Hindawi Publishing Corporation
Mathematical Problems in Engineering
Volume 2013, Article ID 893602, 13 pages
http://dx.doi.org/10.1155/2013/893602

Research Article

Factor of Safety Reduction Factors for Accounting for Progressive Failure for Earthen Levees with Underlying Thin Layers of Sensitive Soils

Adam J. Lobbestael, Adda Athanasopoulos-Zekkos, and Josh Colley

Department of Civil and Environmental Engineering, University of Michigan, 2350 Hayward Street, 1354 GG Brown Laboratory, Ann Arbor, MI 48109, USA

Correspondence should be addressed to Adam J. Lobbestael; ajimlobb@umich.edu

Received 7 August 2013; Accepted 13 October 2013

Academic Editor: Anaxagoras Elenas

The effects of progressive failure on flood embankments with underlying thin layers of soft, sensitive soils are investigated. Finite element analysis allows for investigation of strain-softening effects and progressive failure in soft and sensitive soils. However, limit equilibrium methods for slope stability analysis, widely used in industry, cannot capture these effects and may result in unconservative factors of safety. A parametric analysis was conducted to investigate the effect of thin layers of soft sensitive soils on the stability of flood embankments. A flood embankment was modeled using both the limit equilibrium method and the finite element method. The foundation profile was altered to determine the extent to which varying soft and sensitive soils affected the stability of the embankment, with respect to progressive failure. The results from the two methods were compared to determine reduction factors that can be applied towards factors of safety computed using limit equilibrium methods, in order to capture progressive failure.

1. Introduction

The design and construction of levee systems are often challenging tasks due to the complex nature of the geologic conditions that typically comprise the site of a levee or flood embankment. This complex geology is the result of the processes of deposition and erosion that take place along coasts and riverbanks. The failure of the 17th Street Drainage Canal in New Orleans, during Hurricane Katrina, illustrates the potentially dangerous effects of thin layers of soft and sensitive material beneath a levee [1, 2], which are often found in such geologic environments. The additional load placed on a levee during a storm surge or high-water event has the potential to initiate progressive failure through the layer of soft, sensitive material, causing the levee to fail.

The importance of strain-softening soils and the role they play in progressive failure and delayed collapse of earthen embankments has long been established [3–5]. The potential for progressive failure to occur arises when strain-softening soils are present. As the levee is loaded with a rising water level on the water side, shear strains develop beneath the levee in a nonuniform fashion. The peak shear strength throughout the developing sliding surface is not simultaneously mobilized because the shear stresses that develop within sensitive soils are highly strain dependent. This critical mechanism however cannot be captured by limit equilibrium analysis methods, which are widely used in industry for their ease and simplicity but assume that the stress-strain characteristics of soils forming the slope are nonbrittle [6, 7]. The use of peak strengths, therefore, to model sensitive soils results in an overestimation of the available shear strength and thus an overestimation of the slope stability factor of safety, defined as the ratio of available shear strength to the shear strength at slope failure. The alternative approach, using residual shear strengths in design, has proven to result in an underestimation of the factor of safety [8].

The purpose of this study is to determine the extent to which progressive failure in thin layers of soft sensitive soils underlying levees affects the stability of the levee. Parametric

analyses were conducted in which a levee, founded upon medium stiff clay with an embedded thin layer of soft, sensitive clay, was modeled using both limit equilibrium analysis and finite element analysis. The strength parameters of the thin layer of sensitive soil, as well as the thickness of the layer and the location of the layer beneath the levee, were varied to assess the influence of each parameter on levee stability. The results of the two analysis types were compared to derive reduction factors that can be applied toward limit equilibrium analysis factors of safety, in order to capture progressive failure effects.

The potential effects of progressive failure in sensitive soils have been studied and presented in the literature over the past several decades. Bjerrum [9] addressed progressive failure as a potential factor contributing to the vane shear overestimation of undrained shear strength of clay, which then led to embankment failures. Chirapuntu and Duncan [10] suggested that progressive failure may occur if an embankment is stiffer and stronger than its foundation and presented shear strength reduction factors to account for this. Specifically, the potential for progressive failure through thin layers of soft sensitive soil has been recognized [11]. Up until this point, most of the literature focusing on the effects of progressive failure was primarily qualitative and did not offer guidelines as to how to incorporate them into the design process. More recently, Filz et al. [12] studied the effect of progressive failure through thin layers in the context of lined waste impoundments and showed that accounting for progressive failure is critical in the design of such facilities. They proposed shear strength reduction factors for use in limit equilibrium analysis of lined landfills in order to account for progressive failure.

The issue of incorporating strain softening into geomechanics finite element analyses has been addressed by researchers as early as 1972 [13]. One approach to capturing progressive failure in finite-element analysis is to incorporate true strain-softening constitutive models such as those developed by Duncan and Chang [14], Pietruszczak and Mroz [15], Yoshida et al. [16], and Yoshida et al. [17]. These models capture the contraction of the yield surface and reduce the soil strength parameters in accordance with plastic theory. These models are theoretically robust; however they are complex and pose several challenges with regard to their mathematical formulation. Such models suffer from problems such as mesh dependency and difficulty in monitoring convergence as pointed out by numerous researchers such as de Borst et al. [18], Potts and Zdravkovic [19], Zhou and Randolph [20], Wu and Wang [21], and Galavi and Schweiger [22]. Also, these models require a large number of input parameters and therefore extensive calibration. These issues often combine to hinder such constitutive models from being used in practical applications where strain softening is applicable.

An alternative approach followed by Lo and Lee [23] employed an incremental stress release method to approximately capture strain softening in finite element analysis of slopes. In their approach, the strength of individual elements is iteratively reduced based on the strains developed. The stress is then transferred from the overstressed elements to the surrounding elements. Although this method is approximate, it is relatively simple to implement in terms of the parameters required and the results agree well with field observations. This approach was furthered at the Soil Mechanics Section of Imperial College, London, using the Imperial College Finite Element Program (ICFEP) [24]. The model used for the analysis is a nonlinear strain-softening/hardening model that incorporates a Mohr-Coulomb yield criterion. The model accounts for softening by varying the angle of shearing resistance and cohesion with the calculated deviatoric plastic strain invariant. In this manner, undrained strength is reduced with increasing plastic strain. The model is presented in detail in Potts and Zdravkovic [25]. The approach has been used to analyze numerous documented slope failures (Chingford Embankment, Carsington Dam, Eppingham Dam, Abberton Reservoir, etc.) and hypothetical embankment sections where progressive failure was thought to play a role [24, 26–30]. The results showed good agreement with the field observations and confirmed the validity of the approach. Other authors have followed the approach set out by Dounias et al. [24] and implemented it in "user defined models" in programs such as Plaxis (Gens and Alonso [31]), GeoStudio (Hughes et al. [32], Kelln et al. [33], and Kelln et al. [34]), and TOCHNOG (Troncone [35]). These additional analyses also showed good agreement with field observations further verifying the validity of the approach and confirming the critical effect of progressive failure. This approach has been used extensively and confidence in this approach has grown to the point where it has also been used to predict the likelihood of progressive failure in future projects. For example, this approach was used in a 700 m tall escarpment underwater in the Gulf of Mexico [36] and in the heightening of the Abberton Reservoir embankment [30]. This approach has advantages over true strain-softening models since it is easier to implement and has been shown to achieve a level of accuracy suitable for practical applications.

