EARTHQUAKE ENGINEERING RESEARCH CENTER

STABILIZATION OF POTENTIALLY LIQUEFIABLE SAND DEPOSITS USING GRAVEL DRAIN SYSTEMS

by

H. Bolton Seed
John R. Booker

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Stabilization of Potentially Liquefiable Sand Deposits
Using Gravel Drain Systems

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H. Bolton Seed,¹ and John R. Booker²

INTRODUCTION

In recent years substantial gains have been made in the understanding of the phenomenon of liquefaction of saturated granular materials. It is now widely accepted that if a saturated granular material is subjected to cyclic loading involving the reversal of shear stresses it will tend to compact, and that if the material is unable to drain, this tendency to decrease in volume will lead to an increase in pore water pressure. Ultimately if the cyclic loading is maintained the soil will reach a condition of zero effective stress and, depending on its relative density, will suffer essentially a complete loss of strength (liquefaction) or undergo some degree of strain with little or no resistance to deformation (initial liquefaction with limited strain potential).

Considerable progress has been made in the development of both tests and test procedures to obtain quantitative measures of the stress conditions which lead to these types of soil liquefaction. This development has been accompanied by an associated development of methods of analysis (Seed and Idriss, 1967, 1971) which make use of the test results to evaluate the liquefaction potential of soil deposits in the field, and the methods have been found to provide a useful basis for assessing probable site performance under prescribed earthquake conditions.

¹Prof. of Civil Engrg., University of California, Berkeley, Calif.
²Sr. Lecturer, Dept. of Civil Engrg., University of Sydney, Australia, currently Visiting Scholar, Dept. of Civil Engrg., University of California, Berkeley, Calif.
Much of the work referred to in the preceding paragraph has been restricted to soil behavior under undrained conditions and finds its application in situations in which the redistribution and dissipation of pore water pressure do not have a significant influence on the liquefaction potential of the soil mass. It has been recognized, however, that such mechanisms may be of considerable importance and may have both adverse and beneficial effects. For example, the dissipation of pore water pressures generated in deep soil layers may lead to upward seepage which results in liquefaction of surface layers (Seed and Lee, 1966; Ambraseys and Sarma, 1969; Yoshimi, Yoshiaki and Kuwabara, 1973 and Seed, Martin and Lysmer, 1975). On the other hand, if the pore water pressures generated in a soil mass by cyclic loading can to some extent be dissipated as they are created, then the danger of liquefaction may be averted, Seed et al (1975). In fact it is thought that the better field performance of gravels over sands may be directly attributable to their capacity to dissipate pore water pressures because of their higher permeability (Wong, Seed, and Chan, 1975).

A possible method of stabilizing a soil deposit susceptible to liquefaction is to install a system of gravel or rock drains as shown in Fig. 1(a) so that pore water pressures generated by cyclic loading may be dissipated almost as fast as they are generated. In this paper the one dimensional theory of pore water pressure generation and dissipation developed by Seed, et al (1975) is generalised to three dimensions and applied to the analysis of columnar gravel drains under a variety of earthquake conditions. The results of these analyses are summarised as a series of charts which provide a convenient basis for design considerations.
Fig 1(a) ARRANGEMENT OF GRAVEL DRAIN SYSTEM

Fig 1(b) GRAVEL DRAIN WITH RADIAL DRAINAGE ONLY
BASIC EQUATIONS

In developing the basic equations governing the generation and dissipation of pore water pressure throughout a granular material, it will be assumed that the flow of the pore water is governed by Darcy's Law so that the usual considerations of continuity of flow lead to the equation:

\[
\frac{\partial}{\partial x} \left( \frac{k_H}{\gamma_w} \frac{\partial u}{\partial x} \right) + \frac{\partial}{\partial y} \left( \frac{k_H}{\gamma_w} \frac{\partial u}{\partial y} \right) + \frac{\partial}{\partial z} \left( \frac{k_V}{\gamma_w} \frac{\partial u}{\partial z} \right) = \frac{\partial \varepsilon}{\partial t} .
\]  

(1)

where \( u \) is the excess hydrostatic pore water pressure

\( k_H, k_V \) are coefficients of permeability in the vertical and horizontal directions

\( \gamma_w \) is the unit weight of water

and \( \varepsilon \) is the volume strain, with volumetric reduction being considered positive.

During an interval of time \( dt \) the pore water pressure in an element of soil will undergo a change \( du \), while the element will also be subjected to \( dN \) cycles of alternating shear stress which will cause an additional increase in pore pressure \( \frac{\partial u_g}{\partial N} \cdot dN \), where \( u_g \) is the pore pressure generated by the alternating shear stresses for the appropriate conditions of prior strain history. It therefore follows, considering that the change in bulk stress is negligible, that the volume change \( d\varepsilon \) of the element in time \( dt \) is given by

\[
d\varepsilon = m_{v3} (du - \frac{\partial u_g}{\partial N} \cdot dN)
\]

(2a)

where \( m_{v3} \) is the coefficient of volume compressibility

i.e. \( \frac{\partial \varepsilon}{\partial t} = m_{v3} \left( \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial N} \cdot \frac{\partial N}{\partial t} \right) \)

(2b)
Combining Eqns. (1) and (2b) it is found that:

\[
\frac{\partial}{\partial x} \left( \frac{k_H}{\gamma w} \frac{\partial}{\partial x} \right) + \frac{\partial}{\partial y} \left( \frac{k_H}{\gamma w} \frac{\partial u}{\partial y} \right) + \frac{\partial}{\partial z} \left( \frac{k_H}{\gamma w} \frac{\partial u}{\partial z} \right) = \left[ \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t} \right]
\]

(3)

If the coefficients of permeability are constant and the problem exhibits radial symmetry, Eqn. (3) becomes

\[
\frac{k_H}{\gamma w m v^3} \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} + \frac{k_H}{\gamma w m v^3} \frac{\partial^2 u}{\partial \theta^2} = \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t}
\]

(4)

and for purely vertical drainage reduces to the form developed by Seed et al (1975).

\[
\frac{k_H}{\gamma w m v^3} \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t}
\]

(5)

Under conditions of purely radial drainage as considered in the following section, Eqn. (4) reduces to

\[
\frac{k_H}{\gamma w m v^3} \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} = \frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t}
\]

(6)

In order to evaluate the extent of pore pressure generation and dissipation using this equation, it is necessary to determine \( \partial u_g/\partial N \) and \( \partial N/\partial t \) as well as the soil properties \( k_H \) and \( m v^3 \). The values of \( \partial u_g/\partial N \) can be found from undrained tests as described by Seed et al (1975). For many soils the relationship between \( u_g \) and \( N \) can be expressed for practical purposes in terms of the number of cycles \( N_{\text{L}} \) required to cause initial liquefaction under the given stress conditions in the form (Seed et al, 1975):

\[
\frac{u_g}{\sigma_o'} = \frac{2}{\pi} \arcsin \left( \frac{N}{N_{\text{L}}} \right)^{1/2}\alpha
\]

