A Parametric Study of Embankments on Clay Soils During Earthquake Shaking

Karla I. Reynoso
Utah State University

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A PARAMETRIC STUDY OF EMBANKMENTS ON CLAY SOILS DURING EARTHQUAKE SHAKING

by

Karla I. Reynoso

A thesis submitted in partial fulfillment of the requirements for the degree

of

MASTER OF SCIENCE

In

Civil and Environmental Engineering

Approved:

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UTAH STATE UNIVERSITY
Logan, Utah

2012
ABSTRACT

A Parametric Study of Embankments on Clay Soils During Earthquake Shaking

by

Karla I. Reynoso, Master of Science
Utah State University, 2012

Major Professor: Dr. James Bay
Department: Civil and Environmental Engineering

This study is a parametric evaluation of reduction in undrained shear strength of fine grained soils required to cause failure beneath embankments during earthquake loading. The evaluated parameters are: crust thickness, normalized undrained strength, maximum past pressure, and embankment height. Both finite element and limit equilibrium analyses were used to determine strength reductions that would lead to embankment failure. It was found that reductions of undrained strengths of 55% to 65% would lead to failure during earthquake loading.

The method proposed by Idriss and Boulanger was also used to predict strength reductions for each model over a range of earthquake amplitudes and magnitudes. Idriss and Boulanger predicted strength reductions around 80% which would not lead to collapse of the embankments.

(327 pages)
PUBLIC ABSTRACT

A Parametric Study of Embankments on Clay Soils During Earthquake Shaking

This study is a description and examination of relationships between different parameters in the reduction of the undrained shear strength (type of shear strength in soil mechanics where the rate of loading is much quicker than the rate at which the pore water is able to drain out of the soil due to the action of shearing the soil) of clay soils required to cause failure beneath embankments during earthquake loading. The parameters used in this study are: the thickness of the uppermost desiccated layer of the soil profile, the normalized undrained strength, the historically maximum effective pressure that has been exerted on the soil, and the embankment height. Two approaches were developed to determine strength reductions that would lead to embankment failure: finite element and limit equilibrium analyses. Findings show reductions of undrained strengths of 55% to 65% that would lead to failure during earthquake shaking.

The method proposed by Idriss and Boulanger, geotechnical engineers and members of the American Society of Civil Engineers (ASCE), was also used to predict strength reductions for each soil model over a range of earthquake amplitudes and magnitudes. Idriss and Boulanger predicted strength reductions around 80%, which would not lead to collapse of the embankments.

Karla I. Reynoso
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I dedicate this work to my family for always supporting me and encouraging me to fight for my dreams, that despite the distance I feel you close to me through your unconditional love, and to all my friends back home and here at USU for being my second family, for your unconditional support, for smoothing the rough times with your counseling and for bringing joy to my life with your company, your laughter, or your phone calls every time this journey got harsh. I couldn’t have done it without all of you cheering me up.

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The following symbols are used in this paper:

- $\Delta \sigma = \text{difference between the max past pressure and the pre-consolidation pressure}$
- $K_\alpha = \text{factor relative to static shear stress}$
- $\Sigma_{Msf} = \text{total multiplier from Plaxis}$
- $\gamma = \text{unit weight}$
- $\sigma'_1 = \text{major principal stress}$
- $\sigma'_p = \text{maximum past pressure or overconsolidation pressure}$
- $\sigma'_{vo} = \text{effective vertical stress}$
- $\tau = \text{shear stress}$
- $\tau_{cyc} = \text{cyclic shear stress}$
- $\tau_f = \text{shear stress at failure}$
- $\tau_s = \text{static shear stress}$
- $\varnothing' = \text{angle of internal friction}$
- $AOCR = \text{apparent overconsolidation ratio}$
- $a_{max} = \text{peak horizontal acceleration}$
- $C_{2D} = \text{two-directional cyclic loading factor}$
- $CRR = \text{cyclic resistance ratio}$
- $CSR = \text{cyclic stress ratio}$
- $CTX = \text{cyclic triaxial}$
- $c' = \text{cohesion}$
- $(c/p)_{NC} = \text{undrained strength ratio for normally consolidated clays}$
- $DSS = \text{direct simple shear}$
- $E_{50} = \text{triaxial loading stiffness}$
- $E_{oed} = \text{oedometer stiffness}$
- $E_{ur} = \text{triaxial unloading stiffness}$
- $FEA = \text{finite element analysis}$
- $FS = \text{factor of safety}$
- $g = \text{acceleration of gravity}$
- $K_0^{NC} = \text{K}_0\text{-value for normal consolidation}$
- $LL = \text{liquid limit}$
- $m = \text{power for stress level dependency}$
- $MSF = \text{magnitude scaling factor}$
- $OCR = \text{overconsolidation ratio}$
- $PI = \text{plastic index}$
\[ p_{\text{ref}} \]
= reference pressure;

\[ r_d \]
= stress reduction coefficient;

\[ SEM \]
= scanning electron microscope;

\[ SRF \]
= strength reduction factor;

\[ S_u \]
= undrained shear strength;

\[ UU \]
= unconsolidated undrained;

\[ w_c \]
= natural water content
CHAPTER 1

INTRODUCTION

When materials are loaded or stressed they deform or strain. For clay soils, the critical point is that shear strength during cyclic loading is less than static loading. We do not know how much lower, but when subjected to cyclic loading, just as in the case of earthquake ground shaking, they could develop stresses that may overcome their strength rather quickly leading to instability or failure.

Many studies about sandy soil behavior and liquefaction have been conducted throughout the years. Hence, it is well known how these soils lose their strength during cyclic loading due to earthquakes leading to large soil displacements in the ground that affect the stability of any structure built on it. Clayey soils exhibit strength reductions during cyclic loading. However, studies about clay behavior during earthquake ground motion are not conclusive, and currently there are not tools available to predict the extend of strength reduction in clayey soils during cyclic loading.

The object of this thesis is to evaluate how much strength reduction would be required to cause failure during ground shaking. A parametric study was performed using finite element and limit equilibrium analyses to determine the reduction in strength that would lead to failure for a range in soil parameters. The soil parameters that were varied were: crust thickness, normalized undrained strength, maximum past pressure, and embankment height. It was found that both finite element and limit
equilibrium analysis results are somewhat similar, and that reductions of undrained strengths of 55% to 65% would lead to failure during earthquake loading.

Boulanger and Idriss (2004) have proposed an approach to predict softening of clayey soils during earthquake loading. This approach is based upon limited laboratory testing, and has not been verified for use in the engineering community. The Idriss and Boulanger approach was used to predict clay softening for each of the embankments modeled for ranges of peak ground accelerations of 0.1g, 0.2g, and 0.4g and magnitudes of 5.5, 6.5, 7.5 and 8.5. Idriss and Boulanger predicted softening ranging from 0.76 to 0.89. These levels of softening would not lead to collapse of most embankments.

This thesis comprises six chapters. Chapter 1 is the introduction to the subject to be analyzed. Chapter 2 presents a summary of relevant work previously done in respect to understanding clay behavior during cyclic loading, the methods and approaches used to obtain results. Chapter 3 explains how the clay soil models were created and discretized in finite element and limit equilibrium analyses to obtain the undrained shear strength ($S_u$). Chapter 4 describes the finite element and limiting equilibrium analyses performed on the soil profiles and the comparison between the methods in relation to the strength reductions required to cause failure during earthquake loading. Chapter 5 presents softening and residual strength predictions based on the I. M. Idriss and R. W. Boulanger 2008 monograph “Soil liquefaction during earthquake.” Finally, Chapter 6 presents the conclusions from the analyses.
CHAPTER 2
LITERATURE REVIEW

The behavior of clays during cyclic loading is a matter still wrapped with uncertainties in geotechnical engineering. The issue is to determine if a clayey foundation supporting an embankment would undergo deformations within the profile without any further damage to structures built on it, or would it be stressed to a point where an imminent failure leads to a catastrophe after earthquake shaking.

Most of the previous work related to this subject is empirical, based on field explorations and laboratory testing after several earthquakes where liquefaction or softening of clayey soils has occurred, i.e. the 1999, Chi-Chi, Taiwan Earthquake (Chu et al., 2008). Nonetheless, different approaches have been made to characterize softening in clays (named liquefaction in sands), as summarized below to better understand the current knowledge about this subject.

2.1 The Simplified Procedure

In 1971 Professors H. B. Seed and I. M. Idriss developed and published the “Simplified Procedure” methodology that has become a standard practice to determine the liquefaction of soils (Youd et al., 2001) and has been improved through time by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985).

To estimate the liquefaction resistance of soils, two variables must be calculated: the CSR or stress induced by earthquake loading, and the CRR, which is the
resistance to liquefaction (Seed and Idriss, 1971). The CSR equation is shown as follows:

\[
CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d
\]  

(Eq. 2.1)

where \( a_{max} \) = peak horizontal acceleration at the ground surface due to the earthquake; \( g \) = acceleration of gravity; \( \sigma_{vo} \) and \( \sigma'_{vo} \) are total and effective vertical overburden stresses, respectively; and \( r_d \) = stress reduction coefficient that accounts for the flexibility of the soil profile (i.e., \( r_d = 1 \) corresponds to rigid body behavior).

The CSR is then scaled by a factor of 0.65 to produce a CSR that is considered representative of the most significant cycles over the full duration of loading.

The \( r_d \) value recommended to estimating the CSR may be used by the following equation (Liao and Whitman, 1986):

\[
r_d = 1.0 - 0.00765z, \text{ for } z < 9.15 \text{ m} \]  

(Eq. 2.2)

\[
r_d = 1.174 - 0.0267z, \text{ for } 9.15 \text{ m} < z < 23 \text{ m} \]  

(Eq. 2.3)

where \( z \) = depth below ground surface in meters. Mean values of \( r_d \) calculated from Eq. (2.2) and (2.3) are shown in Fig. 2.1 along with the mean and range of values proposed by Seed and Idriss (1971).

To facilitate computation of \( r_d \) values in engineering practice, T. F. Blake developed an equation easier to program that approximates the curve in Fig. 2.1 as follows (Youd et al., 2001):
where \( z \) = depth beneath ground surface in meters.

\[
\begin{align*}
    r_d &= \frac{1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^{2}} \\
    \text{(Eq. 2.4)}
\end{align*}
\]

**Fig. 2.** \( r_d \) versus depth curves developed by Seed and Idriss (1971) with added mean-value lines plotted from Eq. (2.2) and (2.3) (after Youd et al., 2001).

The common practice to evaluate the liquefaction resistance is through field testing based on standard penetration tests (SPT); cone penetration tests (CPT); shear-wave velocity measurements \( (V_s) \); and Becker penetration test (BPT) for gravelly soil. Advantages and disadvantages of each test are listed in Table 2.1 (Youd et al. 2001).
Table 2. Comparison of advantages and disadvantages of various field tests for assessment of liquefaction resistance (from Youd et al., 2001)

<table>
<thead>
<tr>
<th>Feature</th>
<th>Test Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SPT</td>
</tr>
<tr>
<td>Past measurements at liquefaction site</td>
<td>Abundant</td>
</tr>
<tr>
<td>Type of stress-strain behavior influencing test</td>
<td>Partially drained, large strain</td>
</tr>
<tr>
<td>Quality control and repeatability</td>
<td>Poor to good</td>
</tr>
<tr>
<td>Detection of variability of soil deposits</td>
<td>Good for closely spaced tests</td>
</tr>
<tr>
<td>Soil types in which test is recommended</td>
<td>Non-gravel</td>
</tr>
<tr>
<td>Soil sample retrieved</td>
<td>Yes</td>
</tr>
<tr>
<td>Test measures index or engineering property</td>
<td>Index</td>
</tr>
</tbody>
</table>

The CRR curves (clean-sand based) resulting from the SPT, CPT, and $V_s$ are related to magnitude 7.5 earthquakes. That is why Seed and Idriss (1982) developed “Magnitude Scaling Factors” to correct and adjust these CRR curves to magnitudes of earthquakes smaller or larger than 7.5. This magnitude scaling factor (MSF) affects the factor of safety (FS) against liquefaction as presented below:

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right)^{MSF}$$  \hspace{1cm} (Eq. 2.5)
where CSR = calculated cyclic stress ratio due to earthquake loading; and CRR$_{7.5}$ = cyclic resistance ratio from earthquakes magnitude 7.5 determined from SPT, CPT, or $V_s$ curves.

Other corrections factors were developed by Seed (1983) to account for other site conditions than that of which the simplified procedure was elaborated based on low static shear stresses ($\tau_s$) and low overburden pressures ($\sigma'_{vo}$).

### 2.2 Liquefaction Susceptibility of Fine Grained Soils

The Chinese criteria emerged after liquefaction occurred in fine-grained soils at various sites in China after strong earthquakes. Wang (1979) plotted those CL, CL-ML, and ML soils that liquefied as shown in Fig. 2.2, but did not provide details on how the data was collected neither interpreted, hence it cannot be determined if the soil would behave as sand-like or clay-like, and the use of the water content and liquid limit ratio ($w_c/LL$) to evaluate whether a soil is susceptible to liquefaction or not is misleading (Boulanger and Idriss, 2004).

The Chinese Criteria established that certain clayey soils may develop huge strength loss due to earthquake loading if they present these combined characteristics (Seed and Idriss, 1982):

1) Percent finer than 0.005 mm < 15%

2) Liquid Limit (LL) < 35

3) Water content ($w_c$) > 0.9 x LL

4) Plot above the A-line on the Plasticity chart
Several studies have addressed the liquefaction susceptibility of fine grained soils. Andrews and Martin (2000) used clay defined as grains finer than 0.002mm, and a liquid limit criterion together with a clay content criterion to help address cases where clay sized grains are non-plastic, and non-clay sized grains are plastic. Their conclusions are summarized in Table 2.2.

