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ANALYSIS OF REINFORCED EARTH
STRUCTURES USING PHYSICAL MODELS

by

CRAIG ALBERT HOULE, Hon. B.Sc.

A thesis submitted to the Faculty of
Graduate Studies and Research in partial
fulfillment of the requirements for the degree of

Master of Engineering

Department of Civil Engineering

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ABSTRACT

This investigation has centered on the utilization of two purely two-dimensional physical models to determine the failure modes and corresponding shapes of the failure surfaces occurring in model reinforced earth structures. A steel rod model and a base friction apparatus were used to investigate qualitatively the failure patterns these structures will undergo.

Two distinct modes of failure were discernible in these experiments. These are, Mode 1 in which there is a sufficient volume of soil behind the tensile reinforcing to allow a natural failure surface to form, and Mode 2 where the failure plane forms entirely within the reinforced earth block.

The shape of the failure plane occurring in these tests is greatly different from that which the Rankine theory predicts. The Mode 1 shape is best analysed by using a failure wedge analysis, and the Mode 2 pattern is best represented by a logarithmic spiral.

The suitability of the test apparatus used in this study to evaluate two-dimensional (as opposed to plane strain) problems in cohesionless soils was also evaluated. It was found that both models used in this study were capable of depicting the shapes and modes of failure of reinforced earth structures.

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To my parents,
and for Trudy

TABLE OF CONTENTS

	Page
ABSTRACT	i
ACKNOWLEDGEMENTS	ii
TABLE OF CONTENTS	iii
LIST OF FIGURES	vi
LIST OF TABLES	x
LIST OF SYMBOLS	xi
LIST OF APPENDICES	xiv
CONVERSION OF UNITS	xv
CHAPTER I. INTRODUCTION	1
1.1 General	1
1.2 The Nature of the Problem	2
1.3 Scope and Objectives of the Study	3
1.4 Outline of the Thesis	6
CHAPTER II. REINFORCED EARTH CONCEPT	8
CHAPTER III. THEORETICAL ANALYSIS OF REINFORCED EARTH STRUCTURES	15
3.1 Introduction	15
3.2 Lee's Analysis	15
3.3 Price's Analysis	19
3.4 Banerjee's Analysis	21
3.5 Al-Hussaini and Perry's Analysis	25
3.6 Juran and Schlosser's Analysis	27
3.7 Smith and Wroth's Analysis	29
3.7.1 Failure by Overturning	30
3.7.2 Failure by Tie Pullout	35
3.8 Romstad et al's Analysis	36
3.9 Logarithmic Spiral	45
CHAPTER IV. THEORETICAL CONCEPTS OF PHYSICAL MODELS	47
4.1 Introduction	47

CHAPTER IV. Cont'd.....

4.2 Steel Rod Model	49
4.2.1 Introduction	49
4.2.2 Theory of the Steel Rod Model	49
4.2.3 Stresses in the Steel Rod Model	50
4.3 Base Friction Model	51
4.3.1 Introduction	51
4.3.2 Theory of Base Friction Modelling	52
4.3.3 Stresses in the Base Friction Technique	63
4.3.4 Discussion	65

CHAPTER V. TEST APPARATUS 69

5.1 Steel Rod Model	69
5.2 Base Friction Apparatus	71

CHAPTER VI. MODELLING OF REINFORCED EARTH STRUCTURES . 77

6.1 Steel Rod Model	77
6.1.1 Soil	77
6.1.2 Soil Parameters	77
6.1.3 Tensile Reinforcing Strips	82
6.1.4 Wall Panels	84
6.1.5 Instrumentation	88
6.2 Friction Table	91
6.2.1 Soil	91
6.2.2 Soil Parameters	97
6.2.3 Tensile Reinforcing Strips	98
6.2.4 Wall Panels	98

CHAPTER VII. MODEL WALL TESTS 100

7.1 Steel Rod Model	100
7.1.1 Instrumented Model Reinforced Earth Walls	103
7.1.2 Critical Height Tests	106
7.1.3 Recording of Test Results	110
7.1.4 Test Results	113
7.2 Base Friction Apparatus	122
7.2.1 Construction of Walls	126
7.2.2 Recording of Test Results	127
7.2.3 Test Results	133

CHAPTER VIII. ANALYSIS OF RESULTS 143

8.1 Steel Rod Model	143
8.1.1 Instrumented Tests	143
8.1.2 Coulomb Wedge Analysis	145
8.1.3 Critical Height Ratios	148

CHAPTER VIII. Cont'd.....

8.2 Base Friction Apparatus	150
8.2.1 Logarithmic Spiral	151
8.2.2 Wedge Analysis	153
8.2.3 Effects of Variations in Retaining Wall Geometry	158
8.3 Comparison of Test Results	160

CHAPTER IX. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS . 166

9.1 Summary	166
9.2 Conclusions	166
9.3 Recommendations for Future Work	168

REFERENCES 170

APPENDIX A 180

APPENDIX B 195

APPENDIX C 201

APPENDIX D 202

APPENDIX E 241

LIST OF FIGURES

Figure		Page
1.1	Scope of the Study	4
2.1	Components of Reinforced Earth Structure . .	11
3.1	Failure of Reinforced Earth Structure by Formation of Failure Wedge	22
3.2	Forces Acting on Wedge and Reinforced Zone at Failure	31
3.3	Forces Acting on Wedge using Equilibrium Calculations from data obtained from Load Cells	34
3.4	Assumed Shape of Failure Surface	37
3.5	Components of Failure Wedge	38
4.1	Principle of Base Friction Model	54
4.2	Forces Associated with the Base Friction Model	55
4.3	Limiting Equilibrium for incipient Sliding on an inclined Plane (a) Gravity (b) Base Friction	57
4.4	Dynamic Sliding Without Toppling for Gravity	58
4.5	Force Vector Triangle for Dynamic Sliding (Gravity)	58
4.6	Dynamic Sliding Without Toppling for Base Friction	60
4.7	Solution of Corresponding Velocities for Dynamic Sliding of Base Friction	60
4.8	Dissimilarity of Gravity and Base Friction Models where Momentum is not negligible . .	64
5.1	Detail of Steel Rod Testing Apparatus . . .	70

Figure		Page
5.2	Base Friction Model	73
5.3	Base Friction Apparatus	74
5.4	Motor, Variable Speed Control and Reduction Gears for Base Friction Apparatus	76
6.1	Direct Shear Box Apparatus	79
6.2	Load-Displacement Curve for Direct Shear Test	80
6.3	Direct Shear Test Result for Angle of Internal Friction of steel rods	81
6.4	Modified 60 mm x 60 mm Direct Shear Box . .	83
6.5	Components for Modified Direct Shear Box .	85
6.6	Load-Displacement Curves for Strip/Soil Friction Test	86
6.7	Friction Angle for Soil/Strip Friction Test	87
6.8	Load-Displacement Curve for Soil/Wall Friction Test	89
6.9	Friction Angle for Soil/Wall Friction Test	90
6.10	Strain Gauged Connector in Reinforcing Strip	92
6.11	Instrumentation Monitoring Setup	92
6.12	Components used in Base Friction Tests . .	96
7.1(a)	Cross-section of Reinforced Earth Wall for Mode 1 Configuration	101
7.1(b)	Cross-section of Reinforced Earth Wall for Mode 2 Configuration	102
7.2	Failure of Instrumented Test Wall	105
7.3	Model Reinforced Earth Wall Prior to Testing	107

Figure		Page
7.4	Model Reinforced Earth Wall with Modifications Prior to Testing	108
7.5	Model Reinforced Earth Wall During Construction	111
7.6	Video Camera, and Recording Equipment	112
7.7	Failure of Mode 1 type Model Reinforced Earth Wall	114
7.8	Typical Pattern for Mode 1 type Structures L=275 mm, H=500 mm, S=100 mm	117
7.9	Typical Failure Pattern for Mode 2 type Failures L=275 mm, H=712 mm, S=75 mm	118
7.10	Failure Surface for Mode 1 Configuration L=275 mm, H=525 mm, S=75 mm	118
7.11	Failure Occurring in Mode 1 Structure L=412.5 mm, H=605 mm, S=150 mm	120
7.12	Failures in Mode 2 Configurations (a) L=275 mm, H=600 mm, S=100 mm (b) L=412.5 mm, H=740 mm, S=150 mm	121
7.13	Rotation of Bottom Panel in Mode 1 Configurations (a) L=412.5 mm, H=605 mm, S=150 mm (b) L=412.5 mm, H=635 mm, S=100 mm	123
7.14	Rotation of Bottom Panel in Mode 2 Configuration L=275 mm, H=600 mm, S=100 mm	124
7.15	Construction of a Model Reinforced Earth Wall on the Base Friction Table	128
7.16	Base Friction Apparatus and Camera Strip	129
7.17	Test Setup for Mode 1 Configuration on Base Friction Table. (a) Before Testing, (b) After Failure	130
7.18	Test Setup for the Mode 2 Configuration on Base Friction Table (a) Before Testing, (b) After Failure	131

Figure		Page
7.19	Tilting of the Top and Bottom Panels (a) Start of Test, (b) After Failure	132
7.20	Typical Mode 1 Failure Pattern	139
7.21	Typical Mode 2 Failure Pattern	140
8.1	Effects of Surcharge Load on the Strain in Reinforcing Strips and the Horizontal Yielding of the wall for the Steel Rod Apparatus. Test 4	144
8.2	Failure of Steel Rod Model Test with Instrumentation and Surcharge Load	146
8.3	Shape of Coulomb Failure Wedge in Steel Rod Model (No Surcharge Load).	147
8.4	Normalized Failure Ratio For Two-Dimensional Reinforced Earth Systems	149
8.5	Typical Failure of Mode 2 Structure represented by Logarithmic Spiral	152
8.6	Typical Failure of Mode 1 Structure represented by Failure Wedge	154
8.7	Failure Ratios vs. Time for Friction Table Tests	159
8.8	Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2	161
8.9	Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2	162
8.10	Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2	163
8.11	Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2 . .	164
B.1	Base Friction Apparatus	200

LIST OF TABLES

Table		Page
7.1	Summary of Steel Rod Model Test Results	115
7.2	Summary of Friction Table Test Results . . .	134
8.1	Values of Failure Plane Angles θ_1 and θ_2 for Base Friction Tests, Mode 1.	157

