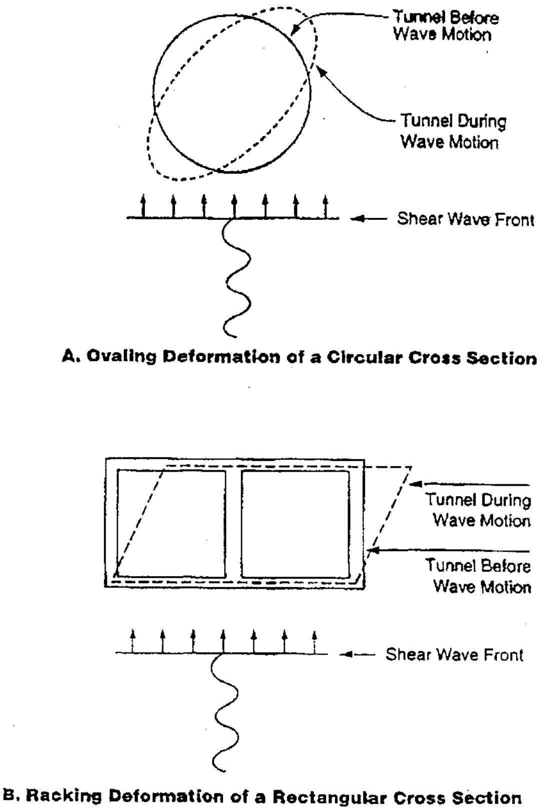


Figure 1 Ovaling and racking deformations

<u>*Step 1.*</u> Estimate the expected free-field ground strains caused by the vertically propagating shear waves of the design earthquakes using the following formula:

$$\gamma_{max} = V_s / C_{se} \qquad (1)$$

where: γ_{max} = maximum free-field shear strain at the elevation of the tunnel
V_s = S-wave peak particle velocity at the tunnel elevation
C_{se} = effective shear wave velocity of ground surrounding the tunnel

Alternatively, the maximum free-field shear strain can be estimated by a more refined free-field site response analysis (e.g., SHAKE, 1972).

It should be noted that the effective shear wave velocity of the vertically propagating shear wave, C_{se} , should be compatible with the level of the shear strain that may develop in the ground at the elevation of the tunnel under the design earthquake shaking.

Step 2. By ignoring the stiffness of the tunnel, which is applicable for tunnels in rock or stiff/dense soils, the lining can be reasonably assumed to conform to the surrounding ground with the presence of a cavity due to the excavation of the tunnel.

The resulting diameter change of the tunnel is:

$$\Delta D_{EQ} = \pm 2\, \gamma_{max}\, (1 - \nu_m)\, D \qquad (2)$$

where: ν_m = Poisson's ratio of the surrounding ground
D = diameter of the tunnel.

Step 3. If the structure is stiff relative to the surrounding soil, then the effects of soil-structure interaction should be taken into consideration. The relative stiffness of the lining is measured by the flexibility ratio, F, defined as follows (Peck, et al., 1972):

$$F = \{E_m\, (1 - \nu_l^2)\, R_l^3\} \,/\, \{6\, E_l\, I_{l,1} (1 + \nu_m)\} \qquad (3)$$

Where: E_m = strain-compatible elastic modulus of the surrounding ground
R_l = nominal radius of the tunnel lining
ν_l = Poisson's ratio of the tunnel Lining
$I_{l,1}$ = moment of inertia of lining per unit width of tunnel

The strain-compatible elastic modulus of the surrounding ground E_m should be derived using the strain-compatible shear modulus G_m corresponding to the effective shear wave propagating velocity C_{se}.

The moment of inertia of the tunnel lining per unit width, $I_{l,1}$, should be determined based on the expected behavior of the selected lining under the combined seismic and static loads, accounting for cracking and joints between segments and between rings as appropriate.

Step 4. The diameter change, ΔD_{EQ}, accounting for the soil-structure interaction effects can then be estimated using the following equation:

$$\Delta D_{EQ} = \pm 1/3\, (k_1\, F\, \gamma_{max}\, D) \qquad (4)$$

where: k_1 = seismic ovaling coefficient

$$= 12\, (1 - \nu_m) \,/\, (2F + 5 - 6\, \nu_m) \qquad (5)$$

The seismic ovaling coefficient curves plotted as a function of F and ν_m are presented in Figure 2.

