FACTA UNIVERSITATIS
Series: **Architecture and Civil Engineering** Vol. 6, N° 2, 2008, pp. 221 - 227
DOI:10.2298/FUACE0802221Z

INFLUENCE OF EARTHQUAKES ON THE STRESS AND STRAIN STATE OF THE SHALLOW TUNNEL STRUCTURES IN SATURATED SOIL OF LOW BEARING CAPACITY

UDC 624.191:624.042.7(045)=111

Elefterija Zlatanović

Faculty of Civil Engineering and Architecture, University of Niš, Serbia
e-mail: elefterija2006@yahoo.com

Abstract. *The paper presents deformations of tunnel structures under the influence of earthquakes. The seismic impact on shallow laid tunnels in saturated soil of low bearing capacity was considered through the analysis of longitudinal and transversal loads. Stress and strain states were specially analyzed.*

Key words: *shallow-laid tunnels, seismic load, stress state, structural deformation.*

1. INTRODUCTION

Underground structures significantly differ from the majority of ground-level structures, by the characteristics in terms of reaction to the seismic influences, namely, they are completely dug-in the soil or rock, and as linear structures, they are characterized by their great length. Underground facilities built in seismically active areas should be designed to withstand both static and seismic loads. So, designing underground buildings resistant to seismic influence has certain aspects that are to a considerable extent different than those of aseismic designing of ground level structures.

In the analysis of seismic load on tunnel structures in the direction of horizontal axis (x) at a depth H in the medium of thickness Hc (which consists of a layer of saturated sand on limestone), the following displacement can be distinguished:

- Displacements in the horizontal plane xOy, which produce axial strain and load caused by bending;
- Displacements in the vertical plane yOz, which lead to distortion of the tunnel cross section.

In most cases it is assumed that the displacements to which the underground facility is exposed to, occur when the object is constructed in a in a free stress field. Interaction of a structure and the environment is not taken into account. This hypothesis assumes that the

Received November 15, 2008

underground facility (tunnel) is of less rigidity in respect to the environment in which it is installed. This hypothesis is questionable in our case, as well in the case of a tunnel in soft soil, therefore, in situations where vibrations periods are slightly higher. At any rate, the hypothesis is always on the side of safety, since structure rigidity (of the tunnel) is acting to reduce the displacement of the environment where it is constructed, which results in less structure strains.

Another hypothesis relates to the ignoring of the effect of inertia. This hypothesis has not been confirmed by testing on numerical models. There is not always continuity of displacement in the contact of the structure and the environment (in our case - the case of submerged tunnel, the continuity certainly exists). If no continuity of displacement on the contact of the structure –environment is assumed and if there is locally concentrated mass in the structure, the effect of inertia must be taken into account. The following analyses, do take into consideration any structure – environment interaction, or influence of the inertia, thus making it quasistatic analysis of the structure wherein the state of strain is analyzed.

2. Analysis of Longitudinal Load

In the horizontal plane *xOy* the following can be distinguished:

– Axial strain parallel to the axis *Ox*;

– Bending strain caused by the displacement components normal to the axis Ox.

2.1. Axial strain cause in the tunnel lining compressive and tensile stress propagating longitudinally by *Vp* velocity (primary, longitudinal seismic wave propagation velocity). Maximum axial strain (max ε_x) can be estimated by the Newmark formula, which relates to deformation in free field:

$$\max \varepsilon_x = \frac{\max Vx}{Vp} \tag{1}$$

where *Vx* is the velocity of vibration parallel to axis *Ox*.

A shear wave, of sine form, of frequency *N*, propagating by the velocity *Vs* (propagation speed of secondary, transversal seismic waves) parallel to the axis *Ox*, shapes the tunnel in the form of a sine, with the wavelength λ_x:

$$\lambda_x = \frac{Vs}{N} \tag{2}$$

2.2. Bending strain is a result of the bending moment and shear stress, where the following relations are valid:

$$M = Er \cdot Ir \frac{d^2 U_y(H)}{dx^2} \tag{3}$$

$$T = Er \cdot Ir \frac{d^3 U_y(H)}{dx^3} \tag{4}$$

Er – modulus of elasticity (Young's modulus) of the tunnel lining

Ir – moment of inertia of the tunnel lining in respect to the axis *Ox*

Uy(*H*) – displacement parallel to the axis *Oy* at the depth *H*, so:

$$M = -\frac{4\pi^2 \cdot Er \cdot Ir}{\lambda_x^{\;2}} \cdot \max Uy(H) \cdot \sin 2\pi \frac{\lambda}{\lambda_x} \tag{5}$$

$$T = -\frac{8\pi^3 \cdot Er \cdot Ir}{\lambda_x^{\;3}} \cdot \max Uy(H) \cdot \cos 2\pi \frac{\lambda}{\lambda_x} \tag{6}$$