(7)

where \( \sigma_o' \) is the initial mean bulk effective stress for triaxial test conditions or the initial vertical effective stress for simple shear conditions and \( \alpha \) is an empirical constant which has a typical value of 0.7. (see Fig. 2)
Fig. 2 RATE OF PORE WATER PRESSURE BUILD-UP IN CYCLIC SIMPLE SHEAR TESTS
(after De Alba et al.)
Thus,

$$\frac{\partial u}{\partial N} = \frac{2\sigma'_o}{\alpha^2 N} \frac{1}{\sin^2 \alpha - \left(\frac{\pi}{2} r_u\right) \cos \left(\frac{\pi}{2} r_u\right)}$$

(8)

where $r_u = \frac{u}{\sigma'_o}$ is the pore pressure ratio.

For practical purposes the irregular cyclic loading induced by an earthquake may be converted to an equivalent number $N_{eq}$ of uniform stress cycles at a stress ratio $\frac{\sigma'_h}{\sigma'_o}$ occurring in some duration of time $t_d$ of earthquake shaking (Seed, Idriss, Makdisi, Banerjee, 1975). Thus

$$\frac{\partial N}{\partial t} = \frac{N_{eq}}{t_d}$$

(9)

In using these results it must be noted that the rate of pore pressure generation $\frac{\partial u}{\partial N}$ depends on the previous cyclic history of the soil and this may be represented approximately by the accumulated pore pressure $u$. Thus for any given point at time $t_1$, the appropriate rate of pore pressure generation $(\frac{\partial u}{\partial N})_{t_1}$ must be determined from Eqn. (8) corresponding to the value of $u$ existing in the soil at that time. By this means the past history of strain cycles may be taken into account with a reasonable degree of accuracy.

**ANALYSIS OF ROCK DRAINS**

As discussed previously, in cases of high liquefaction potential the installation of columnar gravel drains may well provide an efficient method for preventing the development of excessively high pore water pressures. In most practical cases for example the horizontal permeability of a sand will be several times greater than its vertical permeability and the spacing between vertical drains can be made less than the distance required for water to drain vertically to a free surface. Furthermore, many natural
deposits of sand are interspersed with narrow horizontal layers of relatively impermeable silt which may severely inhibit vertical drainage. For these reasons it seems quite likely that the dominant mechanism in the operation of a gravel drain system will often be one of pure horizontal drainage.

Consider, therefore, a network of rock drains as shown in Fig. 1(b) of diameter $2a$ and an effective spacing $2b$ installed in a layer of sand with horizontal permeability, $k$, an initial effective stress $\sigma'_0$ and having characteristics such that it would liquefy after $N_\text{eq}$ uniform stress cycles of magnitude $\gamma_{eq}$ if it were undrained. Suppose also that the layer is subjected to an earthquake consisting of $N_{eq}$ uniform stress cycles of the same magnitude applied over a period of time $t_d$. It will be assumed that the pore pressure generating characteristics of the sand are described by Eqn. 9 and that the filler material in the drains is far more permeable than the surrounding sand layer, so that the excess pore water pressure in the drain is effectively zero. It will also be assumed that the coefficient of compressibility is constant. Examination of experimental data (Lee and Albaiso 1974) shows that this is nearly so for moderate pore pressure ratios and thus if the sand drains are performing their function, this assumption is justified.

Under these assumptions of purely radial flow, the pore pressure ratio, $r_u = u/\sigma'_0$, throughout the sand and drain system depends on the following dimensionless parameters:

$$a/b$$ = a ratio characterizing the geometric configuration of the sand drains

$$\frac{N_{eq}}{N}$$ = a ratio characterizing the severity of the earthquake shaking, in relation to the liquefaction characteristics of
the sand,

\[ T_{bd} = k \cdot \frac{t_d}{\gamma_w m v_3 b^2} \]

relating the duration of the earthquake to the consolidation properties of the sand,

and \( \alpha \) is a parameter characterizing the shape of the pore pressure generation curve, Eqn. 7. (It is found that for many materials \( \alpha = 0.7 \) fits the experimental data well and this value will be adopted throughout this paper).

A finite element program LARF (Liquefaction Analysis for Radial Flow) has been written to solve Eqn. 3 for purely radial flow (see Appendix A).

Before examining the behavior of a gravel drain system it is instructive to consider the behavior of a sand layer when no drain is present. Since only radial drainage is considered and no vertical drainage may occur, the layer will act in an undrained manner. There are two cases to consider. First if the number of equivalent uniform stress cycles induced by the earthquake, \( N_{\text{eq}} \), is less than that required to cause liquefaction, \( N_L \), the excess pore water pressure will rise according to Eqn. 7, until a final value (which remains constant thereafter) is reached when \( t = t_d \) and \( N = N_{\text{eq}} \). Of more practical interest is the case when \( N_{\text{eq}} > N_L \). For this case the excess pore water pressure will rise according to Eqn. 7 until it reaches the value \( \sigma'_{\circ} \) when \( N = N_L \) and \( t = t_u \), where

\[ t_u = \frac{N_L}{N_{\text{eq}}} \cdot t_d \]  

(10)

the sand then developing a condition of initial liquefaction and no further increase in pore pressure being possible.

In order to illustrate the effect of a system of gravel drains, the
Fig. 3 RELATIONSHIP BETWEEN MAXIMUM PORE PRESSURE RATIO AND TIME
case of a sand layer with drains spaced at \(a/b = 0.2\) will be examined in some detail for an earthquake condition where \(N_L\) is equal to one half of \(N_{\text{eq}}\).