LIST OF SYMBOLS

A	Area
AC	Alternating Current
A_s	Cross Sectional Area
B	Foundation Width
B_H	Horizontal Spacing
b	Perimeter of Reinforcing Tie
b_s	Strip Width
c	Shear Strength Parameter
C_u	Coefficient of Uniformity
D,d	Depth
DC	Direct Current
D_R	Relative Density
E	Young's Modulus
e	Void Ratio
F	Force
FP	Failure Plane
FS	Factor of Safety
f	Coefficient of Friction
f_t	Ultimate Tensile Stress of Reinforcing Tie
f_y	Yield Stress
G_s	Specific Gravity
g	Gravity
H	Total Height
Hp	Horsepower

h	Height, Elevation
K	Lateral Earth Pressure Coefficient
K_A	Active Earth Pressure Coefficient
L	Total Length
ℓ	Effective Length of Reinforcing
L_i	Length of Strip Beyond Intersection
L_a	Bond Length
N	Normal Force, Number of Ties
n	Number of Reinforcing Strips per m^2
OD	Outside Diameter
P_h	Horizontal Load
P_v	Vertical Load
P_o	Uniformly Distributed Surcharge Load
R	Resultant
RPM	Revolutions per Minute
r	Radius
S	Spacing, Shear
s	Settlement
S_y	Vertical Spacing of Reinforcing Strips
S_x	Horizontal Spacing of Reinforcing Strips
T_i	Strip Capacity
t	Time, Thickness
U	Initial Velocity
V	Velocity
W	Weight
w	Width of Reinforcing Strip

z_i	Depth of Reinforcing Level
α, β	Angle of Inclination
γ	Unit Weight of Soil
δ	Angle of Wall Friction, Lateral Deflection
ν	Poisson's Ratio
σ	Normal Stress
τ	Shear Stress
ϕ	Angle of Internal Friction
θ^1	Angle of Failure Plane in Reinforced Earth Block
θ^2	Angle of Failure Plane in Unreinforced Soil
ψ	Angle of Friction Between Soil and Reinforcing

LIST OF APPENDICES

Number		Page
A.	Summary of Research Concerned with Reinforced Earth	180
B.	Design and Construction of the Base Friction Apparatus	195
C.	Instrumentation Specifications	201
D.	Failure Plane Determinations for Base Friction Tests	202
E.	Wedge Analysis for Base Friction Test 16	241

CONVERSION OF UNITS

Symbols and Conversion Factors for SI Units

m = metre = 39.37 inches

mm = millimeter = 0.03937 inches

N = Newton = 0.22481 pound force

kN = KiloNewton = 224.81 pound force

Conversion from Imperial to SI Units

1 inch = 25.4 mm

1 foot = 304.8 mm = 0.3048 m

1 lb. force = 4.4482 N

1 kip = 4448.2 N = 4.4482 kN

Conversion of Units for Pressure and Stress

1 psi = 1 lb. force per square inch = 6.8947 kPa

or

1 kPa = 1 kiloNewton per square metre = 0.14504 psi

CHAPTER I

INTRODUCTION

1.1 General

The concept of reinforced earth involves the proportioning and arrangement of a network of metal ties with respect to their lengths and spacing (both horizontal and vertical) for a particular type of backfill material and a specified wall height.

The technique was developed by french Engineer Henri Vidal who later marketed and patented the procedure under the name of the Reinforced Earth Company.

It has long been recognized that it is beneficial to combine the tensile strength of one material with another to improve its properties, such as in reinforced concrete. However, reinforced earth is complicated by the fact that both the shear strength of the soil and the bond strength with the reinforcing are frictional in nature and are thus dependent on the normal effective stress distribution.

Of the many attractive and economical aspects of reinforced earth, some of the more significant ones include the ability of these structures to accept post-construction settlements and seismic forces extremely well. This is due to the fact that since natural material is used

to form the structure, there is a uniform stress distribution on the foundation. Another particularly appealing feature is the relative ease of construction which requires no special skills or equipment. Facets such as these should serve to increase the popularity of this technique and result in more rational and economic design procedures.

At present, the design of reinforced earth structures is based mainly upon theories pertaining to soil mechanics which are unable to account for changes in the failure patterns of the soil due to the presence of tensile reinforcing ties.

This study was undertaken to expand the knowledge concerning reinforced earth by investigating the various modes of failure and the shapes of the failure surfaces for a variety of retaining wall geometries.

1.2 The Nature of the Problem

Previous studies concerning the failure of reinforced earth structures utilized small three-dimensional test apparatus under plane strain conditions. However, in plane strain testing, the intermediate principal stress, σ_2 , can only be reduced by lining of the testing apparatus with a low friction material. The degree to which this is successful depends on many factors such as: the dimensions of the test apparatus, the frictional behaviour of the test soil,

the stress range of the model tests and the friction coefficient between the model material and the test apparatus. Numerous studies (Hausmann, 1978; Lee, 1973) have mentioned that the effects of side-wall friction which are caused by this stress could significantly affect the test results on the behaviour of model reinforced earth structures.

Test results to date have been inconclusive in accurately determining the location of the failure plane without the effects of side-wall friction in the case of model tests, or interference from strain gauge wires in the case of field tests. However, in two-dimensional testing systems, the σ_2 component can be completely eliminated, thus avoiding the possibility of having unaccountable friction forces which may alter the test results.

1.3 Scope and Objectives of the Study

Figure 1.1 is a flow-chart which outlines the scope of this study. It illustrates how the two-dimensional physical models were used with varied spacings and lengths of tensile reinforcing strips along with variations in retaining wall geometry to determine the failure mode and failure surfaces in model reinforced earth structures. The test results were then compared with existing theories to aid in the formulation of the conclusions and recommendations.

✓

Figure 1.1 Scope of the Study

In both types of model experiments, the reinforcing was strong enough not to rupture during all phases of the experiments. This aspect was chosen for several reasons:

(1) There is a heavy safety factor applied to reinforcing strips which are susceptible to corrosion. This results in an increase in the cross sectional area of the reinforcing ties to compensate for the long-term corrosion over the life span of the structure. Thus, the factor of safety against tie breakage or rupture is quite large compared to the safety factor against tie pullout during the early life of the retaining structure.

(2) The relative simplicity in calculating the necessary area required for a metal strip to resist tensile forces.

The primary objectives of this work are:

(1) To determine the shape of the failure planes in model reinforced earth structures having varied wall geometries.

(2) To qualitatively assess the various modes of failure that are possible in reinforced earth systems.

(3) To compare the results obtained in (1) and (2) with existing theories.

(4) To assess the applicability of the two-dimensional models in investigating the behaviour of cohesionless soil masses.

1.4 Outline of the Thesis

A general presentation of the problem regarding the determination of the shape of the failure surface in reinforced earth structures is given in Chapter I.

Chapter II contains a general description of the concept of reinforced earth.

The existing theoretical analyses which pertain to reinforced earth structures is presented in Chapter III.

Chapter IV discusses the theoretical concepts of physical models and justifies the selection of the two models chosen for this study.

A description of the model apparatus including the design and construction of each is given in Chapter V.

Chapter VI contains a description of the components used to model reinforced earth retaining walls for both the physical models used in this study.

In Chapter VII a description of the tests and the recording techniques used for each model is presented. The test results obtained in these investigations is also described.

An analysis and a comparison of the results obtained with existing theories is presented in Chapter VIII.

Summary, conclusions and recommendations for further work are detailed in Chapter IX.

Following Chapter IX, are a list of the references

pertaining to this study and the following Appendices:

Appendix A contains a summary of research concerned with reinforced earth.

Appendix B contains an outline of the design and construction procedure of the base friction apparatus.

Appendix C includes the specifications for the instruments which were used on the steel rod apparatus.

Appendix D contains the results of the failure plane determinations for the tests conducted on the base friction apparatus.

Appendix E includes a sample calculation of Romstad et al's wedge analysis for Test 16 which was conducted on the base friction apparatus.

CHAPTER II

REINFORCED EARTH CONCEPT

Problems involving the stabilization of retaining walls and slopes are of paramount importance in the field of geotechnical engineering. One method which is used to strengthen a soil mass consists of the inclusion of rods, bars, fibers, meshes or nets within the soil to effectively impart tensile strength to the retained mass. This procedure is known as Reinforced Earth or earth reinforcing, and involves the proportioning and arrangement of a network of metal ties with respect to their length and spacing (both horizontal and vertical) for a particular type of backfill material and a specified wall height.

It has long been recognized that it is beneficial to combine the tensile strength of one material with the compressive strength of another to improve its overall properties; such as in reinforced concrete. Soil, particularly sandy or coarser grained granular soil, has little or no tensile strength, but if reinforced with strips, it can transmit tensile forces and the compound material exhibits a tensile strength in the direction of the reinforcement.

The concept of improving soil strength through

the inclusion of materials having tensile properties is not new. Some animals and birds use straw, branches, string, or any other fibrous material mixed with soil to build their habitations. The beneficial effect of plant roots in stabilizing soil has been recognized for some time now, and it is often considered an important part of stabilizing loose surface materials. Preliminary access roads through swampy areas are often constructed on a foundation of small tree trunks and branches, these are called corduroy roads (Lee, 1973). Similar structures were used by early French settlers along Canada's Bay of Fundy. They used a combination of mud and sticks to construct many miles of low dikes to reclaim rich farm land from tidal flats.

Although it is evident that reinforced earth has been used for quite some time, the credit for developing and marketing the concept for modern reinforced soil structures must go to the French engineer, Henri Vidal. Vidal formulated theories from his research and related them to design methods for full scale structures. He was able to prove that this process was significantly different from conventional anchored tie-back systems and thus patented the process under the name "Terre Armee" (Reinforced Earth). Today it has evolved to the point where over 2,200 Reinforced Earth structures, totalling nearly 1,350,000 m² of wall surface, have now been completed in a wide range of climates

and environmental settings in 28 countries (McKittrick, 1979).

Primarily the method was developed for vertical retaining structures, but this concept is now employed to sloping retaining structures, embankment foundation slabs, dams, containment dikes, slide buttresses and bridge abutments (Bartos, 1979). Still other variations of the reinforced soil concept are developed for strengthening earth embankments without retaining structures, for constructing low-cost or expedient roadways and for strengthening the soil beneath foundations (Bartos, 1979).

The initial reinforced earth projects were designed with very high safety factors, but experience led to better understanding of the behaviour of such structures and, therefore, resulted in more rational and economical designs. However, research is still at a very young stage in this field and information is still lacking which could explain the complex behaviour of such soil-reinforcing systems. The late Dr. Kenneth Lee stated "one of the attractive beauties of combining reinforcing elements with soil materials is the number and variety of possible design concepts and applications which come to mind" (Lee, 1978).

