Stability of Embankments Founded on Soft Soil
Improved with Deep-Mixing-Method Columns

ABSTRACT

Foundations constructed by the deep mixing method have been used to successfully support embankments, structures, and excavations in Japan, Scandinavia, the U.S., and other countries. The current state of practice is that design is based on deterministic analyses of settlement and stability, even though deep mixed materials are highly variable. Conservative deterministic design procedures have evolved to limit failures. Disadvantages of this approach include (1) designs with an unknown degree of conservatism and (2) contract administration problems resulting from unrealistic specifications for deep mixed materials.

This dissertation describes research conducted to develop reliability-based design procedures for foundations constructed using the deep mixing method. The emphasis of the research and the included examples are for embankment support applications, but the principles are applicable to foundations constructed for other purposes.

Reliability analyses for foundations created by the deep mixing method are described and illustrated using an example embankment. The deterministic stability analyses for the example embankment were performed using two methods: limit equilibrium analyses and numerical stress-strain analyses. An important finding from the research is that both numerical analyses and reliability analyses are needed to properly design embankments supported on deep mixed columns. Numerical analyses are necessary to address failure modes, such as column bending and tilting, that are not addressed by limit equilibrium analyses, which only cover composite shearing. Reliability analyses are necessary to address the impacts of variability of the deep mixed materials and other system components.

Reliability analyses also provide a rational basis for establishing statistical specifications for deep mixed materials. Such specifications will simplify administration of construction contracts and reduce claims while still providing assurance that the design intent is satisfied.

It is recommended that reliability-based design and statistically-based specifications be implemented in practice now.

LIST OF SYMBOLS

a_s	area replacement ratio
a_t	total area of columns under footing
B	length of prism
B	the center-to-center spacing between rows
c	cohesion
$c_{average}$	average cohesion value
c_{clay}	clay cohesion value
c_{column}	column cohesion value
c'_{soil}	drained effective stress cohesion intercept for soil
c_u	mean undrained shear strength of the untreated soil
c_v	coefficient of consolidation
d	depth
D	deflection
D_i	height of prism
D_{max}	maximum distance
D_{max}	maximum deflection
D_r	relative density
E	Young's Modulus
e	void ratio
E_{50}	secant modulus of elasticity
E_{col}	modulus of elasticity of the column
E_{max}	maximum Young's Modulus
f	density function
FS	factor of safety
FS_{LE}	factor of safety from limit equilibrium
FS_{NM}	factor of safety from numerical methods
G	shear modulus
$G_{average}$	average shear modulus value used for panels and soil between panels
G_{clay}	clay shear modulus value

G_{column}	column shear modulus value
H	horizontal force
H	height of the embankment
h_w	differential ground water level
I	moment of inertia
K	bulk modulus
k	permeability
K_0	effective stress lateral earth pressure coefficient
$K_{average}$	average bulk modulus value used for panels and soil between panels
K_{clay}	clay bulk modulus value
K_{column}	column bulk modulus value
K_u	undrained bulk modulus value
K_w	bulk modulus value of water
k_h	design seismic coefficient
L_s	width of prism
l	length
M	slope of the critical state line
M_{col}	constrained compression modulus of the column
mpc	preconsolidation pressure parameter (FLAC)
$mp1$	reference pressure parameter (FLAC)
M_{soil}	constrained modulus of the soil
m_v	compressibility parameter
mv_1	reference volume parameter (FLAC)
n	number of samples
n	stress concentration ratio
n	centrifugal acceleration factor
N_c	number of calculations
N_v	number of random variables
OCR	over consolidation ratio
P	load
p	mean stress

$p(f)$	probability of failure
$p(s)$	probability of satisfactory performance
$p(u)$	probability of unsatisfactory performance
p_0	initial pressure
P_a	active earth force
PI	plasticity index
P_p	passive earth force
p_p	preconsolidation pressure
q	shear stress
q	embankment pressure
q_{max}	maximum unconfined compression strength
q_u	unconfined compression strength
r	stiffness ratio
s	separation distance
s	center-to-center spacing of columns
S_{2D}	section modulus of the two-dimensional strip
S_{3D}	section modulus of the round column
SR	settlement ratio
S_r	stress ratio appropriate to orientation of failure surface
s_u	undrained shear strength
V	coefficient of variation
V	vertical force
V_{FS}	coefficient of variation of factor of safety
V_s	variation for settlement
w	applied pressure
$w{:}c$	water-to-cement ratio
wc	water content
x_i	measurement values
z	distance from ground surface to failure surface
ΔFS	change in factor of safety,
Δp	change in pressure in the soil

ΔS	change in settlement,
α	inclination of the failure surface
β	reliability index
β	drained cohesion intercept factor
β_F	reliability index for factor of safety
δ	autocorrelation distance
ε	settlement of the soil
ε_s	shear strain
ε_v	volumetric strain
ϕ	friction angle
ϕ'_{col}	drained friction angle of the column
$\phi_{u,col}$	undrained total stress friction angle for column
ϕ_{emb}	embankment friction angle
ϕ_{soil}	undrained friction angle of the soil
ϕ'_{soil}	drained effective stress friction angle for soil
$\phi_{u,soil}$	undrained total stress friction angle for soil
γ	unit weight of treated soil
γ_1	unit weight of the embankment
γ'_{col}	effective unit weight of the column
γ_f^{col}	unit weight of the fictitious layer above the stone column
γ_f^{soil}	unit weight of the fictitious layer above the soil
γ_{soil}	unit weight of the soil
γ_w	unit weight of water
γ_b	buoyant unit weight
η	porosity
η	reduction in clay strength
η_f	slope of the critical state line
κ	slope of the recompression line
λ	slope of the virgin compression line

1. INTRODUCTION

When roadways traverse low-lying areas, embankments may be necessary to bring the roadways up to functional elevations. Such embankments, if placed on highly compressible soft clays or problematic organic soils, may experience long term settlements and edge stability problems. An economical means to improve existing soils in these cases is use of prefabricated vertical drains combined with gradual placement of the embankment fill. This well-established technique can permit construction of embankments on soft ground at a lower construction cost than by using the column-supported embankment technology. However, use of vertical drains and gradual embankment placement requires considerable time for consolidation and strengthening of the soft ground, and this approach can also induce settlement in adjacent facilities, such as would occur when an existing road embankment is being widened.