Some additional recent advances by other researchers include Wu and Wang [21] and Galavi and Schweiger [37] who demonstrated the ability of "nonlocal" approaches as a way of overcoming mesh dependence. Zhou and Randolph [20] investigated large displacement finite element (LDFE) analysis as a way to overcome mesh dependence when modeling cylindrical and spherical penetrometers in strain-softening clays. They made use of remeshing and interpolation technique with small strain (RITSS) implemented in AFENA. Zabala and Alonso [38] successfully modeled progressive failure of the Aznalcóllar Dam using the material point method.

2. Slope Stability Analysis

In order to determine the effects of progressive failure on the stability of a levee with an underlying thin layer of soft sensitive material, slope stability analyses were performed on a series of levee-foundation profiles with each profile being analyzed using both the limit equilibrium analysis method and the finite element analysis method. The profile used in the study is a generic, symmetric levee cross-section constructed on a soft soil foundation. The levee consists of stiff clay with

Table 1: Material parameters used in limit equilibrium analyses.

Material	Soil model	γ_{sat} (kN/m^3)	γ_{unsat} (kN/m^3)	Undrained shear strength (kPa)
Embankment	Undrained ($\varphi = 0$)	18.1	16.5	43.1
Foundation clay	Undrained ($\varphi = 0$)	17.6	15.7	Varied: 35.9–47.8[a]
Thin layer material	Undrained ($\varphi = 0$)	18.1	16.5	Varied: 14.4–19.1[a]

[a]These strengths were varied throughout the parametric analysis. See Table 3 and Figures 3 and 4 for strengths used within range.

Table 2: Material parameters used in finite element analyses.

Material	Soil model	E_{ref} (kPa)	c_{ref} (kPa)	φ (°)	k_h (m/day)	k_v (m/day)
Embankment	Mohr-Coulomb	2215	43.1	0	0.008	0.002
Foundation clay	Mohr-Coulomb	2299	Varied: 35.9–47.8	0	0.08	0.02
Thin layer material	Mohr-Coulomb[a]	Varied[b]	Varied[b]	0	0.08	0.02

[a]A stepwise strength reduction scheme was used in conjunction with the Mohr-Coulomb model to capture strain-softening behavior. See the description of this method in Section 2.2.
[b]These strengths were varied throughout the parametric analysis. See Table 3 and Figures 3 and 4 for strengths used within range.
Unit weights used in finite element modeling are equal to those used in limit equilibrium modeling. Refer to Table 1 for values.

a height of 7.6 meters, 3 : 1 (H : V) slopes, and a 6.1 meter crest width. The foundation soil consists of medium stiff clay, in which a thin layer of soft, sensitive clay is embedded. The depth, at which the thin layer occurs, and the thickness of the layer are two of the parameters varied in the parametric analysis. An example profile from the study is presented in Figure 1. The analyses were conducted using undrained material properties to investigate the short-term behavior of the system under loading by an increase in water level due to a storm surge. For each profile, the factor of safety against slope instability was calculated with the levee under high-water conditions (i.e., water elevation on one side at the levee crest and on the other side at the ground surface and the phreatic surface varying linearly between the toe and the crest as shown in Figure 1).

The limit equilibrium analysis was performed without accounting for the possibility of progressive failure through the thin layer of sensitive soil, that is, assigning peak strength to all soil materials in the embankment foundation. In the finite element analysis an approximate method to capture progressive failure effects was used. Sections 2.1 and 2.2 present the details of the limit equilibrium analyses and the finite element analyses, respectively. Finally, a strain-softening soil model was used in finite difference analysis to perform analysis for validating the simplified finite element approach, as described in Section 6.

2.1. Limit Equilibrium Analysis Modeling. Limit equilibrium analysis (LEA) was performed using the Morgenstern-Price method as implemented in the computer program SLOPE/W [39]. The embankment soil, the thin layer of sensitive clay, and the foundation material above the thin layer were modeled as undrained using a Tresca yield condition ($\varphi_u = 0$). The thin layer of sensitive clay was modeled using the peak undrained shear strength of the material. Preliminary analysis was performed to determine the geometry of the failure surface and showed that the critical failure surface was approximately circular through the upper foundation material and linear along the sensitive clay layer, consistent with field observations of slope stability failures in New Orleans [2]. This observation was also later confirmed with the finite element analyses. Because a portion of the critical failure surface is expected to progress along the thin layer of sensitive clay, the foundation material underlying the thin layer was modeled as impenetrable. The material properties used to model the soils in the limit equilibrium analyses are presented in Table 1. The range of strengths used for the thin layer material is presented in Section 3.

2.2. Finite Element Analysis Modeling. Finite element analysis (FEA) was performed using the finite element code Plaxis 2D [40]. The levee and foundation materials were modeled as undrained with elastoplastic models. The thin layer of sensitive soil was also modeled as elastoplastic, but an approximated strain-softening approach, described later in this section, was used to capture the effects of progressive rupture. Table 2 summarizes the material properties used to model the soils in the finite element analyses.

Initial stress states were generated using the K_o procedure. Plastic analysis of the levee profile was conducted using staged construction, with the levee being placed in five lifts of equal thickness, followed by an incremental increase in the water level on one side of the levee. The water level remained at the ground surface on the opposite side of the levee throughout the staged construction process. Static pore pressures within the profile were calculated using the phreatic level.

At the expense of some accuracy relative to other strain-softening methods, an approximate method was used in this parametric analysis which is simpler to implement. The approximate method used here avoids several of the mathematical implications of true strain-softening models and requires less input parameters. The method consists of using incremental elasto-plastic models, discussed below, and therefore avoids a negative tangent modulus in the constitutive model which prevents violation of the stability criterion and the mesh sensitivity associated with localization and strain softening. The method is similar to those presented by

Lo and Lee [23] and Potts et al. [26] in its approach at tracking plastic strains and reducing shear strength accordingly.

In order to capture the strain-softening behavior of the sensitive material comprising the thin layer embedded in the foundation, a stepwise strength reduction scheme was employed. The thin layer of sensitive soil was divided into several smaller regions (clusters) to allow for local assignment of strength parameters, as shown in Figure 2. These clusters do not represent elements of the mesh but rather regions in which soil is assigned a single set of shear strength and stiffness parameters.

To capture the strain dependence of the thin layer soil strength, the strain development within the layer was tracked through the staged construction process. The calculation process was paused at the conclusion of each incremental loading in order to investigate the strains developed and accordingly reassign strength parameters to the thin layer regions. Depending on the strains within each cluster, a new strength was assigned which better reflects the "new" peak shear strength and the calculation was allowed to proceed to the next incremental loading. This can essentially be thought of as incrementally replacing the soil in small areas with weaker soil. This captured the effect of progressive rupture in the factor of safety calculation, discussed below, since different levels of shear strength were mobilized along the failure surface.

It is important to emphasize that the assigned clusters do not significantly affect or constrain the formation of the shear surface and potential failure surface. This was assured by comparing shear surfaces with and without the clusters. No differences were observed between the two approaches with respect to the formation of the sliding surface.

Following the strain-based strength reduction process, once the model was allowed to reach equilibrium, factors of safety against levee instability were determined using a phi-c reduction methodology employed by PLAXIS, in which the soil strength parameters are successively reduced along the failure surface until failure occurs.