In Fig. 3 the maximum pore pressure ratio \(r_{\text{max}}(t) = \text{maximum value of } u/\sigma_0'\) throughout the layer at time \(t\) is plotted against \(t/t_d\). If the sand had zero permeability so that, \(T_{bd} = 0\), undrained conditions would prevail and the pore pressure ratio would rise to a value of 1 at \(t = t_u = 1/2 t_d\) (for this case); the entire layer would then develop a condition of initial liquefaction at this instant and remain in this condition thereafter since no dissipation could occur. For a sand having a relatively low permeability coefficient, say for example, a value corresponding to \(T_{bd} = 0.2\), the maximum pore pressure ratio rises approximately as it would in the undrained case and initial liquefaction will develop at some time between \(t = t_u\) and \(t = t_d\). The liquefied zone then continues to grow until the end of strong shaking. After this no further excess pore water pressures are generated, the pore pressures that have built up dissipate and the liquefied zone contracts and finally vanishes whereupon the maximum pore pressure ratio drops steadily from the value one down to zero. If the sand had a still greater permeability, corresponding to say \(T_{bd} = 1.25\), the pore pressures build up during the period of strong shaking but the soil does not develop a condition of initial liquefaction although it can be seen that if the earthquake were maintained beyond \(t_d\) liquefaction would eventually occur. On cessation of strong shaking the excess pore pressures dissipate and the pore pressure ratio drops from its greatest value \(r_g\) back to zero. For a still higher permeability corresponding to \(T_{bd} = 5.0\), the pore pressure increases initially but then tends to level off as a stage is reached where the rate of dissipation of pore pressures is almost equal to the rate of their generation.
Similar observations to those described in the previous paragraph hold for a material for which \( \frac{N_{eq}}{N_{eq}} = 5 \). This case can be regarded as similar in every respect to the one described previously except that the intensity of the earthquake is increased. This implies that excess pore pressures are generated at a faster rate and thus it would be expected that the permeability necessary to limit the pore pressure ratio to a specified greatest value \( r_g \) would be increased. This is illustrated by comparison of the results shown in Figs. 3 and 4.

The effect of changing the diameter of the gravel drain is illustrated by considering a material for which \( \frac{N_{eq}}{N_{eq}} = 2 \) for a range of values of \( a/b \) but with a constant value of \( T_{bd} = 1 \). Computed results for this case are shown in Fig. 5. If no drains are present \( a/b = 0 \), there are no drainage boundaries and so the layer behaves in an undrained fashion and liquefies at \( t = t_u = \frac{1}{2} t_d \) and remains liquefied thereafter. For a relatively small drain corresponding to \( a/b = 0.1 \), initial liquefaction is deferred for a period of time but eventually occurs, the liquefied region continuing to grow until the end of the period of strong shaking; thereafter it shrinks and vanishes as a result of pore water pressure dissipation. If a larger diameter drain is introduced corresponding to \( a/b = 0.25 \), initial liquefaction of any part of the layer may be prevented entirely and the greatest pore pressure ratio will be less than one. Note that the diameter of the drain in this region of values of \( a/b \) is quite critical and the greatest value of pore pressure ratio \( r_g \) developed changes rapidly with \( a/b \). Thus, for example, there is a far greater decrease of \( r_g \) when \( a/b \) changes from 0.2 to 0.25 than there is when \( a/b \) changes from 0.25 to 0.3.

In designing a network of gravel drains to prevent liquefaction, it would be helpful to know for a given soil and a given diameter of drain
$N_{eq}/N_2 = 5$

$b/a = 5$

$T_{bd} = \frac{t_d}{\gamma_w \cdot m_3 b^2}$
Fig. 5 EFFECT OF DRAIN DIAMETER AND DRAIN SPACING ON MAXIMUM PORE PRESSURE RATIO

\[ N_{eq}/N_y = 2 \]
\[ T_{bd} = \frac{k}{\gamma_w} \]
\[ = 1.0 \]
what spacing of drains should be chosen to limit the pore pressure ratio to a given greatest value \( r_g \). To facilitate this choice, a series of curves Figs. 6 to 9 have been computed which show the variation of the greatest pore pressure ratio \( r \) developed as a function of the spacing ratio \( a/b \) for values of \( N_{eq}/N_k \) equal to 1, 2, 3, 4, and for a range of values of the parameter \( T_{ad} = \frac{k \cdot t_d}{\gamma_w \cdot m_3 a^2} \). For any particular soil and a given diameter of sand drain, \( N_{eq}/N_k \) and \( T_{eq} \) will be known and thus the value of \( a/b \) corresponding to a given allowable value of \( r_g \) can be read directly from the curves. A similar series of plots showing the greatest average pore pressure ratio in the soil, \( (r) \), are shown in Figs. 10-13.

Example

Suppose, for example, that a soil layer having the properties

\[
k = 10^{-3} \text{ m/sec}
\]

and

\[
m_3 = 2 \times 10^{-6} \text{ ft}^2/\text{lb} = 4.2 \times 10^{-5} \text{ kN/m}^2
\]

is subjected to an earthquake which can be considered as applying 24 uniform stress cycles in a period of 70 seconds and it is known that under undrained conditions, the soil would liquefy under this sequence of stress applications after 12 cycles (i.e. after 35 seconds) so that \( N_{eq}/N_k = 2 \).

If gravel drains of 2' (0.61 m) diameter were considered then, noting that \( \gamma_w = 9.8 \text{ kN/m}^3 \):

\[
T_{ad} = \frac{k}{\gamma_w} \cdot \frac{t_d}{m_3 a^2} = \frac{10^{-5} \times 70}{9.8 \times 4.2 \times 10^{-5} \times (0.305)^2} = 18.3
\]

Now referring to Fig. 7, if a value of \( r_g = 0.6 \) were considered to be allowable, it may be seen that

\[
b/a \leq 0.25
\]
Fig. 6 RELATIONSHIP BETWEEN GREATEST PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{e} = 1$
Fig. 7 RELATIONSHIP BETWEEN GREATEST PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_p = 2$
Fig. 8 RELATIONSHIP BETWEEN GREATEST PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{f} = 3$
Fig. 9 RELATIONSHIP BETWEEN GREATEST PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{g} = 4$
$T_{ad} = \frac{k}{\gamma \omega} \frac{t_d}{m_v \lambda a^2}$

Fig. 10 RELATIONSHIP BETWEEN GREATEST AVERAGE PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{eq} = 1$
Fig. 11 RELATIONSHIP BETWEEN GREATEST AVERAGE PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{p} = 2$

$$T_{ad} = \frac{k}{\gamma_{w}} \frac{t_d}{m v_3 \alpha^2}$$
Fig. 12 RELATIONSHIP BETWEEN GREATEST AVERAGE PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_{f} = 3$

$T_{ad} = \frac{k}{\gamma_w} \frac{t_d}{m v_3 a^2}$
Fig. 13 RELATIONSHIP BETWEEN GREATEST AVERAGE PORE PRESSURE RATIO AND DRAIN SYSTEM PARAMETERS FOR $N_{eq}/N_e = 4$

$$T_{ad} = \frac{k}{\gamma_w} \frac{t_d}{m v_3 a^2}$$
so that a spacing of 8 ft would have to be adopted. A comparison with Fig. 11 shows that a similar spacing would have been adopted if a greatest average pore pressure ratio \( \overline{r} \) = 0.6 had been considered allowable.