**Table 2.2.** Liquefaction susceptibility of silty soils (after Andrews and Martin, 2000)

<table>
<thead>
<tr>
<th>Clay Content</th>
<th>Liquid Limit &lt; 32 $^{(1)}$</th>
<th>Liquid Limit ≥ 32</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay Content &lt; 10%</td>
<td>Susceptible</td>
<td>Further Studies Required (Considering plastic non-clay sized grains - such as Mica)</td>
</tr>
<tr>
<td>Clay Content ≥ 10%</td>
<td>Further Studies Required (Considering non-plastic clay sized grains – such as mine and quarry tailings)</td>
<td>Not Susceptible</td>
</tr>
</tbody>
</table>

Note: Liquid Limit determined by Casagrande-type percussion apparatus.
After the 1999 Kocaeli (Turkey) and Chi-Chi (Taiwan) earthquakes, Seed et al. (2003) provided recommendations regarding liquefiability of soils with significant fine contents. Three zones were identified: A, B and C in the Atterberg Limit Chart, as shown in Fig. 2.3, where Zone A contains soils considered potentially susceptible to cyclically induced liquefaction, Zone B represents soils that may be liquefiable, and Zone C soils (outside Zones A and B) are considered generally not susceptible to liquefaction, but should be checked for potential sensitivity.

**Fig. 2.3** Liquefiable soil types (from Seed et al. 2003)

Bray and Sancio (2006) performed a program of cyclic triaxial (CTX) and cyclic simple shear (CSS) testing on the silty and clayey soils of Adapazari after the Kocaeli earthquake, because it was observed that soils that had liquefied did not meet the Chinese criteria for soils susceptible to liquefaction. Their findings include that:
1) Young, shallow, non-plastic silts and clayey silts of low plasticity index (PI < 12) at \(w_c/LL > 0.85\) can liquefy under significant cyclic loading.

2) Clayey silts and silty clays of moderate plasticity (12 < PI < 18) at \(w_c/LL > 0.80\) can undergo liquefaction when shaken intensely for a significant number of cycles of loading.

3) There may be cases where sensitive soils with PI > 18 undergo severe strength loss as a result of earthquake-induced straining.

Fig. 2.4 presents the proposed fine-grained soil liquefaction susceptibility criteria based on Bray and Sancio (2006) data sets: a) the isotropically consolidated CTX tests, b) field observations and tests in Adapazari (Bray et al., 2004), c) Bray and Sancio (2006) reevaluation of the Bennett et al. (1998) field and index tests from Potrero Canyon for soils that liquefied during the 1994 Northridge earthquake, d) data in China from Wang (1979), and e) some recent observations in Taiwan after the 1999 Chi-Chi earthquake from Chu et al. (2004).

### 2.3 Cyclic Shear Stress and Frequency

In 1989, Ansal and Erken reported a study about the behavior of normally consolidated saturated clays under cyclic shear stresses using cyclic simple shear testing with different amplitudes and frequencies to estimate the response of the soil under earthquake loading. They reported a reduction in the shear strength under cyclic shear stresses as large as 65%. 
The researchers experimented with the cyclic behavior of one-dimensionally consolidated samples using different shear stress amplitudes at the same frequency. The variation of shear strain amplitude and pore pressure versus number of cycles was shown and it was demonstrated a critical level were repeated stress will not lead to failure (Larew and Leornards, 1962). This concept was also validated by Sangrey (1968), Sangrey et al. (1969), France and Sangrey (1977), and Sangrey et al. (1978).
In general, several researchers have reported that for higher cyclic stress levels, large strains would develop and the accumulation of pore pressures would lead to failure (Sangrey et al., 1969, 1978; Sangrey and France, 1980; Koutsoftas, 1978; Matsui et al., 1980).

From the cyclic tests made, the critical cyclic shear stress ratio is approximately equal to 50% of the static shear strength (Ansal and Erken, 1989). However, this result does not compare with others reported by Castro and Christian (1976), Andersen et al. (1980), and Koutsoftas (1978) where the loss in shear strength is not great for larger cyclic strains. Conversely, Thiers and Seed (1969), Taylor and Bacchus (1969), and Lee and Focht (1976), showed a significant decrease in the shear strength if the cyclic strain is large.

Ansal and Erken (1989) also showed that if the number of cycles is small, the applied cyclic stress, even if it is larger than the critical level, would not cause greater harm. Fig. 2.5 shows the effect of number of cycles from tests conducted at 0.1 Hz.

Ansal and Erken (1989) research indicates that the effect of frequency should be taken into account during earthquake motion because under different frequencies the effect of rate of loading is greater during initial cycles. Although, Matsui et al. (1980) and Proctor and Khaffaf (1984) observed no significant variation in the effect of frequency relative to the number of cycles, Yasuhara et al. (1982) reported no significant effect of frequency on cyclic undrained strength.
2.4 Overconsolidation Ratio (OCR)

In 1989, Azzous et al developed the apparent overconsolidation ratio (AOCR) hypothesis where initially cycled NC clays subjected to undrained monotonic shearing acted as overconsolidated clays. This framework led to predictions on cyclic shear behavior. All tests were run with Boston Blue Clays using two-way symmetric cyclic undrained direct simple shear (DSS) with initial OCR as high as 2.

The basis of their predictions of undrained stress-strain-strength behavior was related to the effective stress state prior to the undrained shearing, and the maximum past pressure of the soil on the AOCR. Their conclusions are comparable with other investigations with different types of clays and shearing modes (Matsui and Abe, 1981; Castro and Christian, 1976; Koutsoftas, 1978, Ortigosa et al., 1983).
The AOCR develops after the undrained cyclic shearing is applied on the NC sample, and it increases with the number of cycles. The undrained strength is replaced by the monotonic undrained strength at the actual AOCR, and the relationship between AORC and the number of cycles is what would determine the behavior of overconsolidated clays from results of tests on normally consolidated samples.

The AOCR hypothesis measured in the experiments with Boston Blue clay estimates number of cycles to failure with maximum error of 15%, and cycle shear strain with a maximum error of 60% (Azzous et al., 1989).

### 2.5 Initial Static Shear

In 1996, G. Lefebvre and P. Pfendler performed DSS tests for different values of initial static undrained shear stress on soft clay samples reconsolidated in the laboratory to an overconsolidation ratio (OCR) of 2.2 to investigate its effects before cyclic loading. They observed that this static shear decreases the cyclic resistance, but increases the total undrained shear resistance in soft clays with the rate of loading (Ishihara et al., 1983; Lefebvre and LeBoeuf 1987; Dobry and Vucetic, 1987). Hence, an initial static shear stress generally decreases the cyclic resistance (Seed and Chan, 1966; Goulois et al., 1985; Zimmie and Lien 1986; Andersen, 1988), but may increase the total shear strength combining the static and cyclic shear stresses (Ishihara et al., 1983).

Lefebvre and Pfendler (1996) developed curves demonstrating that the shear strength of intact soft clay degrades rapidly with the number of cycles when there is
no initial static shear stress, but it is compensated by a higher-strength mobilization due to the high strain rate associated with cyclic loading. Also, that the cyclic resistance decreases with increasing initial static shear stress, but a lesser degradation occurs with the number of cycles. Their DSS tests confirmed the increase in \((S_u)\) with strain rate similar to triaxial tests that have been done before (Lefebvre and LeBoeuf, 1987). They showed that at 12 cycles the application of an undrained static shear stress before cycling loading increased the total resistance by 30%, due to the strain rate effect associated with cyclic loading, and to the progressive dissipation of stress and strain reversals as the static shear stress is increased.

### 2.6 Boulanger and Idriss Analytical Procedure

In 2004, Boulanger and Idriss proposed an analytical procedure to evaluate the potential of cyclic failures of clays during earthquake ground motion. They established that sands and clays are different when it comes to predictions of potential strains and loss of strength during earthquake shaking. Moreover, when it comes to low-plasticity silts and clays, it may be necessary to distinguish a “sand-like” or “clay-like” behavior. Thus, they named “cyclic failure” to the onset of high excess pore water pressures and large strains during undrained cyclic loading of clay-like soils.

Atterberg Limits have been used to distinguish between sand-like and clay-like behavior. For engineering practice, it is recommended that fine-grained soils be considered clay-like if they have PI ≥ 7; intermediate if they have 3 < PI < 7; and sand-like if they have PI ≤ 3.
Boulanger and Idriss (2004) determined that the cyclic strength of saturated clays can be expressed as a function of the clay's undrained monotonic shear strength. Fig. 2.6 shows the results for different natural clays with OCR's of 1 to 4.

These authors also stated the effect of initial static shear stress in the cyclic resistance of clayey soils (Seed and Chan, 1966; Goulois et al., 1985; and Andersen et al., 1988) where the static sustained stress and the cyclic stress were both normalized by the soil's monotonic $S_u$. Results show that the cyclic strength decreases with increasing static sustained stress.

They also developed a $K_\alpha$ correction factor from laboratory test data to represent the effects of an initial static shear stress on the cyclic resistance of clays relative to the static shear stress normalized by the undrained shear strength ($\tau_s/S_u$), because they assumed that in seismic design most clay–like soils would have enough time to consolidate under the sustained loading of an embankment or any structure prior an earthquake. They developed the following equation:

$$K_\alpha = 1.344 - \frac{0.344}{\left(1 - \frac{\tau_s}{S_u}\right)^{0.638}}$$

(Eq. 2.6)

The $K_\alpha$ results for the Drammen clay with consolidation under the static shear stress that can be seen in Fig. 2.7 and 2.8 show that the $S_u$ of clay generally increases when it is consolidated under a sustained static shear stress.
Fig. 2.6 Cyclic strength ratios for uniform cyclic loading of five saturated clays: (a) Drammen clay with OCR of 1 and 4, (b) Boston Blue clay with OCR of 1, 1.38, and 2, (c) Cloverdale clay with OCR of 1, (d) St. Alban clay with OCR of 2.2, and (e) Itsukaichi clay with OCR of 1 (from Boulanger and Idriss, 2004).
Fig. 2. 7 $K_\alpha$ versus $(\tau_s/S_u)_{\alpha=0}$ relations for clays based on published data by Goulois et al. (1985), Andersen et al. (1988), and Lefebvre and Pfendler (1996). Note that specimens were not consolidated under the applied static shear stresses, except as otherwise labeled (from Boulanger and Idriss, 2004).

Fig. 2. 8 Derived $K_\alpha$ versus $(\tau_s/S_u)_{\alpha=0}$ relation for clay-like soil consolidated under the static shear stress and the results for NC Drammen clay by Goulois et al. (1985) (from Boulanger and Idriss, 2004).
Boulanger and Idriss used the Seed-Idriss (1971) simplified procedure to estimate the in situ cyclic stress during earthquake loading (Eq. 2.1). They used a MSF to adjust the CSR or the CRR to a common earthquake magnitude (M), conventionally taken as 7.5. They defined it as:

\[
MSF = \frac{\text{CRR}_M}{\text{CRR}_{M=7.5}} \tag{Eq. 2.7}
\]

Boulanger and Idriss (2004) developed a MSF relation for clay-like soils computing the limiting values for ½ cycle at the peak stress (Eq. 8). Fig. 2.9 shows the MSF relationship along with the relationship developed by Idriss (1999) for sands.

\[
MSF = 1.12 \cdot \exp\left(\frac{M}{4}\right) + 0.828, \quad MSF \leq 1.13 \tag{Eq. 2.8}
\]

**Fig. 2.9** MSF for converting a cyclic stress ratio to the equivalent cyclic stress ratio for an Mw=7.5 earthquake (from Boulanger and Idriss, 2004).
Boulanger and Idriss (2004) presented three approaches to evaluate the cyclic strength of clay-like fine-grained soils: 1) through cyclic laboratory testing; 2) by estimating the soil’s monotonic ($S_u$) as a ratio by empirical correlations; and 3) by empirical estimations of the CRR based on the stress history profile.

For the purpose of this paper just the second approach (estimating CRR from the empirical $S_u$ profile) is going to be noted. The relationship for the CRR of clay-like soils in $M=7.5$ earthquakes can be estimated as:

$$CRR_{M=7.5} = 0.8 \cdot \frac{S_u}{\sigma'_{vc}} \cdot K_\alpha \quad (Eq. 2.9)$$

Cyclic failure in clays does not necessarily imply a major risk, but the evaluation of potential deformation, and this is related to the soil’s sensitivity which encompasses it’s liquidity index (LI) and effective consolidation stress. The residual strength will increase and potential strains will decrease with decreasing LI (or $w_c$) or increasing OCR (Boulanger and Idriss, 2004).

2.7 Microfabric of Clayey Soils

In 2006, Gratchev et al. conducted a study of the undrained response of normally consolidated clayey soils to cyclic loading by means of a ring-shear apparatus and scanning electron microscope (SEM) (Osipov et al., 1984) using artificial clay-sand mixtures and natural clayey soils collected from landslides induced by earthquakes. Their study revealed that the plasticity is an important aspect on the liquefaction resistance of the soil and that it is strongly related to certain particles arrangements.
Some previous studies have found that the liquefaction resistance decreased with increasing plasticity (Prakash and Sandoval, 1992; Boulanger et al., 1998). Others, on the contrary, have suggested that an increase in plasticity leads to a higher resistance to liquefaction (Ishihara, 1993; Hyodo et al., 1999; Perlea et al., 1999).

Gratchev et al. (2006) found that adding 7% bentonite to clean sand reduced the liquefaction resistance with failure at lower number of cycles than the clean sand only, but as the bentonite content was increased the resistance to liquefaction also increased concomitantly with the number of cycles. 15% of Bentonite proved to be resistant to liquefaction, but when mixing the clean sand with 15% kaolin resistance dropped very rapidly, triggering liquefaction after two cycles; and when 15% illite was added to the clean sand, liquefaction was triggered, but somewhat higher than with kaolin. These results are plotted in Fig. 2.10 and 2.11. They observed that besides the clay content, the mineralogy also affects liquefaction resistance.

**Fig. 2.10** Results of ring-shear tests on mixtures of sand with kaolin, illite and bentonite plotted as Plasticity Index (PI) and cyclic stress ratio $CSR_{50}$ for the 50$^{th}$ cycle of loading (the numbers next to the marks denote clay content of total weight in %) (from Gratchev et al., 2006).
Fig. 2.11 Results of ring-shear tests plotted as clay against both pore water pressure ratio \( r_{u50} \) and cyclic stress ratio \( CSR_{50} \) for the 50th cycle of loading (from Gratchev et al., 2006).