3. Parametric Analysis

To investigate the effects of various parameters on the performance of flood embankments founded on soils with potential for progressive failure, an embankment was modeled with varying foundation conditions. A list of parameters was identified which potentially affect the degree to which progressive failure occurs. The parameters investigated in this study were (1) depth of the thin layer of sensitive material, (2) peak and residual shear strengths of the thin layer material, (3) thickness of the thin layer, (4) undrained shear strength of the foundation material, and (5) embankment height. This is not intended to be an all-inclusive list of parameters that may affect progressive failure but rather a list of what the authors believe are the most critical parameters. Furthermore, some of the parameters were investigated more extensively than others. A large range of thin layer depths and thin layer strength parameters were investigated, while only two thin layer thicknesses, two foundation material strengths, and three embankment heights were examined.

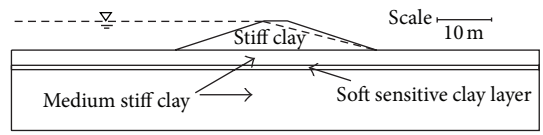

Figure 1: Levee profile used in analyses.

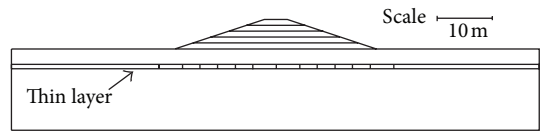

Figure 2: Finite element model geometry showing the thin layer separated into regions.

Three main groups of analyses were performed in the parametric analysis and are presented in tabular form in Table 3.

The first group of analyses was performed for an embankment with a foundation material having an undrained shear strength of 47.8 kPa and an embedded thin layer of sensitive material with a thickness of 0.6 meters. The depth of the thin layer was varied from 1.8 to 10.1 meters, and the stress-strain characteristics of the soil material forming the thin layer were varied as shown in Figures 3(a) and 4(b).

The stress-strain relationship for curve 1(a) is based on the strain-softening parameters for Onsøy Clay presented by Randolph and Andersen [41] and is characteristic of moderately sensitive clays. The remaining stress-strain curves are synthetic and were created by scaling the original relationship (curve 1(a)) to suit the needs of the parametric analysis. The second group of analyses was performed for an embankment with a foundation material having undrained shear strength of 47.8 kPa and an embedded thin layer of sensitive material with a thickness of 0.9 meters. The depth of the thin layer was varied from 2.4 to 15.2 meters, and the stress-strain characteristics of the soil material forming the thin layer were varied as shown in Figures 3(a) and 3(b). The third group of analyses was performed for an embankment with a foundation material having undrained shear strength of 35.9 kPa and embedded thin layer depths of both 0.6 and 0.9 meters. The depth of the thin layer was varied from 5.2 to 10.1 meters, and the stress-strain characteristics of the soil material forming the thin layer were varied as shown in Figure 4. The change in thin layer depth range and thin layer strength parameters between groups 1 and 2 and group 3 was necessary to ensure that enough shear strain was mobilized within the thin layer to induce strain softening and thereby progressive failure.

4. Analyses Results

Each group of analyses was performed using both limit equilibrium analysis (LEA) and finite element analysis (FEA) with the strain-softening approximation. Figure 5 presents

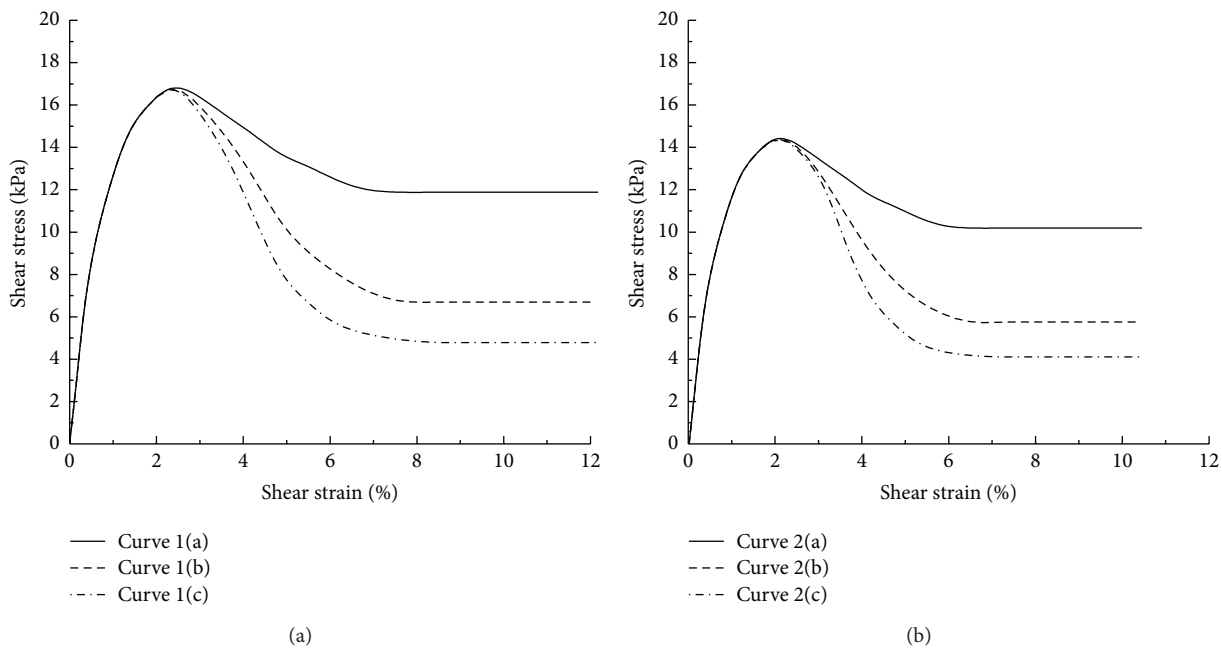

Figure 3: Shear stress-shear strain curves for thin layer material (curves 1 and 2).

Table 3: Parameter combinations investigated in the present study.

Analysis group	Foundation material strength (kPa)	Thin layer thickness (m)	Thin layer depth range (m)	Thin layer curve number
Group 1	47.8	0.6	1.8–10.1	1(a)
			2.4–10.1	1(b)
			1.8–10.1	1(c)
			1.8–10.1	2(a)
			2.4–10.1	2(b)
			1.8–10.1	2(c)
Group 2	47.8	0.9	2.4–12.2	1(a)
			2.4–15.2	1(b)
			2.4–12.2	1(c)
			2.4–12.2	2(a)
			2.4–12.2	2(b)
			2.4–10.1	2(c)
Group 3	35.9	0.6	5.2–10.1	3(a)
			5.2–10.1	3(b)
			5.2–10.1	3(c)
		0.9	5.2–10.1	3(b)
			5.2–10.1	3(c)

the finite element factors of safety plotted against the limit equilibrium factors of safety.

As seen in Figure 5, the vast majority of the data points plot below the 1 : 1 line, indicating that the progressive failure indeed significantly reduces the stability of the embankment. The results from the three analyses groups, for each analysis method (i.e., LEA and FEA), were then compared to determine to what extent each parameter affects the stability of the slope with regard to progressive failure for each analysis type. For each combination of parameters, the resulting factors of safety against slope instability from the limit equilibrium analysis (FS_{LE}) and the finite element analysis (FS_{FE}) were compared.