**EFFECT OF PERMEABILITY OF FILLING MATERIAL**

In the calculation of the previous results it has been assumed that the material within the gravel drain was infinitely permeable. The effect of this assumption can be examined by assuming a finite permeability for this material and again solving Eqn. 5. To simplify the analysis it has been assumed that radial drainage also occurs in the filling material but there is an infinitely pervious pipe at its center. (If no such pipe were present the generated excess pore water pressures within the drain-layer system would redistribute because of their different properties and then dissipate to a final non-zero value, since no pore water could escape.) To illustrate the effect, the case of a sand deposit with \( N_{eq}/N_\phi = 2 \) and \( T_{bd} = 1.25 \) with a spacing ratio \( a/b = 0.2 \) was analyzed for a range of values of \( k_f/k \) where \( k_f \) denotes the permeability of the filling material. For the sake of simplicity it was assumed that the filling material had the same liquefaction characteristics as the sand. The results of the analysis are shown in Fig. 14 and it can be seen that the drain operates perfectly provided it has a permeability of the order of 200 times that of the sand. Thus it would appear that for most sands, medium to fine gravels would provide adequate filling material for the drains.
Fig. 14 EFFECT OF PERMEABILITY OF DRAIN MATERIAL ON RATE OF PORE PRESSURE DISSIPATION
EFFECT OF VERTICAL DRAINAGE

The preceding analyses have been based on the assumption that vertical drainage played an insignificant role in the functioning of the drains. To illustrate this effect, consider the case of a 20 ft layer of the material described in the previous example resting on an impermeable layer and buried beneath a pervious fill 50 ft deep, both materials having a unit weight of 120 lbs/ft³. Suppose also that the vertical permeability of the sand is one third of its horizontal permeability.

It was assumed that 8" diameter sand drains were placed in this material at an effective spacing of 6 ft. The problem was analyzed first assuming that only radial drainage could occur and that the soil had a uniform initial effective stress equal to that at its midsection, and secondly allowing the initial effective stress to vary throughout the layer and allowing both vertical and horizontal drainage to occur. The results of these analyses are compared in Fig. 15, and are virtually indistinguishable within the accuracy of plotting.

Of course there may be many situations in which vertical drainage may have a significant effect but it is conjectured that for many of these cases it will be sufficiently accurate to use the relationship:

\[
\text{Greatest pore pressure ratio} = \frac{\text{Greatest pore pressure ratio for purely radial drainage of midsection}}{\text{Degree of consolidation for purely vertical drainage of the layer}}
\]

Thus the data in Figs. 6 to 13 could still be used to evaluate the efficiency of any proposed drainage system.
Fig. 15 EFFECT OF COMBINED RADIAL AND VERTICAL DRAINAGE ON MAXIMUM PORE PRESSURE RATIO

- Computed for radial drainage only
- ○ Computed for both radial and vertical drainage
CONCLUSION

In many cases, the installation of a drainage system as described in the preceding pages offers an attractive and economical procedure for stabilizing an otherwise potentially liquefiable sand deposit. In fact, the method has already been used in one case involving the construction of stone columns in a relatively loose sand deposit and it is currently being proposed for stabilization of a medium dense sand layer which is known to have developed some degree of liquefaction in a recent earthquake but which appears to be too dense for stabilization by further densification using currently available procedures.

The simplified theory presented in the preceding pages provides a convenient basis for evaluating the possible effectiveness of a gravel drain system in such cases. Where appropriate, additional analyses may readily be made using the computer program LARF but for most practical cases, it is believed that the results presented in Figs. 6 to 13 will provide an adequate basis for design and selection of a suitable drain system for effective stabilization of a potentially liquefiable sand deposit.

ACKNOWLEDGEMENTS

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REFERENCES


### NOTATION

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>2a</td>
<td>Diameter of sand drain.</td>
</tr>
<tr>
<td>2b</td>
<td>Effective spacing of sand drains.</td>
</tr>
<tr>
<td>dt</td>
<td>Increment of time.</td>
</tr>
<tr>
<td>k</td>
<td>Isotropic permeability.</td>
</tr>
<tr>
<td>k_h</td>
<td>Horizontal permeability.</td>
</tr>
<tr>
<td>k_v</td>
<td>Vertical permeability.</td>
</tr>
<tr>
<td>m_v3</td>
<td>Coefficient of volume decrease.</td>
</tr>
<tr>
<td>N</td>
<td>Number of cycles.</td>
</tr>
<tr>
<td>N_eq</td>
<td>Number of uniform stress cycles equivalent to the earthquake.</td>
</tr>
<tr>
<td>N_e</td>
<td>Number of cycles to liquefaction.</td>
</tr>
<tr>
<td>r_u</td>
<td>Pore pressure ratio.</td>
</tr>
<tr>
<td>r_max(t)</td>
<td>Maximum pore pressure ratio at time t.</td>
</tr>
<tr>
<td>(r_g)</td>
<td>Greatest average pore pressure ratio.</td>
</tr>
<tr>
<td>t</td>
<td>Time.</td>
</tr>
<tr>
<td>t_d</td>
<td>Duration of the earthquake.</td>
</tr>
<tr>
<td>t_u</td>
<td>Time at which liquefaction occurs under undrained conditions.</td>
</tr>
<tr>
<td>T_ad</td>
<td>Dimensionless time factor.</td>
</tr>
<tr>
<td>T_bd</td>
<td>Dimensionless time factor.</td>
</tr>
<tr>
<td>α</td>
<td>Parameter describing pore pressure generation in sand.</td>
</tr>
<tr>
<td>ε</td>
<td>Volume strain.</td>
</tr>
<tr>
<td>γ_w</td>
<td>Unit weight of water.</td>
</tr>
<tr>
<td>σº'</td>
<td>Initial mean bulk, or vertical, effective stress.</td>
</tr>
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</table>
APPENDIX A

A computer program LARF, Liquefaction Analysis for Radial Flow, has been written to integrate Eqn. 6 numerically using the finite element approach. The program consists of a main program LARF and 6 subroutines DATA, STIFF, DGNL, SETUP, DUG, SYMSOL. The action of the program is shown schematically in the flow chart, Fig. 16. The finite element discretisation for such problems is well known (see Zienkiewicz, 1971) and so is not detailed here; details of the numerical determination of the source function $\frac{\partial u}{\partial t}$ have been given by Seed et. al. (1975).

Input Details

Data for the program LARF must be input as described in Table 1 and read by means of the subroutine DATA. An illustrative example is given later. The data may be in any set of consistent units.