They concluded that when clay forms open microfabrics bonding to sand particles, it creates low strength connectors prone to liquefaction, while more compact microfabrics with a dominant clay matrix correlates with higher resistance.

Similar results were found when they tested natural soils sampled from landslides produced by the M=6.8 Niigata, Japan Earthquake in 2004. They concluded that the amount and distribution of clay in soil is what determines if it is prone to liquefaction.

2.8 Finite Element Analysis

In recent years, finite element analysis (FEA) has been a numerical technique (based on computer models) commonly developed in geotechnical engineering for finding approximate solutions to actual problems because its ease on handling various types of materials, geometries and boundaries, and showing the distribution of
stresses and displacements. However, this method does not necessarily reveal how 
stresses are influenced by material properties and geometric features, or errors in the 
input data may throw incorrect results. It should be supplemented by experimental 
analysis as possible (Roylance, 2001).

The approached developed herein is based on assumptions related to specific 
parameters and properties of the soil, the subsoil layering, the loading and boundary 
conditions of a clayey foundation in order to determine its behavior regarding 
deformations and stability, and what would be the outcome performance under the 
influence of certain constructed embankment characteristics and the occurrence of 
earthquake loading.

Plaxis is the FEA software used to generate the soil models. This software code 
was developed in 1987, after research studies were done about the Dutch 
Oosterschelde dam at the Technical University of Delft in the Netherlands; first 
enabling elastic-plastic calculations, and then axi-symmetric problems. The software 
develops four stages: Inputs, calculations, outputs and curve plots.

Plaxis is intended to provide a tool for practical analysis of nonlinear finite 
element computations. It is commonly used by geotechnical engineers to develop soil 
models that simulate soil behavior. However, modeling geotechnical problems by 
means of finite element methods involves some inevitable numerical errors, thus its 
accuracy depends on the user regarding how the models are developed, the 
understanding of the models limitations, the parameters selection, and how the 
results are interpreted.
Plaxis has the provision for plane strain stress-displacement as well as safety factor analysis of embankments founded on deposit having any complex soil and pore water pressure conditions. Analysis can be done based on a number of options available, e.g., type of element, coarse mesh or fine mesh, soil models such as Mohr-Coulomb model (MC), Soft Soil Creep model (SSC), Hardening Soil model (HS), etc.

2.9 Limit Equilibrium Analysis

Limit equilibrium analysis methods are conventionally used to determine the equilibrium of a soil mass that may slide down under its own weight (gravity) by the ratio of the resisting forces over the driving forces, defining the FS that it is expressed as follows:

\[ FS = \frac{S}{\tau} \]  

(Eq. 2.10)

where \( S \) is the available shear strength and \( \tau \) is the equilibrium shear stress.

When \( \tau \) is less than \( S \), the FS is greater than 1.0. This means that a stable condition exists; therefore the slope is stable and in equilibrium. When \( \tau = S \), the FS is equal to 1.0. That is, the threshold of critical condition; therefore a failure could be imminent. But, when \( \tau \) is greater than \( S \), the FS is less than 1.0 representing an unstable condition; hence the slope is not stable.

Several procedures of limit equilibrium have been developed using specific assumptions to make the problem statically determine, but they all use the same
A summary of these methods is presented in Table 2.3.

All limit equilibrium methods are capable of expressing soil strength using the Mohr-Coulomb equation in terms of total or effective stress. The equilibrium shear stress is equal to the factorized available shear strength (Eq. 2.11).

\[
\tau = \frac{c' + \sigma' \tan \phi'}{F} = \frac{S_u}{F} \quad \text{(Eq. 2.11)}
\]

where \(c'\) and \(\phi'\) represent the shear strength parameters in terms of effective stresses \((\sigma')\), \(S_u\) is the undrained shear strength, used in short-term conditions in clayey soils, and \(F\) is the factor by which the strength is being reduced.

The available shear strength of any soil profile is dependent of the properties of the soil and the effective normal stresses; while the mobilized shear stress (available shear strength divided by the FS) is dependent of the eternal forces acting on it.

The use of computer programs is a widespread method to develop limit equilibrium analysis for complex and sophisticated scenarios, fast and simple. They can take into account different soil geometries, stratigraphy, shear strength, pore water pressures, and external loads. Moreover, the slip surface with the lowest FS can be found, and plots of the results can be generated using whatever method specified that is contained in the program.
Table 2.3 Summary of Limit Equilibrium methods (after Duncan and Wright, 2005)

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Assumptions</th>
<th>Equilibrium equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infinite Slope</td>
<td>Infinite extent slope. Slip surface parallel to slope face.</td>
<td>𝜎 Forces perpendicular and parallel to slope.</td>
</tr>
<tr>
<td>Logarithmic Spiral</td>
<td>The slip surface is a logarithmic spiral.</td>
<td>𝜎 Moments about center of spiral.</td>
</tr>
<tr>
<td>Swedish circle (Ø = 0)</td>
<td>Circular slip surface. Friction angle = 0.</td>
<td>𝜎 Moments about center of circle.</td>
</tr>
<tr>
<td>Ordinary Method of Slices</td>
<td>Circular slip surface. Forces and sides of the slide are neglected.</td>
<td>𝜎 Moments about center of circle.</td>
</tr>
<tr>
<td>Simplified Bishop</td>
<td>Circular slip surface. Forces on the sides of the slide are horizontal.</td>
<td>𝜎 Forces in vertical direction.</td>
</tr>
<tr>
<td>Force Equilibrium</td>
<td>Assumes inclination of the interslice forces. Assumptions vary with procedure.</td>
<td>𝜎 Forces in horizontal and vertical direction.</td>
</tr>
<tr>
<td>Spencer</td>
<td>Parallel interslice forces. The normal force (N) acts at the center of the base of the slide.</td>
<td>𝜎 Forces in horizontal and vertical direction.</td>
</tr>
<tr>
<td>Morgenstern and Price</td>
<td>Interslice shear force is related to interslice normal force by ( X = \lambda f(x)E ), the normal force (N) acts at the center of the base of the slide (typically).</td>
<td>𝜎 Forces in horizontal and vertical direction.</td>
</tr>
<tr>
<td>Chen and Morgenstern</td>
<td>Interslice shear force is related to interslice normal force by ( X = [\lambda f(x) + f_0(x)]E ); the normal force (N) acts at the center of the base of the slide (typically).</td>
<td>𝜎 Forces in horizontal and vertical direction.</td>
</tr>
<tr>
<td>Sarma</td>
<td>Interslice shear force is related to the interslice shear strength, ( S_u ) by ( X = \lambda f(x)S_u ). The normal force (N) acts at the center of the base of the slide (typically).</td>
<td>𝜎 Forces in horizontal and vertical direction.</td>
</tr>
</tbody>
</table>

Slide is the Rocscience Inc. software, created since 1996, capable of analyzing different types of earth structures, utilized herein to develop the limit equilibrium analysis of embankments constructed over clay soil under certain conditions to obtain
the FS by which the shear strength must be reduced to produce failure. This type of
analysis provides means of estimating the effects of earthquake loading in this type of
soils.

Several soil profiles with varying parameters were generated in Slide. The
details of their characterization and type of analysis are well developed in Chapter 3.
The profiles are the same as those generated in Plaxis in order to make a comparison
between both software results.
CHAPTER 3
DEVELOPMENT OF SUBSURFACE MODELS

Soil models were created in Plaxis defined by 2 layers, a soft clay deposit below a desiccated stiff clay layer (crust), underneath an embankment. Each layer has a specific unit weight ($\gamma$), cohesion ($c'$), and friction angle ($\phi'$), and each model has a specific crust thickness, overconsolidation pressure ($\sigma'_p$), ($c/p)_{NC}$ ratio and height of embankment. The soil profiles are 180 ft wide and 72 ft deep total.

To determine the static $S_u$ of a clayey soil as a function of the in situ effective stress and the overconsolidation pressure, the following equation was utilized:

$$S_u = \left( \frac{c}{\gamma} \right)_{NC} \cdot \sigma'_{vo} \cdot OCR^{0.8} = \left( \frac{c}{\gamma} \right)_{NC} \cdot \sigma'_1 \cdot \left( \frac{\sigma'_p}{\sigma'_1} \right)^0.8$$  \hspace{1cm} (Eq. 3.1)

A Plaxis analysis was generated to obtain the effective stresses in the profiles beneath the embankments. In Plaxis, a hardening soil model was used requiring parameters such as: $\phi'$, $c'$, secant modulus ($E_{50}^{\text{ref}}$), tangent modulus ($E_{oed}^{\text{ref}}$), Young’s modulus for unload-reload ($E_{ur}^{\text{ref}}$), power for stress level dependency ($m$), reference pressure ($p_{\text{ref}}$), and $K_0$-value for normal consolidation ($K_0^{NC}$).

Values of effective normal stresses and shear stresses were obtained after Plaxis at many points throughout the soil profiles. Initial vertical stresses ($\sigma'_{vo}$), maximum past pressures ($\sigma'_p$), and major principal stresses ($\sigma'_1$) were calculated at each of these points to obtain the $S_u$. The following equations were used:
where $\gamma'$ is the effective unit weight and $h$ the depth at which point the stress is being measured.

\[\sigma'_{vo} = \gamma' h \quad \text{(Eq. 3.2)}\]

\[\sigma'_p = \sigma'_{vo} + \Delta\sigma \quad \text{(Eq. 3.3)}\]

where $\Delta\sigma$ is the overconsolidation stress assumed to be 500; 1000; 1500; or 2000 psf.

\[\sigma'_1 = \frac{\sigma_x + \sigma_y}{2} + \frac{1}{2} \sqrt{\left[\left(2\tau_{xy}\right)^2 + \left(\sigma_y - \sigma_x\right)^2\right]} \quad \text{(Eq. 3.4)}\]

where $\sigma_x$, $\sigma_y$, and $\tau_{xy}$ are the normal stresses and shear stress with respect of the rotation angle $\theta$. See Appendix A for more detail.

\[OCR^{0.8} = \left(\frac{\sigma'_p}{\sigma'_1}\right)^{0.8} = \left[\frac{\text{Max}(\sigma'_{vo} + \Delta\sigma, \sigma'_1)}{\sigma'_1}\right]^{0.8} \quad \text{(Eq. 3.5)}\]

Fig. 3.1 presents a typical soil model with crust thickness of 6 ft and embankment height of 10 ft. The $S_u$ values obtained by calculations were plotted against the coordinates $(x, y)$ at which they were calculated. Contour plots were developed showing the variation of the $S_u$ with depth as shown in Fig. 3.2. After the $S_u$ contour plots, the soil profiles were discretized based upon $S_u$ and $\phi' = 0$ or UU definition of undrained strength.
Fig. 3. 1 Typical soil model of a 6 ft crust thickness profile beneath a 10 ft embankment. The distributed B-B load is the representation of the $\sigma'_p$. Arrows pointing down represent the $\sigma'_p$ for the crust, and arrows pointing up represent the $\sigma'_p$ for the soft clay deposit. The A-A line represents the distributed load due to the embankment, but because the B-B distributed load is greater, it looks like a horizontal line.

Fig. 3. 2 Profile contour plot based upon $S_u$ and $\phi'=0$ for a 6 ft crust thickness profile underneath a 10 ft embankment.
3.1 Undrained Strength Model, φ'=0 or UU Definition of Strength

The undrained or UU strength is applicable in this study because in clay soils it is assumed that the rate of loading is greater than the rate at which pore water pressure may dissipate or there is no time for consolidation to occur during the loading period. It is also assumed that the change in total stress during construction does not affect the in situ $S_u$ (Ladd, 1971).

The $S_u$ (Eq. 3.1) is defined by three parameters:

1) The $\sigma'_1$ (Eq. 3.4)

The $\sigma'_1$ values are calculated after the initial effective stresses in the profiles beneath the embankments are obtained from the Plaxis analysis. A hardening soil model criterion was developed in Plaxis, which is a hyperbolic soil model formulated on the basis of hardening plasticity that differs with the Mohr-Coulomb model by the stiffness approach. It assumes isotropic conditions. Strains are calculated using a stress-dependent stiffness and a reference pressure. For both, the crust and the soft clay, the values used in Plaxis for soil stiffness are:

- $E_{50}^{ref}$, a reference stiffness modulus (secant modulus) corresponding to the reference stress ($p^{ref}$), determined from triaxial stress-strain curves for a mobilization of 50% of the peak shear strength, given by the following equation:

$$E_{50}^{ref} = 0.8 \cdot E_{oed}^{ref} \quad \text{(Eq. 3.6)}$$

- $E_{oed}^{ref}$, the tangent stiffness for primary oedometer loading:
where $C_{ce}$ is the compression index set to be 0.25.

- $m$, power for stress-level dependency of stiffness, equal to 1 for soft clays.
- $E_{ref}$ is the reference Young’s modulus for unloading and reloading corresponding to the reference pressure, equal to:

\[
E_{ref} = E_{oed} \cdot \frac{\ln 10}{C_{ce}}
\]  
(Eq. 3. 7)

where $C_{re}$ is the recompression index set to be 0.05.

- $p_{ref}$, reference pressure = 1atm $\approx$ 2,000 psf.
- $\nu_{ur}$, Poisson’s ratio for unloading-reloading. Realistic values of $\nu_{ur}$ are about 0.2.
- $K_{0}^{NC}$, $K_0$-value for normal consolidation, correlated to the friction angle as:

\[
K_{0}^{NC} = 1 - \sin \Phi
\]  
(Eq. 3. 9)

- $R_f$, failure ratio $q_f/q_u$ (derived from the Mohr-Coulomb failure criterion), which should be smaller than 1. $R_f = 0.9$ often is a suitable default setting.

2) Thee $(c/p)_{NC}$ ratio:

One of the most useful ways to obtain the $S_u$ is in terms of the $\tau_f/\sigma'_{vo}$ ratio for normally consolidated clays, also known as $(c/p)_{NC}$ ratio, which is one of the parameters in the parametric model. In this analysis, the $S_u$ was determined using
average values of \((c/p)_{NC}\) ratio to account for the stress induced anisotropy assumed in a perfectly homogeneous isotropic soil, and it varies between 0.20, 0.22, and 0.24.