Figures 6(a)–6(e) show plots of the FS_{FE}/FS_{LE} versus sensitivity (defined as the ratio of peak strength to residual strength) for various combinations of foundation soil shear strength, thin layer depth, thin layer thickness, and thin layer peak shear strength. Figures 7(a)–7(f) show plots of FS_{FE}/FS_{LE} versus thin layer depth for various combinations of foundation soil shear strength, thin layer thickness, thin layer

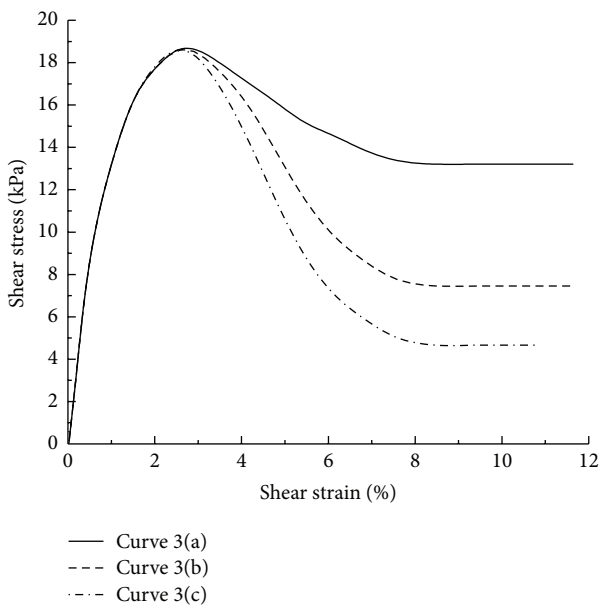

Figure 4: Shear stress-shear strain curves for thin layer material (curve 3).

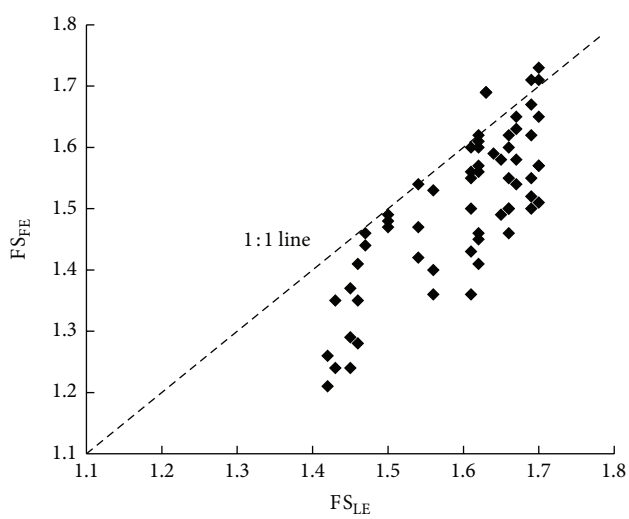

Figure 5: Summary plot of parametric analysis data points.

peak shear strengths, and thin layer sensitivity. From these figures, the relative effect on the potential for progressive failure of each of the parameters investigated can be observed.

4.1. Effect of Thin Layer Sensitivity and Shear Strength. The analyses performed in this study indicate that the peak shear strength and sensitivity are the most influential parameters with regard to progressive failure. It is difficult to discern the effects of the two parameters separately and some of their effects appear to be a function of their combination. Figures 6(a)–6(e) show that, in general, as the sensitivity of the thin layer increases, the extent to which progressive failure influences the embankment stability increases. As the peak strength of the thin layer decreases, the impact of progressive failure on slope stability increases.

4.2. Effect of Thin Layer Depth. It can be readily seen that the depth of the thin layer plays a significant role in the occurrence of progressive failure for the profiles investigated in this study. Comparison of Figure 6(a) with Figure 6(b) and Figure 6(c) with Figure 6(d) shows that as thin layer depth increases the impact of thin layer peak shear strength on progressive failure decreases. Figure 6(e) and Figures 7(a)–7(f) show that as the thin layer depth increases, the ratio of finite element factor of safety to limit equilibrium factor of safety decreases, suggesting that progressive failure has a greater effect for deeper thin layers. This trend continues to a certain thin layer depth, after which the effect of progressive failure diminishes with an increase in thin layer depth. The effect of thin layer depth observed in this study is logical, since as the depth increases, the driving force increases. The resisting force also increases, due to the extended failure surface, but, due to the presence of the embankment, does not entirely compensate for the increased driving force until a certain depth, after which an increase in thin layer depth causes a decrease in progressive failure. It would be expected that for large thin layer depths (beyond those investigated here), progressive failure through the thin layer would play no role, since a more critical failure through the overlying material would develop.

4.3. Effect of Thin Layer Thickness. Comparison of Figure 6(a) with Figure 6(c) and Figure 6(b) with Figure 6(d) shows that the thin layer thickness has a negligible impact on progressive failure. For layers with a thickness that is thin relative to the depth beneath the embankment, it is logical that the thickness would have little impact on progressive failure, since the failure surface through the sensitive material is primarily linear. This however is only true over this narrow range of thin layer thicknesses. It has been shown that for thicker layers of soft and sensitive soils beneath embankments (on the order of 3 meters and greater) the thickness of the layer is a key parameter with respect to the degree to which progressive failure affects the stability of the embankment [42].

4.4. Effect of Foundation Soil Shear Strength. Although not studied in extensive detail, the effect of foundation soil shear strength can be evaluated by comparing Figures 7(a)–7(d) with Figures 7(e)-7(f). The foundation soil shear strength was reduced to produce Figures 7(e) and 7(f) and so in order to allow for the mobilization of adequate shear strains for the initiation of progressive failure, the thin layer peak strength was increased. It appears that as foundation soil shear strength decreases the influence of progressive failure on slope stability increases. Also, progressive failure appears to have significantly less effect at shallower depths for the cases with the lower foundation material strength.

4.5. Effect of Embankment Height. To investigate the effect of embankment height the embankment properties were held constant while the height was varied. The external water