Bending moment and shear stress are maximal for $\lambda = \left(\frac{1}{4} + \frac{k}{4}\right)\lambda_x$, so:

$$\max M = \frac{4\pi^2 \cdot Er \cdot Ir}{\lambda_x^{\;2}} \cdot \max Uy(H) \tag{7}$$

$$\max T = \frac{8\pi^3 \cdot Er \cdot Ir}{\lambda_x^{\;3}} \cdot \max Uy(H) \tag{8}$$

Also, in the function of the acceleration a_y (H) parallel to the axis Oy at the depth H there is:

$$\max M = \frac{Er \cdot Ir}{Vs^2} \cdot \max a_y(H) \tag{9}$$

$$\max T = \frac{2\pi N \cdot Er \cdot Ir}{Vs^3} \cdot \max a_y(H) \tag{10}$$

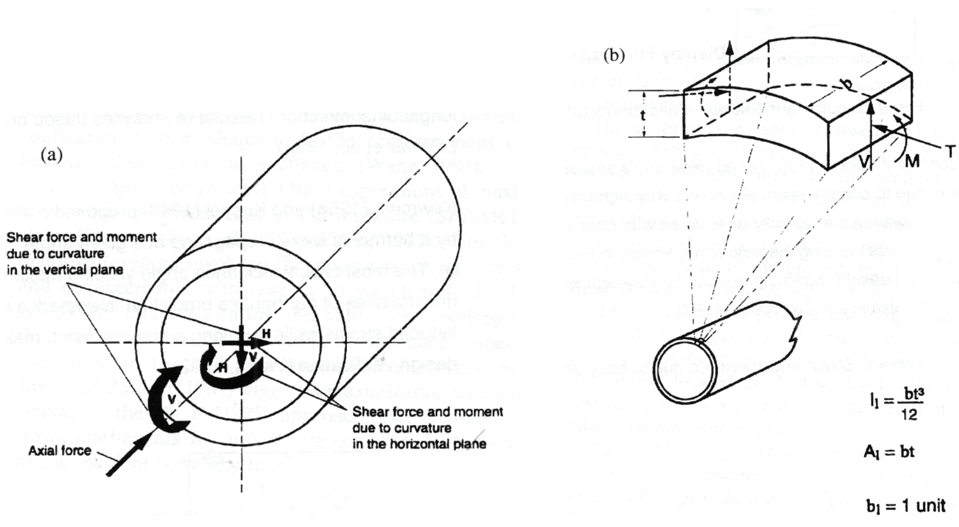

Fig. 1 Forces and moments caused by seismic waves

The paper considers a circular cross section tunnel, 8 m in diameter, with the concrete lining 0.40m thick (concrete elasticity modulus is $Er = 25000$ MPa), whose axis is at the depth of 25m, in the soil layer 50m thick over the rigid bed. Shear wave propagation velocity Vs is 100 m/s and the frequency is $N = 0.5$. The previous formulae for max $a_y(H) = 0{,}2$g result in: max $M = 914$ MNm/m and max $T = 58$ MN/m.

As is demonstrated by this example, it is often necessary to limit the bending moment installing the elastic joints at one quarter of the wavelength $\lambda_x/4$, which would mean, for the preceding example, at every 50m. Resistance of the elastic joints is now calculated by assuming they can withstand shear stresses equal to the shear stress at the corresponding part of the soil.

Kuesel suggests the following rule:

– if max $\varepsilon < 10^{-4}$, the structure is elastic;

– if max $\varepsilon > 10^{-4}$, joints for strain absorption should be designed.

3. Transversal Load Analysis

The following underground structure is considered: its horizontal axis is at the depth of 50m, in the soft soil stratum of Hc thickness. The loads in the plane *yOz* are a result of vertical propagation of shear wave, originating in the rigid bed. For the basic form N_1, the profile is displaced horizontally for $Uy(z)$ and has a form of a quarter of a sine curve (figure 2).

