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Stability analysis of a circular tunnel underneath a fully liquefied soil layer

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ABSTRACT

The present problem deals with the examination of the effect of seismic forces on an unlined circular tunnel in a non-liquefiable general cohesive-frictional soil layer which is located underneath a completely liquefied soil layer. The numerical results are presented in terms of a non-dimensional stability number. The magnitudes of the stability number are determined as a function of thickness of the liquefied layer, tunnel embedment ratio, horizontal pseudo-static seismic forces, friction angle and dilation angle of the soil mass. To conduct this investigation, finite element based lower bound limit analysis is employed. The results are presented in the form of design charts. It is expected that the numerical results presented in this paper would be useful for the practicing engineers.

1. Introduction

Earthquake induced liquefaction can result in complete fluidization of soil, which can severely affect the stability of buried structures, such as, tunnels, pipelines etc. In some cases, the tunnel bearing stratum itself may liquefy, or a layer above the tunnel bearing stratum may shed its strength due to liquefaction, which finally result in to a significant loss in supporting strength of suspended soil mass above the tunnel. Hamada et al. (1996) reported wide scale destruction of lifeline facilities including tunnels from surficial liquefaction. Masaru (1997) described the devastation caused by ground liquefaction on structural members of a municipal subway system. Koseki et al. (1997) studied the effect of tunnel uplift resulting from buoyancy generated from liquefaction of the harboring soil. Chou et al. (2001) used in-situ data to predict possible liquefaction related damages on a shield tunnel and suggested few remedial measures. Pakbaz and Yareevand (2005) used analytical and finite difference methods to study mainly the effects of liquefaction and fault displacement on tunnels using a parameter named as flexibility ratio of the tunnel liner. Yuan et al. (2007) carried out three dimensional dynamic analyses on submarine shield tunnels located in a liquefied soil mass. Azadi and Hosseini (2010a) mainly measured stress states and surface settlements in soils near a tunnel subjected to uplift forces due to soil liquefaction. By using finite difference method, Azadi and Hosseini (2010b) studied large deformational behavior and pore pressure of soils enclosing tunnels resulted from liquefaction. Cilingir and Madabhushi (2011) used numerical and centrifuge modeling to study the effect of embedment depth of a tunnel placed in a liquefiable loose sand layer. Chian et al. (2014) studied the floatation response of a tunnel in a liquefied soil subjected to sinusoidal loading using numerical and centrifuge modeling. Zhuang et al. (2015) used numerical modeling technique to understand the effect of liquefaction on shallow tunnels.

It can be noted that the available research studies are mostly case specific. Also, there is hardly any study which provides detailed design charts to understand the effect of seismic forces on the stability of an unlined circular tunnel when a soil layer above the tunnel bearing stratum has completely liquefied. The present work deals with this problem. The lower bound finite element limit analysis in combination with linear optimization is used for carrying out the analysis. The stability of the tunnel is presented in terms of a non-dimensional stability number. Simple design charts, which are provided in this paper, present the stability number as a function of friction angle and dilation angle of the soil, thickness of the liquefied layer, tunnel embedment ratio and horizontal pseudo-static seismic forces. Note that the inclusion of pseudo-static seismic earthquake acceleration coefficient in conjunction with dilation angle and studying their effects on tunnel stability will help in a more realistic stability analysis of tunnel in a wide range of soils. It should be mentioned here that by using the bound theorems of plasticity, the effect of embedment depth and pseudo-static seismic forces on the stability of unlined circular tunnel in non-liquefied soil were studied by various researchers (Lyamin and Sloan 2000; Yamamoto et al. 2011; Sahoo and Kumar 2012; Chakraborty and Kumar 2013; Banerjee and Chakraborty 2016). Therefore, the results obtained from the present analysis are compared with the results available in literature on the stability of tunnels in absence of any liquefied soil layer (Yamamoto et al. 2011; Sahoo and Kumar 2012; Chakraborty and

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Nomenclature

List of symbols

$[C_1]$	global matrix of equality constraint defined in Eq. (8b)
$[C_2]$	global matrix of inequality constraint defined in Eq. (8c)
$\{a_1\}$	is the global vector defined in Eq. (8b)
$\{a_2\}$	is the global vector defined in Eq. (8c)
c	cohesion of the material in Layer II
H_c	depth of tunnel crown from the base of the liquefied layer
H_b	thickness of the liquefied layer
H	$H_c + H_b$;
E	total number of elements in the domain
D	diameter of the unlined tunnel
D_s	total number of discontinuities in the domain
k_h	horizontal seismic acceleration coefficient
L_H	vertical extent of material domain considered
L_L	horizontal extent of material domain considered
N	total number of nodes in the domain
N_t	total number of nodes along the tunnel lining and along the lower boundary of the liquefied layer
Y	global vector containing all unknown nodal stress and unit weight of the in-situ material
ϕ	angle of internal friction of the material in Layer II
α	angle of inclination of a side of an element with horizontal axis used to describe statically admissible discontinuity
β	angle of inclination of a side of an element with horizontal axis used to describe stress boundary conditions
γ_b	unit weight of the material for the liquefied layer (i.e., Layer I)
γ_c	unit weight of the material in which the tunnel is embedded (i.e., Layer II)
κ	dilative coefficient of the material in Layer II
ψ	dilation angle of the material in Layer II
σ_n	normal stress component normal to any surface
σ_x	normal stress component in x direction
σ_y	normal stress component in y direction
τ_{ns}	shear stress component tangential to any surface in general
τ_{xy}	shear stress component in x-y plane

Kumar 2013; Banerjee and Chakraborty 2016). The proximity of the stress state at any point in the problem domain with respect to the yield is also examined for a few cases.