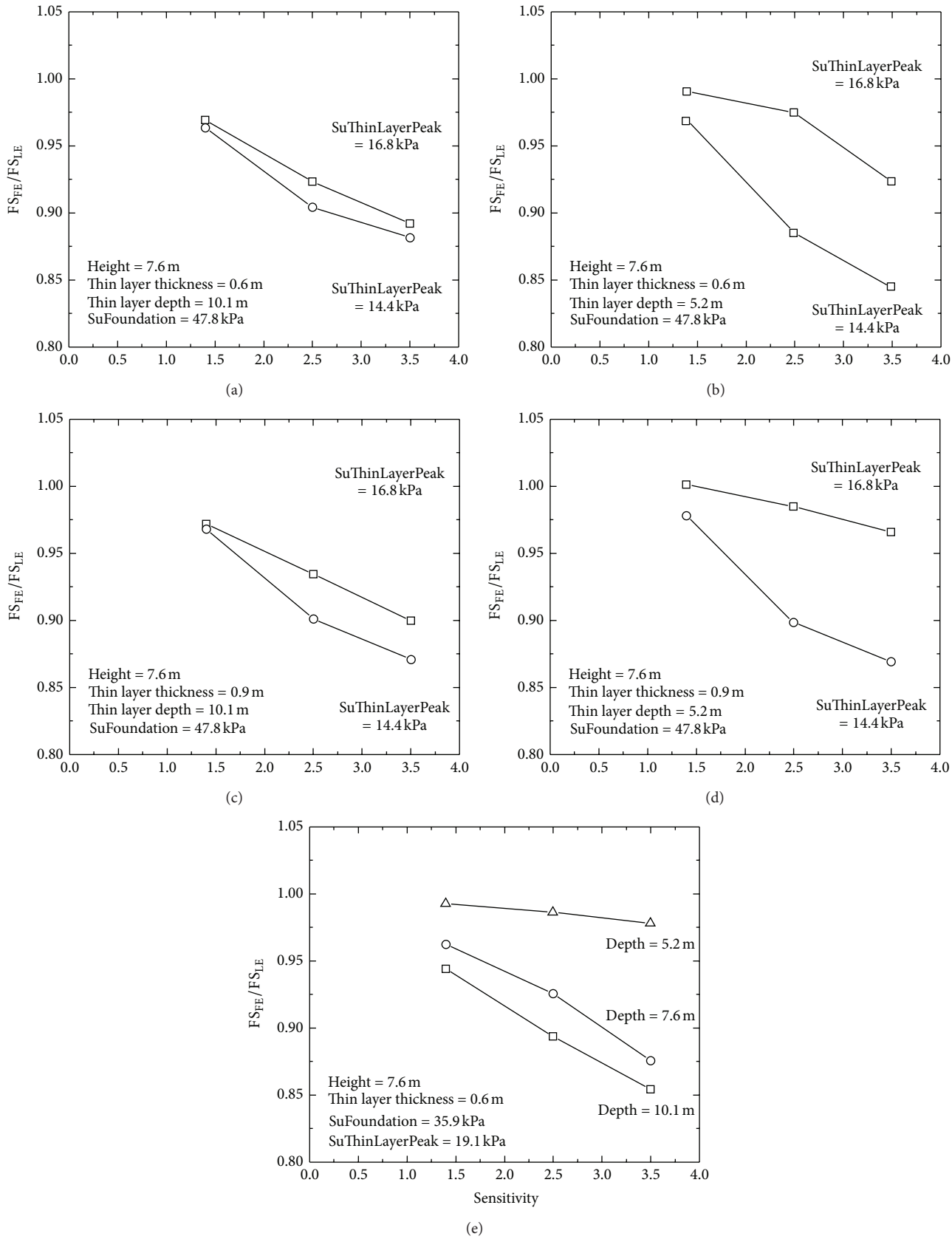

Figure 6: Plots of FS_{FE}/FS_{LE} versus thin layer sensitivity.

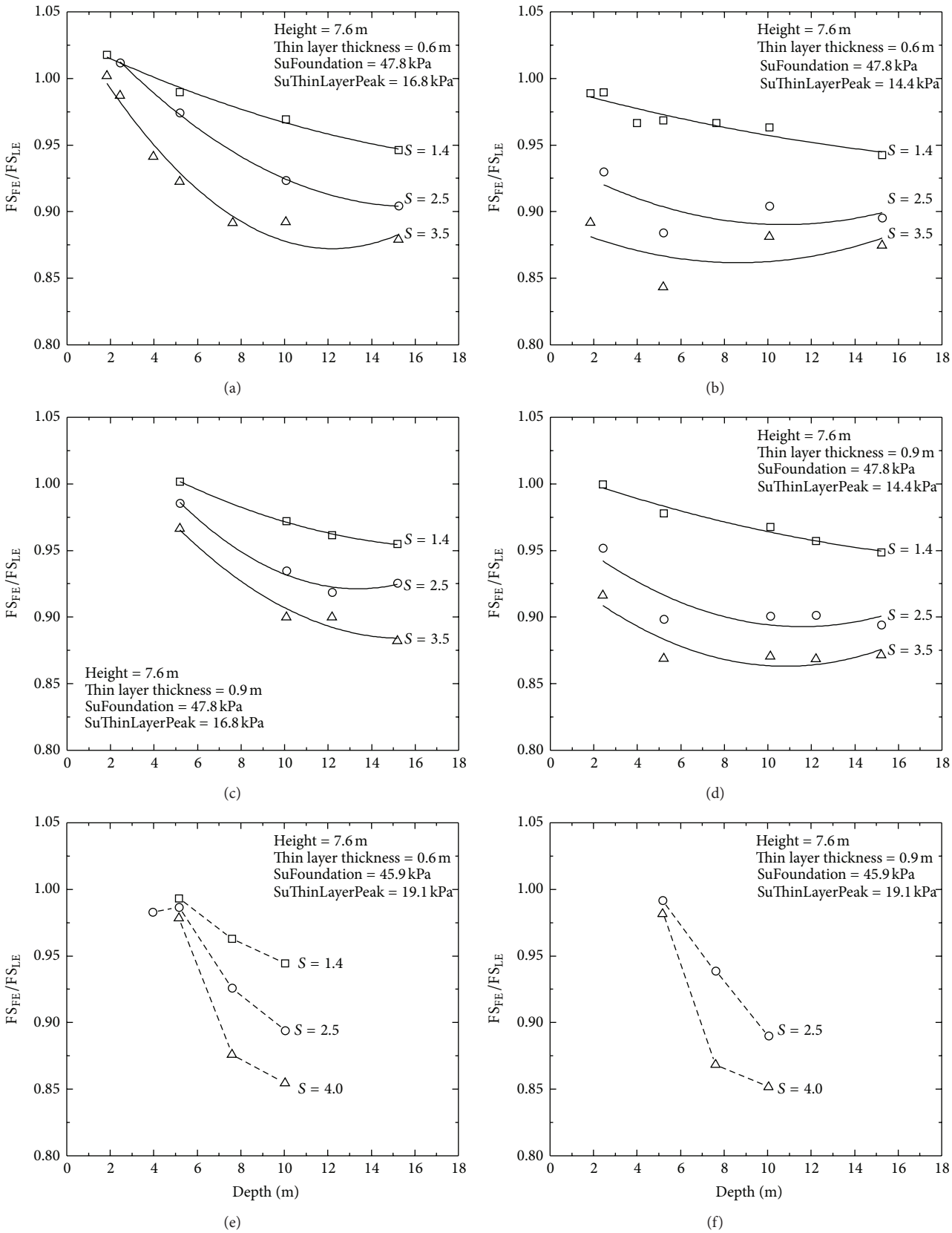

Figure 7: Plots of FS_{FE}/FS_{LE} versus thin layer depth for various sensitivities (S).