The key components of a reinforced earth structure are illustrated in Figure 2.1. The major item is the soil backfill which is placed in layers and compacted as the wall is constructed. This material is so far restricted

FIGURE 2.1. COMPONENTS OF REINFORCED EARTH STRUCTURE

to granular, cohesionless and free-draining soils. The second component consists of the tensile reinforcing strips which extend back into the fill material and hence strengthen the soil. These strips are connected to the face panels. The face panels or skin elements, as they are often called, form the vertical facing of the retaining wall. Theoretically these are non-structural elements; however for esthetic purposes they are often made of pre-cast concrete panels.

The principle of reinforced earth is complicated by the fact that both the shear strength of the soil and the bond strength between the soil and the reinforcing are frictional in nature, and thus are dependent upon the normal effective stress distribution. This is further complicated by the size, geometry, and type of loading of the structure as well as by the types of materials and by other factors (Popescu, 1979).

Many researchers have embarked on extensive research programs using the hypothesis that reinforced earth structures, retaining structures in particular, are analogous to tied-back walls (McKittrick, 1979). Although both methods utilize tie-forces, Rankine failure planes and other topics relevant to earth retaining analysis, the basic mechanics associated with each type are distinctly different. Friction will only develop if there is a change in the

tensile force within the reinforcement. In the case of a pull-out test on a reinforcing strip, or in an actual retaining structure, the reinforcement is subjected to the full pull-out force at one end and no force at the other. In this case, a frictional force will develop to counteract the tensile force and maintain the system in static equilibrium until the maximum frictional force is exceeded. In the case where there is an anchor plate at the end of the strip, the tie is in constant tension, and therefore there is no adhesion between the tie and the soil.

Appendix A contains a summary of published and unpublished research investigations dealing with reinforced earth. It clearly illustrates that there is a lack of information pertaining to the various modes of failure and shape of the failure planes occurring in these structures. Such information is of paramount importance in the design and in the estimation of safety factors. Most of the research to date was conducted using small three dimensional test apparatus. However, experience with retaining wall models indicates that the width to height ratio of the test apparatus should not be less than 1.0 or 1.25 in order to keep the error due to side wall arching (wall friction effect) below 10 percent (Hausmann, 1978). As a consequence, the range of reinforcement configurations which can be tested using the critical height technique without surcharge is very limited.

Many other researchers (Appendix A) conducted experiments on full scale field structures which utilized strain-gauged reinforcing strips. It was postulated that the zone of maximum tension in the reinforcing strips would also correspond to the potential failure surface. However, the effect of the strain gauges and the wires on the friction mobilised between soil and strips could not be accounted for and, as discovered by Felio (1980), can play a significant role in the pull-out strength of the reinforcing.

The many uncertainties associated with these structures prompted the research contained in this thesis. It is an attempt to recognize the modes of failure and shapes of failure planes in reinforced earth structures by using two dimensional models and thus eliminating the effects of side wall friction or interference from strain gauge wires.

CHAPTER III

THEORETICAL ANALYSIS OF REINFORCED EARTH STRUCTURES

3.1 Introduction

For reasons mentioned in section 1.3 of chapter I, the analysis will be restricted to the tie-pullout mode of failure. Furthermore, this mode is the most complicated one to analyse as only the portion of the reinforcing strip extending past the line of the failure plane of the soil backfill will provide frictional anchorage against failure, and hence stabilize the structure. This further emphasizes the importance of determining the shape of the failure surface for the analysis of these structures.

It should be mentioned that in all cases, some movement of the wall is necessary in order to develop friction between the ties and the soil.

3.2 Lee's Analysis

The analysis of reinforced earth structures by Lee et al (1973) for the tie-pullout mode of failure was one of the first of its kind to be published. The calculation for the pullout resistance was based upon the theoretical effective length of the ties, ℓ , or the length of the reinforcing tie which extended beyond the zone of the Rankine failure surface.

The total frictional resistance, F_r , at a depth d , is given by

$$F_r = 2\ell w y d \tan \psi \quad (3.1)$$

where w = the width of the tie

ψ = angle of friction between the tie and the soil

It should be noted that the amount of frictional resistance provided by the edges of the reinforcing tie is considered negligible.

Lee used equation 3.1 to derive three methods of analysis, each producing a different result. The first, or Rankine method, assumes that the total earth pressure force F_e acting over the wall area that is supported by one tie (or one panel in a two-dimensional study) is

$$F_e = S_x S_y \gamma d K_A \quad (3.2)$$

where S_x = horizontal spacing of reinforcing ties

S_y = vertical spacing of reinforcing ties

γ = unit weight of the soil

d = depth

K_A = coefficient of active earth pressure

The forces defined by Equations 3.1 and 3.2 are then

compared at corresponding depths, leading to a factor of safety, FS, with respect to pulling out at that depth of

$$FS = \frac{2lw \tan \psi}{S_x S_y K_A} \quad (3.3)$$

It should be noted that if the effective length, l , was constant, the value of the FS would be independent of depth. However, this would necessitate the use of reinforcing strips of different lengths in order to maintain such a condition. This is highly impracticable. The most critical condition will occur at the topmost level of reinforcing where l is a minimum. Thus, the required total length of tie would be

$$L = \frac{H}{\tan \theta} + \frac{S_x S_y K_A FS}{2lw \tan \psi} \quad (3.4)$$

where H = total height of the retaining wall

θ = angle from the horizontal to the Rankine failure surface

The second method derived by Lee et al (1973) is based on the Coulomb force method which considers overall rather than local stability. Therefore, it is not necessary that all ties extend beyond the failure plane. However, only the summation of the frictional forces for the ties

that do extend beyond the failure surface are considered. Thus, the total maximum frictional resisting force, F_r , for the case in which the ties have a constant length, L , but only part of them extend beyond the theoretical failure plane is given by

$$F_r = 2\gamma S_y w \tan \psi \sum_{i=N}^n i \left[L - (n-i) S_y \tan (45 - \phi/2) \right] \quad (3.5)$$

where i is the summation index counting the number of ties beginning from $i=1$ at the top to $i=n$ at the bottom, and N is the value of i for the first tie from the top to extend beyond the theoretical failure plane.

For this method, the resisting force is equated to the total earth pressure force, and this leads to the following expression for the factor of safety against a tie pullout.

$$FS = \frac{4S_y w \tan \psi}{K_A H^2 S_x} \sum_{i=N}^n i \left[L - (n-i) S_y \tan (45 - \phi/2) \right] \quad (3.6)$$

Finally, the Coulomb moment method is derived from the sum of the moments about the toe of the wall of each tie frictional force, and is compared to the total moment from the lateral earth pressure. The corresponding equation for the factor of safety is

$$FS = \frac{12wS_Y^2 \tan \phi}{K_A H^3 S_x} \sum (n-i) i \left[L-(n-i) S_Y \tan (45-\phi/2) \right] \quad (3.7)$$

Although the examples shown herein have been limited to simple cases, the analyses, especially by the Coulomb methods, can readily be extended to include additional features, such as frictional wall, sloping wall, sloping backfill, surcharge, variable length, and spacing of ties, etc. (Lee et al, 1973).

3.3 Analysis by Price

Price (1975) published design theories related to reinforced earth. For the case of sufficient bond length the analysis is also based on the Coulomb method. However, he disagrees with the assumption that only the portion of the reinforcement projecting beyond the critical failure wedge should be considered to be active in preventing failure due to slippage or pullout. This is based on the composite nature of reinforced earth and should not be compared to the failure plane of a wall retaining unreinforced soil. An analogy is drawn with establishing the failure of plain concrete, and subsequently placing steel beyond any failure areas. "It has been acknowledged for years that concrete and steel work together as a composite material, and reinforced earth has similar mechanical

properties" (Price, 1975). This could be compared with an attempt to design reinforced concrete structures using plain concrete theory.

Based on the above assumptions, Price (1975) recommends an expression for the maximum tension, $T_{\max}$ of

$$T_{\max} = \int_0^{L_a} 2w\psi\gamma H \, dl = 2w\psi\gamma H L_a \quad (3.8)$$

from which, an expression for the bond length, L_a can be derived, yielding

$$L_a > \frac{T_{\max}}{2w\psi\gamma H} \approx \frac{K_A \Delta H}{2w\psi n}$$

where ψ = coefficient of friction between soil
and reinforcing ties

L_a = bond length

w = width of strips

n = number of strips per m'

This is a conservative approach bearing in mind that dilatancy will occur in the area adjacent to the strips, and thereby will increase locally the value of the vertical stress as compared to the overburden value (γh). This increase in stress would therefore increase the total frictional resistance for the strips, and hence compensate

for the fact that in the computation the whole length of the strip is considered for bond resistance. Furthermore, Price (1975) advocates a reduction of the measured friction coefficient by 20 percent to be used in calculations for the design of actual structures.

3.4 Analysis by Banerjee

Banerjee's analysis was one of the more complete assessments of the theoretical behaviour of reinforced earth structures. He has shown that the sum of the tensions produced in the horizontal reinforcing ties through the action of friction and their centre of action opposes the pressure from the retained earth mass, and thus maintains equilibrium.

For bond failure of the reinforcing ties with the subsequent production of a failure wedge in the soil mass, as indicated in Figure 3.1, Banerjee (1975) proposes the following expression

$$\Sigma T = N A_s f_t \quad \text{or} \quad [\psi N L b \{c + (\gamma H + P_o) \tan \phi\}] \quad (3.10)$$

where ΣT denotes the summation of tension in the reinforcing strips

N = Number of ties

A_s = Cross sectional area of ties

FIGURE 3.1 FAILURE OF REINFORCED EARTH
STRUCTURE BY FORMATION OF
FAILURE WEDGE (AFTER BANERJEE,
1975)

f_t = The ultimate tensile stress of the tie

ψ = Coefficient of friction between the soil and the tie

L = Length of the ties

b = perimeter of a tie

c = Apparent cohesion of the soil

P_o = Uniformly distributed surcharge load

Therefore,

$$\begin{aligned}\Sigma T &= \sum_{i=1}^N \psi b [L - (H - Z_i) \tan \beta] [c + (\gamma Z_i + P_o) \tan \phi] \\ &= \sum_{i=1}^N \psi b [(Lc + LP_o \tan \phi - HP_o \tan \phi \tan \beta - Hc \tan \beta) \\ &\quad + Z_i (\gamma L \tan \phi - \gamma H \tan \phi \tan \beta + c \tan \beta + P_o \tan \phi \tan \beta) \\ &\quad + Z_i^2 \gamma \tan \phi \tan \beta] \quad (3.11)\end{aligned}$$

If the vertical spacing of the strips is equal, the expression is

$$\begin{aligned}\Sigma T &= \psi b [(Lc + LP_o \tan \phi - HP_o \tan \phi \tan \beta - Hc \tan \beta) \\ &\quad + S_y \left(\frac{N^2 + N}{2} \right) (\gamma L \tan \phi - \gamma H \tan \phi \tan \beta + c \tan \beta + P_o \tan \phi \tan \beta) \\ &\quad + S_y^2 \frac{(N^2 + N)(2N + 1)}{6} \gamma \tan \phi \tan \beta] \quad (3.12)\end{aligned}$$

The value of β can only be determined if ΣT is known. However, it has been determined that the summation, ΣT , is not very sensitive to the value of β (Banerjee, 1975), and thus as a first approximation β can be assumed to be equal to $45-\phi/2$, and therefore, ΣT can be evaluated using Equation 3.12.