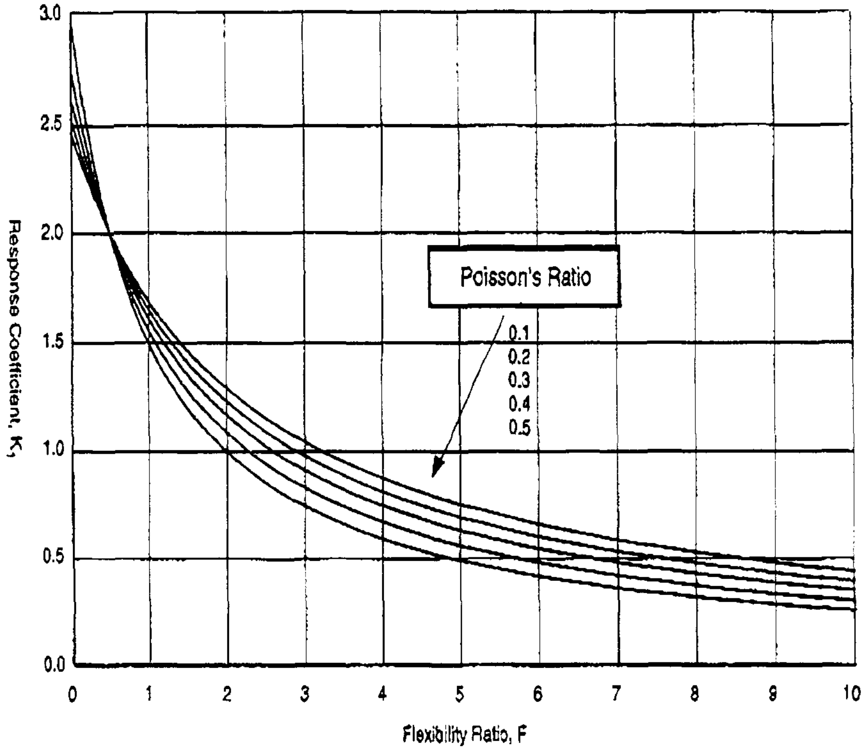


Figure 2: Seismic ovaling coefficient, K_1

The resulting bending moment induced maximum fiber strain, ε_m, and the axial force (i.e., thrust) induced strain, ε_T, in the lining can be derived as follows:

$$\varepsilon_m = \{ (1/6)\, k_1 [\, E_m / (1 + \nu_m)]\, R_l^{\,2}\ \gamma_{max} \} (t_l / 2) / (E_l I_{l,1}) \qquad (6)$$

$$\varepsilon_T = \{ (1/6)\, k_1 [\, E_m / (1 + \nu_m)]\, R_l\ \gamma_{max} \} / (E_l t_l) \qquad (7)$$

where: t_l = thickness of the lining.

The solutions presented in Equations 4 through 7 assume that a full slippage condition exists along the soil/lining interface, which allows normal stresses (without normal separation) but no tangential shear force. The full-slippage assumption yields slightly more conservative results in estimating the diameter change and bending strain but significantly lower values of thrust-induced strain than the no-slippage condition. Therefore, Equation 7 should not be used unless a full-slippage mechanism is incorporated in the design. Instead, the no-slippage condition should be assumed in deriving the thrust-induced strain as follows (Wang, 1993):

$$\varepsilon_T = \{ k_2 [E_m / 2(1 + \nu_m)] R_l \ \gamma_{max} \} / (E_l t_l) \qquad (8)$$

where: $k_2 = 1 + \{ F[(1 - 2\nu_m) - (1 - 2\nu_m)C] - \_ (1 - 2\nu_m)^2 + 2\} /$
$\{F[(3 - 2\nu_m) + (1 - 2\nu_m)C] + C[5/2 - 8\nu_m + 6\nu_m^2] + 6 - 8\nu_m\}$ (9)

C = compressibility ratio
$= [E_m (1 - \nu_c^2) R_l] / [E_c t (1 + \nu_m)(1 - 2\nu_m)]$ (10)

The seismically induced strains due to the ovaling effect need to be combined with strains resulting from non-seismic loading, and then checked against the allowable strain limits consistent with the performance goal established for the design of the tunnel lining.