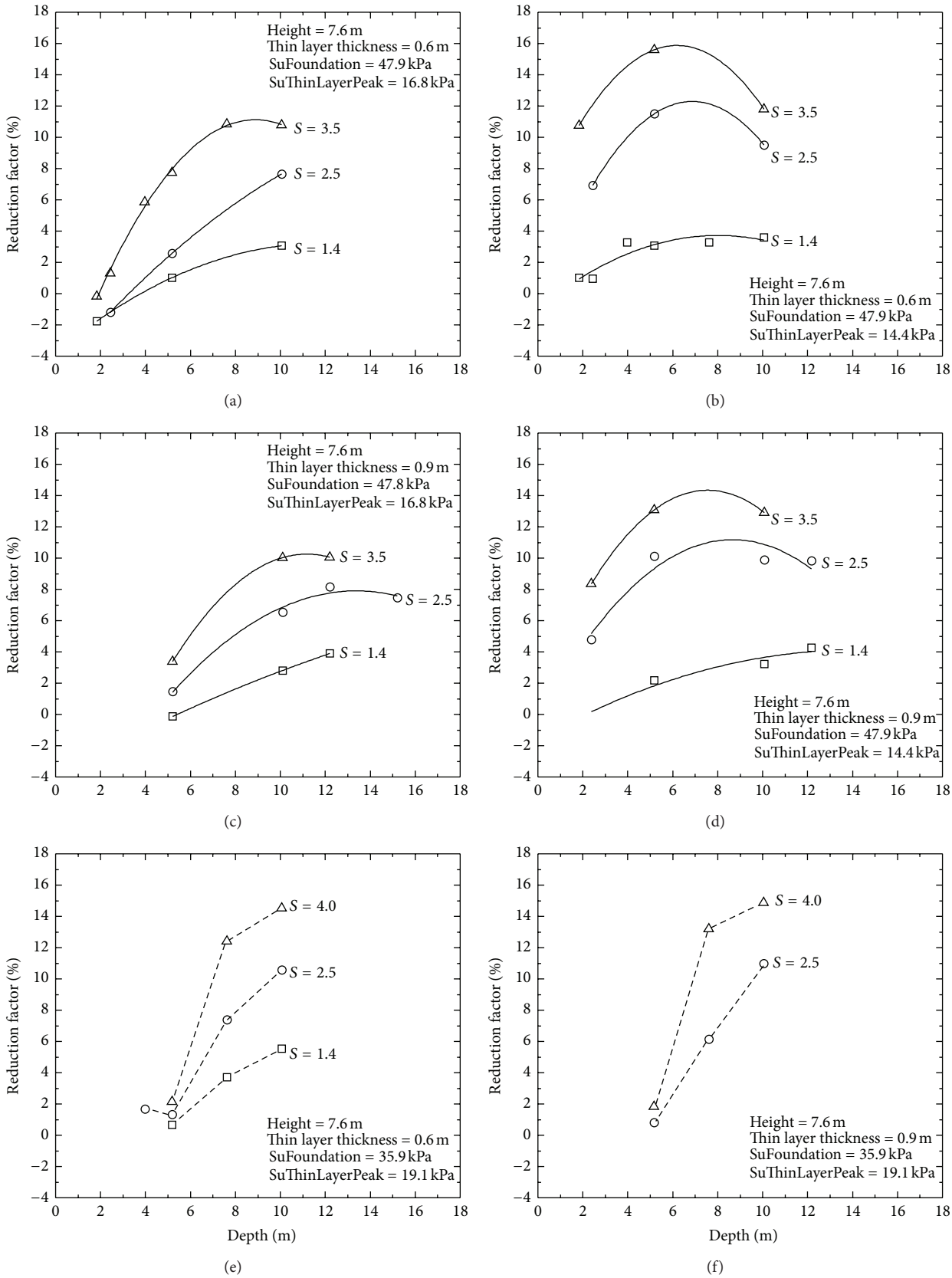

Figure 8: Plots of reduction factor versus thin layer depth.

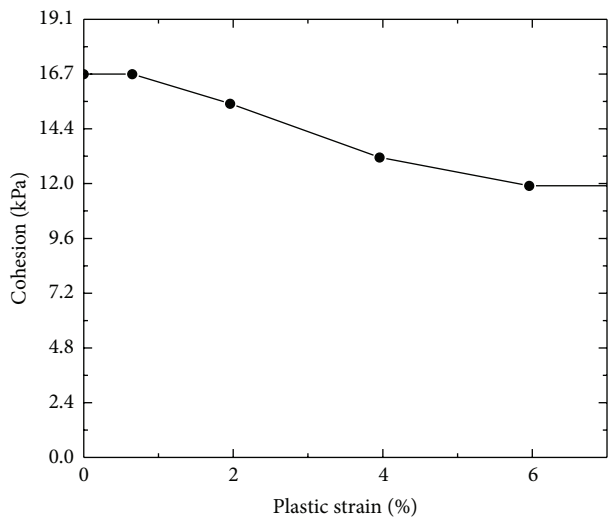

Figure 9: Variation of thin layer cohesion with plastic strain for FLAC analysis.

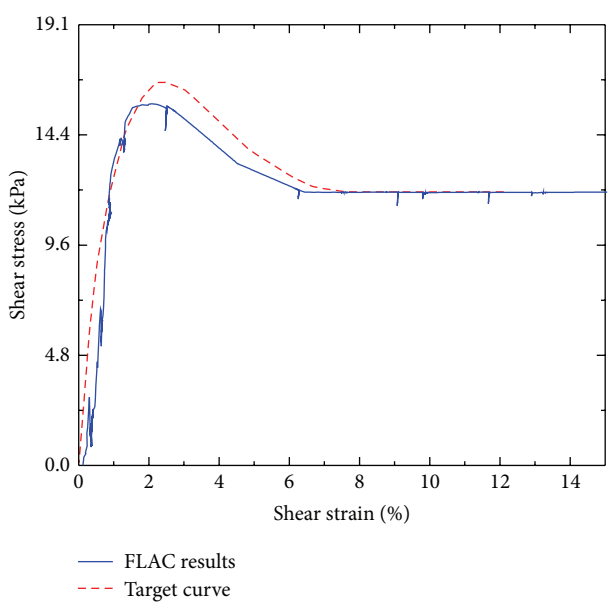

Figure 10: Computed and target stress-strain values within the thin layer material for the FLAC comparison analysis.

level at which strain softening occurred was recorded and the factors of safety were calculated using the finite element method, with approximate modeling of strain softening, and the limit equilibrium method. Table 4 shows the results of this study.

The numerical analysis showed that regardless of embankment height the mechanism of failure is still the same (i.e., progressive failure initiating within the thin layer beneath the toe of the downstream face and running along the thin layer). The effect of embankment height on the influence of progressive failure is, somewhat intuitively, that as embankment height increases the external load at which progressive failure occurs decreases.

Table 4: Analyses Results for different embankment heights.

Slope height (ft)	Water level at progressive failure (ft)	FS_{FE}	FS_{LE}	FS_{FE}/FS_{LE}
25	No progressive failure	1.82	1.67	1.09
30	20	1.48	1.51	0.98
35	10	1.25	1.34	0.93

As seen in Figures 6 and 7 some of the FS_{FE}/FS_{LE} ratios are slightly greater than 1.0. This should not be interpreted as the effects of progressive failure *increasing* the factor of safety against slope stability. Rather, it is an indication of the range of error that arises when comparing the two methods when the factors of safety are essentially equal. This occurs for cases when progressive failure either barely initiates or does not initiate at all (for shallow thin layer depths and relatively strong thin layer parameters).

5. Factor of Safety Reduction Factors

The percent decrease in the limit equilibrium factor of safety required to match the finite element (with approximate strain softening) factor of safety was designated as the necessary reduction factor to account for progressive failure effects when using limit equilibrium analysis. The behavior and trends of the reduction factor, based on the combination of parameters, were studied and the resulting observations and correlations are presented in the following section. The reduction factor (RF) is defined as

$$RF = \frac{FS_{LE} - FS_{FE}}{FS_{LE}} \cdot 100\%. \quad (1)$$

Figures 8(a)–8(f) show plots of RF against thin layer depth for a combination of embankment and soil properties. The plots can be used to determine the necessary reduction in factor of safety (from limit equilibrium analysis) to account for strain softening, for properties matching those for a specific plot. A negative reduction factor should be neglected, since including strain softening should not lead to an increase in factor of safety. Instead it should be assumed that in those cases the role of strain softening is negligible and the factor of safety from limit equilibrium analysis does not need to be modified.