For a purely granular soil with no cohesion, β can be expressed as

$$\beta = \tan^{-1} \sqrt{X} \quad (3.13)$$

where

$$X = N A_s f_t / (0.5 \gamma H^2 + P_o H) \quad (3.14)$$

and hence a more precise value for a minimum factor of safety can be obtained. Therefore,

$$\begin{aligned} \Sigma T = & \psi b [(L P_o \tan \phi - H P_o \tan \beta \tan \phi) \\ & + S_y \frac{(N^2 + N)}{2} (\gamma L \tan \phi - \gamma H \tan \phi \tan \beta + P_o \tan \phi \tan \beta) \\ & + S_y^2 \frac{(N^2 + N)(2N + 1)}{6} \gamma \tan \phi \tan \beta] \end{aligned} \quad (3.15)$$

This expression can be greatly simplified for the situation where there is no uniform surcharge loading on the top of the retaining structure. Therefore, Equation 3.15 may be written as (for $P_o = 0$)

$$\begin{aligned} \Sigma T = & \psi b \left[S_y \frac{(N^2 + N)}{2} (\gamma L \tan \phi - \gamma H \tan \phi \tan \beta) \right. \\ & \left. + S_y^2 \frac{(N^2 + N)(2N + 1)}{6} \gamma \tan \beta \tan \phi \right] \end{aligned} \quad (3.16)$$

Considering the equilibrium of the surface XY in Figure 3.1, inclined at an angle β with the vertical, the total sliding force P can be evaluated from

$$P = (0.5\gamma H^2 + P_o H) \sin \beta. \quad (3.17)$$

and the total resisting force F can be expressed as

$$\begin{aligned} F = & (0.5\gamma H^2 + P_o H) \sin \beta \tan \beta \tan \phi + (\cos \beta \tan \phi + \sin \beta) \Sigma T \\ & + cH \sec \beta \end{aligned} \quad (3.18)$$

Therefore, the factor of safety FS can now be determined as

$$FS = \frac{F}{P}$$

where the term ΣT in Equation 3.18 may be evaluated from Equations 3.16, 3.15, 3.12 or 3.11, whichever is applicable to the internal conditions of the retaining structure.

3.5 Al-Hussaini and Perry

Al-Hussaini and Perry (1978, ref. 3) conducted experiments on full-scale reinforced earth walls to examine the behaviour of these structures during construction and

at failure. The factor of safety against pullout failure, FS, can be expressed as

$$FS = \frac{\tau_f}{\tau_m}$$

where τ_f is the shear stress between the tie and the soil at failure and τ_m is the maximum frictional stress along the tie. Also, the resisting force due to friction T_e may be expressed as

$$T_e = \frac{4}{3} w l (P_o + \gamma d) \frac{\tan \psi}{FS} \quad (3.19)$$

where ψ = friction angle between the soil and the tie

P_o = surcharge pressure

d = depth below the surface of the fill

γ = unit weight of the fill

w = width of the tie

l = effective length of the tie

The maximum tensile force, T_d at a depth d , is equal to the force generated by earth pressure at that depth; thus

$$T_d = K_A (P_o + \gamma d) S_x S_y \quad (3.20)$$

where S_x and S_y are the horizontal and vertical spacing of the ties. Therefore, combining Equations 3.19 and 3.20 leads to the following factor of safety, FS, with respect to tie pullout

$$FS = \frac{4lw \tan \psi}{3K_A S_x S_y} \quad (3.21)$$

3.6 Juran and Schlosser

Research on bidimensional and tridimensional models of reinforced earth walls conducted at the Laboratoire Central des Ponts et Chaussées (Paris) by Juran and Schlosser (1978) reveals that the classical retaining walls limit analysis design methods presently used, or proposed, are inadequate to describe the failure mechanism of models, and lead to a large underestimation of the models ultimate resistance. Considering the internal stability of a typical reinforced earth wall and in order to prevent failure by pullout, the maximum pullout force, $T_{\max}$ must be less than the total frictional resistance, of the reinforcing strip. This resistance may be expressed as

$$F = \int_0^{l_a} 2w\psi\sigma_v dl \quad (3.22)$$

where l_a = effective length of the tie resisting pullout

σ_v = vertical stress acting on the strip

and w and ψ are as previously defined. Since the tensile forces developed in the ties are not at a maximum at the wall face, hence $l_a < L$. Therefore, the second criteria with respect to tie pullout is

$$\frac{F}{S_y S_x} = \frac{2\psi_w l a}{S_y S_x} \int_0^l \frac{a}{v} \frac{dl}{a} \geq \frac{T_{\max}}{S_y S_x} \quad (3.23)$$

where S_y and S_x are the vertical and horizontal spacing of the strips respectively.

The formulation of an appropriate design method for predicting the tensile forces in the reinforcement strips and stresses within the soil requires knowledge of the initial stress conditions in the backfill. This depends essentially on the compaction history of the fill, the constitutive equations for the soil, the effect the reinforcing has on the stress-strain relationship of the soil, the failure criterion of the reinforced earth material and on the "correct" or at least realistic boundary conditions representing the actual interaction between the reinforced embankment and the face panels (Juran and Schlosser, 1978). Based on the analysis of the equilibrium of forces acting on a reinforced earth wedge with the failure surface passing through the toe of the wall, and which is defined by the geometrical position of the maximum tensile faces in the strips, the method yields

$$T_{\max} = K_A \gamma H S_x S_y \quad (3.24)$$

The above equation assumes that the distribution of the total active forces is linearly increasing with depth.

If the failure is caused by strip pullout, then the ultimate critical height of the retaining wall is a function of the length of the reinforcing ties ($H_{crit} = f(L)$). Therefore, in order to determine the values of the ultimate critical height, H_{crit} , and the corresponding limit length, ℓ_a , for which this condition exists, the following equations may be used

$$H_{crit} = \frac{R_T t w}{K_A \gamma S_x S_y} \quad (3.25)$$

where R_T = resistance of the reinforcing strip to tension

t = thickness of the reinforcing strip

and K_A , γ , S_y and S_x are as defined earlier, and

$$L = \ell_a = \frac{K_A}{2\psi} \frac{S_y}{w/S_x} \quad (3.26)$$

However, it is worth noting that the theoretical ultimate critical height obtained using the above equation was approximately one half the actual critical height obtained in experimental tests.

3.7 Smith and Wroth

Smith and Wroth (1978) conducted experiments on a 200 mm wide glass-walled tank (see Appendix A) using

radiographs (X-rays) to detect the failure surfaces.

However, since their tests were carried out in a narrow tank, the frictional force exerted by the side walls could be significant, and seriously affect their results.

3.7.1 Failure by Overturning

If the reinforcement is closely spaced, it can be assumed that the reinforced zone behaves as a rigid block (Smith and Wroth, 1978 and Smith and Bransby, 1976). In this case, failure will occur by overturning of the reinforced block, and the pattern of reinforcement remains rectangular. This indicates that there is little deformation in the reinforced zone during collapse. This pattern of failure suggests that the walls could be analysed by considering a Coulomb wedge behind the reinforced block. Figure 3.2 shows a section through such a reinforced earth wall. By resolving forces horizontally and vertically, considering the wedge ABE as a free body, the following two equations are obtained

$$R \cos (\theta + \phi) = Q \cos \phi \text{ and}$$

$$R \sin (\theta + \phi) + Q \sin \phi = \frac{1}{2} \gamma H^2 \tan \theta$$

solving for Q

$$Q = \frac{\frac{1}{2} \gamma H^2 \tan \theta}{(\cos \phi \tan (\theta + \phi) + \sin \phi)} \quad (3.27)$$

FIGURE 3.2 FORCES ACTING ON WEDGE AND REINFORCED
ZONE AT FAILURE (AFTER SMITH AND WROTH, 1978)

where the forces R and Q are defined in Figure 3.2

The overturning of block BCDE can occur when the normal force, N, does not act within the middle-third of the foundation base, DE. For an extreme case, and assuming the maximum possible value of H/L for any postulated stress distribution on DE, the force N is taken to act at the corner, D (ie. $X=L$). Now resolving forces horizontally and vertically for the block BCDE one will obtain

$$S = Q \cos \phi \quad \text{and}$$

$$N = \gamma HL + Q \sin \phi \quad (3.28)$$

where the forces S and N are given in Figure 3.2

In the initial analysis Smith and Bransby assumed that the force Q always acts at a height of $1/3 H$ from the base. Therefore, taking moments about point E (Figure 3.2) one obtains

$$(Q \cos \phi) (1/3 H) + \gamma HL (L/2) - NX = 0$$

substituting for N from Equation 3.28, and setting $X=L$, will yield

$$Q(L \sin \phi - 1/3 H \cos \phi) = -\frac{1}{2} \gamma HL^2$$

substituting Q from Equation 3.27 in the above relationship the value of the critical H/L ratio is found to be a

solution of the equation

$$1/3(H/L)^2 \tan \theta - (H/L) \tan \theta \tan \phi - \tan (\theta + \phi) - \tan \phi = 0 \quad (3.29)$$