Column-supported embankments are constructed over soft ground to accelerate construction, improve embankment stability, control total and differential settlements, and protect adjacent facilities. The columns that extend into and through the soft ground can be of several different types: driven piles, vibro-concrete columns, deep-mixing-method columns, stone columns, etc. The columns are selected to be stiffer and stronger than the existing site soil, and if properly designed, they can prevent excessive movement of the embankment. If accelerated construction is necessary, or if adjacent existing facilities need to be protected, then column-supported embankments may be an appropriate technical solution. Column-supported embankments are in widespread use in Japan, Scandinavia, and the United Kingdom, and they are becoming more common in the U.S. and other countries. The column-supported embankment technology has potential application at many soft-ground sites, including coastal areas where existing embankments are being widened and new embankments are being constructed.

The cost of column-supported embankments depends on several design features, including the type, length, diameter, spacing, and arrangement of columns. Geotechnical design engineers establish these details based on considerations of load transfer, settlement, and stability. A report by Filz and Stewart (2005) addresses the load transfer and settlement issues. Established procedures are available for analyzing the stability of embankments supported on driven piles and on stone columns. Stability analysis methods for embankments supported on piles and stone columns are presented in appendices to this report. In Japan and Scandinavia, transportation embankments on soft ground are often supported on columns installed by deep mixing methods (DMM). This technology is also finding more frequent application in other countries, including the United States. Limit equilibrium methods are used for analyzing the stability of embankments supported on deep-mixing-method columns, but these methods only reflect composite shearing through the columns and soil, and they do not reflect the more critical failure modes of column bending and tilting that can occur when the columns are strong.

The primary emphasis of this research dissertation is on stability of embankments supported on columns installed by the deep mixing method because (1) new embankments at the I-95/U.S. Route 1 interchange, in Alexandria, Virginia, were being designed using columns installed by the deep mixing method at the time this research was initiated and (2) more uncertainty exists in the literature about this case than for embankments supported on driven piles or stone columns.

In the deep mixing method, stabilizers are mixed into the ground using rotary mixing tools to increase the strength and decrease the compressibility of the ground. The various techniques that constitute the deep mixing method originated independently in Sweden and Japan in the 1960's (Porbaha 1998). Bruce (2001) defines the deep mixing method as "the methods by which materials of various types, but usually of cementatious nature, are introduced and blended into the soil through hollow, rotated shafts equipped with cutting tools, and mixing paddles or augers, that extend for various distances above the tip." In the dry method of deep mixing, dry lime, cement, fly ash, and/or other stabilizers are delivered pneumatically to the mixing tool at depth. In the wet method of deep mixing, cement-water slurry is introduced through the hollow stem of the mixing tools. Since its inception, deep mixing has been widely used to improve the strength and reduce the compressibility of soft soils.

There is a range of behavior covered by driven piles, stone columns, and deep-mixed columns used to support embankments over soft soils. Stone columns and piles can be seen as the two ends of the spectrum, with the behavior of lime, lime-cement, and soil-cement columns somewhere in between. The differences in behavior between these foundation types are due to column strength, stiffness, and ability to transmit a bending moment. Driven piles are essentially designed to carry the full weight of the embankment and transmit all loads to deeper soils. Stability analyses of embankments founded on stone columns are performed by using a composite shear strength based on the shear strength of the soil, the shear strength of the columns, and the area replacement ratio. This approach reasonably approximates composite behavior when the column type is granular. Stone columns are only somewhat stronger than in-situ material, and they are unable to transmit a bending moment along their length.

Currently, standard practice is to analyze the stability of embankments founded on ground treated with deep-mixed columns by using a composite shear strength based on the shear strength of the soil, the shear strength of the columns, and the area replacement ratio (CDIT 2002, EuroSoilStab 2002, Broms 1999, Kivelo 1998, Wei et al. 1990). In concept, this is the same approach as used for stone columns, however, there are several constraints placed on the analysis of deep-mixed foundations.

Although shear failure is the assumed failure mode for existing methods of analysis, recent research by Kitazume et al. (1996) reveals that deep-mixed columns can fail in several different modes, including bending and tilting failure (CDIT 2002). Kivelo (1998) investigated the ultimate shear strength of lime-cement columns considering bending, but his approach is not currently incorporated into stability analysis procedures, mainly owing to lack of complete understanding of this phenomenon (Porbaha 2000). When slope stability is a concern for embankments founded on soil reinforced with columns, CDIT (2002) recommends that numerical analyses be performed concurrent with slope stability analyses to investigate displacements. Displacements from numerical analyses may provide designers an indication of whether failure mechanisms other than shear failure may occur.

A principle outcome of this research is to recommend that engineers use numerical stress-strain analyses to calculate the stability of embankments supported on columns installed by the deep mixing method. Such analyses do reflect the multiple realistic failure mechanisms that can occur when strong columns are installed in weak ground. Detailed recommendations for performing numerical analyses in this application are presented.

Another important recommendation is that engineers should use reliability analyses in conjunction with numerical analyses of the stability of embankments supported on deep-mixing-method columns. Reliability analyses are necessary because deep-mixed materials are highly variable and because typical variations in clay strength can induce abrupt tensile failure in the columns, unless properly accounted for in design.

An additional benefit of reliability-based design is that it permits rational development of statistically based specifications for constructing deep-mixed materials. Such specifications can reduce construction contract administration problems because they allow for some low strength values while still provide assurance that the design intent is being met.

The primary purpose of this research is to develop reliable procedures that geotechnical engineers can use to analyze the stability of column-supported embankments. Although this research focused on columns installed by the deep mixing method, the stability analysis methods presented here are also expected to apply to vibro-concrete piles.

2. LITERATURE REVIEW

2.1. Introduction

This literature review was based on searches using Compendex®, Web of Science®, and other search engines, including those supported by the American Society of Civil Engineers, the Federal Highway Administration, and the Swedish Geotechnical Institute. The contents of relevant journals and conference proceedings were also surveyed. Much information has been published on the use of deep-mixing-method columns to support roadway embankments and similar structures. Five hundred literature sources have been collected for this study that include case histories, research, current state of the art reports, and design methodologies. However, many of these sources deal primarily with settlement control rather than issues related to the edge stability of these systems. Because this research is concerned with the edge stability of embankments founded on deep-mixing-method columns, articles related to issues of stability rather than settlement receive the most attention in this literature review. This chapter presents information published on this topic grouped into the following five categories; (1) installation and overview of deep-mixing columns, (2) property values and variability of deep-mixing-method materials, (3) analysis methods for stability of embankments supported on deep-mixed columns, (4) case histories and previous research, and (5) related columnar technologies. Obviously, some sources cover more than one of these categories.

2.2. Installation and overview of deep-mixed columns

Deep mixing includes a broad range of installation methods and technologies, most of which are proprietary. Several sources published in the last few years are devoted to construction equipment, installation, application, and cost of these technologies.