6. Strain-Softening Model Validation

In order to validate the stepwise reduction method used to account for strain softening in the parametric analysis, a comparison analysis was conducted using the finite difference software FLAC 2D [43]. The sensitive thin layer material was modeled using a built-in strain softening model which is a Mohr-Coulomb formulation that accounts for strain softening by varying the model strength parameters as a function of plastic strain. The comparison analysis was conducted for a 25-foot levee with a 2-foot thick thin layer located at 17 feet below the levee. The thin layer had a peak strength of 16.75 kPa and a sensitivity of 1.4. All material

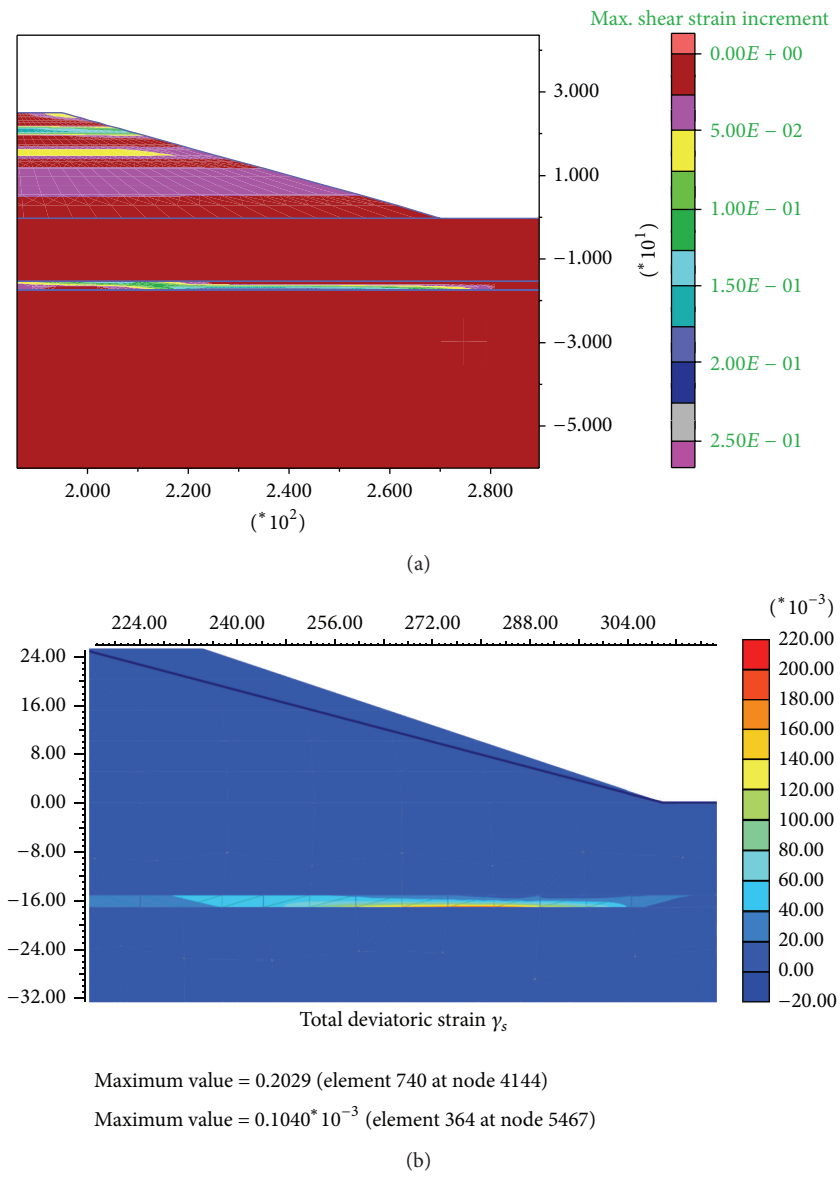

Figure 11: Shear strain contours at high water level in (a) FLAC and (b) PLAXIS.

properties and staged construction sequences were the same as for the analyses conducted in PLAXIS, except for the thin layer model, for which the cohesion was defined as a function of plastic strain, as shown in Figure 9.

The results of the FLAC analysis, using the built-in strain-softening model, were used to validate the stepwise strength reduction technique used in the PLAXIS analyses. Figure 10 presents the stress strain curve from the FLAC analysis calculated within the thin layer, at the location of maximum shear strain. The target stress strain curve for the thin layer material is also plotted for comparison, and it can be seen that the FLAC analysis accurately captures the strain-softening behavior of the sensitive material. Figures 11(a) and 11(b) present the shear strain contours at the high water level in FLAC and PLAXIS, respectively. The shear strains from the two analyses have very similar extents within the thin layer and have maximum values at roughly the same locations. Also, the magnitudes of the strains for the two analyses are very close, with the FLAC analysis having a maximum value of approximately 22.5% and the PLAXIS analysis having a maximum value of approximately 20.3%.

7. Concluding Remarks and Recommendations

A parametric analysis was conducted to investigate the effect of thin layers of soft, sensitive soils on the stability of flood embankments and to propose factor of safety reduction factors that can be applied to factors of safety computed using limit equilibrium analysis. The results of the parametric analysis emphasize the importance of accounting for progressive

failure in thin layers of soft sensitive soils and the inadequacy of limit equilibrium methods with respect to accounting for progressive failure. From the parametric analysis presented here, it can be concluded that progressive failure can have a significant effect on the stability of flood embankments with underlying thin layers of sensitive material. A host of parameters (i.e., thin layer depth, thin layer thickness, thin layer peak and residual shear strength, foundation soil shear strength, and embankment height) have been shown to contribute to the development and likelihood of progressive failure. However, the parameters that mostly affect the reduction factors recommended herein are the depth of the sensitive, soft layer and the peak shear strength and sensitivity of the sensitive, soft layer. The reduction factors shown in Figure 8 are recommended when performing limit equilibrium slope stability analysis of embankments with geometries and soil properties similar to the ones included in this study.

Additional analysis is needed to investigate the effect of soil type of the levee foundation materials surrounding the sensitive soft layer, as well as the effect of inclined layering, to be able to provide similar recommendations for a wider range of levees.

Acknowledgments

This material is based in part on work supported by the National Science Foundation under Grant no. CMMI-1030159 and the authors are grateful for their support. Any opinions, findings, and conclusions or recommendations expressed here are those of the authors and do not necessarily reflect the views of the National Science Foundation. The authors would also like to express their thanks to the Horace Rackham School of Graduate Studies at the University of Michigan for their support provided by the Rackham Merit Fellowship for Mr. Lobbestael.

References

[1] J. D. Rogers, G. P. Boutwell, D. W. Schmitz et al., "Geologic conditions underlying the 2005 17th Street Canal levee failure in New Orleans," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, vol. 134, no. 5, pp. 583–601, 2008.

[2] R. B. Seed, R. G. Bea, A. Athanasopoulos-Zekkos et al., "New orleans and hurricane katrina. III: the 17th Street drainage canal," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 134, no. 5, pp. 740–761, 2008.

[3] K. Terzaghi and R. B. Peck, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, New York, NY, USA, 1948.

[4] A. W. Skempton, "Long term stability of clay slopes," *Geotechnique*, vol. 14, no. 2, pp. 77–101, 1964.

[5] R. B. Peck, "Stability of natural slopes," *Journal of Soil Mechanics and Foundations Division*, vol. 93, pp. 403–436, 1967.

[6] J. M. Duncan and S. G. Wright, "The accuracy of equilibrium methods of slope stability analysis," *Engineering Geology*, vol. 16, no. 1-2, pp. 5–17, 1980.

[7] J. M. Duncan, "State of the art: limit equilibrium and finite-element analysis of slopes," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 122, no. 7, pp. 577–596, 1996.

[8] A. W. Skempton, "Slope stability of cuttings in Brown London Clay," in *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering*, vol. 3, pp. 261–270, 1977.

