SEISMIC-LIQUEFACTION ASSESSMENT METHODS IN CONJUNCTION WITH PORE PRESSURE DEVELOPMENT AND DISSIPATION

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ABSTRACT

The assessment of potential liquefaction on the performance of a given structure requires assessment of site-specific factors with a certain degree of accuracy. In sensitive or high-hazard structures, such as dams, extreme seismic events are to be considered (ANCOLD, 1998), such as those with a return period in excess of 30,000 years compared with typical return period of 452 years for buildings.

Although site-specific earthquake parameters may be assessed by a seismologist for bedrock conditions, the seismic wave may be either attenuated or amplified by the overlying soil profile to a dam foundation. Liquefaction may be generated by transient pore pressures as the soil particles try to re-arrange themselves in a denser packing. Given the characteristics of the foundation hydrogeology and nature of the earthquake, concurrent dissipation of the generated pore pressures may occur, thus mitigating the consequences. Despite the recognition of potential benefits, in most cases undrained conditions are assumed (ANCOLD, 1998).

The authors present a methodology suitable for practical applications. A seismic ground response analysis, based on total stress approach, will be carried out in conjunction with a transient seepage analysis in the time domain. The analysed soil profile consists of an 11 m thick layered deposit of saturated clay, silt and sand overlaying a siltstone. Effects of liquefaction in the upper sand layer, as well as pore pressure development and dissipation, will be assessed.

1 INTRODUCTION

Worldwide, fewer than 30 dams have failed completely during earthquakes [USCOLD, 2000]. These were primarily tailings or hydraulic fill dams, or relatively small embankments of questionable design. In most cases, the damage has occurred as a result of liquefaction.

Seismic liquefaction refers to a sudden loss in stiffness and strength of soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented or curtailed by the presence of water in the pores that cannot escape, it leads to a rise in pore water pressure and a resulting drop in effective stress. If the effective stress drops to zero (100% pore water pressure rise), the strength and stiffness also drop to zero, and the soil behaves as a heavy liquid. However, unless the soil is very loose, it will dilate and regain some stiffness and strength as it strains. If this strength is sufficient it will prevent a flow slide from occurring, but may still result in excessive displacements. When large displacements occur (cyclic mobility) at low effective (normal) stress, soil would dilate thus picking up some strength close to the undrained residual strength, which is the value used by some of the dam fraternity when a post-earthquake liquefaction assessment is required (ANCOLD, 1998). As outlined above, this ignores the possible dissipation / re-distribution of pore pressures and also what happens when only partial-liquefaction occurs (ie, undrained pore pressure is less than the effective pore pressure).

In addition, generated pore pressures and their redistribution could result in heaving / piping under high hydraulic gradient and internal erosion, with the loss of fine particles in clay gravel/sand (e.g., Sellmeijer (1988). If such failure mechanisms are initiated by the liquefaction-induced pore pressures, water-retaining structures such as dams may fail. Therefore, it is important to understand the dissipation of pore pressures and their seepage paths in addition to the generation of pore pressures under undrained conditions.

Except for several well-known cases, few dams have been tested by ground motion equivalent to their Design Basis Earthquake (USCOLD, 1999). Conversely, a few dams have experienced significant damage under moderate shaking, which demonstrates the importance of seismic assessment of dams.

This paper determines the ground response analysis of a complex foundation, based on linear analysis and the pore water generation and dissipation in the foundation due to earthquakes.

2 METHOD

Rather than adopting a generalised assessment method, the ground response analysis was done to determine the site-specific shear stress history at different layers of soil under a given earthquake.. The equivalent number of uniform cycles representing the irregular stress history was determined by the method proposed by Seed *et al.* (1975a). Then, the critical stress ratio (CSR) of different layers of soil was determined based on ground response analysis. The relation between CSR required causing liquefaction (i.e. cyclic resistance ratio, CRR) and number of uniform stress cycles provides the means to convert the irregular stress time series into an equivalent number of uniform stress cycles. Then the rate of pore pressure generation and dissipation in different layers can be determined by the method proposed by Seed *et al.* (1975b). This procedure is simple and the results are acceptable for practical purposes.