2.3. Property values and variability of deep-mixing-method materials

The engineering properties of soils stabilized by the deep mixing method are influenced by many factors including the water, clay, and organic contents of the soil; the type, proportions, and amount of binder materials; installation mixing process; installation sequence and geometry; effective in-situ curing stress; curing temperature; curing time; and loading conditions. Given all the factors that affect the strength of treated soils, the Japanese Coastal Development Institute of Technology (CDIT 2002) states that it is not possible to predict within a reasonable level of accuracy the strength that will result from adding a particular amount of reagent to a given soil, based on the in-situ characteristics of the soil. Consequently, mix design studies must be performed using soils obtained from a project site. Laboratory preparation and testing of specimens is discussed by Jacobson et al. (2005) for the dry method and by Filz et al (2005) for the wet method. Even relatively modest variations in binder materials may result in greatly different properties of the mixture. Also, field mixed and cured materials will differ from laboratory mixed and cured material. Construction contractors have experience relating the strength of laboratory mixed and cured specimens to the strength of field mixed and cured materials. Furthermore, engineering properties of mixtures are time dependent, due to long-term pozzolanic processes that occur when mixing cement or lime with soil. Design is generally based on the 28-day strength of the mixture.

Most strength and stiffness information about deep-mixed materials comes from unconfined compression tests. Numerous studies (Miura et al. 2002, CDIT 2002, Shiells et al. 2003, Dong et al. 1996, Matsuo 2002, Bruce 2001, Jacobson et al. 2005, EuroSoilStab 2002, Takenaka and Takenaka 1995, Hayashi et al. 2003) show that the unconfined compressive strength, q_u, of deep-mixed materials increases with increasing stabilizer content, increasing mixing efficiency, increasing curing time, increasing curing temperature, decreasing water content of the mixture, and decreasing organic content of the base soil. One interesting interaction of these factors is that increasing the water content of the mixture can increase mixing efficiency, so that in the case of low-water-content clays, adding water to the mixture can increase the mixture strength (McGinn and O'Rourke 2003). Nevertheless, it remains true that, for thoroughly mixed materials, a decrease in the water-to-cement ratio of the mixture produces an increase in the unconfined compressive strength.

For soils treated by the dry method of deep mixing, values of unconfined compressive strength may range from about 2 to 400 psi, and for soils treated by the wet method of deep mixing, values of the unconfined compressive strength may range from about 20 to 4,000 psi (Japanese Geotechnical Society 2000, Baker 2000, Jacobson et al. 2003). For a wet deep-mixing project at the Oakland Airport in California, the minimum and average values of unconfined compressive strength were specified to be 100 and 150 psi, respectively (Yang et al. 2001). For the I-95/Route 1 interchange project, which also employed the wet method of deep mixing, the minimum and average values of unconfined compressive strength were specified to be 100 and 160 psi, respectively (Shiells et al. 2003, Lambrechts et al. 2003).

Variability of the unconfined compressive strength can be expressed in terms of the coefficient of variation, which is the standard deviation divided by the mean. Values of the coefficient of variation in the published literature for deep-mixed materials range from about 0.15 to 0.75 (Kawasaki et al. 1981, Honjo 1982, Takenaka and Takenaka 1995, Unami and Shima 1996, Matsuo 2002, Larsson 2005).

Secant values of Young's modulus of elasticity determined at 50% of the unconfined compressive strength, E_{50}, have been related to the unconfined compressive strength, q_u. For the dry method of deep mixing, values of the ratio of E_{50} to q_u have been reported in the range from 50 to 250 (Baker 2000, Broms 2003, Jacobson et al. 2005). For the wet method of deep-mixing, values of the ratio of E_{50} to q_u have been reported in the range from 75 to 1000 (Ou et al. 1996).

Reported values of Poisson's ratio for deep-mixed material ranges from 0.25 to 0.50 (CDIT 2002, McGinn and O'Rourke 2003, Terashi 2003, Porbaha et al. 2005).

The unit weight of soils treated by deep mixing is not greatly affected by the treatment process. For the dry method of deep mixing, Broms (2003) reports that the unit weight of stabilized organic soil with high initial water content can be greater than the unit weight of untreated soil and that the unit weights of inorganic soils are often reduced by dry mix stabilization. The Japanese CDIT (2002) reports that the total unit weight of the dry-mixed soil increases by about 3% to 15% above the unit weight of the untreated soil. CDM (1985) indicates that, for soils treated by the wet method of deep mixing, the change in unit weight is negligible. However, at the Boston Central Artery/Tunnel Project, McGinn and O'Rourke (2003) report that a significant decrease in unit weight occurred because the initial unit weight of the clay was relatively high and water was added to pre-condition the clay before mixing.

2.4. Analysis methods for stability of embankments supported on deep-mixed columns

Two reports published recently summarize most of what is currently known about embankments founded on DMM columns, and how these systems are implemented in practice: EuroSoilStab (2002), and CDIT (2002). A third report by Broms (2003) covers much of this material, and includes many ideas about the behavior of DMM columns that have been published over the past twenty-five years. All three of these references describe slope stability analysis methods that are based on limit equilibrium.

The stability of ground improved with deep-mixing-method columns is often analyzed using a short term, undrained analysis because the shear strengths of the columns and the soil between columns generally increase over time. A composite, or weighted average, undrained shear strength is used to evaluate slope stability for ground improved with columns, assuming complete interaction between stiff columns and the softer surrounding soil (Broms and Boman 1979). Embankment stability is typically analyzed assuming a circular shear surface as shown in Figure 2-1.

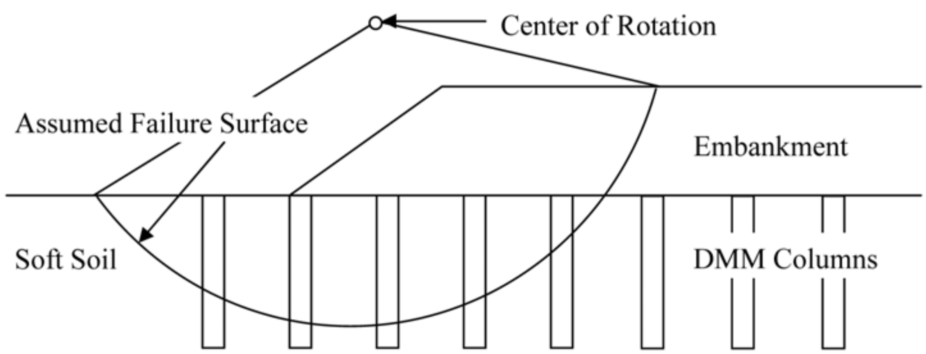

Figure 2-1. Circular sliding surface (after Broms and Boman 1979)

Broms and Boman (1979) recommended the composite shear strength analysis for lime piles, which are softer than the lime-cement and soil-cement columns in use today. The interaction of these stiffer and more brittle columns with the surrounding unstabilized soil may be different from the interaction in ground stabilized with lime columns, so the applicability of a composite shear strength analysis to ground stabilized with lime-cement and soil-cement columns is uncertain (Broms 1999). High shear forces and bending moments can be transferred to the stiffer columns, causing the columns to fail in bending or by tilting (Kivelo and Broms 1999). Back calculations of a failed embankment in Scandinavia indicated that column strengths would need to be only 10% of the actual measured column strengths for shearing to account for the failure (Broms 2003). This indicates that other column failure modes were involved in the embankment failure.