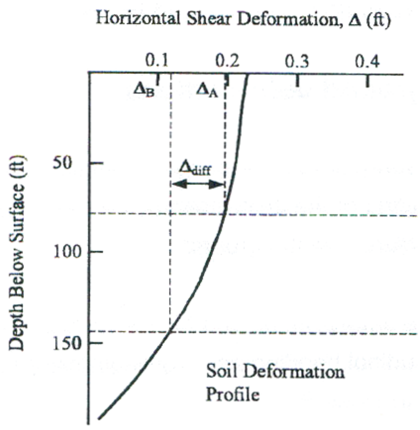

Fig. 2 Propagation of shear waves from rigid bed

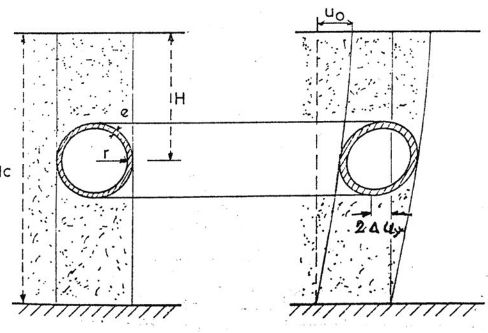

Fig. 3 Distortion of circular cross-section of the tunnel

This displacement creates a distortion in the cross-section of the tunnel. The difference between the horizontal displacement of the lowest point in the lining of the circular tunnel and the lowest point in the cross section of the circular tunnel is $2\Delta Uy$ (figure 3):

$$2\Delta U_y = \left(\frac{dU_y}{dz}\right)\cdot 2r \qquad (z = H) \tag{11}$$

Maximum value is:

$$\max \Delta U_y = \frac{2}{\pi}\cdot\frac{Hc}{Vs^2}\cdot a_0 \cdot r\sin\left(\frac{\pi}{2}\cdot\frac{H}{Hc}\right) \tag{12}$$

Load of the tunnel lining depends on its rigidity. If the tunnel lining is a thin circular ring of *e* thickness, the tunnel lining rigidity is expressed through the following two moduli:

– Compressive rigidity modulus

$$Ksc = \frac{Er \cdot e}{(1-\nu_r^{\ 2}) \cdot r} \tag{13}$$

– Bending rigidity modulus

$$Ksf = 9\frac{Er \cdot Ir}{(1-\nu_r^{\ 2}) \cdot r^3} \tag{14}$$

where ν_r is Poisson's coefficient of tunnel lining.

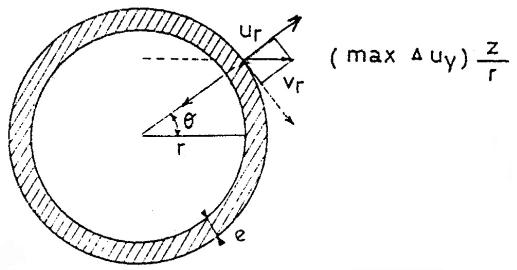

Fig. 4 Radial stresses σ_r and shear stresses $\tau_{r\theta}$ analysis

Radial stress σ_r and **shear stress $\tau_{r\Theta}$** (figure 4) act on the top surface of the ceiling and they can be determined with the aid for the following differential equations:

$$Ksc \cdot \left(\frac{d^2V_r}{d\theta^2} + \frac{dU_r}{d\theta} \right) = -r \cdot \tau_{r\theta} \tag{15}$$

$$Ksf \cdot \left(\frac{d^4U_r}{d\theta^4} + 2\frac{d^2U_r}{d\theta^2} + U_r \right) = r \cdot \sigma_r \tag{16}$$

where the radial and tangential components on the top surface of the ceiling *Ur* and *Vr* are presented by the terms:

$$Ur = -\frac{1}{2}\max(\Delta U_y) \cdot \sin 2\theta \tag{17}$$

$$Vr = \max(\Delta U_y) \cdot \sin^2 \theta \tag{18}$$

It should be emphasized that the stresses are positive (radial stress is positive if its direction is turned away from the ceiling), and that at the top of the tunnel the horizontal displacement *Uy* is linear.

Integration of the given differential equations yields the following values:

$$\sigma_r = \frac{1}{2}\frac{\max(\Delta U_y)}{r}(Ksc - Ksf) \tag{19}$$

$$\tau_{r\theta} = -\frac{\max(\Delta U_y)}{r}\cdot Ksc \tag{20}$$

Normal span *A* is obtained from the term:

$$A = \alpha \cdot r \cdot \sin 2\theta \tag{21}$$

Bending moment *M* is obtained from the term:

$$M = \beta \cdot r^2 \cdot \sin 2\theta \tag{22}$$

where:

$$\alpha = \frac{1}{2}(Ksc + \frac{1}{3}Ksf)\frac{\max(\Delta U_y)}{r} \tag{23}$$

$$\beta = \frac{1}{6}Ksf\frac{\max(\Delta U_y)}{r} \tag{24}$$

Normal stress and bending moment have maximum value for $\theta = \frac{\pi}{4} + k\pi$.