Output Details

The output of program LARF consists of the values of the maximum pore pressure ratio, average pore pressure ratio and greatest pore pressure ratio as well as the values of the excess pore pressures at all the node points.
Fig. 16 FLOW CHART FOR LARF
### TABLE 1
SEQUENCE FOR READING DATA

<table>
<thead>
<tr>
<th>No. of DATA CARDS</th>
<th>DESCRIPTION</th>
<th>FORMAT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Read unit weight of water $\text{GAMAW}$.</td>
<td>F10.4</td>
</tr>
<tr>
<td>2</td>
<td>Read duration of earthquake $\text{TD}$.</td>
<td>F10.4</td>
</tr>
<tr>
<td>3</td>
<td>Read initial vertical effective stress $\text{ESV}$.</td>
<td>F10.4</td>
</tr>
<tr>
<td>4</td>
<td>Read the number of nodes $\text{NR}$.</td>
<td>I4</td>
</tr>
<tr>
<td>5</td>
<td>Read the node number ($J$), $R$ coordinate $R(J)$, number of equivalent cycles $\text{ENE}(J)$, and number of cycles to liquefaction $\text{ENL}(J)$ for each node.</td>
<td>I4,3F10.4</td>
</tr>
<tr>
<td>6</td>
<td>Read number of different time steps, $\text{NINT}$.</td>
<td>I4</td>
</tr>
<tr>
<td>7</td>
<td>Read number of time steps $\text{NDELT}(I)$, and magnitude of that time step $\text{DELT}(I)$.</td>
<td>I4,F10.4</td>
</tr>
<tr>
<td>8</td>
<td>Read element number $\text{JR}$, horizontal permeability of the element $\text{PERMR}(\text{JR})$ and compressibility of the element $\text{COMX}(\text{JR})$.</td>
<td>I4,2E12.6</td>
</tr>
</tbody>
</table>
Special Details

At the moment the program is set up to cope with
(a) A maximum number of 50 nodes NRA
(b) A maximum number of 10 different time increments NINTA.

If desired, this can be changed by altering the statements

\[
\begin{align*}
&\text{NRA} = 50 \quad \text{to say} \quad \text{NRA} = 100 \\
&\text{NINTA} = 10 \quad \text{to say} \quad \text{NINTA} = 20
\end{align*}
\]

and the replacement, in the common blocks and dimension statements, of

\[
\begin{align*}
50 & \quad \text{by} \quad 100 \\
49 & \quad \text{by} \quad 99 \\
10 & \quad \text{by} \quad 20
\end{align*}
\]

The finite element nodes should be numbered from the drain outwards.
It will always be assumed that the innermost node is free to drain and
thus has no excess pore pressure and that the outer-most node lies on an
impermeable boundary. The elements are bounded by pairs of adjacent nodes
and are numbered from the inner-most to the outer-most.

Example

To illustrate the use of LARF, consider the drain system discussed in
the earlier example. Suppose, in the finite element discretisation, that
7 equally-spaced nodes are chosen and that the solution is calculated at 30
distinct times, the first 20 at 3.5 second intervals, the subsequent 10 at
five second intervals. Force, distance and time will be expressed in terms
of k Newtons, meters and seconds respectively.

It will be assumed for definiteness that the initial effective stress
has a value of 100 units although it should be noted that the pore pressure
ratios are independent of this value.
The above problem should be regarded merely as illustrative. In any practical situation, the error due to choice of time step should be reduced to an acceptable level by an independent study (series of trial runs) examining the effect of changing the step size.
LISTING AND SAMPLE DATA FOR PROGRAM LARF

PROGRAM LARF(INPUT,OUTPUT)
COMMON/NDATA/NSR,NRA,K(50),ESV,NM
COMMON/TDATA/NINT,NINTA,DELT(10),NDELT(10)
COMMON/EDATA/NR,PERM(49),COMP(49),GAMMA
COMMON/EOO/ENE(50),ENL(50),TD
DIMENSION AA(50,2),AAD(50,2),DIAG(50,2)
DIMENSION U(50),BO(50),UGT(50)
MRA=50
NINT=10
NRAMB=NR-1
NR=NRMB
M=2

READ INPUT DATA
CALL DATA
ICH=1
IF(N.GT.NA) GOTO 19
ICH=2
IF(N.GT.NA) GOTO 19

SET UP STIFFNESS MATRIX
CALL STIFF(AA,NA,MA)

INITIALISE PORE PRESSURES
DO 210 I=1,N
U(I)=0.0
210 CONTINUE
T=0.0
RGREAT=0.0
DO 200 JT=1,NINT
DT=DELT(JT)
NT=NDELT(JT)
KKK=1

SET UP APPROXIMATING EQUATIONS
CALL SETUP(ALPHA,DT,AA,DIAG,U,AAD,MA,MA)
DO 200 IT=1,NT
T=T+DT

CALCULATE LOAD VECTOR
CALL LVECT(ALPHA,DT,T,AA,DIAG,U,UGT,BO,MA,MA)

SOLVE APPROXIMATING EQUATIONS
CALL SYMSESS(AAD,BO,NM,MA,MA,KKK)
KKK=2

CALCULATE MAXIMUM,AVERAGE AND GREATEST PORE PRESSURE RATIOS
RMAX=0.0
DO 10 I=1,N
U(I)=BO(I)
AR=U(I)/ESV
IF(AR.GT.RMAX) RMAX=AR
10 CONTINUE
IF(RMAX,GT,RGREAT) RGREAT=RMAX
AREA=0.0
SUM=0.0
DO 15 J=2,N
QUANT=0.5*(R(J)+R(J-1))*(R(J)-R(J-1))
AREA=AREA + QUANT
15 SUM=SUM+0.5*(U(J)+U(J-1))*QUANT
UAV=SUM/AREA/ESV

OUTPUT PORE PRESSURE RATIOS

PRINT 53,T,MAX,UAV,RGREAT
93 FORMAT(//2EX:,TIME = *,F10.4//)
2* MAXIMUM PORE PRESSURE RATIO = *,F10.4/
1* AVERAGE PORE PRESSURE RATIO = *,F10.4/
3* GREATEST PORE PRESSURE RATIO = *,F10.4/
PRINT 94
94 FORMAT(//,'EXCESS PORE WATER PRESSURE(I)**',/)
PRINT 95((I,U(I)),I=1,N)
95 FORMAT(5(2X,I4,F10.4))
200 CONTINUE
GOTO 39

DIAGNOSTICS FOR LARF

19 CONTINUE
PRINT 29,ICH
29 FORMAT(/**DIAGNOSTICS FOR STILL ICH=*,14)
39 CONTINUE

END

SUBROUTINE DATA
COMMON/NDATA/NP,NRA.R(50),ESV,N,M
COMMON/TDATA/NINT,NINTA,DELT(10),NDELT(10)
COMMON/DDATA/MR,NRA.PERM(49),COMX(49),GAMAU
COMMON/EO/DEN(50),ENL(50),TD