Kivelo (1998) investigated the ultimate shear strength of lime-cement columns considering bending, but his approach is not currently incorporated into stability analysis procedures, mainly owing to lack of complete understanding of this phenomenon (Porbaha 2000). Rather, limitations are placed on the shear strengths used in the stability analysis. Furthermore, the use of isolated vertical deep-mixed columns is avoided under certain circumstances.

In order to stabilize embankments, columns are often overlapped in rows constructed perpendicular to the centerline of the embankment. These rows of columns, also referred to as panels, are shown in Figure 2-2. Panels are often connected with longitudinal walls, sometimes referred to as shear walls. Such column arrangements are analyzed with the same stability analysis as isolated columns.

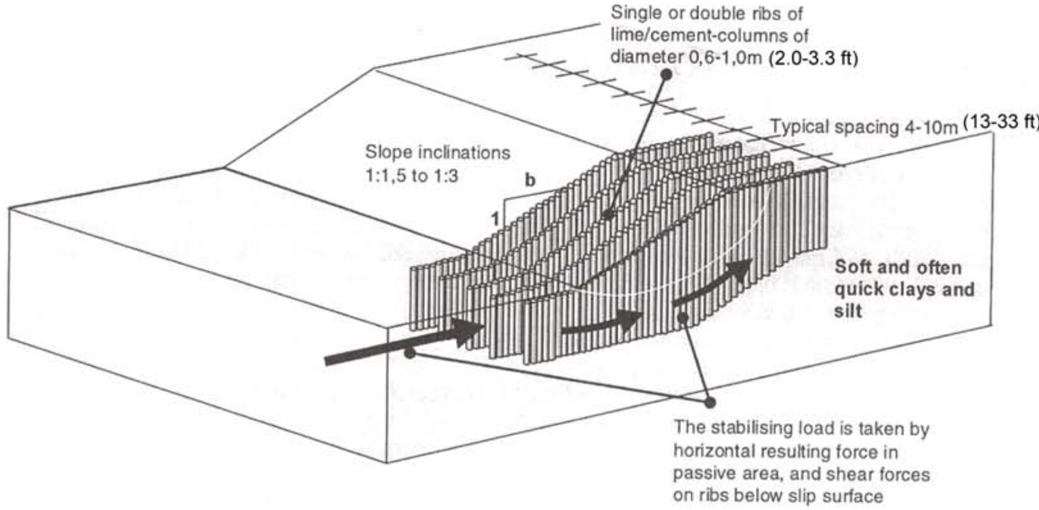

Figure 2-2. Slope stabilization with lime-cement columns (Watn et al. 1999)

2.5. Case histories and previous research performed

Published work in this area
consists of case histories of field performance, centrifuge test results, numerical analyses, state-of-practice reports, and other types of scholarly discussions on column behavior.

Although many case histories have been published illustrating the settlement reduction achieved with the use of DMM columns, few case histories document instances where these foundation systems exhibit slope stability problems. Terashi (2003) mentions that bending failure of DMM columns has happened in a couple of unreported cases. Broms (2003) evaluates two case histories in Scandanavia where he believes progressive stability failure has occurred. Kivelo (1998) also documents these two failures, and the original case histories were published in Swedish by Jacklin et al. (1994) and Arner et al. (1996).

An embankment constructed at the Island of Orust, Sweden failed during the final stage of construction. The six-meter-high embankment was placed on a one-meter crust overlying seven to twenty meters of soft marine clays. The embankment was constructed on 2.0-ft-diameter lime/cement columns extended to a depth of 49-ft and arranged in a square pattern with 3.3 and 5.9-ft spacing. These arrangements result in area replacement ratios of 27 and 12 percent, respectively. The embankment experienced up to a meter of settlement, which was attributed to an edge stability failure.

A test embankment at Norral, Sweden experienced large vertical and horizontal displacements. These deflections were also attributed to an edge stability failure. The 26-foot-high embankment was placed on a three-foot crust overlying 25 to 30 feet of soft clay and organic silt. The embankment was constructed on 2.0-ft and 2.6-ft diameter lime/cement columns arranged in a square pattern beneath the center of the embankment, with panels used for the side slopes of the embankment. The 2.0-ft columns had a center-to-center spacing of 3.6-ft and the 2.6-ft columns had a center to center spacing of 4.6-ft. These arrangements result in area replacement ratios of 23 and 26 percent, respectively.

A list of field tests, centrifuge tests, and numerical analysis of embankments founded on columns is included as Table 2-3. Unfortunately, only a few of these investigations considered the slope stability of embankments founded on deep-mixed columns. Field tests included in this table use means other than columns to prevent slope failure, and they concentrate on settlement reduction. However, three series of centrifuge tests have been performed to evaluate the performance of DMM columns used to improve soft clay subject to embankment type loading: (1) Miyake et al. (1991), (2) Kitazume et al. (1996), and (3) Inagaki et al. (2002).