Smith and Wroth (1978) used a similar approach, but load cells placed along zone DE in their model tests indicated that the point of action of force N at failure was at the mid-point of the foundation. Furthermore, equilibrium calculations using the load cell data indicated that the point of action of Q was much lower than the assumed height of H/3. Therefore, as illustrated in Figure 3.3, it is assumed that the force R acts at a height of H/3, and the lines of action for the forces R, W and Q will meet at the same point on the shear surface AE in order to satisfy equilibrium conditions. The force Q acts at a distance of Y above point E and this distance is given by

$$Y = 1/3H(1 - \tan \theta \tan \phi) \quad (3.30)$$

Thus, for various values of ϕ and θ the height of action, Y, above the base can be evaluated in terms of the overall height H of the wall. Furthermore, by assuming values for the point of action of the normal force, N on the base (X/L ratio), and taking moments about E, the following relationship is obtained for $Y = 0.16H$ and $X/L = 0.5$

$$Q \cos \phi (0.16H) + \gamma HL(L/2) - N(L/2) = 0$$

Now substituting for N from Equation 3.28 this equation

FIGURE 3.3 FORCES ACTING ON WEDGE USING
EQUILIBRIUM CALCULATIONS
FROM DATA OBTAINED FROM LOAD
CELLS (AFTER SMITH AND WROTH,
1978)

can be expressed as

$$\frac{\frac{1}{2}\gamma H^2 \tan \theta}{(\cos \phi \tan(\theta + \phi) + \sin \phi)} (\cos \phi)(0.16H) + \gamma HL(L/2) - (\gamma HL + Q \sin \phi)(L/2) = 0$$

Finally, substituting for Q from Equation 3.27 yields

$$\frac{\frac{1}{2}\gamma H^2 \tan \theta (\cos \phi)(0.16H)}{(\cos \phi \tan(\theta + \phi) + \sin \phi)} - \frac{\frac{1}{2}\gamma H^2 \tan \theta (\sin \phi)(L/2)}{(\cos \phi \tan(\theta + \phi) + \sin \phi)} = 0$$

This expression can be solved for H/L to yield

$$\frac{H}{L} = \frac{0.5\gamma \tan \theta \sin \phi}{0.16\gamma \tan \theta \cos \phi} \quad (3.31)$$

Now, if for a typical example $\phi = 50^\circ$ and $\theta = 23^\circ$ are substituted in the above equation one obtains

$$H/L = 3.125 \tan \phi$$

It should be noted that this expression will change when different values of ϕ , θ and $N = X/L$ are selected.

3.7.2 Failure by Tie Pullout

The failure by tie pullout adopted by Smith and Wroth (1978) is based upon a variation of the equation proposed by Lee et al (1973) (Equation 3.3). The major drawback to this expression is that the factor of safety does not vary with the height of the wall, and is thus of no use for predicting the failure heights in the model tests. Therefore, the factor of safety with respect to

tie pullout obtained in the same way as Equation 3.3 is thus given by

$$FS = \frac{W L \tan \psi}{K_A S_y S_x (1 + K_A d^2 / L^2)} \quad (3.32)$$

This expression leads to a variation in safety factor with the height of the wall, and thus the predicted failure heights can be compared with experimental values (Smith and Wroth, 1978).

3.8 Romstad et al

An analysis involving a composite wedge derived from the shape of the failure surface was predicted by a finite element model carried out by Romstad et al (1978). The analysis proceeds by dividing the area into a component rectangle and two triangles (Figure 3.4) and determining the maximum resultant normal and shear forces at collapse. The assumption that the failure plane can be represented by two straight lines intersecting at the boundary between the backfill soil and the reinforced block is utilized in this analysis. The main failure block AEDC (Figure 3.4) can be divided up into individual components as shown in Figure 3.5. The weight of element (1) is

$$W_1 = \gamma_1 H_1 L = \gamma (H - H_1) L = \gamma_1 L^2 (H/L - \tan \theta_1)$$

FIGURE 3.4 ASSUMED SHAPE OF FAILURE SURFACE (AFTER ROMSTAD ET AL, 1978)

FIGURE 3.5 COMPONENTS OF FAILURE WEDGE

thus, the normal force is assumed to be given by

$$N_1 = \frac{1}{2} K \gamma_1 H_1^2 = \frac{1}{2} K \gamma_1 L^2 (H/L - \tan \theta_1)^2$$

The shear at the back edge of the wall is taken to be some fraction of the normal force

$$r N_1 = \frac{1}{2} r K \gamma_1 L^2 (H/L - \tan \theta_1)^2$$

and from equilibrium they state

$$S_1 = N_1$$

where γ_1 = unit weight of the reinforced earth mass

L = strip length

K = assumed coefficient of lateral earth pressure

r = a fraction of the resultant normal force

which must be less than $\tan \phi$

The weight of element (2) is

$$W_2 = \frac{1}{2} \gamma_1 H_2 L = \frac{1}{2} \gamma_1 L^2 \tan \theta_1$$

and from equilibrium,

$$S_2 = (W_1 + W_2 - r N_1) \sin \theta_1 + S_1 \cos \theta_1$$

$$N_2 = (W_1 + W_2 - r N_1) \cos \theta_1 - S_1 \sin \theta_1$$

The weight of element (3) is

$$W_3 = \frac{1}{2} \gamma_2 H_1 L_1 = \frac{1}{2} \gamma_2 H_1^2 \cot \theta_2 = \frac{1}{2} \gamma_2 L^2 (H/L - \tan \theta_1)^2 \cot \theta_2$$

where γ_2 = unit weight of the backfill material.

From equilibrium of the body, they then determine

$$S_3 = (W_3 + rN_1) \sin \theta_2 - N_1 \cos \theta_2 \quad \text{and} \quad (3.33)$$

$$N_3 = (W_3 + rN_1) \cos \theta_2 + N_1 \sin \theta_2 \quad (3.34)$$

The total frictional force developed along both surfaces is then given by

$$\begin{aligned} S_2 + S_3 = & (W_1 + W_2) \sin \theta_1 + W_3 \sin \theta_2 - rN_1 (\sin \theta_1 - \sin \theta_2) \\ & + S_1 \cos \theta_1 - N_1 \cos \theta_2 \end{aligned} \quad (3.35)$$

"The capacity to resist these applied shear forces is a function of the frictional capacity of the soil, the resistance of the reinforcing strips and cohesion of the soil material within the reinforced earth and the backfill" (Romstad et al, 1978). The resistance capacity of the soil may be written

$$S_r = C_1 L_2 + C_2 L_3 + N_2 \tan \phi_1 + N_3 \tan \phi_2 \quad (3.36)$$

where C_1 and C_2 represent the cohesion of the reinforced earth and backfill material respectively, ϕ_1 and ϕ_2 the associated friction angles and N_{2s} the resultant normal force on the soil in the reinforced earth mass.

Assuming that the strip resultants are horizontal at failure, or that significant bending of the strips does not occur prior to failure such that the resultant strip

forces are parallel to the failure surface yields a total resistance capacity of

$$\begin{aligned}
 S_r = & C_1 L_2 + C_2 L_3 + (N_2 + \sum_i T_{i \max} \sin \theta_1) \tan \phi_1 + N_3 \tan \phi_2 + \\
 & \sum_i T_{i \max} \cos \theta_1 = C_1 L_2 + C_2 L_3 + ((W_1 + W_2 - r N_1) \cos \theta_1 - S_1 \sin \theta_1 \\
 & + \sum_i T_{i \max} \sin \theta_1) \tan \phi_1 + ((W_3 + r N_1) \cos \theta_2 + N_1 \sin \theta_2) \tan \phi_2 \\
 & + \sum_i T_{i \max} \cos \theta_1 \quad (3.37)
 \end{aligned}$$

where $\sum_i T_{i \max}$ is the total resisting capacity of the

strips. They assume that failure is to occur when the maximum applied shear force (Equation 3.35) equals the shear resistance capacity given in Equation 3.37. For simplicity they assume $\phi_1 = \phi_2$ to obtain

$$\begin{aligned}
 & (W_1 + W_2)(\sin \theta_1 - \cos \theta_1 \tan \phi) + W_3(\sin \theta_2 - \cos \theta_2 \tan \phi) - r N_1 \\
 & (\sin \theta_1 - \sin \theta_2 + (\cos \theta_1 - \cos \theta_2) \tan \phi) + S_1(\cos \theta_1 + \sin \theta_1 \tan \phi) \\
 & - N_1(\cos \theta_2 + \sin \theta_2 \tan \phi) - C_1 L_2 - C_2 L_3 - \sum_i T_{i \max}(\cos \theta_1 + \sin \theta_1 \\
 & \tan \phi) = 0 \quad (3.38)
 \end{aligned}$$

Finally, collecting terms in the appropriate manner yields a quadratic equation of the form

$$\begin{aligned}
& [(\sin \theta_2 - \cos \theta_2 \tan \phi) \cot \theta_2 - rK(\sin \theta_1 - \sin \theta_2 - (\cos \theta_1 - \cos \theta_2) \\
& \tan \phi) + K(\cos \theta_1 + \sin \theta_1 \tan \phi - (\cos \theta_2 + \sin \theta_2 \tan \phi))] (H/L - \tan \theta_1)^2 \\
& + 2(\sin \theta_1 - \cos \theta_1 \tan \phi) (H/L - \tan \theta_1) + (\sin \theta_1 - \cos \theta_1 \tan \phi) \tan \theta_1 \\
& - \frac{2\gamma L^2 \tan \phi}{\gamma L^2} (\sin \theta_1 \tan \phi + \cos \theta_1) - \frac{2}{\gamma L^2} (c_1 L_2 + c_2 L_3) = 0 \quad (3.39)
\end{aligned}$$

where $L_2 = L \sec \theta_1$

$L_3 = L \csc \theta_2 (H/L - \tan \theta_1)$

Furthermore, they state that the backfill shear coefficient, r can be obtained from the equilibrium condition in element (3) by assuming that at failure the applied shear S_3 (Equation 3.33) must equal the shear capacity of the soil along the chosen failure slope. Using this assumption, they solve Equations 3.33 and 3.34 for N_3 and r for the case of zero cohesion in the backfill and the result is

$$r = -\frac{\cot \theta_2}{K} + \frac{\cos \theta_2 + \sin \theta_2 \tan \phi}{\sin \theta_2 - \cos \theta_2 \tan \phi} \quad (3.40)$$

If r is calculated to be greater than $\tan \phi$ then it should be set equal to $\tan \phi$ and θ_2 will then equal 90 degrees.