READ DATA

READ THE UNIT WEIGHT OF WATER

READ 71,GAMAU
71 FORMAT(8F10.4)
PRINT 106,GAMAU
106 FORMAT(//,' UNIT WEIGHT OF WATER = *,F10.4//)

READ DURATION OF THE EARTHQUAKE

107 FORMAT(/** DURATION OF EARTHQUAKE = *,F10.4//)
READ 71,TD
PRINT 107,TD

READ THE INITIAL VERTICAL STRESS

READ 71,ESV
PRINT 103,ESV
103 FORMAT(/** INITIAL EFFECTIVE STRESS =*,F10.4//)

READ THE NUMBER OF NODES

READ 1,NP
PRINT 101,NP
101 FORMAT(/** NUMBER OF NODES =*,14//)
ICH=1
ICH=ICH+1
IF(NK.GT.NRA) GOTO 19
NR=NR-1
C
C    READ THE NODAL DATA

PRINT 102
102 FORMAT(/** NODE R COORDINATE EQUIVALENT NUMBER*/
    *"# OF CYCLES"
    *"# NUMBER *.16X","NUMBER OF CYCLES TO LIQUEFACTION"
   DO 66 J=1,HR
   READ 72,J,R(J),ENE(J),ENL(J)
   PRINT 73,J,R(J),ENE(J),ENL(J)
72 FORMAT(14.5F10.4)
73 FORMAT(3X,14.5X,F10.4,F10.4,F10.4,F10.4)
66 CONTINUE
C
C    READ TIME STEP DATA

READ 1,HINT
PRINT 115,HINT
115 FORMAT(/** NUMBER OF DIFFERENT TIME STEPS =*.I4/)
    *"ICH=1"
    IF(HINT.GT.1) GOTO 19
    DO 17 I=1,HINT
   READ 5,NDLT(I),DELT(I)
5 FORMAT(14.5F10.4)
PRINT 117,NDLT(I),DELT(I)
117 FORMAT(*"NUMBER OF STEPS = *.I4 TIME INCREMENT =*.F10.4"
17 CONTINUE
C
C    READ THE ELEMENT DATA

PRINT 105
105 FORMAT(/** ELEMENT NUMBER PERMEABILITY COMPRESSIBILITY*/)
   DO 15 JR=1,MR
   READ 79,J,RFMR(JR),COMX(JR)
79 FORMAT(14.2E12.5)
   PRINT 104,J,RFMR(JR),COMX(JR)
104 FORMAT(8X,14.5X,E12.5,4X,E12.5)
15 CONTINUE
10 CONTINUE
1 CONTINUE
1 FORMAT(214)
RETURN
19 CONTINUE
PRINT 29,ICH
29 FORMAT(/** DIAGNOSTIC SUBROUTINE DATA ICH =*.I4*
       END
       SUBROUTINE STIFF(AA,NA,MA)
       COMMON/NDATA/1R,NRA,1R(50),ESV,N,M
       COMMON/EDATA/1R,NRA,PERMR(49),COMX(49),GAMAU
       DIMENSION AA(N,MA)
C
C       SET UP STIFFNESS MATRIX A

DO 16 I=1,HR
   DO 16 J=1,HR
16   AA(I,J)=0.0
   DO 40 JR=1,MR
   DR=PERMR(JR)+0.5*DR
   RBAR=R(JR)+0.5*DR
   AH=PERMR(JR)*RBAR/DR/GAMAU
C
C
CALCULATE THE LOAD VECTOR
SUBROUTINE LVECT(ALPHA,DT,T,AA,DIAJ,UGT,B0,NA,MA)
COMMON/NDATA/HR,HR,R(50),ESV,N,M
COMMON/EDATA/MINT,MINTA,DELT(18),HDELT(18)
COMMON/EDATA/MR,MRN,PERM(49),COMX(49),GAMAU
COMMON/EOD/EINE(50),ELEN(50),TD
DIMENSION NA(NA,MA),BO(NA),U(NA)
DIMENSION UGT(NA),DIAJ(NA,MA)
EPS'i.01
ESO=ESV
DO 240 K=1,MR
ES=ESO-U(K)
IF(ES.LT.EPS) U(K)=ESO
UAV=U(K)
CALL DUG(EINE,K),ENL(K),TR,UGT,ESV,T,UGH
UGT(K)=UGH
IF(ES.LT.EPS) UGT(K)=0.0
240 CONTINUE
DO 300 I=1,N
IM=I-1
SUN=0.0
DUN=0.0
LIM=M+1-I
IF(LIM.LT.1) LIM=1
IF(LIM.LT.IM) GOTO 777
DO 330 K=LIM,IM
DUN=DUN+DIAG(K,1-K+1)U(K)+UGT(K))
320 SUN=SUN+AR(K,1-K+1)*U(K)
330 CONTINUE
LIM=N+1-H
IF(LIM.LT.N) LIM=N
DO 320 K=1,LIM
DUN=DUN+DIAG(K,1-K+1)U(K)+UGT(K))
320 SUN=SUN+AR(K,1-K+1)*U(K)
300 BO(I)=SUN+DT*ALPHA
BO(I)=0.0
RETURN
END
SUBROUTINE DGNL(DIAJ,BO,NA,MA)
COMMON/NDATA/HR,HR,R(50),ESV,N,M
COMMON/EDATA/MR,MRN,PERM(49),COMX(49),GAMAU
DIMENSION DIAJ(NA,MA)
DIMENSION U(NA)
DQ 15 I-1
DQ 15 J-I
15 DIAJ(I,1)=0.0
DQ 220 JR=1,MR
DR=MR(JR)-I-5
RBAR=AR(JR)+5.5*MR
IA=JR
ID=IA+1
QUANT=.5*COMX(JR)*DR*RBAR
DIAJ(IA,1)=DIAJ(IA,1)+QUANT
DIAG(IB,1)=DIAG(IB,1)+QUANT
CONTINUE
RETURN
END
SUBROUTINE SETUP(ALPHA,DT,AA,DIAG,U,AAQ,NA,MA)
COMMON/KDATA/NR,NRA,R(50),ESV,N,M
COMMON/EQD/ENE(50),ENL(50),TD
DIMENSION AA(NA,MA),AAQ(NA,MA),DIAG(NA,MA)