Table 2-3. Research into the behavior of column-supported embankments

Reference	Improvement Method	Test/Analysis	Loading
Alamgir et al. (1996)	Columnar inclusion	FEM	Embankment settlement
Almeida et al. (1985)	Staged construction	Centrifuge	Embankment
Asaoka et al. (1994)	SCP	Field case/FEM	Tank settlement
Aubeny et al. (2002)	Piles with geotextile	FEM	Embankment
Bai et al. (2001)	Soil -cement columns	FEM	Embankment settlement
Baker (1999)	Lime-cement columns	FEM	Embankment settlement
Balaam and Booker (1985)	Stone columns	FEM	Vertical
Ekstrom et al. (1994)	Soil -cement columns	Test fill	Settlement
Enoki et al. (1991)	SCP	Lab triaxial test	Composite shear
Han and Gabr (2002)	Piles	FDM	Settlement
Greenwood (1991)	Stone columns	Load tests	Vertical
Ilander et al. (1999)	Lime-cement columns	Test embankment/FEM	Settlement
Inagaki et al. (2002)	Cement columns	Centrifuge/FEM	Embankment
Jagannatha et al. (1991)	Stone columns	Load tests	Vertical
Jones et al. (1990)	Piles with geotextile	FEM	Embankment
Kaiqui (2000)	Cement columns	FEM	Embankment
Karastanev et al. (1997)	(Kitazume tests)		
Kempfert et al. (1997)	Piles with geotextile	FEM	Embankment settlement
Kempton et al. (1998)	Piles with geotextile	FDM	Embankment settlement
Kimura et al. (1983)	SCP	Centrifuge	Vertical
Kitazume et al. (2000)	Cement columns	Centrifuge/FEM	Vertical and lateral
Kitazume et al. (1996)	SCP	Centrifuge	Vertical and backfill
Long and Bredenberg (1999)	Lime-cement columns	FEM	Deep excavations
Miyake et al. (1991)	Cement columns	Centrifuge/FEM	Embankment
Rogbeck et al. (1998)	Piles with geotextile	FDM	Embankment settlement
Russel and Pierpoint (1997)	Piles with geotextile	FDM	Embankment settlement
Takemura et al. (1991)	SCP	Centrifuge	Embankment
Terashi et al. (1991)	SCP	Centrifuge/Full scale	Tank settlement
Terashi and Tanaka (1983)	Soil-cement columns	FEM	Settlement
Watabe et al. (1996)	SCP & pile	Centrifuge	Retaining structure

notes: SCP is sand compacted pile, FEM is finite element method, FDM is finite difference method

Miyake et al. (1991) performed centrifuge tests to model edge stability of an embankment supported by columns. Clay slurry was poured into the model box and consolidated with self-weight under a centrifugal acceleration of 80g. Cylindrical holes were excavated with a thin walled sampler and replaced with polyvinyl chloride bars intended to represent cement-treated soil columns. These tests were performed to investigate the effect that location of the treated zone has on embankment stability. The three patterns of improvement are listed in Table 2-4, and a schematic diagram is shown in Figure 2-3.

Centrifuge tests performed on the three improvement configurations illustrate the advantage of improving soil beneath the full height of the embankment rather than beneath the side slopes. Improvement pattern (a) located beneath the slope experienced large lateral deformations while cases (b) and (c) experienced no significant movement in the tests.

Table 2-4. Miyaki et al. (1991) test cases

Case No.	Diameter of Columns (in)	Number of Columns	Replacement Ratio (%)	Pattern
1	0.59	64	49.6	a
2	0.59	64	49.6	b
3	0.59	15+49	23.2, 38.0	c

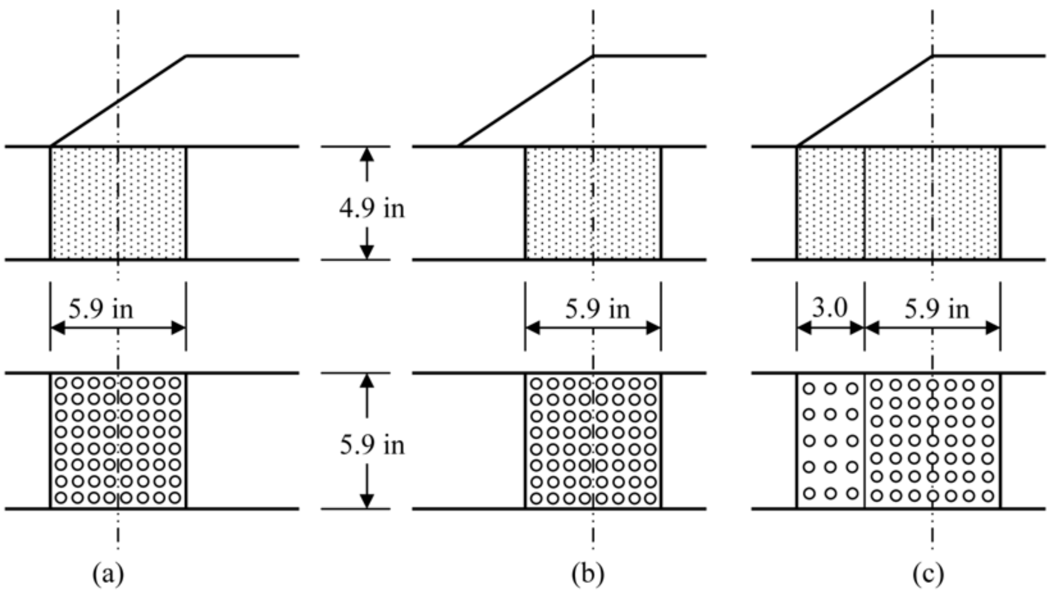

Figure 2-3. Miyaki et al. (1991) centrifuge test schematics

Kitazume et al. (1996) performed centrifuge tests using an inclined load on soft clay improved with soil-cement dowels that illustrate the effect failure modes other than slip surfaces have on stability. A drainage layer of sand was placed at the bottom of the model test box. Clay slurry was poured into forms at both ends of the model test box and consolidated under load on the laboratory floor. The columns were cast outside the model test box using soil-cement slurry and, after curing, they were arranged in a rectangular pattern in the middle of the box. Clay slurry with higher water content was then pumped between columns. The schematic is included as Figure 2-4.

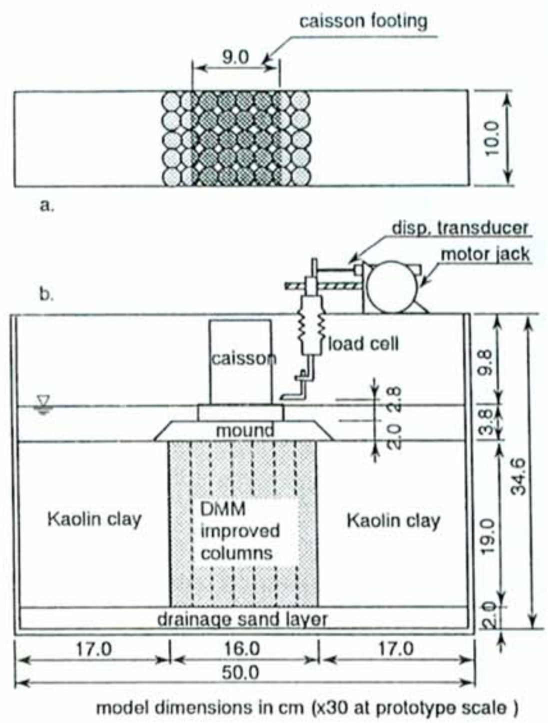

Figure 2-4. Model schematic (Kitazume et al. 1996)

Racking, or tilting of columns and bending failure in the columns can be seen in the photographs of the centrifuge test results shown in Figure 2-5. The horizontal and vertical loads causing failure in the experiments were compared to the loads causing failure from a slope stability analysis. The experimental loads were much smaller than the failure loads from the slope stability analyses. When bending failure was included in the slope stability analyses, the results matched experiments much more closely, as shown in Figure 2-6.