The strip capacity per unit width of soil at any level i , assuming the failure surface passes through the end of the strip at level n , is determined by

$$T_i = \frac{2wL_i\psi H_i}{S_x} \leq \frac{A_s f_y}{S_x} \quad (3.41)$$

where w = strip width

L_i = length of strip beyond intersection

ψ = coefficient of friction between soil
and strip

H_i = height of soil above level i

S_x = horizontal spacing of reinforcing strips

A_s = cross-sectional area of strips

f_y = yield stress of strips

This expression assumes a linear increase with depth of the vertical stress distribution which, of course, is a function of the amount of overburden.

Furthermore, they simplify the solution by averaging the strip reinforcement capacity over the vertical height through which the failure plane passes. This means that the capacity provided by the reinforcement can be treated as a continuous function having units of stress. The capacity is obtained by dividing Equation 3.41 by the vertical spacing of the reinforcement to obtain

$$T_x = \frac{2w\psi}{S_y S_x} H_i L_i \leq \frac{A_s f_y}{S_y S_x} \quad (3.42)$$

Further manipulation of the expression to account for the level, X_y , measured from the base of the wall where the resistance changes from strip yielding to strip pullout, and integrating to the level at the back of the wall intersected by the failure plane, will yield an expression for the total resisting capacity of the strips.

$$\frac{\sum_i T_{i \max}}{\gamma L^2} = \left[\frac{a_1 X_n}{\gamma L^2} \right] \left[\frac{1}{2} + \left(\frac{a_2}{a_1} - 1 \right) \frac{X_y}{X_n} + \left(\frac{L}{H} \right) (\tan \theta_1) \left(-\frac{1}{6} + \frac{1}{2} \left(\frac{X_y}{X_n} \right)^2 - \frac{1}{3} \left(\frac{X_y}{X_n} \right)^3 \right) + \frac{1}{2} \left(\frac{X_y}{X_n} \right)^2 \right] \quad (3.43)$$

The determination of the nondimensional parameters can be obtained from

$$\frac{a_1 X_n}{\gamma L^2} = C_1 \frac{H}{L} \tan \theta_1 \quad \text{and}$$

$$\frac{a_2}{a_1} = C_2 \frac{L}{H} \quad \text{where}$$

$$C_1 = \frac{2w\psi L}{S_y S_x} \quad (3.44)$$

$$C_2 = \frac{A_f s_y}{2w\psi \gamma L^2} \quad (3.45)$$

Finally, the nondimensional parameter X_y/X_n can be obtained from

$$\frac{x_y}{x_n} = \frac{1 + \frac{H/L}{\tan \theta_1} - \sqrt{\left[1 + \frac{H/L}{\tan \theta_1}\right]^2 - \frac{4H/L}{\tan \theta_1} \left(1 - \frac{C_2}{H/L}\right)}}{2} \quad (3.46)$$

which is derived from the equality condition

$$a_1 \left\{1 - \frac{x_y}{H}\right\} \left\{1 - \frac{x_y}{x_n}\right\} = a_2 \quad (3.47)$$

Therefore, all of the design values required in a reinforced earth wall are now defined and the problem can be solved completely in a nondimensional form.

3.9 Logarithmic Spiral

The shape of the failure surface occurring in retaining walls and slopes can be approximated by the logarithmic spiral method which was first described by Rendulic, and which is well publicized in the literature. This procedure takes the curvature of the sliding surface into consideration.

The logarithmic spiral is based on the principle that the resultant force makes an angle of ϕ with the tangent to the spiral and its line of action passes through the centre of the spiral. The logarithmic spiral is given by the following equation

$$r = r_0 \exp(\theta \tan \phi) \quad (3.48)$$

where r = radius of the spiral at any angle θ
from r_0

r_0 = starting radius

ϕ = angle of internal friction of the soil

θ = angle between r and r_0 , in radians

and will be fitted as closely as possible to the observed failure plane. The log spiral extends to a point where the slope of the spiral is $45^\circ + \phi/2$. After mathematically determining the centroid of the composite shape of the failure mass the active earth pressure, calculated from Rankine's earth pressure theory and the weight of the sliding wedge, can be calculated.

From these two forces acting through the centroid of the failure section the resultant of the weight of the sliding soil mass and the active earth pressure with respect to the straight portion of the rupture surface can be determined. The wall is stable if the line of action of the resultant R is located to the left of the centre of the logarithmic spiral (Broms, 1978).

CHAPTER IV

THEORETICAL CONCEPTS OF PHYSICAL MODELS

4.1 Introduction

The use of models in Engineering practice is unquestionably invaluable. However, for the optimum of information to be extracted from the use of models, their limitations must be realized. Furthermore, any model that is not of the actual size (full scale) and that doesn't exactly duplicate all the field conditions will have limitations. Thus, in the use of these particular models, it is hoped that the limitations are clearly understood. It can be argued that many problems can be solved reasonably well by elastic or plastic solutions, finite element analysis or mathematical models. However, there are many areas in Geotechnical Engineering which need to be assessed qualitatively as well as mathematically. Therefore, models such as the Steel Rod Model and the Base Friction Table are necessary to verify the assumptions made in a mathematical analysis. Furthermore, physical models are particularly useful in identifying the mode of failure and can therefore guide the more detailed and frequently more expensive computer studies.

Generally, gravity loads can be simulated in at least four ways: (1) the model can be built while it lies

flat or on an inclined plane and then tilted to increase gravity up to one g ; (2) the model can be spun in a centrifuge; (3) the model can be constructed of a material with a relatively high density such as steel rods. This model will be discussed in the next section; and (4) the model can be subjected to base friction as detailed later in the thesis.

Tilting experiments require lateral constraints, and are therefore susceptible to side wall friction effects. These will be particularly pronounced if the height to width ratio of the model is large.

Centrifuge models are the most complex and, correspondingly, expensive. Most centrifuges are capable of providing accelerations of up to one hundred times that of gravity, and therefore, it is theoretically possible to fail actual prototype materials, avoiding the inevitable distortions in model materials (Bray J., 1981). However, the testing space is limited to relatively small dimensions, and the models also require side constraints, resulting in an intermediate principal stress which could be quite significant. The main difficulty with centrifuge modelling is the remoteness of the testing domain, which can be monitored only by electronic means.

Therefore, of the different models mentioned, only the last two types are purely two dimensional models (as

opposed to plane strain) and were deemed suitable for the study of deformations and failure modes of reinforced earth structures.

4.2 Steel Rod Model

4.2.1 Introduction

The "steel rod" model is an apparatus which simulates the two dimensional behaviour of a cohesionless soil mass by using steel rods. In the study at hand, rods 100 mm in length and having three different diameters (3.2 mm, 4.8 mm and 6.4 mm) were used. Due to the high unit weight and the rigidity of the steel rods, relatively high retaining walls can be modelled without using any lateral supports normal to the plane of the model. This system has been proven to be an excellent mechanical analog to a granular soil, and is very suitable for investigating plane strain problems such as lateral earth pressures against retaining walls (Holtz and Broms, 1978).

Furthermore, most model tests are plagued by the problem of extremely low confining (overburden) stresses as compared to field conditions. This problem is somewhat alleviated by the steel rods due to their relatively high unit weight.

4.2.2 Theory of the Steel Rod Model

In the steel rod apparatus a soil mass is simulated

by steel rods stacked up in a parallel array. This arrangement represents a granular medium that will behave in a two dimensional fashion. The use of these horizontal metal rods eliminates the need for side walls perpendicular to the direction of movement, and hence eliminates the problem of side wall friction, which is difficult if not impossible to account for in other models.

It is assumed that there is a linear relationship between properties of the model and the prototype structure. That is, for simple problems in static plastic and elastic equilibrium, the dimensionless ratios, known as π terms, are the same in both model and prototype. This can be expressed as

$$\pi(\text{strength}) = \frac{S}{\gamma L} \quad (4.1)$$

and

$$\pi(\text{rigidity}) = \frac{E}{\gamma L} \quad (4.2)$$

where L is a linear dimension, γ is the unit weight of the material and S and E are the shear strength and modulus of elasticity respectively.

4.2.3 Stresses in the Steel Rod Model

The two dimensional stresses in the steel rod model correspond to those for a cohesionless soil mass. That is, the vertical stress, σ_v , is given by

$$\sigma_v = \gamma H \quad (4.3)$$

where H is the depth and γ is the unit weight of the material. Similarly, the horizontal stress can be determined from

$$\sigma_h = \gamma H K_A \quad (4.4)$$

where γ and H are as defined above, and K_A is the coefficient of active earth pressure of the model material.

In summary, the high unit weight, the rigidity of the "soil", the absence of side wall friction and the linear scaling factors, make the steel rod model well suited to simulate reinforced earth retaining structures.

4.3 Base Friction Model

4.3.1 Introduction

The Base Friction Table technique simulates gravity forces in the plane of a two dimensional physical model by drag forces acting along its base. These forces are supplied by the dynamic friction between a moving belt and the back of the model. The development of these frictional shear forces will simulate gravity loading in the model.

The first documented use of this technique was given by Erguvanli and Goodman (1972). They used a stationary sandpaper sheet across which the model material was pushed in order to study the effects of slope behaviour in jointed

rock masses having two sets of joints. Horizontal machines on a larger scale which utilize a continuous belt on rollers, were first introduced by Golder and Associates in Vancouver around 1976. Since then, various institutions, such as the University of California at Berkeley (Goodman, 1976), Imperial College London (Bray, 1981), and the University of Alberta (Baumgartner, 1980) have used them very successfully. In all these cases the devices were used for the study of two dimensional problems of rock behaviour. This study on hand is the first case in which the technique is employed to simulate the behaviour of cohesionless soil, in particular, the modes of failure associated with reinforced earth structures.

The base friction technique is particularly attractive because the deformation of the soil can take place at a relatively slow rate and can be controlled by the belt speed. Thus, a high speed camera, generally a necessity to photograph the movements in the steel rod model tests, is not required for the base friction model. Also, for the base friction model, the gravity forces can be turned off at any point of the test by stopping the belt, therefore "freezing" the behaviour of the model. Thus, a still camera is generally sufficient to capture the event.