3 Racking of rectangular tunnels

Racking deformations are defined as the differential sideways movements between the top and bottom elevations of the rectangular structures, shown as Δ_s in Figure 3. The resulting material strains in the lining associated with the seismic racking deformation, Δ_s, can be derived by imposing the differential deformation on the structure in a structural frame analysis.

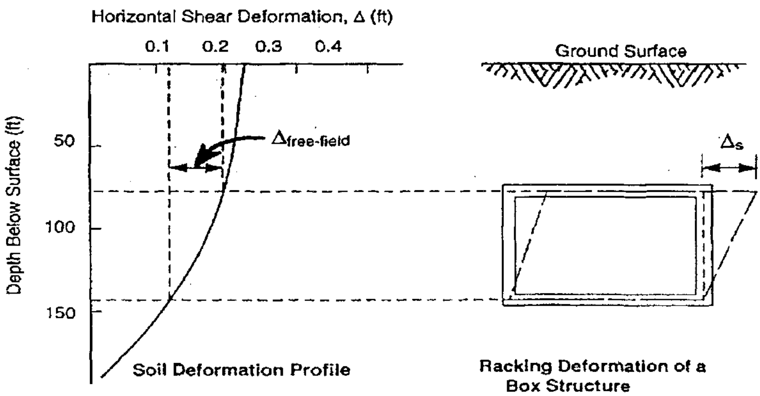


Figure 3: Racking deformations of a rectangular tunnel

The procedure for determining Δ_S, taking into account the soil-structure interaction effects, is presented below (Wang, 1993).

Step 1. Estimate the free-field ground strains γ_{max} (at the structure elevation) caused by the vertically propagating shear waves of the design earthquakes, see disccusion presented earlier. Determine $\Delta_{free\text{-}field}$, the differential free-field relative displacements corresponding to the top and the bottom elevations of the rectangular structure by:

$$\Delta_{free\text{-}field} = h \bullet \gamma_{max} \quad (11)$$

where: h = height of the structure

Step 2. Determine the racking stiffness, K_S, of the structure from a structural frame analysis. For practical purposes, the racking stiffness can be obtained by applying a unit lateral force at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The structural racking stiffness is defined as the ratio of the applied force to the resulting lateral displacement. In performing the structural frame analysis, it is important to use appropriate moment of inertia, taking into account the potential development of cracked section, particularly for the vertical walls.

Step 3. Determine the flexibility ratio, F_{rec}, of the proposed design of the structure using the following equation:

$$F_{rec} = (G_m / K_S) \bullet (w/h) \quad (12)$$

where: w = width of the structure
G_m = average strain-compatible shear modulus of the surrounding ground

The flexibility ratio is a measure of the relative racking stiffness of the surrounding ground to the racking stiffness of the structure. The derivation of F_{rec} is schematically depicted in Figure 4.

Step 4. Based on the flexibility ratio obtained form Step 3 above, determine the racking ratio, R_{rec}, for the structure using Figure 5 or the following expression:

$$R_{rec} = 4(1 - \nu_m) / \{ [(3 - 4 \nu_m) / F_{rec}] + 1 \} \quad (13)$$

The racking ratio is defined as the ratio of actual racking deformation of the structure to the free-field racking deformation in the ground. The triangular points in Figure 5 were data generated by performing a series of dynamic finite element analyses on a number of cases with varying soil and structural properties, structural configurations, and ground motion characteristics. Five different types of rectangular tunnel geometries are studied in the dynamic finite