If the following example is observed, where $r = 4$m and $e = 0.4$m, $Er = 25000$ MPa and $\nu_r = 0.15$, the rigidity moduli are $Ksc = 2046$ MPa and $Ksf = 16$ MPa, so the result is:

$$\max M = 2{,}67\frac{\max(\Delta U_y)}{r}\text{, expressed in MNm/m} \tag{25}$$

$$\max A = 1025{,}5\frac{\max(\Delta U_y)}{r}\text{, expressed in MN/m}^2 \tag{26}$$

For the conditions $Hc = 50$m and $H = 25$m, $Vs = 100$m/s and for the maximum acceleration on the surface $a_0 = 0.5$g, $\max(\Delta Uy) = 1.1$cm is obtained.

4. CONCLUSION

Tunnel structure deformation decreases with the increase of the thickness of layer above the tunnel and the deep-laid tunnels are safer and less sensitive to earthquakes, as opposed to the shallow-laid tunnels.

Tunnel structures built in the soil are more prone to damage in respect to the structures which are in the bedrock.

From the analysis, it is clear that the loads increase in intensity with the decrease of the velocity of secondary seismic waves (shear waves), that is in cases when the soil is less compacted (tenuous).

Presence of water in the soil with low bearing capacity (soft soil) further aggravates soil response to seismic influence. Namely, when subjected to action of seismic waves, such deposits are prone to liquefaction (soil flow), which results in floating of the tunnel structure on the water saturated subsoil.

REFERENCES

1. Hashash Y., Hook J., Schmidt B., Yao C.: Seismic Design and Analysis of Underground Structures, Elsevier Science, 2001.
2. Lukić, D.: Prilog rešavanju problema seizmičke ugroženosti podzemnih objekata - dinamička teorija, Izgradnja, 1991, No. 11/91, pp. 15- 18, UDK: 624.035:550.34,
3. Lukić, D.: Postupak približnog rešavanja jednačina poprečnih oscilacija podzemnih linijskih objekata, Izgradnja, 1993, No. 9/93, pp. 18- 19, UDK: 624.035.001,
4. Lukić, D.: Dopunsko seizmičko sekundarno naponsko stanje, Prvi Jugoslovenski simpozij o tunelima, Brioni: Društvo za tunele Jugoslavije, 24-26 novembar, 1988, pp. 123- 128,
5. Lukić, D.: Prilog rešavanju problema seizmičkih kolebanja podzemnih objekata, Sedmi Jugoslovenski simpozij za mehaniku stene i podzemne radove, Beograd: Društvo za mehaniku stena i podzemne radove Jugoslavije, oktobar, 1989, pp. 129- 130,
6. Lukić, D.: Aseizmičko građenje u Beogradu, Kongres konstruktera, Vrnjačka Banja: Društvo konstruktera, 1-3 novembar, 2000, pp. 289- 293,
7. Anagnosti, P.,Lukić, D.: A seismic impact on underground structures, Međunarodni simpozijum o zemljotresnom inženjerstvu ISEE 99, Budva: Društvo za zemljotresno inženjerstvo Crne Gore, 1999, pp. 259- 266,
8. Anagnosti P.: Seizmički uticaji na podzemne konstrukcije, Zbornik radova sa savetovanja "Zemljotresno inženjerstvo i inženjerska seizmologija", Sokobanja, 2008.
9. Adme Z.: Analysis of NATM Tunnel Responses Due to Earthquake Loading in Various Soils
10. Kontogianni V., Stiros S.: Earthquakes and Seismic Faulting – Effects on Tunnels, Turkish Journal on Earth Sciences, 2003.
11. Parra J., Hackert C., Ghosh A.: Dynamic Response of a Fractured Tunnel to Seismic Waves
12. Bairaktaris D., Frondistou-Yannas S., Kalles D., Stathaki A.: Intelligent Monitoring of Seismic Damage in Reinforced Concrete Tunnel Linings
13. Newmark N., Rosenblueth E.: Osnovi zemljotresnog inženjerstva, Građevinska knjiga, Beograd, 1987.
14. Zlatanović E., Lukić D.: Seizmičko opterećenje na plitko položene podzemne objekte, Zbornik radova Građevinskog fakulteta, Subotica, 2007.
15. Web: www.sciencedirect.com

STANJE NAPONA I DEFORMACIJA USLED UTICAJA ZEMLJOTRESA NA PLITKO POLOŽENE TUNELSKE OBJEKTE U ZASIĆENOM SLABONOSIVOM TLU

Elefterija Zlatanović

U radu se prikazuju deformacije tunelskih konstrukcija usled delovanja zemljotresa. Predmet razmatranja bili su seizmički uticaji na plitko položene tunele u zasićenom tlu slabih karakteristika kroz analizu podužnih i transverzalnih opterećenja. Posebno je analizirano naponsko i deformacijsko stanje.