4.3.2 Theory of Base Friction Modelling

The theory of base friction modelling is illustrated

in Figure 4.1. The model is loaded by pulling the continuous belt past the restraint bar at a velocity $\vec{V}$. The drag along the base of the model is resisted by a force F_b , the base friction force; and the sum of all the base friction forces in the model is equal and opposite to the total reaction force at the restraint bar (Figure 4.2).

$$\text{Therefore, } \int_{\text{Area}} d\vec{F}_{bf} = \vec{F}_B \quad (4.5)$$

The conditions of limiting equilibrium for a block on an inclined plane for both gravity and base friction loadings is illustrated in Figure 4.3. Under the influence of the forces of gravity, the weight of the block, W , is equal to the resultant (reaction) force R of the block on the plane; since the block is at rest. Limiting conditions for sliding are achieved when the resultant, R , is inclined at an angle, ϕ with the normal to the plane and intersects the plane within the limits of the blocks base (Bray J., 1981). Referring to Figure 4.3(a), the conditions of limiting equilibrium with regard to incipient sliding is:

$$\alpha = \phi \leq \arctan (b/h) \quad (4.6)$$

where α = inclination of the plane

ϕ = friction angle between the model and the surface

FIGURE 4.1 PRINCIPLE OF BASE FRICTION MODEL

Figure 4.2 Forces Associated with the Base Friction Model

b = width of the model

h = thickness of the model

Similarly, in the base friction model, as shown in Figure 4.3(b) for conditions of limiting equilibrium against sliding, the block remains stationary while the belt moves under it in a direction parallel to the inclination of the plane. The base friction force, F_b , is the only force applied to the block, and acts in a direction parallel to the plane. This produces a reaction (R) at an angle α with the normal to the plane. Therefore, Equation 4.6 can also be applied to the condition of limiting equilibrium at the moment of incipient sliding.

For a block situated on an inclined plane, sliding will not be initiated until the inclination of the plane equals or exceeds ϕ , the angle of friction between the block and the plane. This is illustrated in Figure 4.4. For a block moving with a constant velocity on this surface, D'Alembert's Principle applies. It stipulates that there is an inertial resistance force directed up the plane and passing through the centroid of the block. The normal reaction to the inclined plane acts at an angle ϕ to the normal of the base, and must also pass through the centroid of the block in order to satisfy equilibrium conditions.

The resolution of these forces is best accomplished with a force triangle. (Figure 4.5). From the law of

(a) Gravity

(b) Base Friction

Figure 4.3 Limiting Equilibrium for incipient
Sliding on an inclined Plane
(a) Gravity (b) Base Friction
(after Bray, 1981)

Figure 4.4 Dynamic Sliding Without Toppling for Gravity, $\alpha > \phi$, $\phi < \arctan (b/h)$

$$(90 - \alpha) + (90 + \phi) + z = 180$$

$$z = \alpha - \phi$$

Figure 4.5 Force Vector Triangle for Dynamic Sliding (Gravity) (after Bray, 1981)

sines, one obtains

$$\frac{-W\ddot{u}}{g \sin(\alpha-\phi)} = \frac{W}{\sin(90+\phi)} = \frac{R}{\sin(90-\alpha)} \quad (4.7)$$

Since one is interested in a constitutive relationship involving the downslope velocity, the weight and gravity, the first two terms of the equation are considered only.

$$\text{Thus, } \frac{-W\ddot{u}}{g \sin(\alpha-\phi)} = \frac{W}{\sin(90+\phi)} \quad (4.8)$$

$$\text{or } \frac{\ddot{u}}{g} = \frac{\sin(\alpha-\phi)}{\cos \phi} \quad (4.9)$$

If the same block (Figure 4.4) is considered under the action of a base friction force, as illustrated in Figure 4.6, the model material on the inclined plane is constrained by the barrier and thus remains stationary (after a time required to establish the initial displacements accompanying the start up of the belt) (Bray J., 1981). If the stability is such that the block remains at rest, then the equilibrium in the direction parallel to the belt velocity requires that

$$dF_b = \int_0^Z dT \quad (4.10)$$

where Z = the direction parallel to the belt

dT = the increment of the drag force under a differential dz , or $dT = \psi_b \gamma t d_x d_z$

Figure 4.6 Dynamic Sliding Without Toppling for Base Friction, $\alpha > \phi$, $\phi < \arctan(b/h)$ (after Bray, 1981)

Figure 4.7 Solution of Corresponding Velocities for Dynamic Sliding of Base Friction

where ψ_b = coefficient of friction between the
model and belt

γ = unit weight

t = thickness

If, however, the model moves down the inclined plane, for $\phi > \psi_b$, the direction of slip between the block and the belt is defined by the vector difference ($\vec{V}_r$) of the absolute velocities of the block ($\vec{u}$) and the belt ($\vec{V}$) (Bray J., 1981). Since the base friction force ($\vec{F}_b$) of the belt on the block is parallel to the direction of slip in their plane of contact, and is also the force transmitted from the barrier to the block, it is therefore parallel to $\vec{V}_r = \vec{V} - \vec{u}$. As Bray J., (1981) states "If the block moves with constant velocity, it experiences no inertia force; hence, $\vec{F}_b$ is the only body force applied to the block and must be collinear and opposite to the resultant reaction ($\vec{R}$) between the block and the inclined plane. If the block undergoes motion, $\vec{R}$ is inclined at ϕ degrees with the normal to the plane." Therefore, the directions of $\vec{V}_r$ and $\vec{F}_b$ are best determined by the velocity triangle in Figure 4.7. Again, applying the law of sines yields

$$\frac{\vec{u}}{\sin(\alpha - \phi)} = \frac{V}{\sin(90 - \phi)} = \frac{V_r}{\sin(90 - \alpha)} \quad (4.11)$$

As before, since one is interested in the initial velocity and induced velocity relationship, the final term can also be neglected from the equation.

$$\text{Therefore, } \frac{\dot{u}}{V} = \frac{\sin(\alpha - \phi)}{\cos \phi} \quad (4.12)$$

In comparing this result with Equation 4.9, it should be evident that the equation of motion for slip in the base friction realm is identical to the acceleration of gravity of a proto-type structure. At limiting equilibrium, $\dot{u} = \ddot{u} = 0$ and the magnitudes and directions of forces are correct. As the block acquires a constant velocity, the direction of the base friction force becomes different from that of gravity by an amount of $(\alpha - \phi)$. This introduces no distortion since $\vec{F}_b$ acts parallel to the vector sum of the weight ($\vec{W}$) and the inertia force ($-W/g \vec{u}$) (Bray J., 1981).

A similar approach, simulating gravity with base friction, can be made for the combined effects of dynamic sliding with toppling, dynamic toppling without sliding and for the motion of a block under pure rotation about a corner. These analyses are used to simulate rock falls and do not occur in problems associated with earth retaining structures.

In the above comparison the friction table was tilted through an angle of α to facilitate comparison

with a block on an inclined plane. In theory, α can vary from 0° to 90° , but in practice α should be less than ϕ in order to avoid slipping.

It should be noted that the base friction method cannot duplicate the correct response when the moving body acquires horizontal momentum since there is no mechanism for persistence of translation and rotation after "impact", or change in friction of the sliding surface. This aspect is best exemplified in Figure 4.8. Under gravitational influence the block in Figure 4.8(a) will describe a parabolic trajectory, whereas the block in Figure 4.8(b) acted on by base friction will follow the motion of the belt once the rotation has occurred that allows it to become free of the model.

4.3.3 Stresses associated with the Base Friction Technique

For a model of constant thickness and having a total height, H , (measured from the constraint barrier to the top of the model), the reaction on the barrier can be calculated from Equation 4.5

$$F_b = - \int_H^0 dF_b = - \int_H^0 t \gamma \tan \phi \, dh \quad (4.13)$$

$$\text{or} \quad F_b = \gamma H t \tan \phi \quad (4.14)$$

Furthermore, the reaction to the total base friction

Figure 4.8 Dissimilarity of Gravity and Base Friction Models where Momentum is not negligible (after Goodman, 1976).

force at any elevation from the barrier h , is

$$F_b(h) = \gamma(H-h)t \tan \phi \quad (4.15)$$

From this, it is evident that the base friction force increases linearly with height and on the friction angle ϕ , the unit weight γ and the thickness t of the model.

It should be noted that the rate of increase is independent of the belt velocity. Similarly, the vertical stress at a depth $(H-h)$ is

$$\sigma = \gamma \tan \phi (H-h) \quad (4.16)$$

The stress reaches a maximum value at the elevation of the barrier or,

$$\sigma_{\max} = \gamma \tan \phi H \quad (4.17)$$

4.3.4 Discussion

The base friction technique accentuates the deformation behaviour of a model to failure at a slow rate of particle displacement. This is achieved by mobilizing drag or friction forces to represent gravity in a two dimensional model.

Although inertia forces are present, in systems undergoing slow movements, they may be considered negligible compared to gravitational or drag forces.

Frictional forces are transmitted through contacts, and the establishment or loss of contact depends on the relative displacements between the model and the belt.

The relative displacements are dependent on the acceleration in the real structure and on belt velocity in the base friction model. For any configuration that is linked kinematically during the failure process, the velocity diagrams in the model and the acceleration diagrams in the real structure (under gravity) should be geometrically similar (Bray J., 1981). This means that the model should pass through the same sequence of configurations as the real structure. However, if different parts of the model move completely independently, their relative rates of movement will probably be inconsistent with those in an actual structure acted upon by gravity. Thus, if acquired accelerations are small, this divergence should also be small.

For the actual case of the failure of a structure large inertial forces exist and will alter the balance of forces. The base friction model may then give misleading results since the relative movements of independent moving parts of the model may be wrong, and the effects of momentum and of impact will be absent. However, this aspect is more prevalent in the types of models that the base friction technique was formerly used to depict, such as for the study

of rock behaviour in underground stopes, or in tunnels etc. For these problems the model material is commonly made of sand, flour and oil combinations with discontinuities and rock fabrics simply cut into the model. Since the model may fail along these discontinuities or fracture to form new ones, the trend is somewhat different from that in this particular study. Furthermore, this study involved models of uniform mass with no induced discontinuities and with a rather insignificant area of tensile reinforcing strips.

For rock mechanics models where a gallery or any such opening is first removed from the system before failure is initiated, the effects of momentum could in fact be significant. However, for the study of reinforced granular soil where the model was packed to approximately the same void ratio each time, and the soil had no large openings, this effect would certainly be negligible.

Two assumptions implicit in the base friction theory are worth mentioning here. The friction coefficient between the belt and the model remains constant throughout the testing. This assumption is valid, for the same belt was used throughout the entire duration of this study and no difference in the surface abrasion could be detected. The end and side restraints of the apparatus do not contribute to the stability of the model. This aspect will be dealt with later on in the thesis.

In conclusion, the base friction model is a powerful tool to study the deformation behavioural response of two dimensional problems affected by gravity. As long as its limitations are understood, it can be used very successfully to simulate the behaviour of prototype structures. In comparison with other methods for simulating gravity, the technique is found to be convenient, inexpensive and revealing.