2.1 GROUND RESPONSE ANALYSIS

2.1.1 Introduction

A ground response analysis has been undertaken using this data at the bedrock level to determine the seismicity data (response spectrum and time histories) throughout the foundation strata and the ground surface, mainly the accelerations and shear stresses. The computer code “SHAKE2000-release 2009” has been employed for analysis purposes.

SHAKE is a one dimensional finite difference code capable of simulating the dynamic response of a system of homogeneous and visco-elastic layers of infinite extent (Schnabel *et al.,* 1972). The software computes the response of a semi-infinite horizontally layered soil deposit overlying a uniform half-space, when subjected to vertically propagating shear waves. The bottom layer is the half-space, and the source of the input based ground motion. Dynamic shear modulus or shear wave velocity, critical damping ratio, density and thickness are parameters which have to be specified for definition of each layer. Furthermore, the correlations of shear modulus and damping ratio versus shear strain should be defined for strain-dependent dynamic properties of each layer..

Since formulation of SHAKE2000 is based on total stress analysis, the pore pressure generation and development in the domain cannot be determined. Thus a simplified procedure has been employed, based on Seed and Booker (1977), to model the pore water pressure generation and development in the liquefiable materials. The pore pressure generation model is based on the assumption of undrained conditions with a particular initial effective stress, and as a result there is some uncoupling effect between the pore pressure generation and transient seepage. This would underestimate the beneficial effect of the pore pressure dissipation to some degree.

2.1.2 Relationship between strain and Shear Modulus reduction and damping ratio

Sensitivity analysis for different shear modulus reduction relations was carried out, and finally the shear modulus reduction relationships proposed by Sun *et al.* (1988) for clayey and sandy materials and Schnabel (1973) for rock were used for analysis (Figures 1).

The damping ratio versus shear strain relationships proposed by Seed and Idriss (1970) and Golesorkhi (1989) have been used for clayey and sandy layers, respectively (Figure 2).

2.2 PORE PRESSURE GENERATION AND DEVELOPMENT

2.2.1 Governing Equations

Darcy’s Law defines the governing equation for pore water pressure generation and dissipation throughout a granular material. The three dimensional form of this equation, considering that the change in bulk stress to be negligible is as follows:

$$\frac{\partial}{\partial x}(\frac{k_H}{\gamma_w}\frac{\partial u}{\partial x})+\frac{\partial}{\partial y}(\frac{k_H}{\gamma_w}\frac{\partial u}{\partial y})+\frac{\partial}{\partial z}(\frac{k_V}{\gamma_w}\frac{\partial u}{\partial z})=m_{v3}[\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}] \qquad (1)$$

where u is the excess k_H and k_V are coefficients of permeability in the horizontal and vertical directions respectively, m_{v3} is the coefficient of volume compressibility, γ_w is the unit weight of water, dt is the interval of time, dN is the number of uniform cycles of alternating shear stress, u_g is the pore pressure generated by the alternating shear stresses for the appropriate conditions of prior strain history.

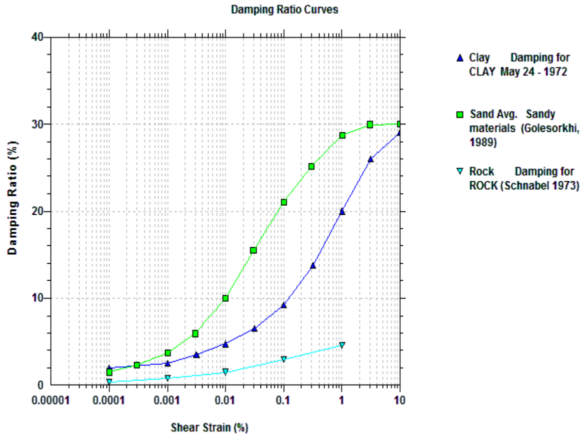

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Figure 1: Shear modulus reduction.