3) The OCR (Eq. 3.5):

The OCR depends on \(\Delta \sigma\), which is also another parameter in the parametric model that varies between 500; 1,000; 1,500; and 2,000 psf.

Fig. 3.3 is a representation of Plaxis’s generated mesh after the load of the embankment is applied, and the triangles at which initial stresses are calculated.

![Deformed mesh of the baseline profile under a 10 ft embankment](image-url)
### 3.2 Parameters in Parametric Study

The assumed parameters for this parametric study are shown in Table 3.1.

**Table 3.1 Parameters of the parametric study**

<table>
<thead>
<tr>
<th>Crust Thickness (ft)</th>
<th>(c/p)_{NC}</th>
<th>Δσ (psf)</th>
<th>Height of embankment (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.20</td>
<td>500</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>0.22</td>
<td>1,000</td>
<td>15</td>
</tr>
<tr>
<td>12</td>
<td>0.24</td>
<td>1,500</td>
<td>20</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>2,000</td>
<td>25</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>30</td>
</tr>
</tbody>
</table>

To narrow the study, a baseline was chosen from which parameters were varied. The baseline for each embankment height is defined below. Table 3.2 shows the combination of parameters modeled for each embankment height, and thus, 40 models were created.

**Baseline:**

- \( \left( \frac{c}{\lambda p} \right)_{NC} = 0.22 \)
- Crust thickness = 6 ft
- \( \Delta \sigma = 1,500 \) psf
Table 3.2 Combination of parameters for the soil models

<table>
<thead>
<tr>
<th>(c/p)_{NC}</th>
<th>Crust thickness (ft)</th>
<th>∆σ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.22</td>
<td>6</td>
<td>1,500</td>
</tr>
<tr>
<td>0.20</td>
<td>6</td>
<td>1,500</td>
</tr>
<tr>
<td>0.22</td>
<td>2</td>
<td>1,500</td>
</tr>
<tr>
<td>0.22</td>
<td>6</td>
<td>500</td>
</tr>
<tr>
<td>0.22</td>
<td>6</td>
<td>1,000</td>
</tr>
<tr>
<td>0.22</td>
<td>6</td>
<td>2,000</td>
</tr>
<tr>
<td>0.22</td>
<td>12</td>
<td>1,500</td>
</tr>
<tr>
<td>0.24</td>
<td>6</td>
<td>1,500</td>
</tr>
</tbody>
</table>

3.3 Soil Models for Finite Element and Limit Equilibrium Analyses

Each soil profile was modeled using discretized profiles based on the undrained case. The variation of the $S_u$ with depth, as previously shown in Fig. 3.2, was defined differently for the stiff and the soft clay layers.

It is commonly seen soft clay deposits that have an upper clay layer highly overconsolidated due to desiccation. It is difficult to obtain representative engineering properties of clays preconsolidated by desiccation. Laboratory determination of strength, compressibility, and stress history properties are usually scattered and biased by the type of test used (Al-Layla, 1970; O’Neill and Reese, 1972). For this analysis the $S_u$ of the crust was assumed to be 1,000 psf at the top and decreasing until meeting the same $S_u$ value at the beginning of the soft clay layer. Conversely, the $S_u$ of
the soft clay profile was defined according to Skempton’s work concerned with understanding short-term failures involving soft clays (Skempton, 1945), which identify the surface crust and below the $S_u$ profile increasing with depth, and thus showing its proportionality to the increase in effective overburden stress as shown in Fig. 3.4. The $S_u$ contour plots developed after the Plaxis analysis present $S_u$ increasing with depth. However, due to the variability of the $S_u$ contour plots’ linearity some layers were assigned with an average $S_u$ value.

Fig. 3.4 Properties of post-glacial clay from near Malaben, Sweden (after Skempton, 1948)
A vertical-faced embankment was assumed. The depth of water table corresponds to the limit between the crust and the soft layers. Table 3.3 shows the soil properties assumed for modeling in finite element and limit equilibrium analyses.

**Table 3.3** Soil properties for modeling

<table>
<thead>
<tr>
<th>Soil parameters</th>
<th>Crust</th>
<th>Soft clay</th>
<th>Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ</td>
<td>105 pcf</td>
<td>100 pcf</td>
<td>125 pcf</td>
</tr>
<tr>
<td>c'</td>
<td>250 psf</td>
<td>0 psf</td>
<td>-</td>
</tr>
<tr>
<td>φ'</td>
<td>20°</td>
<td>26°</td>
<td>-</td>
</tr>
<tr>
<td>σ'(_p)(^{(1)})</td>
<td>σ'(_{vo}) + 5000 psf (highly overconsolidated),</td>
<td>σ'(_{vo}) + Δσ psf (varying stress increments)</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: The σ'\(_p\) was generated in Plaxis by applying and removing the loads, and then resetting displacements to zero before applying the load of the embankments.

### 3.3.1 Finite Element Modeling

In Plaxis, the same stiffness moduli were used in the discretized profiles as in the initial soil models. Fig. 3.5 shows a typical discretized model for a baseline with a 10 ft embankment.

### 3.3.2 Limit Equilibrium Modeling

Limit equilibrium analysis does not use soil stiffness. Fig. 3.6 shows a typical discretized model for a baseline with a 10 ft embankment in Slide.
Fig. 3.5 Typical discretized soil model of a baseline with a 10 ft embankment. The distributed B-B load is the representation of the $\sigma'_p$. Arrows pointing down represent the $\sigma'_p$ for the crust, and arrows pointing up represent the $\sigma'_p$ for the soft clay deposit. The A-A line represents the distributed load due to the embankment, but because the B-B distributed load is greater, it looks like a horizontal line.

Fig. 3.6 Typical discretized soil model of a baseline with a 10 ft embankment in Slide. The distributed load represents the load due to the embankment. The rectangle represents the grid that seeks for the slip surface with the lowest FS.
CHAPTER 4

FINITE ELEMENT AND LIMIT EQUILIBRIUM ANALYSES

Finite element and limit equilibrium analyses were performed for each of the discretized models (discussed in Chapter 3) to determine how much strength reduction they would undergo prior to failure, and if the profiles are prone to develop severe deformations (instability/failure) from strength reductions due to cyclic loading. The same discretized models were analyzed using both, the FEA software Plaxis, and the limit equilibrium analysis software Slide. Results between the finite element and limit equilibrium approaches were compared, and found to be similar.

The analyses were based on the undrained case where $\phi'=0$ and $c'=S_u$. It was found that tall, marginally stable embankments have a SRF in the range between 0.65 – 1.0, and short, stable embankments have a SRF in the range between 0.38 – 0.70.

4.1 Plaxis Analyses

A hardening soil model was developed to account for limiting states of stress defined by $c'$, and $\phi'$. The soil stiffness is stress dependent and is defined by triaxial loading stiffness ($E_{50}$), triaxial unloading stiffness ($E_{ur}$), oedometer stiffness ($E_{oed}$), and a power for stress-level dependency of stiffness (m). The values used for the stiffness moduli are the same values used for the initial models (as discussed in Chapter 3).

The discretized soil profiles were used as inputs in Plaxis to compute the FS. A phi-$c'$ reduction analysis was executed, where the strength parameters $\tan \phi'$ and $c'$ of
the soil are reduced incrementally until an imminent failure occurs (Nordal and Glaamen, 2004).

The strength reduction is controlled by the total multiplier ($\Sigma Msf$). This parameter increases in a step-by-step procedure until failure occurs. The FS is then defined as the value of $\Sigma Msf$ at failure.

$$\tau = \frac{\sigma' \tan \phi' + c'}{\Sigma Msf} \quad (Eq. 4.1)$$

The $\Sigma Msf$ is set to 1 at the start of a calculation to set all material strength to their unreduced values. In the phi-c reduction calculation, the $Msf$ is used to specify the increment of the strength reduction of the first calculation step. Therefore, the strength reduction factor (SRF) is the inverse of the $\Sigma Msf$ at failure.

After the first Plaxis analysis determined a SRF for a soil profile, the crust strength was multiplied by $\frac{1}{SRF}$. This procedure was repeated until there was no significant change in SRF because during cyclic loading, it is hypothesized that only the saturated soft clay will undergo strength reduction, not the stiff desiccated crust. Therefore, an iterative procedure was used to adjust the crust strength.

The SRFs were obtained from plots $\Sigma Msf$ vs. displacement. Values of SRF were determined for 0.5 ft of displacement and for asymptotic displacement. Collapse plots of SRF vs. embankment heights are shown in Fig. 4.1 to 4.6.

Some of the models did not reached the phi-c’ reduction stage because the stress induced by the embankments produced great deformations and instability within the foundation that leaded to failure.
4.2 Slide Analyses

The Slide analyses used the same discretized profiles that were used in Plaxis analyses to compute the critical slip circle with the lowest FS. However, limit equilibrium analysis does not use soil stiffness. The development of these profiles was discussed in Chapter 3. The material properties, the distributive loads (embankments), and groundwater table location were assigned as in Plaxis to determine the effect of the combined variables on the FS of the slope using the undrained case model ($\phi'=0$ and $c'=S_u$). The crust strength was incrementally increased in the same manner as it was used for the Plaxis analyses. Fig. 4.7 shows the critical slip circle for the baseline of a 10 ft embankment.

![Graph showing Plaxis SRF vs. embankment height at 0.5 ft of displacement for varying crust thicknesses](image)

**Fig. 4.1** Plaxis SRF vs. embankment height at 0.5 ft of displacement for varying crust thicknesses
**Fig. 4.2** Plaxis SRF vs. embankment height at 0.5 ft of displacement for varying $(c/p)_{NC}$ ratio

**Fig. 4.3** Plaxis SRF vs. embankment height at 0.5 ft of displacement for various maximum past pressures
**Fig. 4.4** Plaxis SRF vs. embankment height at asymptotic displacement for varying crust thicknesses

**Fig. 4.5** Plaxis SRF vs. embankment height at asymptotic displacement for varying $(c/p)_{NC}$ ratio
**Fig. 4.6** Plaxis SRF vs. embankment height at asymptotic displacement for various maximum past pressures

**Fig. 4.7** Critical slip circle for the baseline of a 10 ft embankment
The Morgenstern-Price procedure (Morgenstern and Price, 1965) was the limit equilibrium method chosen to analyze the soil models. It requires satisfying equilibrium of forces and moments acting on individual blocks. It considers only the moment equations of individual slices, and it assumes that the shear forces between slices are related to the normal force as using the following equation:

\[ X = \lambda f(x)E \]  

(Eq. 4. 2)

where \( X \) and \( E \) are the vertical and normal forces between slices, \( \lambda \) is an unknown scaling factor that is solved for as part of the unknowns, and \( f(x) \) is an assumed function that has prescribed values at each slide boundary (Duncan and Wright, 2005). The inter-slice force function may be: constant, half-sine, clipped-sine, trapezoid or specified. Fig. 4.8 shows a body diagram of a slice using the Morgenstern-Price method. Fig. 4.9 shows typical inter-slice functions (i.e. \( f(x) \)). In this project, the half-sine function was used in Slide for the inclination angles (\( \delta_i \)) of forces (\( E_i \)) acting between the blocks.

Fig. 4.8 Free body diagram of slice using the Morgenstern-Price method. \( W_i \) is the line of action of weight of block. \( M \) is the center of the \( i-th \) segment. \( N_i \) is the normal force acting at \( M \). \( \delta_i \) is the inclination of forces \( E_i \) acting between blocks.
The SRFs were defined by the inverse of the FS. Plots of SRF vs. embankment height are shown in Fig. 4.10 to 4.12.
Fig. 4. 11 Slide SRF vs. embankment height for varying \((c/p)_{NC}\) ratio

Fig. 4. 12 Slide SRF vs. embankment height for various maximum past pressures
4.3 Comparison Between Finite Element Analysis and Limit Equilibrium Analysis

A FEA uses a stress-strain model to calculate the deformations in the soil profile, thus the calculated FS is the overall expression of the stability based upon stress-strain behavior. A limit equilibrium analysis utilizes only the stresses at failure to sum forces and moments on an assumed slip surface within the soil profile, and it does not take into account the stress-strain relationship. For both analysis methods, a global equilibrium condition is satisfied using static equilibrium equations to find the FS.

Another difference between the two approaches is that FEA uses equations of equilibrium compatibility and constitutive relationships to correctly solve the statically indeterminate problems, whereas the limit equilibrium analysis requires assumptions to make the problem statically determine and to balance the number of equations and unknowns. Furthermore, in Plaxis it is possible to pick the $\Sigma M_{sf}$ at a desired displacement, unlike Slide that provides an overall FS for a critical slip surface at collapse.

A nearly circular failure was evident in the FEA, and assumed in the limit equilibrium analysis. The location of the critical circle is similar for both approaches, although it varied in depth. Results show that failure circles were deeper in Plaxis, maybe because Plaxis takes into account the foundation deformations, while Slide just analyses limiting soil strength.

Slide's SRFs for short embankments (10–15 ft) are closer to Plaxis’s SRFs at asymptotic displacement. Conversely, Slide’s SRFs for tall embankments (> 15 ft) are
closer to Plaxis’s SRFs at 0.5 ft displacement. This could be because higher embankments represent higher SRF; therefore they get closer to those at 0.5 ft of displacement, which are greater than those at asymptotic displacement. However, for the 30 ft embankments the SRFs from Plaxis and Slide are similar at 0.5 ft displacement, and also at asymptotic displacement.

The % differences between Plaxis and Slide regarding critical circle depth, and SRFs at 0.5 ft displacement and at asymptotic displacement are shown in Table 4.1 to 4.5. The failure circle depth and the SRFs % differences were calculated using Eq. 4.3 and Eq. 4.4 respectively. Positive values mean that Plaxis results are greater than Slide’s, negative values mean otherwise.

\[
\% = 100 \cdot \frac{Depth_{Plaxis} - Depth_{Slide}}{Depth_{Plaxis}} \quad \text{(Eq. 4.3)}
\]

\[
\% = 100 \cdot \frac{SRF_{Plaxis} - SRF_{Slide}}{SRF_{Plaxis}} \quad \text{(Eq. 4.4)}
\]

Soil profiles marked as “Failed” are the ones that did not reach the end of construction of the embankment due to severe deformations.