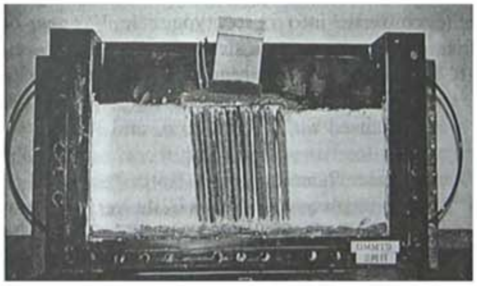 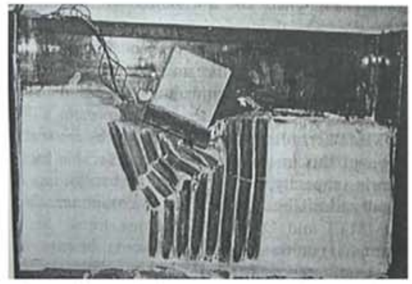

Figure 2-5. Column tilting and column bending (Kitazume et al. 2000)

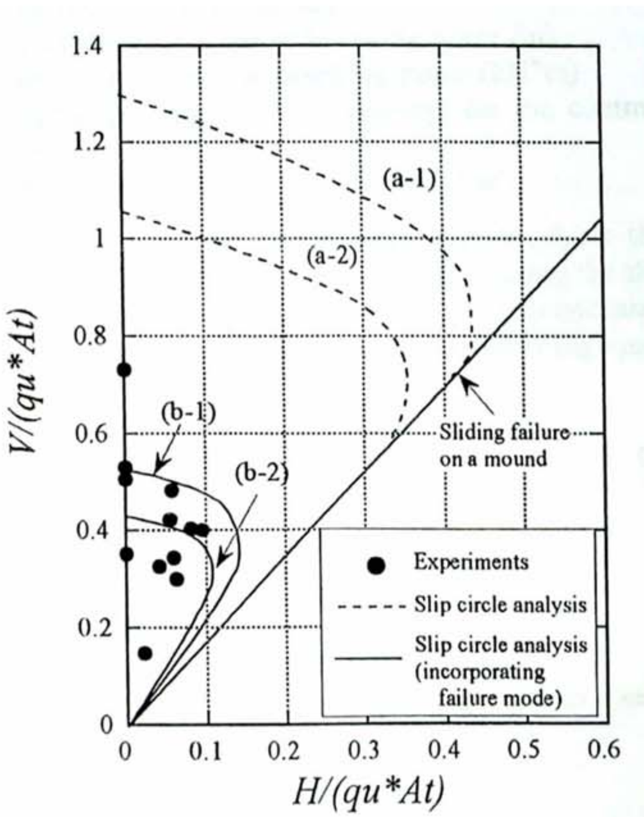

Figure 2-6. Slip circle analysis with and without bending failure mode

(after Kitazume et al 2000)

The dashed line (a-1) in Figure 2-6 corresponds to an undrained slope stability analysis assuming a circular shear surface with the shear strength of the columns equal to one-half the unconfined compressive strength. Due to the potential for progressive failure of these foundations, Terashi et al. (1983) recommend using residual shear strengths for stability analyses. Kitazume et al.

(2000) used a residual strength that is 80 percent of the design strength based on the work of Tatsuoka et al. (1983). The dashed line (a-2) incorporates this residual strength. Still, measured failure loads were much smaller than indicated by this analysis. The solid lines (b-1) and (b-2) represent the results of stability analyses incorporating bending failure for the design and residual strengths, respectively. It can be seen that line b-2 is in reasonable agreement with the experimental results. Kitazume et al. (2000) include a rough approximation of bending failure in the stability analyses. The method of stability analysis used to develop lines (b-1) and (b-2) in Figure 2-6 is not intended as a predictive measure of failure.

Finite element analyses were also used to model the experiments. Kitazume et al. (2000) used Mohr-Coulomb properties for the clay and elastic properties for the columns. This approach could not model bending failure in the columns, but it did effectively model column tilting. The researchers determined that tilting failure depended on clay strength outside the zone improved by columns.

Inagaki et al. (2002) performed centrifuge tests to show the effect that deep-mixing columns had on soft clay foundations for embankments. A schematic diagram of their tests is included as Figure 2-7. The centrifuge model was created by pouring clay slurry over a compacted sand base and allowing the clay to consolidate under normal gravity. Columns were installed in the model by coring holes in the clay and filling them with soil-cement slurry. Three experiments were performed with different column geometries used to support the slope. Two cases were run with columns extending down to the base sand layer. These tests showed bending failure of the interior columns and tilting failure of columns at the toe. One case was configured with columns stopping short of the base sand layer. These “floating” columns did not show bending or tilting failure; however, they did experience larger lateral deflections.

Inagaki et al. (2002) performed numerical analyses of the centrifuge tests using a finite element model. The soft clay was modeled using a water-soil coupled, elasto-plastic model known as the Sekiguchi/Ohta Model. This is a modified cam-clay model that incorporates soil anisotropy. The embankment and columns were modeled as elastic materials. The two-dimensional plane-strain model was able to match column deflections very closely.