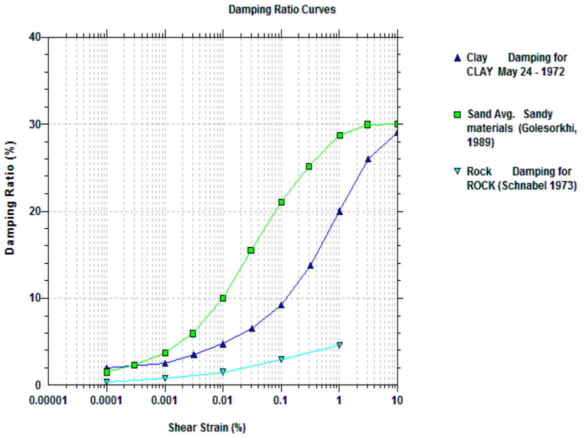


Figure 2: Dampening ratio.

For purely vertical drainage, Equation (1) becomes:

$$\frac{\partial}{\partial z}\left(\frac{k_V}{\gamma_w}\frac{\partial u}{\partial z}\right) = m_{v3}\left[\frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}\right] \qquad (2)$$

or

$$\frac{\partial u}{\partial t} = \frac{\partial u_g}{\partial t} + \frac{1}{m_{v3}}\left[\frac{\partial}{\partial z}\left(\frac{k_V}{\gamma_w}\frac{\partial u}{\partial z}\right)\right] \qquad (3)$$

The first term on right hand side of the Equation (3) determines the rate of pore water pressure generation due to shaking and the second term reveals the rate of pore water pressure dissipation due to drainage through the soil domain. The method proposed by Seed *et al.* (1975b) is used to determine the first term. In this regard, if the excess pore pressure ratio, r_u, is expressed as the ratio of the peak excess pore water pressure generation by the earthquake u to the initial effective overburden pressure, σ'_{V0}, and the cycle ratio r_N as the ratio of the number of applied stress cycles N to the accumulative number of cycles required to cause liquefaction, N_l, then the following equation can be used to relate pore pressure generation to cyclic ratio:

$$r_N = \left[\frac{1}{2}(1 - \cos \pi\, r_u)\right]^{\alpha} \qquad (4)$$

where α is a function of soil properties and test conditions, with a typical value of 0.7 for sands. By inverting Equation (4), the pore pressure ratio r_u maybe expressed in term of the cycle ratio r_N by the relationship:

$$r_u = \frac{1}{2} + \frac{1}{\pi}\arcsin\left(2r_N^{1/\alpha} - 1\right) \qquad (5)$$

Over a small increment Δt, the term $q = \partial u_g / \partial t$ in Equation (3) can be approximated by the expression:

$$q = \frac{u_{gl} - u_0}{\Delta t} \qquad (6)$$

in which u_0 is the excess pore pressure at time t_0 and u_{gl} is the excess pore pressure that would develop at time $t_l = t_0 + \Delta t$ by shaking, without considering any drainage.

It is possible to evaluate u_{gl} by the mentioned procedure. Indeed, at time $t = t_i$ when the pore pressure ratio is $r_u = u/\sigma'_{V0}$, the value of cyclic ratio, $r_{N_i} = N_i / N_l$ (N_i is the number of applied cycles until time t_i), can be determined from Equation (4). Then at time $t = t_{i+1}$, the value of the cyclic ratio if the system were undrained would be:

$$r_{N_{i+1}} = r_{N_i} + \Delta r_N \qquad (7)$$

where Δr_N is the increase in cyclic ratio for undrained conditions over the time increment Δt and is defined by:

$$\Delta r_N = \frac{\Delta t}{t_d} \cdot \frac{N_{eq}}{N_l} \qquad (8)$$

in which t_d is the duration of strong earthquake shaking, N_{eq} is the number of uniform stress cycles equivalent to earthquake and N_l is the number of cycles required to cause liquefaction under undrained condition.