C SET UP APPROXIMATING EQUATIONS
C
ALPHA=0.5
CALL DGNLCDIAG.U,NA,MA
DO 350 I=1,M
DO 360 J=1,M
360 AAQ(I,J)=AA(I,J)*DT*BETA+DIAG(I,J)
350 CONTINUE
K=1
DO 430 J=1,M
L=K-J+1
IF(L.GT.0) AAQ(L,J)=0.0
430 CONTINUE
RETURN
END
SUBROUTINE DUGENE,ENE,TD,U,ESV,TT,UGH)
C CALCULATE SOURCE TERM
C
UGH=0.0
IF(TT.GT.TD) GOTO 25
ALPHA=0.7
BETA=1.0/ALPHA
PI=3.14159265
XX=1.0
RU=U/ESV
ARG=0.5*(1.0-COS(PI*RU))
RMO=ARG**ALPHA
BR=DT/ENE/ENE
RHI=RNO+BR
ARG=2.0*RHI**BETA-1.0
IF(ARG.GT.XX) ARG=XX
RU=0.5+ASIN(ARG)/PI
UGH=RU*ESV-U
25 CONTINUE
RETURN
END
SUBROUTINE SYMSOL(A,B,NN,MM,NA,MA,KKK)
DIMENSION A(NA,MA),B(NA)
C SOLVE SIMULTANEOUS EQUATIONS
C
EPS=0.000001
ICH=1
IF(NN.GT.NA) GOTO 19
ICH=2
IF(MM.GT.MA) GOTO 19
ICH=K(KK.GT.1) GOTO 2600
1000 DO 280 N=1,NN
DO 280 L=2,MM
ICH=3
IF(ABS(A(N,I)).LT.EPS) GOTO 19
C=A(N,L)/A(N,1)
I=N+L-1
IF(NN-1) 260.240.240
240 J=1
DO 250 K=L,M1
J=J+1
250 A(I,J)=A(I,J)-C*A(N,K)
260 A(N,L)=C
280 CONTINUE
2000 DO 290 N=1,NN
DO 285 L=2,MM
I=N+L-1
IF(NN-1) 280.285.285
285 B(I)=B(I)-A(N,L)*B(N)
290 B(N)=B(N)/A(N,1)
N=NN
300 N=NN-1
IF(NN) 350.500.350
350 DO 400 K=2,MM
L=N+K-1
IF(NN-L) 400.370.370
370 B(N)=B(N)-A(N,K)*B(L)
400 CONTINUE
GOTO 390
500 RETURN
19 CONTINUE
PRINT 29,ICH
29 FORMAT(*DIAGNOSTICS FOR SYMBOL*,I4)
END

9.8
70.0
100.0
7
1 0.3850 24.0 12.0
2 0.4375 24.0 12.0
3 0.5100 24.0 12.0
4 0.6125 24.0 12.0
5 0.7150 24.0 12.0
6 1.0675 24.0 12.0
7 1.2200 24.0 12.0
20 3.5
18 5.0
1+0.10606E-04=0.42000E-04
2+0.10606E-04=0.42000E-04
3+0.10606E-04=0.42000E-04
4+0.10606E-04=0.42000E-04
5+0.10606E-04=0.42000E-04
6+0.10606E-04=0.42000E-04

**END OUTPUT
**END OUTPUT
SAMPLE OF OUTPUT FROM PROGRAM LARF

UNIT WEIGHT OF WATER = 9.8000

DURATION OF EARTHQUAKE = 70.0000

INITIAL EFFECTIVE STRESS = 100.0000

NUMBER OF NODES = 7
<table>
<thead>
<tr>
<th>NODE NUMBER</th>
<th>R COORDINATE</th>
<th>EQUIVALENT NUMBER OF CYCLES</th>
<th>NUMBER OF CYCLES TO LIQUEFACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.3050</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>2</td>
<td>0.4575</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>3</td>
<td>0.6100</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>4</td>
<td>0.7625</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>5</td>
<td>0.9156</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>6</td>
<td>1.0683</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
<tr>
<td>7</td>
<td>1.2200</td>
<td>24.0000</td>
<td>12.0000</td>
</tr>
</tbody>
</table>

NUMBER OF DIFFERENT TIME STEPS = 2

NUMBER OF STEPS = 20 TIME INCREMENT = 3.5000
NUMBER OF STEPS = 10 TIME INCREMENT = 5.0000

ELEMENT NUMBER PERMEABILITY COMPRESSION

<table>
<thead>
<tr>
<th>ELEMENT NUMBER</th>
<th>PERMEABILITY (E-04)</th>
<th>COMPRESSION (E-04)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
<tr>
<td>2</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
<tr>
<td>3</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
<tr>
<td>4</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
<tr>
<td>5</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
<tr>
<td>6</td>
<td>1.0E+00</td>
<td>4.2E+00</td>
</tr>
</tbody>
</table>

TIME = 3.5000
MAXIMUM PORE PRESSURE RATIO = 1.213
AVERAGE PORE PRESSURE RATIO = 1.1674
GREATEST PORE PRESSURE RATIO = 1.218

EXCESS PORE WATER PRESSURE

<table>
<thead>
<tr>
<th>ELEMENT NUMBER</th>
<th>PORE WATER PRESSURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.8287</td>
</tr>
<tr>
<td>2</td>
<td>0.3737</td>
</tr>
<tr>
<td>3</td>
<td>0.2200</td>
</tr>
<tr>
<td>4</td>
<td>1.4170</td>
</tr>
<tr>
<td>5</td>
<td>1.9231</td>
</tr>
<tr>
<td>6</td>
<td>1.2162</td>
</tr>
<tr>
<td>7</td>
<td>1.1733</td>
</tr>
</tbody>
</table>

TIME = 7.0000
MAXIMUM PORE PRESSURE RATIO = 1.946
AVERAGE PORE PRESSURE RATIO = 1.625
GREATEST PORE PRESSURE RATIO = 1.946

EXCESS PORE WATER PRESSURE

<table>
<thead>
<tr>
<th>ELEMENT NUMBER</th>
<th>PORE WATER PRESSURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.2612</td>
</tr>
<tr>
<td>2</td>
<td>19.4558</td>
</tr>
<tr>
<td>3</td>
<td>14.1674</td>
</tr>
<tr>
<td>4</td>
<td>17.0520</td>
</tr>
<tr>
<td>5</td>
<td>18.5596</td>
</tr>
<tr>
<td>TIME</td>
<td>MAXIMUM PORE PRESSURE RATIO</td>
</tr>
<tr>
<td>------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>10.5000</td>
<td>0.2459</td>
</tr>
<tr>
<td>14.0000</td>
<td>0.2920</td>
</tr>
<tr>
<td>17.5000</td>
<td>0.3283</td>
</tr>
<tr>
<td>21.0000</td>
<td>0.3583</td>
</tr>
<tr>
<td>24.5000</td>
<td>0.3845</td>
</tr>
</tbody>
</table>