Comparing the SRF vs. embankment height plots between Plaxis and Slide, it is evident that for the same embankments that did not reach the end of construction in the Plaxis analysis, they also had FS at or below 1.0 in the slide analysis. Just a few cases where the Plaxis analysis resulted in SRFs really close to 1.0, SRFs from Slide were just above 1.0. Those cases are shown in Table 4.6.
Table 4. 1 Percentage difference between FEA and limit equilibrium analysis relative to the failure circle depth and SRF for soil profiles under a 10 ft embankment

<table>
<thead>
<tr>
<th>(c/p)_{NC}</th>
<th>Thickness (ft)</th>
<th>Δσ (psf)</th>
<th>Failure circle depth</th>
<th>SRF_{0.5}</th>
<th>SRF Asymptotic</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>6.00</td>
<td>1,500</td>
<td>22.5</td>
<td>20.2</td>
<td>1.9</td>
</tr>
<tr>
<td>0.24</td>
<td>6.00</td>
<td>1,500</td>
<td>9.8</td>
<td>24.0</td>
<td>1.5</td>
</tr>
<tr>
<td>0.22</td>
<td>2.00</td>
<td>1,500</td>
<td>15.0</td>
<td>9.9</td>
<td>0.7</td>
</tr>
<tr>
<td>0.22</td>
<td>12.00</td>
<td>1,500</td>
<td>15.0</td>
<td>37.8</td>
<td>8.4</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>500</td>
<td>3.3</td>
<td>22.2</td>
<td>-3.0</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,000</td>
<td>23.7</td>
<td>23.2</td>
<td>2.1</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,500</td>
<td>9.8</td>
<td>24.1</td>
<td>2.5</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>2,000</td>
<td>22.0</td>
<td>20.6</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 4. 2 Percentage difference between FEA and limit equilibrium analysis relative to the failure circle depth and SRF for soil profiles under a 15 ft embankment

<table>
<thead>
<tr>
<th>(c/p)_{NC}</th>
<th>Thickness (ft)</th>
<th>Δσ (psf)</th>
<th>Failure circle depth</th>
<th>SRF_{0.5}</th>
<th>SRF Asymptotic</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>6.00</td>
<td>1,500</td>
<td>12.5</td>
<td>6.3</td>
<td>-6.6</td>
</tr>
<tr>
<td>0.24</td>
<td>6.00</td>
<td>1,500</td>
<td>27.0</td>
<td>7.6</td>
<td>-6.6</td>
</tr>
<tr>
<td>0.22</td>
<td>2.00</td>
<td>1,500</td>
<td>42.1</td>
<td>-8.0</td>
<td>-9.6</td>
</tr>
<tr>
<td>0.22</td>
<td>12.00</td>
<td>1,500</td>
<td>18.8</td>
<td>19.9</td>
<td>-12.0</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>500</td>
<td>3.4</td>
<td>15.3</td>
<td>9.0</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,000</td>
<td>32.3</td>
<td>2.9</td>
<td>-9.7</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,500</td>
<td>31.3</td>
<td>9.6</td>
<td>-6.5</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>2,000</td>
<td>20.6</td>
<td>7.8</td>
<td>-5.6</td>
</tr>
</tbody>
</table>
Table 4. 3 Percentage difference between FEA and limit equilibrium analysis relative to the failure circle depth and SRF for soil profiles under a 20 ft embankment

| $(c/p)_N$ | Thickness (ft) | Δ$σ$ (psf) | Difference (%) | | |
|-----------|----------------|------------|---------------|---|
|           |                |            | Failure circle depth | SRF$_{0.5}$ | SRF (Asymptotic) |
| 0.20      | 6.00           | 1,500      | 10.3          | -5.4        | -7.9 |
| 0.24      | 6.00           | 1,500      | 32.3          | -4.5        | -7.3 |
| 0.22      | 2.00           | 1,500      | -             | Failed      | Failed |
| 0.22      | 12.00          | 1,500      | 30.0          | -7.9        | -7.8 |
| 0.22      | 6.00           | 500        | -             | Failed      | Failed |
| 0.22      | 6.00           | 1,000      | 30.0          | -10.1       | -11.7 |
| 0.22      | 6.00           | 1,500      | 32.3          | -15.7       | -19.0 |
| 0.22      | 6.00           | 2,000      | 45.2          | 0.0         | -5.8 |

Table 4. 4 Percentage difference between FEA and limit equilibrium analysis relative to the failure circle depth and SRF for soil profiles under a 25 ft embankment

| $(c/p)_N$ | Thickness (ft) | Δ$σ$ (psf) | Difference (%) | | |
|-----------|----------------|------------|---------------|---|
|           |                |            | Failure circle depth | SRF$_{0.5}$ | SRF (Asymptotic) |
| 0.20      | 6.00           | 1,500      | -             | Failed      | Failed |
| 0.24      | 6.00           | 1,500      | 33.3          | -8.3        | -10.0 |
| 0.22      | 2.00           | 1,500      | -             | Failed      | Failed |
| 0.22      | 12.00          | 1,500      | 25.0          | -6.8        | -10.2 |
| 0.22      | 6.00           | 500        | -             | Failed      | Failed |
| 0.22      | 6.00           | 1,000      | -             | Failed      | Failed |
| 0.22      | 6.00           | 1,500      | 60.7          | -6.0        | -6.0 |
| 0.22      | 6.00           | 2,000      | 56.0          | -4.7        | -7.3 |
Table 4.5 Percentage difference between FEA and limit equilibrium analysis relative to the failure circle depth and SRF for soil profiles under a 30 ft embankment

<table>
<thead>
<tr>
<th>(c/p)$_{NC}$</th>
<th>Thickness (ft)</th>
<th>Δσ (psf)</th>
<th>Failure circle depth</th>
<th>SRF$_{0.5}$</th>
<th>SRF (Asymptotic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>6.00</td>
<td>1,500</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.24</td>
<td>6.00</td>
<td>1,500</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.22</td>
<td>2.00</td>
<td>1,500</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.22</td>
<td>12.00</td>
<td>1,500</td>
<td>31.4</td>
<td>-10.1</td>
<td>-9.3</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>500</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,000</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>1,500</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>0.22</td>
<td>6.00</td>
<td>2,000</td>
<td>-</td>
<td>Failed</td>
<td>Failed</td>
</tr>
</tbody>
</table>

Because both analyses results are similar, an average SRF can be obtained from them. Fig. 4.13 to 4.15 show SRF vs. embankment height for varying crust thickness, (c/p)$_{NC}$ ratio and Δσ as an average from both FEA and limit equilibrium analyses.

Table 4.6 Soil models showing SRFs close to 1.0 in Plaxis, but above 1.0 in Slide

<table>
<thead>
<tr>
<th>(c/p)$_{NC}$</th>
<th>Embankment height (ft)</th>
<th>Crust thickness (ft)</th>
<th>Δσ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.22</td>
<td>20</td>
<td>6</td>
<td>1,000</td>
</tr>
<tr>
<td>0.24</td>
<td>25</td>
<td>6</td>
<td>1,500</td>
</tr>
<tr>
<td>0.22</td>
<td>25</td>
<td>6</td>
<td>1,500</td>
</tr>
<tr>
<td>0.22</td>
<td>30</td>
<td>12</td>
<td>1,500</td>
</tr>
</tbody>
</table>
**Fig. 4.13** Average SRF vs. embankment height from Plaxis and Slide for varying crust thicknesses.

**Fig. 4.14** Average SRF vs. embankment height from Plaxis and Slide for varying \((c/p)_{NC}\) ratio.
Fig. 4. 15 Average SRF vs. embankment height from Plaxis and Slide for various maximum past pressures
I. M. Idriss and R. W. Boulanger in their 2008 monograph “Soil Liquefaction during Earthquake” discussed the behavior of saturated clays and plastic silts during earthquakes. They mentioned that ground failures in clay and plastic silt deposits are less common than in saturated sand deposits, but they have been observed during earthquakes such as in the 1999 Chi-Chi Earthquake in Taiwan (Chu et al., 2004, 2007; Boulanger and Idriss, 2004), in the 1985 Michoacan earthquake in Mexico (Mendoza and Auvinet, 1988; Zeevaert, 1991), in the 1999 Kocaeli earthquake in Turkey (Bray et al., 2004; Martin et al., 2004, Yilmaz et al., 2004) and in the 2001 Bhuj earthquake in India (Bardet et al., 2002); and also that clays type of stress-strain behavior could lead to significant ground deformation during cyclic loading that can be difficult to differentiate from ground displacements caused by liquefaction of sands.

Idriss and Boulanger addressed the cyclic strength of clays and plastic silts and the consequences of cyclic softening in clay-like fine grained soils. Based on their approach predictions have been made as to whether each embankment model will soften, or collapse under a range of earthquake loads. In the Idriss and Boulanger approach the cyclic resistance ratio (CRR) is the softened (reduced) undrained shear strength divided by the vertical effective stress, and the cyclic stress ratio (CSR) is the earthquake induced shear stress divided by the vertical effective stress. Seismic softening occurs when the CSR is greater than the CRR. Collapse will occur when reduced undrained shear strength is less than the in situ shear stress.
5.1 The Cyclic Stress Ratios for Embankment Models

The CSR was calculated using the following equation:

\[ CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \left( \frac{a_{\text{max}}}{g} \right) \left( \frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d \]  
(Eq. 5.1)

The 0.65 factor is the reference stress level (i.e., the percentage of the peak shear stress) taken arbitrary to represent the number of equivalent uniform loading cycles produced by an earthquake. The ratio between the peak horizontal acceleration and the acceleration of gravity \((a_{\text{max}}/g)\) is the stress induced by an earthquake, which was assumed to be 0.1, 0.2, and 0.4 g. The \(\sigma_{vo}\) and \(\sigma'_{vo}\) are the total and effective vertical stresses from Plaxis analysis. The \(r_d\) factor is the stress reduction coefficient that accounts for the flexibility of the soil profile, and it was calculated with Eq. 5.2.

\[ r_d = \frac{1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2} \]  
(Eq. 5.2)

where \(z\) is the depth beneath the ground surface, or the depth beneath the top of the embankment, in meters, depending on the point at which it was calculated.

5.2 The Cyclic Resistance Ratios for Embankment Models

The CRR was calculated for 4 different magnitude earthquakes, 5.5; 6.5; 7.5; and 8.5 using the following equation:

\[ CRR_M = 0.8 \cdot \frac{S_u}{\sigma'_{vo}} \cdot MSF \cdot K_\alpha \]  
(Eq. 5.3)
The 0.8 factor was estimated by Boulanger and Idriss (2004) from triaxial and DSS testing $\tau_{cyc}/S_u$ ratios, and an adjustment factor for the effects of two-directional cyclic loading ($C_{2D}$). The $S_u$ is the undrained strength from the discretized models. The $\sigma'_v$ is the effective stress from Plaxis analysis. The MSF factor that accounts for the average number of equivalent uniform loading cycles depending on the earthquake magnitude, distance and site conditions was calculated with Eq. 5.4. The $K_\alpha$ factor that accounts for the effects of an initial static shear stress was calculated with Eq. 5.5.

\[
MSF = 1.12 \cdot \exp\left(\frac{-M}{4}\right) + 0.828
\]  
(Eq. 5.4)

\[
K_\alpha = 1.344 - \frac{0.344}{\left(1 - \frac{T_s}{S_u}\right)^{0.638}}
\]  
(Eq. 5.5)

5.3 The Factor of Safety against Softening

The FS against softening is defined as the ratio between the CRR and the CSR for a given magnitude earthquake. FS were calculated (Eq. 5.6) for each magnitude earthquakes: 5.5, 6.5, 7.5, and 8.5, and peak ground acceleration: 0.1, 0.2, and 0.4g. Whenever the FS is less than 1.0, softening will occur. Fig. 5.1 shows a typical FS contour plot of a baseline model with a 10 ft embankment.

\[
FS_{Soften} = \frac{CRR_M}{CSR}
\]  
(Eq. 5.6)
The FS vary according to the change in parameters, peak ground acceleration and magnitude earthquake. The contour plots show that the higher the $\Delta\sigma$, the bigger the crust thickness and the higher the c/p ratio, the higher the factor of safety. Also, they show that the greater the earthquake magnitude, the greater the peak acceleration, and the higher the embankment, the lower the factor of safety. Even though, greater c/p ratios represent higher FS; for greater c/p ratios with lower $\Delta\sigma$ or higher embankment, the FS is lower. Range of FS values for varying peak ground acceleration are shown in Table 5.1. See Appendix B for all contour plots.
Table 5.1 Range of values of FS against softening

<table>
<thead>
<tr>
<th>Embankment height (ft)</th>
<th>Factors of safety(^{(1)})</th>
<th>0.1g</th>
<th>0.2g</th>
<th>0.4g</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.1g</td>
<td>0.2g</td>
<td>0.4g</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>1.0 – 1.8</td>
<td>0.6 – 1.4</td>
<td>0.4 – 0.8</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>0.4 – 1.8</td>
<td>0.2 – 1.4</td>
<td>0.4 – 0.8</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>0.2 – 1.8</td>
<td>0.2 – 1.4</td>
<td>0.2 – 0.8</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>0.2 – 1.8</td>
<td>0.2 – 1.4</td>
<td>0.2 – 0.6</td>
</tr>
<tr>
<td>30(^{(2)})</td>
<td></td>
<td>0.2 – 1.8</td>
<td>0.2 – 1.4</td>
<td>0.2 – 0.8</td>
</tr>
</tbody>
</table>

Notes: 1. Ranges are slightly higher for c/p = 0.24 and crust thickness of 12 ft
2. For the 30 ft embankment just the 12 ft crust thickness is stable

5.4 The Factor of Safety against Collapse

The FS against collapse is the ratio of the normalized residual undrained strength and the normalized static shear stress. FS were calculated (Eq. 5.7) after each magnitude earthquakes: 5.5, 6.5, 7.5, and 8.5. Whenever the FS is less than 1.0, collapse will occur. Fig. 5.2 shows a typical FS contour plot of a baseline model with a 10 ft embankment. See Appendix B for all models.

\[
FS_{\text{collapse}} = \frac{0.8 \times \frac{S_u}{\sigma_v^*} \times MSF}{\frac{\tau_s}{\sigma_v^*}} = \frac{0.8 \times S_u \times MSF}{\tau_s} \quad (\text{Eq. 5.7})
\]

The \(S_u\) is the undrained strength from the discretized models. The MSF was calculated with Eq. 5.4. The 0.8 factor was estimated by Boulanger and Idriss (2004) as it was previously mentioned. The \(\tau_s\) is the in situ shear stress from Plaxis analysis.
The FS against collapse vary according to the change in parameters and magnitude earthquake. It represents the residual undrained strength in the soil after earthquake loading. Range of FS values are shown in Table 5.2. See Appendix B for all contour plots.