By calculating Δr_N from Equation (8) and $r_{N_{i+1}}$ from Equation (7), r_u can be determined from Equation (5) and the u_{gl} can be determined from:

$$u_{gl} = r_u\, \sigma'_{V0} - u \qquad (9)$$

A numerical solution by finite element or finite difference methods can be used for discretisation of the problem. For an explicit finite difference approximation, the solution would be stable if the following condition was satisfied (Remson *et al.* 1971):

$$\frac{k_V}{\gamma_w m_{v3}} \frac{\Delta t}{(\Delta z)^2} \leq 0.5 \tag{10}$$

However, an unconditionally-stable finite difference discretisation can be used, similar to that adopted for complex heat generation / transfer analysis (Vitharana, 1995).

3 SAMPLE

3.1 INPUT DATA

As an example of the suggested approach, the case shown in Figure 3 has been analysed. In this case, a 15.0 m soil profile comprised of 5 different layers was modelled. The assumed properties of each layer are given on Table 1.

Table 1: Assumed properties for soil layers

Soil Layer	Depth, Z (m)	Shear wave velocity (m/s)	Permeability (m/s)	Compressibility (m^2/kN)
Clay1	0.0 to 2.0	100	1.0e-7	4.2e-5
Sand1	2.0 to 5.0	220	1.5e-4	3.1e-5
Clay2	5.0 to 7.0	230	1.e-7	4.2e-5
Sand2	7.0 to 11.0	160	1.e-3	3.1e-5
Clay3	11.0 to 15.0	250	1.0e-7	4.2e-5
Bedrock	> 15.0	1500	---	---

The component H1 of Mammoth Lakes earthquake with scale factor of 1.0 was used for ground response analysis (Figure 4).

3.2 RESULTS OF GROUND RESPONSE ANALYSIS

The ground response analysis for this soil profile was done using SHAKE2000. The results consist of distribution of CSR in depth, and shear stress and acceleration time histories in pre-defined location in soil profile.

Figures 5 and 6 show the acceleration time histories in Sand1 and Sand2 layers, respectively. The duration of strong earthquake shaking, t_d, can be determined from Figures 4-6. Here, the strong motion defines an acceleration of more than 35% of peak acceleration of time history. Therefore, t_d is assumed as 15 seconds for component H1 of the Mammoth Lakes earthquake.

Figures 7, 8 and 9 show the shear stress time histories at Sand1 and Sand2 Layers and bedrock, respectively. The number of uniform stress cycles equivalent to earthquake at stress level of $0.65\tau_{max}$, N_{eq}, can be determined based on these figures and the proposed method by Seed *et al.* (1975a), where τ_{max} is the maximum calculated shear stress generated in a given layer. These numbers are equal to 3.5, 2.8 and 4.4 for Sand1 and Sand2 layers and bedrock, respectively.

The other important parameter in evaluation of pore water pressure is the number of cycles required to cause liquefaction when the stress level is equal to $0.65\,\tau_{max}$, i.e. N_l. This parameter can be obtained from cyclic shear or cyclic triaxial laboratory tests. In this paper, the relation between CSR and N_l provided by Seed *et al.* (1975a) was used to determine $N_{l,}$ as shown in Figure 11. The CSR can be determined by ground response analysis or by simplified procedures (Seed and Idriss, 1971), (Idriss, 1999). Figure 10 shows the results of CSR at different depths of soil profile prepared by SHAKE 2000. It is assumed that the amounts of CRR for both sandy layers are equal or less than CSR, i.e. both sandy layers are potentially liquefiable.