**EXCESS PORE WATER PRESSURE**

<table>
<thead>
<tr>
<th>TIME</th>
<th>MAXIMUM PORE PRESSURE RATIO</th>
<th>AVERAGE PORE PRESSURE RATIO</th>
<th>GREATEST PORE PRESSURE RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.5000</td>
<td>11.2629</td>
<td>24.8932</td>
<td>23.4170</td>
</tr>
<tr>
<td>14.0000</td>
<td>12.5708</td>
<td>29.2007</td>
<td>27.3695</td>
</tr>
<tr>
<td>17.5000</td>
<td>13.9745</td>
<td>32.7966</td>
<td>30.6286</td>
</tr>
<tr>
<td>21.0000</td>
<td>14.9653</td>
<td>35.8541</td>
<td>33.3855</td>
</tr>
<tr>
<td>24.5000</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TIME = 28.8088
MAXIMUM PORE PRESSURE RATIO = 0.4073
AVERAGE PORE PRESSURE RATIO = 0.3299
GREATEST PORE PRESSURE RATIO = 0.4073

EXCESS PORE WATER PRESSURE(1)

1  0.  2  15.8430  3  25.6709  4  31.9117  5  35.7654
6  37.6296  7  38.4496

MAXIMUM PORE PRESSURE RATIO = 0.4073
AVERAGE PORE PRESSURE RATIO = 0.3299
GREATEST PORE PRESSURE RATIO = 0.4073

TIME = 31.5000
MAXIMUM PORE PRESSURE RATIO = 0.4275
AVERAGE PORE PRESSURE RATIO = 0.3448
GREATEST PORE PRESSURE RATIO = 0.4275

EXCESS PORE WATER PRESSURE(1)

1  0.  2  16.7143  3  27.0680  4  33.7039  5  37.8346
6  48.0631  7  48.7348

MAXIMUM PORE PRESSURE RATIO = 0.4275
AVERAGE PORE PRESSURE RATIO = 0.3448
GREATEST PORE PRESSURE RATIO = 0.4275

TIME = 35.0000
MAXIMUM PORE PRESSURE RATIO = 0.4455
AVERAGE PORE PRESSURE RATIO = 0.3568
GREATEST PORE PRESSURE RATIO = 0.4455

EXCESS PORE WATER PRESSURE(1)

1  0.  2  17.4468  3  28.2630  4  35.2845  5  39.6660
6  42.0352  7  42.7504

MAXIMUM PORE PRESSURE RATIO = 0.4455
AVERAGE PORE PRESSURE RATIO = 0.3568
GREATEST PORE PRESSURE RATIO = 0.4455

TIME = 38.5000
MAXIMUM PORE PRESSURE RATIO = 0.4617
AVERAGE PORE PRESSURE RATIO = 0.3715
GREATEST PORE PRESSURE RATIO = 0.4617

EXCESS PORE WATER PRESSURE(1)

1  0.  2  18.0637  3  29.3601  4  36.6906  5  41.2954
6  43.7349  7  44.5507

MAXIMUM PORE PRESSURE RATIO = 0.4617
AVERAGE PORE PRESSURE RATIO = 0.3715
GREATEST PORE PRESSURE RATIO = 0.4617

EXCESS PORE WATER PRESSURE(1)

1  0.  2  18.6417  3  30.3244  4  37.9582  5  42.7642
6  45.3793  7  46.1714
<table>
<thead>
<tr>
<th>TIME</th>
<th>MAXIMUM PORE PRESSURE RATIO</th>
<th>AVERAGE PORE PRESSURE RATIO</th>
<th>GREATEST PORE PRESSURE RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>42.0000</td>
<td>0.4764</td>
<td>0.3929</td>
<td>0.4764</td>
</tr>
<tr>
<td>45.5000</td>
<td>0.4900</td>
<td>0.3934</td>
<td>0.4900</td>
</tr>
<tr>
<td>49.0000</td>
<td>0.5024</td>
<td>0.4031</td>
<td>0.5024</td>
</tr>
<tr>
<td>52.5000</td>
<td>0.5143</td>
<td>0.4195</td>
<td>0.5143</td>
</tr>
<tr>
<td>56.0000</td>
<td>0.5243</td>
<td>0.4206</td>
<td>0.5243</td>
</tr>
</tbody>
</table>

EXCESS PORE WATER PRESSURE (1)

<table>
<thead>
<tr>
<th>TIME</th>
<th>MAXIMUM PORE PRESSURE RATIO</th>
<th>AVERAGE PORE PRESSURE RATIO</th>
<th>GREATEST PORE PRESSURE RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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EXCESS PORE WATER PRESSURE (1)

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**Maximum Pore Pressure:**
- 1.5543
- 3.4725
- 6.5450
- 6.5450
- 13.9203

**Average Pore Pressure:**
- 3.0334
- 5.0874
- 7.4344
- 7.5544
- 12.4308

**Greatest Pore Pressure:**
- 3.5352
- 7.3930
- 12.4308
- 12.4308
- 21.3519

**Excess Pore Water Pressure:**
- 21.1862
- 34.6954
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- 50.2000

**Time:**
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EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS


EERC 68-1  Unassigned

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EERC 69-7  "Rock Motion Accelerograms for High Magnitude Earthquakes," by H. B. Seed and I. M. Idriss - 1969 (PB 187 940)


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"Hysteretic Behavior of Ductile Moment Resisting Reinforced Concrete Frame Components," by V. V. Bertero and E. P. Popov - 1975

"Relationships Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong Earthquakes," by H. Bolton Seed, Ramesh Murarka, John Lysmer and I. M. Idriss - 1975


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EERC 76-8  "Cyclic Shear Tests on Concrete Masonry Piers," R. L. Mayes, Y. Omote and R. W. Clough - 1976


EERC 76-10  "Stabilization of Potentially Liquefiable Sand Deposits Using Gravel Drain Systems," by H. Bolton Seed, and John R. Booker - 1976
In many cases, the installation of a system of vertical columnar drains offers an attractive and economical procedure for stabilizing an otherwise potentially liquefiable sand deposit. In fact, the method has already been used in one case involving the construction of stone columns in a relatively loose sand deposit and it is currently being proposed for stabilization of a medium dense sand layer which is known to have developed some degree of liquefaction in a recent earthquake but which appears to be too dense for stabilization by further densification using currently available procedures.

The report presents a simplified theory which provides a convenient basis for evaluating the possible effectiveness of a gravel drain system in such cases. Where appropriate, additional analyses may readily be made using the computer program LARF described in the report but for most practical cases, it is believed that the results presented in chart form will provide an adequate basis for design and selection of a suitable drain system for effective stabilization of potentially liquefiable sand deposits.