Table 5.2 Range of values of FS against collapse

<table>
<thead>
<tr>
<th>Embankment height (ft)</th>
<th>Factors of safety(^{(1)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1.2 – 2.5</td>
</tr>
<tr>
<td>15</td>
<td>1.0 – 1.8</td>
</tr>
<tr>
<td>20</td>
<td>1.0 – 1.8</td>
</tr>
<tr>
<td>25</td>
<td>1.0 – 1.8</td>
</tr>
<tr>
<td>30</td>
<td>1.0 – 1.6</td>
</tr>
</tbody>
</table>

Note: Ranges are slightly higher for c/p = 0.24 and crust thickness of 12 ft
5.5 Summary

Idriss and Boulanger predicted strength reductions between 0.77 and 0.89 along the failure surface. This is significantly less than the strength reductions required for failure found in Chapter 4. It is also significantly higher than laboratory strength reductions measured by Ansal and Erken (1989), Thiers and Seed (1969), Taylor and Bacchus (1969), and Lee and Focht (1976).

Deformations will occur in the soil after earthquake loading, but that does not imply an imminent failure/collapse of the embankments according to what is shown in the contour plots (see Appendix B).
CHAPTER 6

CONCLUSIONS

The purpose of this thesis was to determine how much strength reduction due to earthquake loading is developed within a clay profile beneath an embankment given the previous discussed strength parameters and site characteristics.

The predictions made herein were not verified with field trial embankments or case studies. Also, it is worth mentioning that not all factors that control shear strength can be represented through these methods, similarly, the characterization of the site, the geology, and the soil properties. The analysis is restricted to typical values of c/p ratio, $\sigma'_p$ (psf), crust thickness (ft), and height of embankment (ft).

The obtained results suggested that:

- Tall, marginally stable embankments have a SRF in the range between 0.65 – 1.0.
- Short, stable embankments have a SRF in the range between 0.38 – 0.70.
- FEA vs. limit equilibrium analysis are comparable even though they represent different approaches. According to these methods:
  - For short embankments, Slide’s SRFs are closer to Plaxis’s SRFs for asymptotic displacement.
  - For tall embankments, Slide’s SRFs are closer to Plaxis’s SRFs at 0.5 ft displacement.
  - For the 30 ft embankments the SRFs from Plaxis and Slide are similar at 0.5 ft and at asymptotic displacement.
• Idriss and Boulanger predicted FS against softening between 0.2 and 1.8 for both, tall and short embankments.

• Idriss and Boulanger predicted FS against collapse greater than 1.0 for both, tall and short embankments.

• Idriss and Boulanger’s strength reductions values (~ 0.80) vs. previous laboratory results are significantly lower.

• None of the constructed embankments will collapse according to Idriss and Boulanger’s method, because the residual strength will be enough to resist the stresses.

  Further research ought to provide new knowledge about clay behavior taking into account that engineering judgment will always play a crucial role.
REFERENCES


APPENDICES
APPENDIX A

Mohr Circle Stress Theory
From the Mohr Circle stress theory:

\[
\sigma'_1 = \frac{\sigma_x + \sigma_y}{2} + \frac{1}{2}\sqrt{(2\tau_{xy})^2 + (\sigma_y - \sigma_x)^2}
\]

(Eq. A.1)

where \(\sigma'_1\) is the major principal stress, \(\sigma_x\), \(\sigma_y\), and \(\tau_{xy}\) are the normal stresses and shear stress with respect of the rotation angle \(\theta\).

The major principal stress is determined by constructing a two-dimensional Mohr's circle because the normal stresses \(\sigma_x\), \(\sigma_y\), and the shear stress \(\tau_{xy}\) are known. In soil mechanics, normal stresses are considered positive when they are in compression, and shear stresses are considered positive when they rotate counterclockwise around the point being considered.

The Mohr’s circle of stress for a state of plane stress, or plane strain is created when two points are plot in the \(\sigma_n:\tau_n\) space corresponding to the known stress components on both perpendicular planes, i.e. A \((\sigma_y, \tau_{xy})\) and B\((\sigma_y, \tau_{xy})\) see Fig.A.1. Then, the points are connected by a straight line and meet the midpoint O of the \(\sigma_n\) axis. Finally, the Mohr’s circle is drawn with diameter \(\overline{AB}\) and centre at O.

The radius \(R\) of the circle is:

\[
R = \sqrt{\frac{1}{2} (\sigma_x - \sigma_y)^2 + \tau_{xy}^2}
\]

(Eq. A.2)

and the coordinates of its centre are:
\[ C = \left[ \frac{1}{2} (\sigma_x + \sigma_y), 0 \right] \]  \hfill (Eq. A. 3)

The Mohr’s circle intersects the \( \sigma_n \) axis at two end points called principal stresses; \( \sigma_1 \), the maximum normal stress \( (\sigma_{\text{max}}) \); and \( \sigma_2 \), the minimum normal stress \( (\sigma_{\text{min}}) \). \( \theta \) is the angle between the maximum normal stress and the \( x \)-axis. See Fig. A.1.

**Fig. A. 1** Mohr’s circle for plane stress and plane strain conditions.
APPENDIX B

FS contour graphs

(Profile name nomenclature: c/p-embankment height-crust thickness-overconsolidation pressure)
Fig. B. 1  FS for profile 0.20-10-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 2  FS for profile 0.20-10-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 3  FS for profile 0.20-10-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 4  FS for profile 0.20-10-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 5 FS for profile 0.22-10-2-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 6 FS for profile 0.22-10-2-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 7 FS for profile 0.22-10-2-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 8 FS for profile 0.22-10-2-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 9** FS for profile 0.22-10-6-500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 10** FS for profile 0.22-10-6-500 with 6.5 magnitude earthquake @0.1g
Fig. B. 11 FS for profile 0.22-10-6-500 with 7.5 magnitude earthquake @0.1g

Fig. B. 12 FS for profile 0.22-10-6-500 with 8.5 magnitude earthquake @0.1g
Fig. B. 13 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.1g

Fig. B. 14 FS for profile 0.22-10-6-1000 with 6.5 magnitude earthquake @0.1g
Fig. B. 15 FS for profile 0.22-10-6-1000 with 7.5 magnitude earthquake @0.1g

Fig. B. 16 FS for profile 0.22-10-6-1000 with 8.5 magnitude earthquake @0.1g
Fig. B. 17 FS for profile 0.22-10-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 18 FS for profile 0.22-10-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 19 FS for profile 0.22-10-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 20 FS for profile 0.22-10-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 21 FS for profile 0.22-10-6-2000 with 5.5 magnitude earthquake @0.1g

Fig. B. 22 FS for profile 0.22-10-6-2000 with 6.5 magnitude earthquake @0.1g
Fig. B. 23 FS for profile 0.22-10-6-2000 with 7.5 magnitude earthquake @0.1g

Fig. B. 24 FS for profile 0.22-10-6-2000 with 8.5 magnitude earthquake @0.1g
Fig. B. 25 FS for profile 0.22-10-12-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 26 FS for profile 0.22-10-12-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 27 FS for profile 0.22-10-12-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 28 FS for profile 0.22-10-12-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 29** FS for profile 0.24-10-6-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 30** FS for profile 0.24-10-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 31 FS for profile 0.24-10-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 32 FS for profile 0.24-10-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 33 FS for profile 0.20-10-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 34 FS for profile 0.20-10-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 35 FS for profile 0.20-10-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 36 FS for profile 0.20-10-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 37 FS for profile 0.22-10-2-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 38 FS for profile 0.22-10-2-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 39 FS for profile 0.22-10-2-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 40 FS for profile 0.22-10-2-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 41 FS for profile 0.22-10-6-500 with 5.5 magnitude earthquake @0.2g

Fig. B. 42 FS for profile 0.22-10-6-500 with 6.5 magnitude earthquake @0.2g
Fig. B. 43 FS for profile 0.22-10-6-500 with 7.5 magnitude earthquake @0.2g

Fig. B. 44 FS for profile 0.22-10-6-500 with 8.5 magnitude earthquake @0.2g
Fig. B. 45 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.2g

Fig. B. 46 FS for profile 0.22-10-6-1000 with 6.5 magnitude earthquake @0.2g
Fig. B. 47 FS for profile 0.22-10-6-1000 with 7.5 magnitude earthquake @0.2g

Fig. B. 48 FS for profile 0.22-10-6-1000 with 8.5 magnitude earthquake @0.2g
Fig. B. 49 FS for profile 0.22-10-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 50 FS for profile 0.22-10-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 51 FS for profile 0.22-10-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 52 FS for profile 0.22-10-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 53 FS for profile 0.22-10-6-2000 with 5.5 magnitude earthquake @0.2g

Fig. B. 54 FS for profile 0.22-10-6-2000 with 6.5 magnitude earthquake @0.2g
Fig. B. 55 FS for profile 0.22-10-6-2000 with 7.5 magnitude earthquake @0.2g

Fig. B. 56 FS for profile 0.22-10-6-2000 with 8.5 magnitude earthquake @0.2g
Fig. B. 57 FS for profile 0.22-10-12-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 58 FS for profile 0.22-10-12-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 59 FS for profile 0.22-10-12-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 60 FS for profile 0.22-10-12-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 61 FS for profile 0.24-10-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 62 FS for profile 0.24-10-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 63 FS for profile 0.24-10-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 64 FS for profile 0.24-10-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 65 FS for profile 0.20-10-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 66 FS for profile 0.20-10-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 67 FS for profile 0.20-10-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 68 FS for profile 0.20-10-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 69 FS for profile 0.22-10-2-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 70 FS for profile 0.22-10-2-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 71 FS for profile 0.22-10-2-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 72 FS for profile 0.22-10-2-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 73 FS for profile 0.22-10-6-500 with 5.5 magnitude earthquake @0.4g

Fig. B. 74 FS for profile 0.22-10-6-500 with 6.5 magnitude earthquake @0.4g
Fig. B. 75 FS for profile 0.22-10-6-500 with 7.5 magnitude earthquake @0.4g

Fig. B. 76 FS for profile 0.22-10-6-500 with 8.5 magnitude earthquake @0.4g
Fig. B. 77 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.4g

Fig. B. 78 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.4g
Fig. B. 79 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.4g

Fig. B. 80 FS for profile 0.22-10-6-1000 with 5.5 magnitude earthquake @0.4g
Fig. B. 81 FS for profile 0.22-10-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 82 FS for profile 0.22-10-6-1500 with 6.5 magnitude earthquake @0.4g
**Fig. B. 83** FS for profile 0.22-10-6-1500 with 7.5 magnitude earthquake @0.4g

**Fig. B. 84** FS for profile 0.22-10-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 85 FS for profile 0.22-10-6-2000 with 5.5 magnitude earthquake @0.4g

Fig. B. 86 FS for profile 0.22-10-6-2000 with 6.5 magnitude earthquake @0.4g
Fig. B. 87 FS for profile 0.22-10-6-2000 with 7.5 magnitude earthquake @0.4g

Fig. B. 88 FS for profile 0.22-10-6-2000 with 8.5 magnitude earthquake @0.4g
**Fig. B. 89** FS for profile 0.22-10-12-1500 with 5.5 magnitude earthquake @0.4g

**Fig. B. 90** FS for profile 0.22-10-12-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 91 FS for profile 0.22-10-12-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 92 FS for profile 0.22-10-12-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 93 FS for profile 0.24-10-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 94 FS for profile 0.24-10-6-1500 with 6.5 magnitude earthquake @0.4g
**Fig. B. 95** FS for profile 0.24-10-6-1500 with 7.5 magnitude earthquake @0.4g

**Fig. B. 96** FS for profile 0.24-10-6-1500 with 8.5 magnitude earthquake @0.4g
**Fig. B. 97** FS for collapse for profile 0.20-10-6-1500 with 5.5 magnitude earthquake

**Fig. B. 98** FS for collapse for profile 0.20-10-6-1500 with 6.5 magnitude earthquake
Fig. B. 99 FS for collapse for profile 0.20-10-6-1500 with 7.5 magnitude earthquake

Fig. B. 100 FS for collapse for profile 0.20-10-6-1500 with 8.5 magnitude earthquake
Fig. B. 101 FS for collapse for profile 0.22-10-2-1500 with 5.5 magnitude earthquake

Fig. B. 102 FS for collapse for profile 0.22-10-2-1500 with 6.5 magnitude earthquake
Fig. B. 103 FS for collapse for profile 0.22-10-2-1500 with 7.5 magnitude earthquake

Fig. B. 104 FS for collapse for profile 0.22-10-2-1500 with 8.5 magnitude earthquake
Fig. B. 105 FS for collapse for profile 0.22-10-6-500 with 5.5 magnitude earthquake

Fig. B. 106 FS for collapse for profile 0.22-10-6-500 with 6.5 magnitude earthquake
Fig. B. 107 FS for collapse for profile 0.22-10-6-500 with 7.5 magnitude earthquake

Fig. B. 108 FS for collapse for profile 0.22-10-6-500 with 8.5 magnitude earthquake
Fig. B. 109 FS for collapse for profile 0.22-10-6-1000 with 5.5 magnitude earthquake