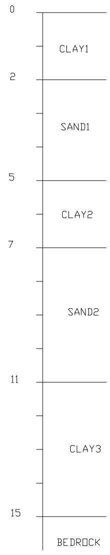


Figure 3: Assumed soil profile

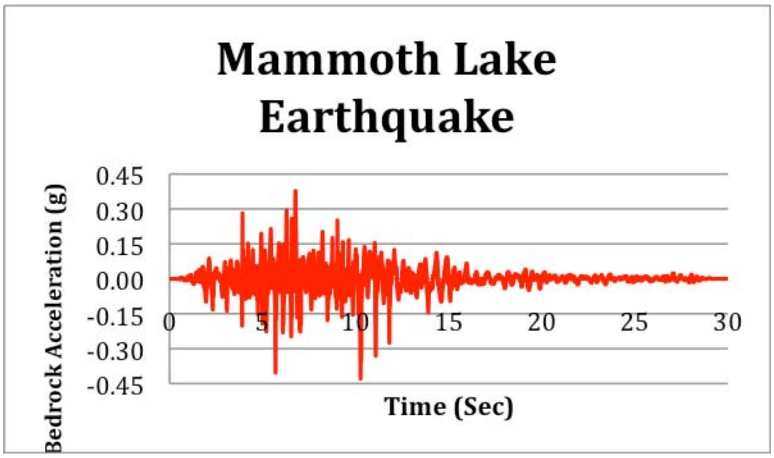


Figure 4: Time history of component H1 of Mammoth Lakes Earthquake at bedrock (scale factor =1)

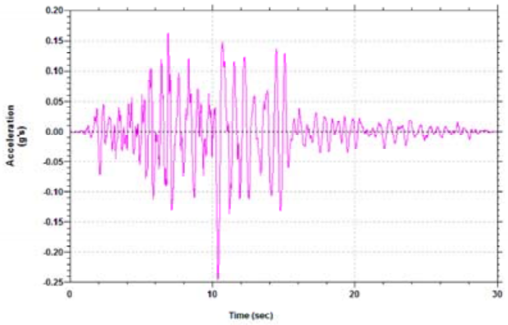


Figure 5: Acceleration time history on SAND1 layer

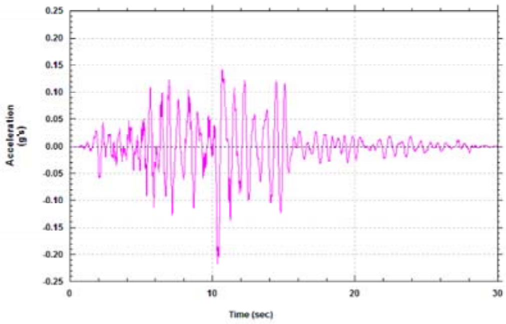
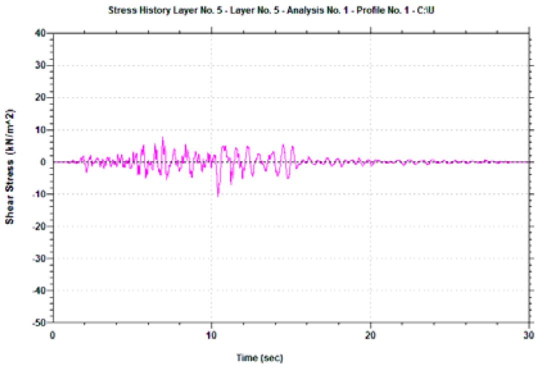


Figure 6: Acceleration time history on SAND2 layer



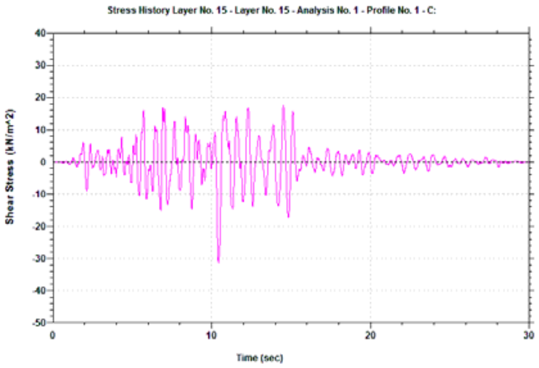


Figure 7: Shear stress time history on the SAND1 layer. Figure 8: Stress time history at SAND2 layer.

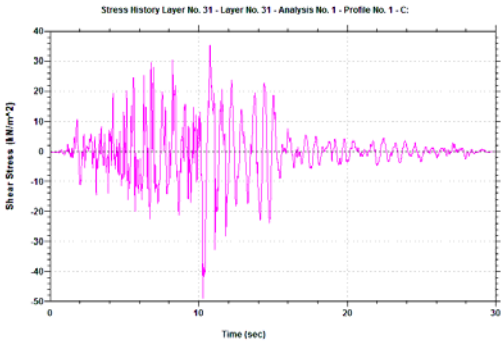


Figure 9: Shear stress time history on the bedrock.