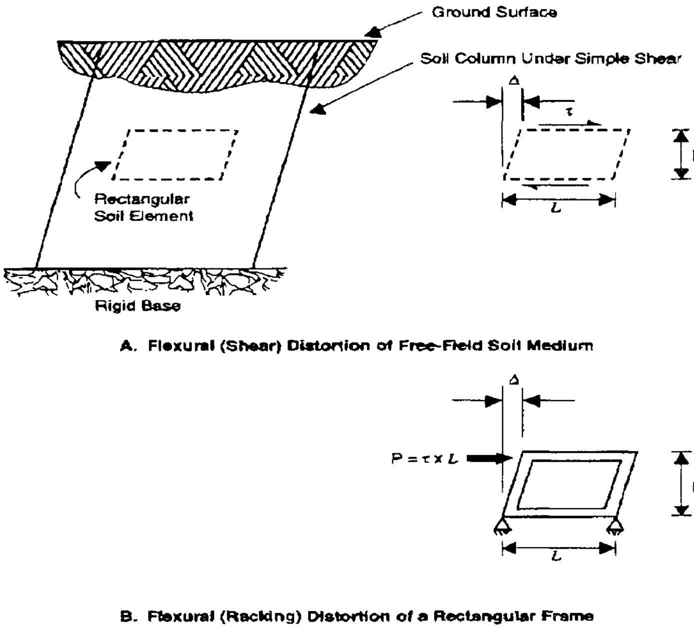


Figure 4: Relative stiffness of soil vs. rectangular frame

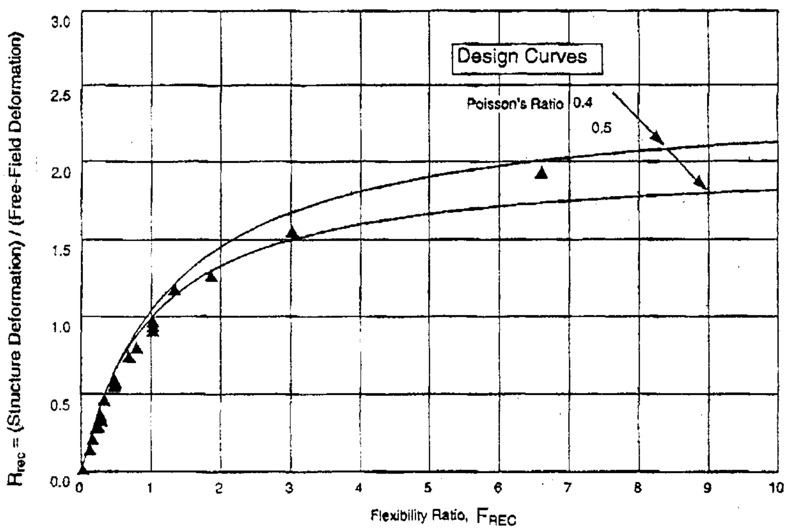


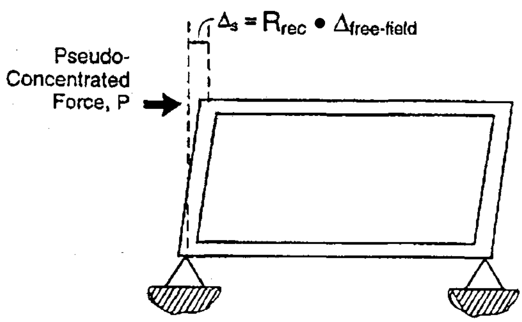
Figure 5: Racking ratio between structure and free-field

element analysis, including one-barrel, one-over-one two-barrel, and one-by-one twin-barrel configurations. As indicated in the figure, if $F_{rec} = 1$, the structure is considered to have the same racking stiffness as the surrounding ground and therefore the racking distortion of the structure is about the same as that of the ground in the free field. When F_{rec} is approaching zero, representing a perfectly rigid structure, the structure does not rack regardless of the distortion of the ground in the free field. For $F_{rec} > 1.0$ the structure becomes flexible relative to the ground and the racking distortion will be magnified in comparison to the shear distortion of the ground in the free field. This magnification effect is not caused by the effect of dynamic amplification. Rather, it is attributed to the fact that the ground has a cavity in it as opposed to the free field condition.

Step 5. Determine the racking deformation of the structure, Δ_s, using the following relationship:

$$\Delta s = R_{rec} \bullet \Delta_{free\text{-}field} \tag{14}$$

Step 6. The seismic demand in terms of internal forces as well as material strains can be calculated by imposing Δ_s upon the structure in a frame analysis as depicted in the following figure.



Frame Analysis Modeling of Racking Deformations

Figure 6

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4 Conclusions

A rational and consistent procedure, using a deformation-based approach, is presented for seismic design of tunnels under vertically propagating shear waves. The ovaling effect on circular tunnels and racking effect on rectangular tunnels are found to be highly dependent on the relative stiffness between the tunnel lining and the surrounding ground, making soil-structure interaction one of the most important factors in the seismic design and evaluation for tunnel structures. When a tunnel structure is more flexible than the ground, the tunnel lining will experience amplified ovaling/racking distortions in comparison to the shear distortions of the ground in the free field. On the other hand, when a tunnel is stiffer than the ground it tends to resist the ground deformations, resulting in smaller lining distortions compared to those produced in the ground.

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