Fig. B. 110 FS for collapse for profile 0.22-10-6-1000 with 6.5 magnitude earthquake
Fig. B. 111 FS for collapse for profile 0.22-10-6-1000 with 7.5 magnitude earthquake

Fig. B. 112 FS for collapse for profile 0.22-10-6-1000 with 8.5 magnitude earthquake
Fig. B. 113 FS for collapse for profile 0.22-10-6-1500 with 5.5 magnitude earthquake

Fig. B. 114 FS for collapse for profile 0.22-10-6-1500 with 6.5 magnitude earthquake
Fig. B. 115 FS for collapse for profile 0.22-10-6-1500 with 7.5 magnitude earthquake

Fig. B. 116 FS for collapse for profile 0.22-10-6-1500 with 8.5 magnitude earthquake
**Fig. B. 117** FS for collapse for profile 0.22-10-6-2000 with 5.5 magnitude earthquake

**Fig. B. 118** FS for collapse for profile 0.22-10-6-2000 with 6.5 magnitude earthquake
**Fig. B. 119** FS for collapse for profile 0.22-10-6-2000 with 7.5 magnitude earthquake

**Fig. B. 120** FS for collapse for profile 0.22-10-6-2000 with 8.5 magnitude earthquake
Fig. B. 121 FS for collapse for profile 0.22-10-12-1500 with 5.5 magnitude earthquake

Fig. B. 122 FS for collapse for profile 0.22-10-12-1500 with 6.5 magnitude earthquake
Fig. B. 123 FS for collapse for profile 0.22-10-12-1500 with 7.5 magnitude earthquake

Fig. B. 124 FS for collapse for profile 0.22-10-12-1500 with 8.5 magnitude earthquake
**Fig. B. 125** FS for collapse for profile 0.24-10-6-1500 with 5.5 magnitude earthquake

**Fig. B. 126** FS for collapse for profile 0.24-10-6-1500 with 6.5 magnitude earthquake
Fig. B. 127 FS for collapse for profile 0.24-10-6-1500 with 7.5 magnitude earthquake

Fig. B. 128 FS for collapse for profile 0.24-10-6-1500 with 8.5 magnitude earthquake
Fig. B. 129 FS for profile 0.20-15-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 130 FS for profile 0.20-15-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 131 FS for profile 0.20-15-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 132 FS for profile 0.20-15-6-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 133** FS for profile 0.22-15-2-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 134** FS for profile 0.22-15-2-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 135 FS for profile 0.22-15-2-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 136 FS for profile 0.22-15-2-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 137 FS for profile 0.22-15-6-500 with 5.5 magnitude earthquake @0.1g

Fig. B. 138 FS for profile 0.22-15-6-500 with 6.5 magnitude earthquake @0.1g
Fig. B. 139 FS for profile 0.22-15-6-500 with 7.5 magnitude earthquake @0.1g

Fig. B. 140 FS for profile 0.22-15-6-500 with 8.5 magnitude earthquake @0.1g
Fig. B. 141 FS for profile 0.22-15-6-1000 with 5.5 magnitude earthquake @0.1g

Fig. B. 142 FS for profile 0.22-15-6-1000 with 6.5 magnitude earthquake @0.1g
Fig. B. 143 FS for profile 0.22-15-6-1000 with 7.5 magnitude earthquake @0.1g

Fig. B. 144 FS for profile 0.22-15-6-1000 with 8.5 magnitude earthquake @0.1g
Fig. B. 145 FS for profile 0.22-15-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 146 FS for profile 0.22-15-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 147 FS for profile 0.22-15-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 148 FS for profile 0.22-15-6-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 149** FS for profile 0.22-15-6-2000 with 5.5 magnitude earthquake @0.1g

**Fig. B. 150** FS for profile 0.22-15-6-2000 with 6.5 magnitude earthquake @0.1g
Fig. B. 151 FS for profile 0.22-15-6-2000 with 7.5 magnitude earthquake @0.1g

Fig. B. 152 FS for profile 0.22-15-6-2000 with 8.5 magnitude earthquake @0.1g
Fig. B. 153 FS for profile 0.22-15-12-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 154 FS for profile 0.22-15-12-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 155 FS for profile 0.22-15-12-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 156 FS for profile 0.22-15-12-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 157** FS for profile 0.24-15-6-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 158** FS for profile 0.24-15-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 159 FS for profile 0.24-15-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 160 FS for profile 0.24-15-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 161 FS for profile 0.20-15-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 162 FS for profile 0.20-15-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 163 FS for profile 0.20-15-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 164 FS for profile 0.20-15-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 165 FS for profile 0.22-15-2-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 166 FS for profile 0.22-15-2-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 167 FS for profile 0.22-15-2-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 168 FS for profile 0.22-15-2-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 169 FS for profile 0.22-15-6-500 with 5.5 magnitude earthquake @0.2g

Fig. B. 170 FS for profile 0.22-15-6-500 with 6.5 magnitude earthquake @0.2g
Fig. B. 171 FS for profile 0.22-15-6-500 with 7.5 magnitude earthquake @0.2g

Fig. B. 172 FS for profile 0.22-15-6-500 with 8.5 magnitude earthquake @0.2g
Fig. B. 173 FS for profile 0.22-15-6-1000 with 5.5 magnitude earthquake @0.2g

Fig. B. 174 FS for profile 0.22-15-6-1000 with 6.5 magnitude earthquake @0.2g
Fig. B. 175 FS for profile 0.22-15-6-1000 with 7.5 magnitude earthquake @0.2g

Fig. B. 176 FS for profile 0.22-15-6-1000 with 8.5 magnitude earthquake @0.2g
Fig. B. 177 FS for profile 0.22-15-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 178 FS for profile 0.22-15-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 179 FS for profile 0.22-15-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 180 FS for profile 0.22-15-6-1500 with 8.5 magnitude earthquake @0.2g
**Fig. B.181** FS for profile 0.22-15-6-2000 with 5.5 magnitude earthquake @0.2g

**Fig. B.182** FS for profile 0.22-15-6-2000 with 6.5 magnitude earthquake @0.2g
Fig. B. 183 FS for profile 0.22-15-6-2000 with 7.5 magnitude earthquake @0.2g

Fig. B. 184 FS for profile 0.22-15-6-2000 with 8.5 magnitude earthquake @0.2g
Fig. B. 185 FS for profile 0.22-15-12-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 186 FS for profile 0.22-15-12-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 187 FS for profile 0.22-15-12-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 188 FS for profile 0.22-15-12-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 189 FS for profile 0.24-15-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 190 FS for profile 0.24-15-6-1500 with 6.5 magnitude earthquake @0.2g
**Fig. B. 191** FS for profile 0.24-15-6-1500 with 7.5 magnitude earthquake @0.2g

**Fig. B. 192** FS for profile 0.24-15-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 193 FS for profile 0.20-15-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 194 FS for profile 0.20-15-6-1500 with 6.5 magnitude earthquake @0.4g
**Fig. B. 195** FS for profile 0.20-15-6-1500 with 7.5 magnitude earthquake @0.4g

**Fig. B. 196** FS for profile 0.20-15-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 197 FS for profile 0.22-15-2-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 198 FS for profile 0.22-15-2-1500 with 6.5 magnitude earthquake @0.4g
**Fig. B. 199** FS for profile 0.22-15-2-1500 with 7.5 magnitude earthquake @0.4g

**Fig. B. 200** FS for profile 0.22-15-2-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 201 FS for profile 0.22-15-6-500 with 5.5 magnitude earthquake @0.4g

Fig. B. 202 FS for profile 0.22-15-6-500 with 6.5 magnitude earthquake @0.4g
Fig. B. 203 FS for profile 0.22-15-6-500 with 7.5 magnitude earthquake @0.4g

Fig. B. 204 FS for profile 0.22-15-6-500 with 8.5 magnitude earthquake @0.4g
Fig. B. 205 FS for profile 0.22-15-6-1000 with 5.5 magnitude earthquake @0.4g

Fig. B. 206 FS for profile 0.22-15-6-1000 with 6.5 magnitude earthquake @0.4g
Fig. B. 207 FS for profile 0.22-15-6-1000 with 7.5 magnitude earthquake @0.4g

Fig. B. 208 FS for profile 0.22-15-6-1000 with 8.5 magnitude earthquake @0.4g
Fig. B. 209 FS for profile 0.22-15-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 210 FS for profile 0.22-15-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 211 FS for profile 0.22-15-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 212 FS for profile 0.22-15-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 213 FS for profile 0.22-15-6-2000 with 5.5 magnitude earthquake @0.4g

Fig. B. 214 FS for profile 0.22-15-6-2000 with 6.5 magnitude earthquake @0.4g
Fig. B. 215 FS for profile 0.22-15-6-2000 with 7.5 magnitude earthquake @0.4g

Fig. B. 216 FS for profile 0.22-15-6-2000 with 8.5 magnitude earthquake @0.4g
**Fig. B. 217** FS for profile 0.22-15-12-1500 with 5.5 magnitude earthquake @0.4g

**Fig. B. 218** FS for profile 0.22-15-12-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 219 FS for profile 0.22-15-12-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 220 FS for profile 0.22-15-12-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 221 FS for profile 0.24-15-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 222 FS for profile 0.24-15-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 223 FS for profile 0.24-15-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 224 FS for profile 0.24-15-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 225 FS for collapse for profile 0.20-15-6-1500 with a 5.5 magnitude earthquake

Fig. B. 226 FS for collapse for profile 0.20-15-6-1500 with a 6.5 magnitude earthquake
**Fig. B. 227** FS for collapse for profile 0.20-15-6-1500 with a 7.5 magnitude earthquake

**Fig. B. 228** FS for collapse for profile 0.20-15-6-1500 with an 8.5 magnitude earthquake
Fig. B. 229 FS for collapse for profile 0.22-15-2-1500 with a 5.5 magnitude earthquake

Fig. B. 230 FS for collapse for profile 0.22-15-2-1500 with a 6.5 magnitude earthquake
Fig. B. 231 FS for collapse for profile 0.22-15-2-1500 with a 7.5 magnitude earthquake

Fig. B. 232 FS for collapse for profile 0.22-15-2-1500 with an 8.5 magnitude earthquake
Fig. B.233 FS for collapse for profile 0.22-15-6-500 with a 5.5 magnitude earthquake

Fig. B.234 FS for collapse for profile 0.22-15-6-500 with a 6.5 magnitude earthquake
Fig. B. 235 FS for collapse for profile 0.22-15-6-500 with a 7.5 magnitude earthquake

Fig. B. 236 FS for collapse for profile 0.22-15-6-500 with an 8.5 magnitude earthquake
**Fig. B. 237** FS for collapse for profile 0.22-15-6-1000 with a 5.5 magnitude earthquake

**Fig. B. 238** FS for collapse for profile 0.22-15-6-1000 with a 6.5 magnitude earthquake
Fig. B. 239 FS for collapse for profile 0.22-15-6-1000 with a 7.5 magnitude earthquake

Fig. B. 240 FS for collapse for profile 0.22-15-6-1000 with an 8.5 magnitude earthquake
Fig. B. 241 FS for collapse for profile 0.22-15-6-1500 with a 5.5 magnitude earthquake

Fig. B. 242 FS for collapse for profile 0.22-15-6-1500 with a 6.5 magnitude earthquake
Fig. B. 243 FS for collapse for profile 0.22-15-6-1500 with a 7.5 magnitude earthquake

Fig. B. 244 FS for collapse for profile 0.22-15-6-1500 with an 8.5 magnitude earthquake
**Fig. B. 245** FS for collapse for profile 0.22-15-6-2000 with a 5.5 magnitude earthquake

**Fig. B. 246** FS for collapse for profile 0.22-15-6-2000 with a 6.5 magnitude earthquake
Fig. B. 247 FS for collapse for profile 0.22-15-6-2000 with a 7.5 magnitude earthquake

Fig. B. 248 FS for collapse for profile 0.22-15-6-2000 with an 8.5 magnitude earthquake
Fig. B. 249 FS for collapse for profile 0.22-15-12-1500 with a 5.5 magnitude earthquake

Fig. B. 250 FS for collapse for profile 0.22-15-12-1500 with a 6.5 magnitude earthquake
Fig. B. 251 FS for collapse for profile 0.22-15-12-1500 with a 7.5 magnitude earthquake

Fig. B. 252 FS for collapse for profile 0.22-15-12-1500 with an 8.5 magnitude earthquake
Fig. B. 253 FS for collapse for profile 0.24-15-6-1500 with a 5.5 magnitude earthquake

Fig. B. 254 FS for collapse for profile 0.24-15-6-1500 with a 6.5 magnitude earthquake
**Fig. B. 255** FS for collapse for profile 0.24-15-6-1500 with a 7.5 magnitude earthquake

**Fig. B. 256** FS for collapse for profile 0.24-15-6-1500 with an 8.5 magnitude earthquake
Fig. B. 257 FS for profile 0.20-20-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 258 FS for profile 0.20-20-6-1500 with 6.5 magnitude earthquake @0.1g
**Fig. B. 259** FS for profile 0.20-20-6-1500 with 7.5 magnitude earthquake @0.1g

**Fig. B. 260** FS for profile 0.20-20-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 261 FS for profile 0.22-20-6-1000 with 5.5 magnitude earthquake @0.1g

Fig. B. 262 FS for profile 0.22-20-6-1000 with 6.5 magnitude earthquake @0.1g
Fig. B. 263 FS for profile 0.22-20-6-1000 with 7.5 magnitude earthquake @0.1g

Fig. B. 264 FS for profile 0.22-20-6-1000 with 8.5 magnitude earthquake @0.1g
**Fig. B. 265** FS for profile 0.22-20-6-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 266** FS for profile 0.22-20-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 267 FS for profile 0.22-20-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 268 FS for profile 0.22-20-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 269 FS for profile 0.22-20-6-2000 with 5.5 magnitude earthquake @0.1g