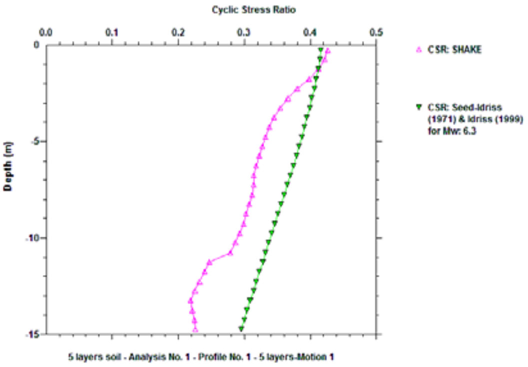


Figure 10: CSR in different depths of the soil profile.

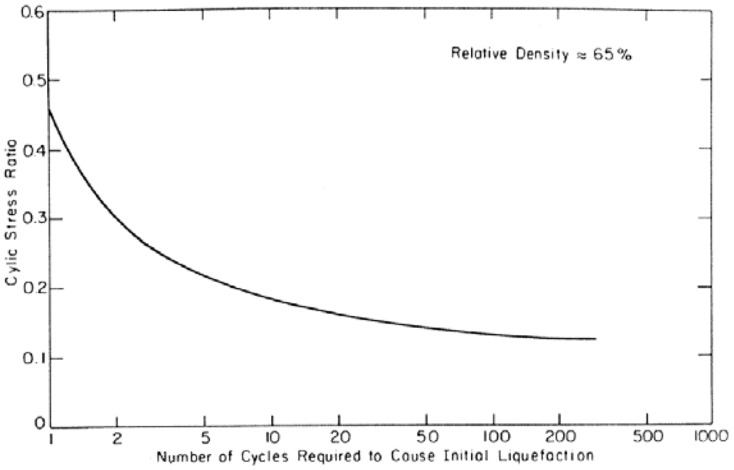


Figure 11: Representative curve for relationship between Cyclic stress ratio to cause liquefaction versus number of uniform loading cycles (Seed *et al,* 1975a)

Based on Figure 10, the average amount of CSR for upper and lower sandy layer are 0.36 and 0.30 according to results of SHAKE. Therefore, the amounts of N_l for these two layers are 1.5 and 2, respectively based on Figure 11.

3.3 RESULTS OF PORE PRESSURE GENERATION AND DEVELOPMENT

A computer code was prepared for calculations of pore pressure generation and development based on formulation discussed on Section 2.2.1.

The pore pressure generation and development of the mentioned soil profile were modelled. Two cases were considered for the analysis. Case 1 models the normal condition of the ground during and after earthquake, whereas Case 2 models the ground when the first layer is replaced with a drain material with k=1.0e-2 m/s.

The results of analysis are shown in Figures 12 and 13. Figure 12 shows that in the existing condition the pore pressure in both sandy layers is increased during the earthquake to its maximum value, i.e. $r_u=1$. After the earthquake, the dissipation of pore pressure can be seen.

Figure 13 shows the condition in which the upper clay layer is replaced with the drain material. Therefore, the upper sandy layer can be drained from the top. As shown in Figure 13, this method can be effective for the upper sandy layer but it does not affect the lower sandy layer.

Build-up of pore pressure is associated with a decrease in soil stiffness and confining pressure, and hence an increase in strain. The progressive stiffness degradation of the soil may be evaluated from the mentioned pore pressure analysis, by assuming that the shear modulus of sands at low strains is proportional to the square root of the existing confining pressure σ_v' (Poulos, 1988), as follows:

$$\frac{\text{G at time t}}{\text{G at time 0}} \approx \left(\frac{\sigma_v' \text{ at time t}}{\sigma_v' \text{ at time 0}}\right)^{0.5} \tag{11}$$

Which means that the sandy soil can lose all of its strength when $r_u=1$. Also, the shear modulus is an influential parameter in ground response analysis, reduction of this parameter can reduce the ground response.