2. Problem definition

By using the lower bound finite element limit analysis technique with the incorporation of pseudo-static horizontal seismic body forces, it is required to find out the effect of seismic forces on an unlined circular tunnel of diameter D when a soil layer (denoted as Layer I) of depth H_b and located above the tunnel bearing stratum has completely liquefied [refer Fig. 1(a)]. It is important to note that the mobilized shear strength for a liquefied layer of soil is extremely unreliable, hence it is considered as zero for the present problem. This should provide $\gamma H_c/c$ values which eliminate this uncertainty and estimate safer (conservative) design parameters. The embedment depth of the tunnel from the lower boundary of the liquefied layer (CD) is H_c [in Layer II, as indicated in Fig. 1(a)]. The schematic diagram for the problem is presented in Fig. 1(a). The unit weight of the fully saturated, liquefied soil layer is assumed as $\gamma_b = 20\,\text{kN/m}^3$ and reduced strength of this layer is assumed to be zero (as fully liquefied). Hence the load transferred on the lower layer is assumed to act as a uniformly distributed load ($\gamma_b \times H_b\,\text{kN/m}^2$). The unit weight of the soil stratum in which the tunnel is present and which is unsusceptible towards liquefaction is defined as a variable γ_c. The value of $k_h \times \gamma_c$ as a pseudo-static body force acts on the tunnel bearing stratum and represents the horizontal earthquake force. The soil mass is assumed to follow the Mohr-Coulomb yield criterion. Given that the Layer I is liquefied [see Fig. 1(a)], for the present problem γ_c is to be maximized for obtaining the maximum supported load on the tunnel before the soil stratum (i.e., Layer II) starts to yield as per the stress envelope provided by the Mohr-Coulomb yield surface. The effect of earthquake on the tunnel is expressed in terms of stability number ($\gamma_c H_c/c$). As the unit weight is maximized in the given problem, the $\gamma_c H_c/c$ obtained are the maximum admissible values; i.e., a higher design value of $\gamma_c H_c/c$ compared to that obtained from the present analysis indicates yielding of the material and vice-versa. The lower bound limit analysis with the consideration of both associated and non-associated flow rule (Palmer 1966; Drescher and Detournay 1993; Sloan 2013) is utilized for the present study. The value of dilative coefficient (κ), where $\kappa = \psi/\phi$ is varied from 0.0 to 1.0 for understanding the effect of non-associated flow rule on $\gamma_c H_c/c$; here ψ and ϕ indicate the dilation angle and friction angle of the material, respectively. The material in the Layer II is considered as homogeneous cohesive-frictional (c-ϕ) soil continuum without any directional anisotropy. The lower bound formulation developed by Sloan (1988) and γ maximization formulation developed by Chakraborty and Kumar (2013) are combined to construct the present numerical form. Note that due to the lower bound devising, the $\gamma_c H_c/c$ calculated will be a lower (i.e., conservative or safe) estimate of the true $\gamma_c H_c/c$.

It should be mentioned here that by knowing the values of γ_c from the charts provided in this paper, the maximum depth up to which an unsupported circular tunnel of a given diameter can withstand the overlying stresses due to overburden and seismic loading can be easily determined. After applying a certain factor of safety, a practicing engineer can then decide about the depth of embedment of the tunnel and the corresponding tunnel diameter. It should be noted here that that the assumption of an unlined tunnel is a simplification which helped in a more generalized study of the problem.

3. Mesh details and boundary conditions

The soil domain in which the tunnel is present and which is unsusceptible towards liquefaction (i.e., Layer II) is discretized with the help of three noded triangular elements. It should be mentioned here that the lower bound limit analysis is different from the displacement based numerical analysis where generally boundary conditions like roller, hinge or rigid support are assigned. In the lower bound limit analysis, the boundary conditions refer to the known stresses in the domain. The stress boundary conditions defining the problem are indicated in Fig. 1(a). Along the interface (CD) between Layer I and Layer II, the weight of the liquefied soil layer is introduced as a uniformly distributed load of normal stress acting in downward direction with a magnitude of $\gamma_b \times H_b\,\text{kN/m}^2$ [refer Fig. 1(a)]. The tunnel lining is assumed to be absent, hence the corresponding shear and normal stresses along the tunnel boundary would be zero ($\sigma_n = \tau_{ns} = 0$). Note that along (i) the horizontal line AB, and (ii) the vertical lines AD and BC of the domain, the states of stresses are controlled by the shear strength of the soil mass. Hence, no separate boundary conditions are required to be imposed along these boundaries.

The vertical extent ($L_H = 8D$ to $12D$) and the horizontal stretch ($L_L = 20D$ to $60D$) of the domain for the tunnel analysis are chosen in such a way that the effect of the domain size on the $\gamma_c H_c/c$ is absent on further extension of the domain size [refer Fig. 1(a)]. Adequate convergence study is carried out to finalize the finite element mesh for different physical conditions. The finite element mesh used for a specific case of the given problem with $H_c/D = 3$, $H_b/D = 1$, $\phi = 30^\circ$ and