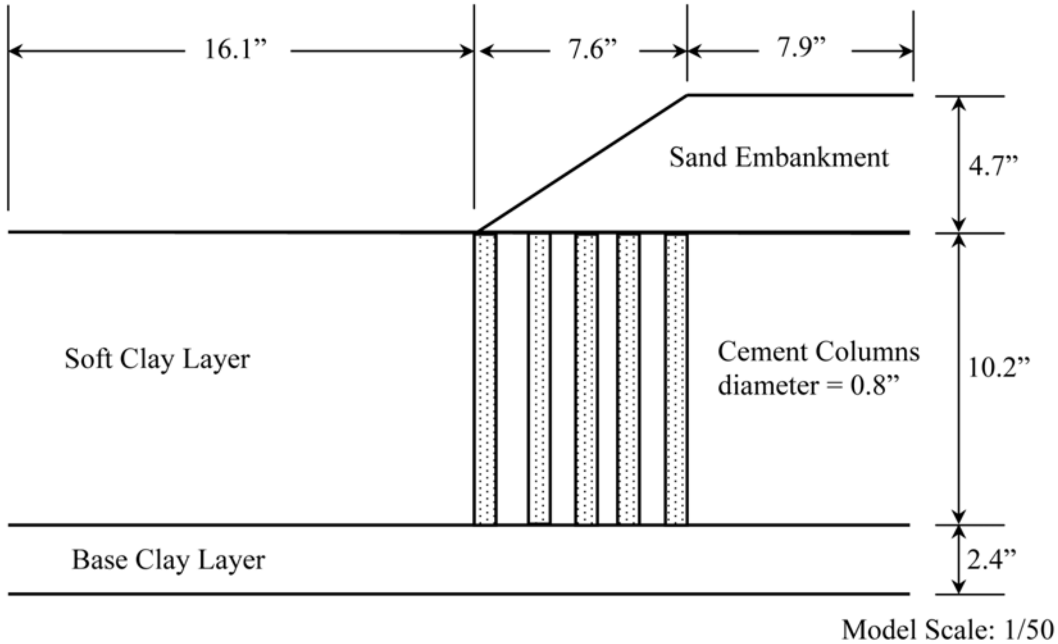

Figure 2-7. Inagaki et al. (2002) centrifuge test schematic

A research dissertation (Kivelo 1998) addressed the edge stability of embankments founded on lime-cement columns, and incorporated moment capacity into a limit equilibrium analysis with a non-circular shear surface. He resolved the limiting horizontal force acting along the shear surface that could be carried by columns based on several possible failure mechanisms shown in Figure 2-8. These failure mechanisms were originally developed by Broms (1972) when he addressed the use of piles to stabilize slopes. Failure mode d in Figure 2-8 represents flow of the soil around intact columns.

Kivelo (1998) provides equations for these failure modes; however, there are some obstacles to implementing them in a slope stability analysis. The presence of columns changes the location of the critical failure surface, so the equations need to be incorporated into a computer program to search for the critical failure surface. The equations determine a horizontal force applied at the location of the failure surface. It is very conservative to only include the horizontal component of the resisting force acting along the shear surface for steep inclinations of the failure surface. The failure modes based on bending failure require values of the allowable bending capacity for columns, and there is substantial uncertainty in the bending capacity of deep-mixed columns. Kivelo (1998) presents a method to determine the bending capacity of columns based on the plasticized area of the column, but does not explain how to find that area. The bending capacity is determined using an assumption that the columns have no tension capacity.

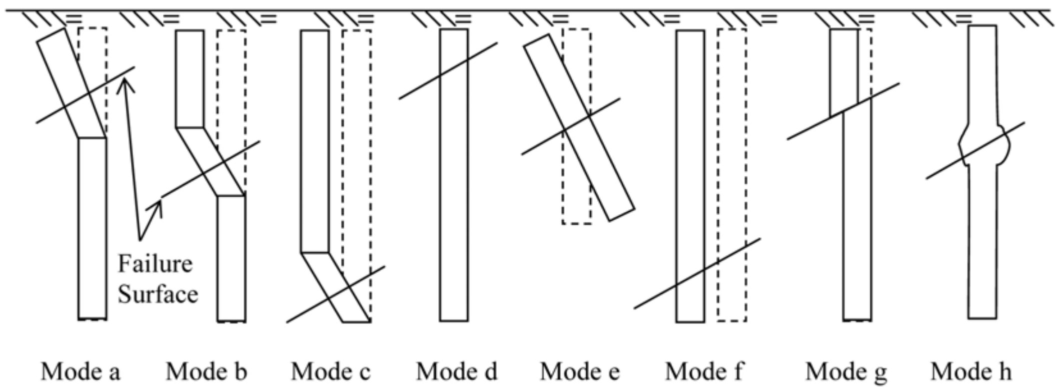

Figure 2-8. Failure mechanisms for deep-mixing-method columns (Kivelo 1998)

Kivelo and Broms (1999) state that embankments founded on soft soils and improved with brittle, stiff columns will fail in a progressive manner. This behavior was noted in the centrifuge tests performed by Kitazume et al. (1996). As the failing soil mass begins to strain, stresses concentrate in the first row of columns. If the first row of columns fails to arrest the sliding mass, load is then concentrated on the second row of columns. Broms (2003) applies a method to evaluate progressive failure to the two documented failures in Scandinavia mentioned above. Although this method is interesting, it has not been validated by testing or investigated by other authors.

2.6. Related columnar technologies

There is much information in the literature that can be applied to the edge stability of columns used to support embankments that is written about technologies other than deep mixing. These technologies include stone columns used to support embankments, pile-supported embankments, and piles used to stabilize slopes.

2.6.1. Stone columns

Stone columns and sand columns, which are similar ground improvement technologies, result in a vertical column of material that is stronger than the surrounding native soil. These methods of ground improvement have been used successfully to enable embankment construction over deposits of soft soil. An FHWA manual (Barksdale and Bachus 1983) clearly summarizes design methodology for stone columns used for embankment construction, including edge stability analyses. Sand columns, which have been used in Japan, follow the same design methodologies as are used for stone columns.

It is useful to understand the work that has been done with stone columns because the design methods are very similar to those used for deep-mixed columns. Concepts such as area replacement ratio, composite strength, and stress concentration apply to both technologies. Some of the concepts related to analysis of embankments on stone columns have been applied in this research to limit equilibrium or numerical analyses of the stability of embankments founded on deep-mixed columns.

2.6.2. Pile-supported embankments

Piles are designed to carry the full weight of the embankment. Filz and Stewart (2005) covers the design aspects required to transfer embankment loads to the pile foundation with a bridging layer. Many case histories are in the literature for geotextile reinforced, pile supported (GRPS) embankments. These case histories have been summarized by Aubeny and Briaud (2003) and included here as Table 2-5.

Piled raft foundations are somewhat similar to pile supported embankments. Numerous articles have been published on piled raft foundations, which are not covered here, other than to include the modulus values recommended by Poulos (2002) for soil beneath the raft, based on the work of Decourt (1989,1995). The soil Young's modulus below the raft is twice the value N from a standard penetration test in MPa (20.9 times *N* in tsf). The soil Young's modulus along the piles and below the piles is three times the value *N* in MPa (31.3 times *N* in tsf).