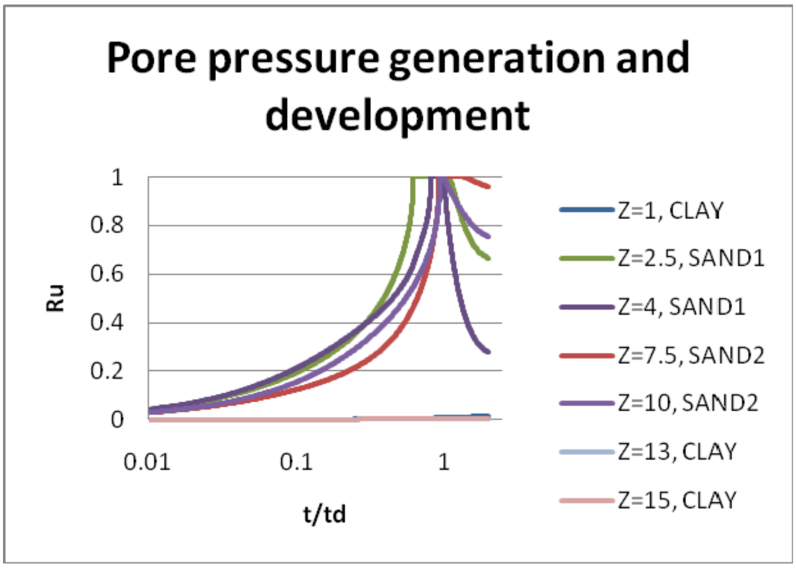


Figure 12- Case 1 - pore pressure generation condition existing condition

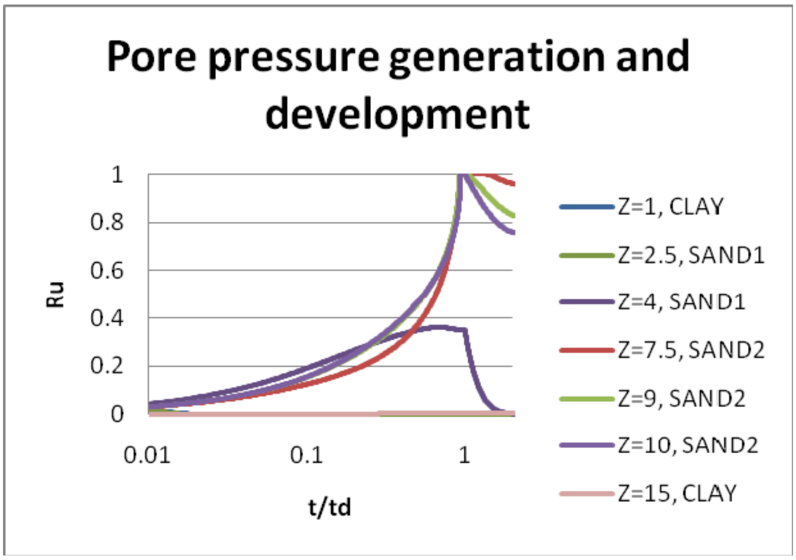


Figure 13: Case 2 - pore pressure generation condition with drain.

4 CONCLUSIONS

- The ground response of a dam site is presented using SHAKE 2000 and pore water pressure generation based on Seed *et al.* (1975a, b). Subsequently, the generation and concurrent dissipation of pore water pressure during an earthquake was modelled using a transient seepage analysis.
- An assessment of the concurrent dissipation of pore pressure is important, as the resulting beneficial effects can be considered in the design process. The traditional method recommended by some design practices for determining liquefaction-induced instability, with undrained residual shear strength parameters, can be conservative in cases where concurrent seepage can occur. This also ignores the other unfavourable situations such as heaving and piping, which are critical for structures such as earth dams on sandy foundations.
- The methodology presented here is simple and useful for practical applications. Currently, parametric studies are being undertaken for typical Australian conditions.

5 DISCLAIMER

The information and findings contained in this paper represent the views of the Authors and not necessarily those of Sinclair Knight Merz.

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