Fig. B. 270 FS for profile 0.22-20-6-2000 with 6.5 magnitude earthquake @0.1g
**Fig. B. 271** FS for profile 0.22-20-6-2000 with 7.5 magnitude earthquake @0.1g

**Fig. B. 272** FS for profile 0.22-20-6-2000 with 8.5 magnitude earthquake @0.1g
Fig. B. 273 FS for profile 0.22-20-12-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 274 FS for profile 0.22-20-12-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 275 FS for profile 0.22-20-12-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 276 FS for profile 0.22-20-12-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 277 FS for profile 0.24-20-6-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 278 FS for profile 0.24-20-6-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 279 FS for profile 0.24-20-6-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 280 FS for profile 0.24-20-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 281 FS for profile 0.20-20-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 282 FS for profile 0.20-20-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 283 FS for profile 0.20-20-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 284 FS for profile 0.20-20-6-1500 with 8.5 magnitude earthquake @0.2g
**Fig. B. 285** FS for profile 0.22-20-6-1000 with 5.5 magnitude earthquake @0.2g

![Graph](image)

**Fig. B. 286** FS for profile 0.22-20-6-1000 with 6.5 magnitude earthquake @0.2g

![Graph](image)
Fig. B. 287 FS for profile 0.22-20-6-1000 with 7.5 magnitude earthquake @0.2g

Fig. B. 288 FS for profile 0.22-20-6-1000 with 8.5 magnitude earthquake @0.2g
Fig. B. 289 FS for profile 0.22-20-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 290 FS for profile 0.22-20-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 291 FS for profile 0.22-20-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 292 FS for profile 0.22-20-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 293 FS for profile 0.22-20-6-2000 with 5.5 magnitude earthquake @0.2g

Fig. B. 294 FS for profile 0.22-20-6-2000 with 6.5 magnitude earthquake @0.2g
Fig. B. 295 FS for profile 0.22-20-6-2000 with 7.5 magnitude earthquake @0.2g

Fig. B. 296 FS for profile 0.22-20-6-2000 with 8.5 magnitude earthquake @0.2g
Fig. B. 297 FS for profile 0.22-20-12-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 298 FS for profile 0.22-20-12-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 299 FS for profile 0.22-20-12-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 300 FS for profile 0.22-20-12-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 301 FS for profile 0.24-20-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 302 FS for profile 0.24-20-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 303 FS for profile 0.24-20-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 304 FS for profile 0.24-20-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 305 FS for profile 0.20-20-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 306 FS for profile 0.20-20-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 307 FS for profile 0.20-20-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 308 FS for profile 0.20-20-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 309 FS for profile 0.22-20-6-1000 with 5.5 magnitude earthquake @0.4g

Fig. B. 310 FS for profile 0.22-20-6-1000 with 6.5 magnitude earthquake @0.4g
Fig. B. 311 FS for profile 0.22-20-6-1000 with 7.5 magnitude earthquake @0.4g

Fig. B. 312 FS for profile 0.22-20-6-1000 with 8.5 magnitude earthquake @0.4g
Fig. B. 313 FS for profile 0.22-20-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 314 FS for profile 0.22-20-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 315 FS for profile 0.22-20-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 316 FS for profile 0.22-20-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 317 FS for profile 0.22-20-6-2000 with 5.5 magnitude earthquake @0.4g

Fig. B. 318 FS for profile 0.22-20-6-2000 with 6.5 magnitude earthquake @0.4g
Fig. B. 319 FS for profile 0.22-20-6-2000 with 7.5 magnitude earthquake @0.4g

Fig. B. 320 FS for profile 0.22-20-6-2000 with 8.5 magnitude earthquake @0.4g
Fig. B. 321 FS for profile 0.22-20-12-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 322 FS for profile 0.22-20-12-1500 with 6.5 magnitude earthquake @0.4g
**Fig. B. 323** FS for profile 0.22-20-12-1500 with 7.5 magnitude earthquake @0.4g

**Fig. B. 324** FS for profile 0.22-20-12-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 325 FS for profile 0.24-20-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 326 FS for profile 0.24-20-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 327 FS for profile 0.24-20-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 328 FS for profile 0.24-20-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 329 FS for collapse for profile 0.20-20-6-1500 with a 5.5 magnitude earthquake

Fig. B. 330 FS for collapse for profile 0.20-20-6-1500 with a 6.5 magnitude earthquake
Fig. B. 331 FS for collapse for profile 0.20-20-6-1500 with a 7.5 magnitude earthquake

Fig. B. 332 FS for collapse for profile 0.20-20-6-1500 with an 8.5 magnitude earthquake
**Fig. B. 333** FS for collapse for profile 0.22-20-6-1000 with a 5.5 magnitude earthquake

**Fig. B. 334** FS for collapse for profile 0.22-20-6-1000 with a 6.5 magnitude earthquake
**Fig. B. 335** FS for collapse for profile 0.22-20-6-1000 with a 7.5 magnitude earthquake

**Fig. B. 336** FS for collapse for profile 0.22-20-6-1000 with an 8.5 magnitude earthquake
Fig. B. 337 FS for collapse for profile 0.22-20-6-1500 with a 5.5 magnitude earthquake

Fig. B. 338 FS for collapse for profile 0.22-20-6-1500 with a 6.5 magnitude earthquake
Fig. B. 339 FS for collapse for profile 0.22-20-6-1500 with a 7.5 magnitude earthquake

Fig. B. 340 FS for collapse for profile 0.22-20-6-1500 with an 8.5 magnitude earthquake
Fig. B. 341 FS for collapse for profile 0.22-20-6-2000 with a 5.5 magnitude earthquake

Fig. B. 342 FS for collapse for profile 0.22-20-6-2000 with a 6.5 magnitude earthquake
Fig. B. 343 FS for collapse for profile 0.22-20-6-2000 with a 7.5 magnitude earthquake

Fig. B. 344 FS for collapse for profile 0.22-20-6-2000 with an 8.5 magnitude earthquake
Fig. B. 345 FS for collapse for profile 0.22-20-12-1500 with a 5.5 magnitude earthquake

Fig. B. 346 FS for collapse for profile 0.22-20-12-1500 with a 6.5 magnitude earthquake
Fig. B. 347 FS for collapse for profile 0.22-20-12-1500 with a 7.5 magnitude earthquake

Fig. B. 348 FS for collapse for profile 0.22-20-12-1500 with an 8.5 magnitude earthquake
**Fig. B. 349** FS for collapse for profile 0.24-20-6-1500 with a 5.5 magnitude earthquake

**Fig. B. 350** FS for collapse for profile 0.24-20-6-1500 with a 6.5 magnitude earthquake
**Fig. B. 351** FS for collapse for profile 0.24-20-6-1500 with a 7.5 magnitude earthquake

**Fig. B. 352** FS for collapse for profile 0.24-20-6-1500 with an 8.5 magnitude earthquake
**Fig. B.353** FS for profile 0.22-25-6-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B.354** FS for profile 0.22-25-6-1500 with 6.5 magnitude earthquake @0.1g
**Fig. B. 355** FS for profile 0.22-25-6-1500 with 7.5 magnitude earthquake @0.1g

**Fig. B. 356** FS for profile 0.22-25-6-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 357** FS for profile 0.22-25-6-2000 with 5.5 magnitude earthquake @0.1g

**Fig. B. 358** FS for profile 0.22-25-6-2000 with 6.5 magnitude earthquake @0.1g
Fig. B. 359 FS for profile 0.22-25-6-2000 with 7.5 magnitude earthquake @0.1g

Fig. B. 360 FS for profile 0.22-25-6-2000 with 8.5 magnitude earthquake @0.1g
Fig. B. 361 FS for profile 0.22-25-12-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 362 FS for profile 0.22-25-12-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 363 FS for profile 0.22-25-12-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 364 FS for profile 0.22-25-12-1500 with 8.5 magnitude earthquake @0.1g
**Fig. B. 365** FS for profile 0.24-25-6-1500 with 5.5 magnitude earthquake @0.1g

**Fig. B. 366** FS for profile 0.24-25-6-1500 with 6.5 magnitude earthquake @0.1g
**Fig. B. 367** FS for profile 0.24-25-6-1500 with 7.5 magnitude earthquake @0.1g

**Fig. B. 368** FS for profile 0.24-25-6-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 369 FS for profile 0.22-25-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 370 FS for profile 0.22-25-6-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 371 FS for profile 0.22-25-6-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 372 FS for profile 0.22-25-6-1500 with 8.5 magnitude earthquake @0.2g
**Fig. B. 373** FS for profile 0.22-25-6-2000 with 5.5 magnitude earthquake @0.2g

**Fig. B. 374** FS for profile 0.22-25-6-2000 with 6.5 magnitude earthquake @0.2g
**Fig. B. 375** FS for profile 0.22-25-6-2000 with 7.5 magnitude earthquake @0.2g

**Fig. B. 376** FS for profile 0.22-25-6-2000 with 8.5 magnitude earthquake @0.2g
Fig. B. 377 FS for profile 0.22-25-12-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 378 FS for profile 0.22-25-12-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 379 FS for profile 0.22-25-12-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 380 FS for profile 0.22-25-12-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 381 FS for profile 0.24-25-6-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 382 FS for profile 0.24-25-6-1500 with 6.5 magnitude earthquake @0.2g
**Fig. B. 383** FS for profile 0.24-25-6-1500 with 7.5 magnitude earthquake @0.2g

**Fig. B. 384** FS for profile 0.24-25-6-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 385 FS for profile 0.22-25-6-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 386 FS for profile 0.22-25-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 387 FS for profile 0.22-25-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 388 FS for profile 0.22-25-6-1500 with 8.5 magnitude earthquake @0.4g
**Fig. B. 389** FS for profile 0.22-25-6-2000 with 5.5 magnitude earthquake @0.4g

**Fig. B. 390** FS for profile 0.22-25-6-2000 with 6.5 magnitude earthquake @0.4g
**Fig. B. 391** FS for profile 0.22-25-6-2000 with 7.5 magnitude earthquake @0.4g

**Fig. B. 392** FS for profile 0.22-25-6-2000 with 8.5 magnitude earthquake @0.4g
Fig. B. 393 FS for profile 0.22-25-12-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 394 FS for profile 0.22-25-12-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 395 FS for profile 0.22-25-12-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 396 FS for profile 0.22-25-12-1500 with 8.5 magnitude earthquake @0.4g
**Fig. B. 397** FS for profile 0.24-25-6-1500 with 5.5 magnitude earthquake @0.4g

**Fig. B. 398** FS for profile 0.24-25-6-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 399 FS for profile 0.24-25-6-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 400 FS for profile 0.24-25-6-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 401 FS for collapse for profile 0.22-25-6-1500 with a 5.5 magnitude earthquake

Fig. B. 402 FS for collapse for profile 0.22-25-6-1500 with a 6.5 magnitude earthquake
**Fig. B. 403** FS for collapse for profile 0.22-25-6-1500 with a 7.5 magnitude earthquake

**Fig. B. 404** FS for collapse for profile 0.22-25-6-1500 with an 8.5 magnitude earthquake
Fig. B. 405 FS for collapse for profile 0.22-25-6-2000 with a 5.5 magnitude earthquake

Fig. B. 406 FS for collapse for profile 0.22-25-6-2000 with a 6.5 magnitude earthquake
Fig. B. 407 FS for collapse for profile 0.22-25-6-2000 with a 7.5 magnitude earthquake

Fig. B. 408 FS for collapse for profile 0.22-25-6-2000 with an 8.5 magnitude earthquake
Fig. B. 409 FS for collapse for profile 0.22-25-12-1500 with a 5.5 magnitude earthquake

Fig. B. 410 FS for collapse for profile 0.22-25-12-1500 with a 6.5 magnitude earthquake
Fig. B. 411 FS for collapse for profile 0.22-25-12-1500 with a 7.5 magnitude earthquake

Fig. B. 412 FS for collapse for profile 0.22-25-12-1500 with an 8.5 magnitude earthquake
Fig. B. 413 FS for collapse for profile 0.24-25-6-1500 with a 5.5 magnitude earthquake

Fig. B. 414 FS for collapse for profile 0.24-25-6-1500 with a 6.5 magnitude earthquake
**Fig. B. 415** FS for collapse for profile 0.24-25-6-1500 with a 7.5 magnitude earthquake

**Fig. B. 416** FS for collapse for profile 0.24-25-6-1500 with an 8.5 magnitude earthquake
Fig. B. 417 FS for profile 0.22-30-12-1500 with 5.5 magnitude earthquake @0.1g

Fig. B. 418 FS for profile 0.22-30-12-1500 with 6.5 magnitude earthquake @0.1g
Fig. B. 419 FS for profile 0.22-30-12-1500 with 7.5 magnitude earthquake @0.1g

Fig. B. 420 FS for profile 0.22-30-12-1500 with 8.5 magnitude earthquake @0.1g
Fig. B. 421 FS for profile 0.22-30-12-1500 with 5.5 magnitude earthquake @0.2g

Fig. B. 422 FS for profile 0.22-30-12-1500 with 6.5 magnitude earthquake @0.2g
Fig. B. 423  FS for profile 0.22-30-12-1500 with 7.5 magnitude earthquake @0.2g

Fig. B. 424  FS for profile 0.22-30-12-1500 with 8.5 magnitude earthquake @0.2g
Fig. B. 425 FS for profile 0.22-30-12-1500 with 5.5 magnitude earthquake @0.4g

Fig. B. 426 FS for profile 0.22-30-12-1500 with 6.5 magnitude earthquake @0.4g
Fig. B. 427 FS for profile 0.22-30-12-1500 with 7.5 magnitude earthquake @0.4g

Fig. B. 428 FS for profile 0.22-30-12-1500 with 8.5 magnitude earthquake @0.4g
Fig. B. 429 FS for collapse for profile 0.22-30-12-1500 with a 5.5 magnitude earthquake

Fig. B. 430 FS for collapse for profile 0.22-30-12-1500 with a 6.5 magnitude earthquake
**Fig. B. 431** FS for collapse for profile 0.22-30-12-1500 with a 7.5 magnitude earthquake

**Fig. B. 432** FS for collapse for profile 0.22-30-12-1500 with an 8.5 magnitude earthquake