[9] L. Bjerrum, "Embankments on soft ground," in *Proceedings of the Specialty Conference on Earth and Earth Supported Structures*, vol. 2 of *ASCE*, pp. 1–54, 1972.

[10] S. Chirapuntu and J. M. Duncan, "The role of fill strength in the stability of embankments on soft clay foundations," Final Report Contract S-76-6, University of California at Berkeley, Berkeley, Calif, USA, 1976.

[11] G. A. Leonards, "Investigation of failures," *Journal of the Geotechnical Engineering Division*, vol. 108, no. 2, pp. 187–246, 1982.

[12] G. M. Filz, J. J. B. Esterhuizen, and J. Michael Duncan, "Progressive failure of lined waste impoundments," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 127, no. 10, pp. 841–848, 2001.

[13] K. Höeg, "Finite element analysis of strain-softening clay," *Journal of the Soil Mechanics and Foundations Division*, vol. 98, no. 1, pp. 43–58, 1972.

[14] J. M. Duncan and C. Y. Chang, "Nonlinear analysis of stress and strain in soils," *Journal of Soil Mechanics and Foundations Division*, vol. 96, no. 5, pp. 1629–1653, 1970.

[15] S. Pietruszczak and Z. Mroz, "Finite element analysis of deformation of strain-softening materials," *International Journal for Numerical Methods in Engineering*, vol. 17, no. 3, pp. 327–334, 1981.

[16] N. Yoshida, N. R. Morgenstern, and D. H. Chan, "A failure criterion for stiff soils and rocks exhibiting softening," *Canadian Geotechnical Journal*, vol. 27, no. 2, pp. 195–202, 1990.

[17] N. Yoshida, N. R. Morgenstern, and D. H. Chan, "Finite-element analysis of softening effects in fissured, overconsolidated clays and mudstones," *Canadian Geotechnical Journal*, vol. 28, no. 1, pp. 51–61, 1991.

[18] R. de Borst, L. J. Sluys, H.-B. Muhlhaus, and J. Pamin, "Fundamental issues in finite element analyses of localization of deformation," *Engineering Computations*, vol. 10, no. 2, pp. 99–121, 1993.

[19] D. M. Potts and L. Zdravkovic, *Finite Element Analysis in Geotechnical Engineering: Application*, Thomas Telford, London, UK, 2001.

[20] H. Zhou and M. F. Randolph, "Computational techniques and shear band development for cylindrical and spherical penetrometers in strain-softening clay," *International Journal of Geomechanics*, vol. 7, no. 4, pp. 287–295, 2007.

[21] S. Wu and X. Wang, "Mesh dependence and nonlocal regularization of one-dimensional strain softening plasticity," *Journal of Engineering Mechanics*, vol. 136, no. 11, pp. 1354–1365, 2010.

[22] V. Galavi and H. F. Schweiger, "Nonlocal multilaminate model for strain softening analysis," *International Journal of Geomechanics*, vol. 10, no. 1, pp. 30–44, 2010.

[23] K. Y. Lo and C. F. Lee, "Stress analysis and slope stability in strain-softening materials," *Geotechnique*, vol. 23, no. 1, pp. 1–11, 1973.

[24] G. T. Dounias, D. M. Potts, and P. R. Vaughan, "Finite element analysis of progressive failure: two case studies," *Computers and Geotechnics*, vol. 6, no. 2, pp. 155–175, 1988.

[25] D. M. Potts and L. Zdravkovic, *Finite Element Analysis in Geotechnical Engineering: Theory*, Thomas Telford, London, UK, 1999.

[26] D. M. Potts, G. T. Dounias, and P. R. Vaughan, "Finite element analysis of progressive failure of Carsington embankment," *Geotechnique*, vol. 40, no. 1, pp. 79–101, 1990.

[27] G. T. Dounias, D. M. Potts, and P. R. Vaughan, "Analysis of progressive failure and cracking in old British dams," *Geotechnique*, vol. 46, no. 4, pp. 621–640, 1996.

[28] D. M. Potts, N. Kovacevlc, and P. R. Vaughan, "Delayed collapse of cut slopes in stiff clay," *Geotechnique*, vol. 47, no. 5, pp. 953–982, 1997.

[29] N. Kovacevic, K. G. Higgins, D. M. Potts, and P. R. Vaughan, "Undrained behaviour of brecciated Upper Lias Clay at empingham dam," *Geotechnique*, vol. 57, no. 2, pp. 181–195, 2007.

[30] N. Kovacevic, D. W. Hight, D. M. Potts, and I. C. Carter, "Finite-element analysis of the failure and reconstruction of the main dam embankment at Abberton Reservoir, Essex, UK," *Geotechnique*, vol. 63, no. 9, pp. 753–767, 2012.

[31] A. Gens and E. E. Alonso, "Aznalcóllar dam failure—part 2: stability conditions and failure mechanism," *Geotechnique*, vol. 56, no. 3, pp. 185–201, 2006.

[32] D. Hughes, V. Sivakumar, D. Glynn, and G. Clarke, "A case study: delayed failure of a deep cutting in lodgement till," *Geotechnical Engineering* , vol. 160, no. 4, pp. 193–202, 2007.

[33] C. Kelln, J. Sharma, and D. Hughes, "A finite element solution scheme for an elastic-viscoplastic soil model," *Computers and Geotechnics*, vol. 35, no. 4, pp. 524–536, 2008.

[34] C. Kelln, J. Sharma, D. Hughes, and J. Graham, "An improved elastic-viscoplastic soil model," *Canadian Geotechnical Journal*, vol. 45, no. 10, pp. 1356–1376, 2008.

[35] A. Troncone, "Numerical analysis of a landslide in soils with strain-softening behaviour," *Geotechnique*, vol. 55, no. 8, pp. 585–596, 2005.

[36] N. Kovacevic, R. J. Jardine, D. M. Potts, C. E. Clukey, J. R. Brand, and D. R. Spikula, "A numerical simulation of underwater slope failures generated by salt diapirism combined with active sedimentation," *Geotechnique*, vol. 62, no. 9, pp. 777–786, 2012.

[37] V. Galavi and H. F. Schweiger, "Nonlocal multilaminate model for strain softening analysis," *International Journal of Geomechanics*, vol. 10, no. 1, pp. 30–44, 2010.

[38] F. Zabala and E. E. Alonso, "Progressive failure of aznalcó llar dam using the material point method," *Geotechnique*, vol. 61, no. 9, pp. 795–808, 2011.

[39] GEO-SLOPE International, *Stability Modeling with SLOPE/W, 2007*, Calgary, Alberta, Canada, 2008.

[40] R. Brinkgreve and W. Broere, "PLAXIS 2D Version 9 finite element code for soil and rock analyses," The Netherlands, 2008.

[41] M. F. Randolph and K. H. Andersen, "Numerical analysis of T-bar penetration in soft clay," *International Journal of Geomechanics*, vol. 6, no. 6, pp. 411–420, 2006.

[42] A. Lobbestael and A. Athanasopoulos-Zekkos, "A parametric analysis of the effects of progressive failure on embankments founded on soft sensitive soils," in *Proceedings of the Geo-Frontiers: Advances in Geotechnical Engineering*, pp. 3669–3678, Dallas, Tex, USA, March 2011.

[43] Itasca Consulting Group, *FLAC—Fast Lagrangian Analysis of Continua, Version 7.0 User's Guide*, Itasca, Minneapolis, Minn, USA, 2011.