Table 2-5. Case histories for GRPS embankments (from Aubeny and Briaud 2003)

Case No.	Reference	Application	Soil condition	Pile type	Geosynthetic type	Design parameters
1	Reid et al. (1983)	Near bridge abutment	Soft clay	Concrete displacement pile	Membrane (paraweb)	H=10m, s=3.5-4.5m, a=1.1-1.5m, P_c=5-14%, N=1
2	Barksdale et al. (1983)	Railway	Very soft peat	Rigid stone columns	Fabric	H=7.6m,s=1.6-2.2m,d=0.51-0.56m,T=0, P_c =6-8%, N=1
3	Jones et al. (1990)	Railway	Very soft alluvium and peat.	Semi-precast concrete pile.	Geotextile "Paralink"	H=3-5m, s=2.75m,a=1.4m, T=0.5m, P_c=20%, N=1
4	Tsukada et al. (1993)	Street pavement	Peat	Concrete pile	Geogrid "Tensar SS2"	H=1.5m, s=2.1m, d=0.8m, P_c =11%, N=1,T=0
5	Holtz et al. (1993)	Pavement	Uniform grey clay	Timber pile	Geotextile "multifilament	H=5-6m,s=1.5m,a=1m, P_c=44%, N=3
6	Bell et al. (1994)	Toll Plaza	Highly compressible peat and estuarine clay	VCC The columns.	Tensar SS2 geogrid	H=2.5-6.0m,s=2.2-2.7m, d=0.4m, N=2
7	Card et al. (1995)	Docklands Light Railway (DLR)	Silty organic clay, Peat and Clay/sand	Driven or continuous flight augured piles	Biaxial Tensar SS2 geogrid	H=2.5-3m,s=3m, a=1m, N=3, d=0.45m.
8	Topolnicki (1996)	Highway and tramway	Loose fill, peat Organic clay	VCC	Geogrid "Tensar SS1" and 'Tensar SS2"	H<1.5m,s=1.8-2.5m, d=0.55m, P_c =9-17%, N=2-3, T=0
9	Brandl et al. (1997)	Railway	Peat and organic silt	Driven pile	Geogrid	H>2m, s=1.90m,d=0.118m, a=1.0m, P_c =35%, N=3.
10	Geo-Institute (1997)	Highway embankment bridge abutment	A mixture of soft clays, silts and sands with bands of peat	VCC/Stone columns	Geotextile	H<7m, s=1.6m for VCC and s=2.2m for stone column. N=1.
11	Jenner et al (1998)	Bypass	Peat and soft silty alluvial strata	VCC	Tensar SS1 and Tensar SS2 geogrid	H=4-7m,s=2.05-2.35, d=0.45m,a=0.75m,N=2-3.
12	Rogbeck et al. (1998)	Full scale testing	Loose silt and fine sand	Precast concrete pile	Geogrid	H=1.7m, s=2.4m, a=1.2m, P_c=25%,N=1
13	Kuo, et al. (1998)	MSE Walls	Very soft waste clay	Timber pile.	Geotextile	H=6m,s=1.5m,d=0.3m, P_c=3%, N=2.
14	Alzamora et al. (2000)	Segmental retaining walls	0 to 1 blow count organic silt and clay	Jet grout column	Uniaxial Geogrid	H=2-8.2m, s=3m, d=1.2m, P_c =13%, N=3

Note: H- embankment fill height; s-pile spacing at centers; d-pile diameter; a- cap width; T- cap thickness; c-cushion thickness; e-efficacy (%); P_c- percent coverage of pile caps; N- number of geosynthetic layers; VCC- vibro concrete column; Efficacy- defined percentage of the embankment load carried by pile cap.

2.6.3. Piles used to stabilize slopes

Viggiani (1981) provides a summary of the use of piles to stabilize landslides. Piles have been used in slopes for over 100 years, even though a well established and widely used design procedure is still to be developed. Although Vigianni notes that some failures have occurred (Root 1958, Baker and Marshall 1958), there have also been successes (DeBeer and Wallays 1970, Ito and Matsui 1975, Sommer 1977, Fukuoka 1977). Essentially, the design of these piles is a three step process; (1) determine the shear force required to increase the factor of safety by the desired amount, (2) evaluate the maximum shear force that each pile can receive from the sliding soil and transmit to the stable underlying soil, (3) select the type and number of piles and their most appropriate locations on the slope.

Step (1) generally uses limit equilibrium to determine the resisting force required. Generally, piles are used when only a small increase in stability is required. If the factor of safety is one, it is possible to evaluate the shear force needed to increase the factor of safety by the desired amount. If the factor of safety is anything other than one, Hutchinson (1977) describes the difficulty of assessing existing stability and improvement.

There are several different approaches used to determine the resisting force in step (2):

- Consider the piles as cantilevers provided they penetrate into stable soil 1/3 their length (Baker and Yoder , 1958).
- Calculate the force based on rupture conditions, and estimate a range of yield values for the pile-soil interaction (DeBeers and Wallays 1970, DeBeer 1949, Brinch Hansen 1961)
- Apply a theory of plastic deformations or viscous flow. This determines the force acting on piles in a row when soil is forced to squeeze between piles. Ito and Matsui (1977) determined the soil force acting on piles above the critical surface as a function of soil strength, pile diameter, spacing, and position.
- Introduce a coefficient of horizontal subgrade reaction, which determines pile-soil interaction (Fukuoka 1977).
- Evaluate the ultimate load of a vertical pile acted upon by a horizontal load for cohesive soil (Broms 1964).

Hassiotis and Chameau (1984) present a procedure to analyze piles used to stabilize slopes that determines forces acting on piles above the critical surface based on plastic equilibrium and determines forces acting on piles below the critical surface based on subgrade reaction. The authors also provide a computer program to perform these calculations.

Reese et al. (1992) present a four step method to analyze drilled shafts used to stabilize slopes;

1) Estimate loads due to earth pressures.
2) Assess the resistance of soil below the sliding surface.
3) Estimate the response of the drilled shaft – above and below the sliding surface.
4) Estimate factor of safety for the slope reinforced with drilled shafts.

Vigianni expands on the approach taken by Broms (1964) to investigate the pile resistance in a soft layer of known thickness, overlying a firm underlying soil. The shear surface is assumed to be between the two layers, and both the force acting on the upper part of the pile and the force acting on the lower part of the pile can be determined using Equation 2-1.

$$p_y = k\, c\, d\, L \tag{2-1}$$

where p_y is the horizontal load in the pile, d is the pile diameter, L is the length of pile either above or below the failure surface, c is the cohesion either above or below the failure surface, and k is a bearing capacity factor. It is interesting to note that Reese (1958) found k to be about 2 at the soil surface and to increase with depth until reaching a constant value at $3d$. Broms (1964) uses a simplified pattern with p_y equal to zero at the ground surface and increasing with depth until reaching a constant value at $1.5d$. When this type of failure mode is applied to deep-mixing-method columns, Kivelo (1998) recommends using a k value of 9.

Broms (1972) presented multiple failure mechanisms for piles, for which equations were later applied to deep-mixed columns used to support embankments (Kivelo 1998) and summarized in Section 2.5.