Most of the theoretical aspects contained herein evolved from the work by Goodman (1976) and Br and Goodman (1981).

CHAPTER V

TEST APPARATUS

5.1 Steel Rod Model

The steel rods used for the two dimensional simulation of a cohesionless soil mass required a frame to contain the stacked up rods in a vertical plane. This frame consisted of four aluminum U-channels which were fastened together by brackets at each corner of the frame. The beam forming the rigid bottom (foundation or base beam) of this apparatus consisted of a 150 mm wide by 90 mm deep by 15 mm thick aluminum channel section 1.83 m in length. A 50 mm wide slot was cut into the middle of this to accommodate a series of load cells. The function of these cells will be discussed in detail in the next section. The two upright (vertical) supports and the reaction beam (top) of the frame were made of 130 mm wide by 60 mm deep by 10 mm thick aluminum channels, 1.59 m and 1.5 m in length respectively. These were fastened together using aluminum angle brackets along with eight 12 mm bolts per connection to ensure absolute rigidity of the frame. Furthermore, the reaction beam was designed in such a fashion that it could be adjusted to various heights above the base (foundation) beam. Figure 5.1 shows the details

Figure 5.1 Detail of Steel Rod Testing Apparatus (after Scalzitti, 1979)

of the testing apparatus. The entire frame was clamped to a solid wooden table in order to provide a convenient working height. Also, the frame was anchored to the wall at the top to prevent the possibility of overturning.

A model footing (illustrated in Figure 5.1) was also used in some of the tests in order to investigate the effect of a strip load on the deformation behaviour of the reinforced soil. The footing load was applied through the gearjack assembly mounted to the reaction beam, and the applied vertical loads were measured using a proving ring. The footing consisted of two rigid steel plates, comprising the shaft and the base, and were welded together perpendicular to each other. The width of the base of the footing was chosen as 150 mm. The dimension of the footing normal to the testing plane was the same as the length of the steel rods (100 mm).

5.2 Base Friction Apparatus

The design and construction of the base friction apparatus was carried out without the aid of any previously made design drawings, specifications or helpful descriptive notes. Although some reference to the use of the friction table for the study of rock slope behaviour was published (Goodman, 1976; Bray, 1981 and Baumgartner, 1979), little information was given to construction details. Therefore,

a brief description of the key elements used in the apparatus will be contained in this section, and more complete construction procedure is contained in Appendix B.

The apparatus was designed in order that the model area (belt) was at the same height as a standard table top. Figure 5.2 is a scale drawing giving a complete perspective of the base friction apparatus. The entire frame, including the legs were made of aluminum angles and are bolted together.

The surface of the table was made of 19 mm thick plywood with a 1.6 mm thick sheet of galvanized steel glued to the top of it with epoxy cement. This ensured an absolutely smooth and level surface for the belt to run over. When it was positioned on top of the frame, the surface of the galvanized plate was at an elevation slightly above that of the top of the rollers, to ensure that there was no sagging of the belt even if it was pulled taut. Figure 5.3 shows a photograph of the apparatus in operation.

The belt was custom made by the 3M Corporation. It was 1.32 m wide and 2.1 m in circumference, and consisted of No. 220 grit emery on a resin reinforced cloth base. This material is unaffected by liquids, hence modelling of cohesive soils is possible.

The two rollers were made from 102 mm O.D. steel

BASE FRICTION MODEL
FIG. 5.2

Figure 5.3 Base Friction Apparatus

tubing with a 3.2 mm wall thickness. The front roller provided the belt drive, and the rear roller was responsible for tensioning the belt. Crowning of the rollers was not necessary since belt wandering was minimal, due to the low testing speeds. Torque was effectively transmitted from the drive roller to the belt, therefore slippage did not occur and thus no frictional material was required on the surface of the roller.

The roller rotation was supplied by an electric motor rated for 1/3 h.p. at 1725 R.P.M. This motor was connected to two sets of reduction gears (50:1 and 10:1 gear ratios), and a variable speed control (Figure 5.4), and therefore the belt speed could be completely varied. The motor was fastened to a separate frame which rested on the floor. The motor-drive unit was independent of the table in order that no vibrations could be transferred from the motor to the frame. This was absolutely necessary to prevent any disturbance of the soil model.

The apparatus functioned flawlessly throughout the complete series of tests.

Figure 5.4 Motor, Variable Speed Control and
Reduction Gears for Base Friction
Apparatus

CHAPTER VI

MODELLING OF REINFORCED EARTH STRUCTURES

6.1 Steel Rod Model

6.1.1 Soil

For the two dimensional modelling of an idealized granular, cohesionless soil mass, metal rods have proved to be an excellent mechanical analog (Holtz and Broms, 1978). For this particular study, steel rods 100 mm in length and of three diameters (3.2, 4.8 and 6.4 mm) were selected. The soil was modelled using quantities of different diameter rods in the ratio of 2:2:1, a combination which performed remarkably well and resulted in void ratios corresponding to realistic field values for granular fills generally specified for use in reinforced earth retaining walls. The total quantity of rods required was in the order of 32,000 pieces. These were cut on a reciprocating saw and deburred by hand (Scalzitti, 1979).

6.1.2 Soil Parameters

To measure the "in situ" representative unit weight of the steel rods in the model, the total weight and area occupied by the stacked rods was measured. The average unit weight of this material with very little

2

4

deviation was 61.8 kN/m^3 . Similarly, the measured representative void ratio, e , was 0.21 for the model "soil".

The frictional properties of the "soil" (steel rods) was determined in a conventional Wykenham Farrance Direct Shear Box apparatus (Figure 6.1). The steel rods had to be cut to a length of 59 mm to fit inside the shear box. The ends of the rods were de-burred to ensure that no "binding" would occur. The rods were randomly arranged in the box using the same 2:2:1 ratio mentioned previously. Since the shear strength is a function of density and packing, every attempt was made to ensure that the sample tested for the angle of internal friction, ϕ , was representative of the soil mass used in the tests. Typical load-displacement curves for various normal loads are illustrated in Figure 6.2. An average of the maximum shearing loads, disregarding anomalous or peak values was selected for each curve, and these peak values were used in the standard calculations for determining ϕ . Figure 6.3 shows the value of the internal friction angle to be 33° . Correcting for the change in area of the direct shear box during horizontal displacement had a negligible effect on the friction angle. The value obtained compared well with the results obtained by Scalzitti (1979). He conducted similar tests and found a value of ϕ for such an arrangement of steel rods to be 33.7° .

Figure 6.1 Direct Shear Box Apparatus

Figure 6.2

Figure 6.3

6.1.3 Tensile Reinforcing Strips

The reinforcing ties used in these model tests were made from 0.04 mm thick sheet steel. They were cut to a width of 40 mm, a value based upon preliminary design calculations (Bowles, 1977). Three different strip lengths (275, 412.5 and 550 mm) were selected. The significance of these lengths will be dealt with in the next chapter of the thesis. These reinforcing strips had an ultimate tensile strength of 6.4 kN, sufficient that they would not break during any phase of a test.

In order to fully evaluate the stability of model reinforced earth walls and to design actual structures, it is necessary to determine the value of the friction mobilized between the tensile reinforcing strip and the soil mass. This was accomplished by modifying a standard 60 x 60 mm direct shear box as illustrated in Figure 6.4. A wooden block was cut to fill the bottom portion of the box, and a section of the actual reinforcing strip was glued to the top of the block. Thus, when the upper portion of the shear box was filled with the steel rods as explained in section 6.1.2 and sheared across the surface of the metal strip, the friction angle, ψ , between the soil and the strip could be determined. The direct shear test is a better method than a pullout test for determining the friction coefficient because no dilatancy

Figure 6.4

Modified 60mm x 60mm Direct Shear Box Test

occurs. Figure 6.5 shows the wooden block in the bottom half of the shear box and the steel rods used in the test. The results obtained from these tests are illustrated in Figures 6.6 and 6.7. The value for the friction angle, ψ , between the steel rods and the tensile reinforcing strips is about 7 degrees.

In a model wall test one end of the strip was fastened with a small "wing-bolt" to the individual wall panel simulating a typical prototype connector. The wall panels are discussed in the following section.

6.1.4 Wall Panels

The wall panels, theoretically, are a non-structural part of a reinforced earth wall. Their function is to maintain the integrity of the structure by preventing soil from becoming dislodged from the wall, and in addition they provide a surface to connect the reinforcing strips to. However, for esthetic and economic reasons they are commonly made of precast concrete panels.

In order to duplicate the wall components used in the model study as closely as possible with those used in a prototype structure, individual wooden panels were selected to simulate the precast concrete elements. These were made of plywood, 16 mm thick and 100 mm wide, that is, equivalent in length to the steel rods. Three different lengths of panels, 150, 100 and 75 mm were selected

Figure 6.5 Components for Modified
Direct Shear Box

Figure 6.6

Figure 6.7

STRIP/SOIL FRICTION TEST

in this investigation. The surfaces of panels in contact with one another were lined with teflon tape to reduce the effects of friction.

A small hole was drilled through the centre of each panel to accommodate the Wing-bolt used to connect the reinforcing ties.

In the design of actual earth retaining structures, assumptions are often made regarding the value of the back-wall friction angle, δ . For this particular study, an estimate of this value was obtained from direct shear box tests (Figure 6.4). In these tests a wooden wall panel filled the bottom half of the shear box, and tests were conducted as discussed in section 6.1.3. Figures 6.8 and 6.9 give the results of these tests, and show the value of δ to be about 17 degrees.

6.1.5 Instrumentation

The first series of tests in the steel rod model contained an elaborate instrumentation package. This included a strain-gauged connector at the middle of the reinforcing strip, load cells at the base of the stacked rods, horizontal wall deflection indicators and devices to measure the footing load and the settlement.

The strain gauged connector strip was made from 3.2 mm thick steel, 76 mm long, 40 mm wide and was necked

Figure 6.8

WALL/SOIL FRICTION TEST

Figure 6.9

at both sides in order to prevent stress concentrations (Figure 6.10). The actual strain gauge configuration was a full bridge setup with dummy gauges, that completely negated the effects of bending. Therefore, only tensile strains were measured in the reinforcing strips.

Precision universal load cells with a capacity of 44.5 kN were used to measure the vertical loads at the base of the foundation. Three of these cells were located beneath the aluminum channel. Each cell measured the vertical load over a 50 mm wide portion of the base. Both the strain gauged strips and the load cells were monitored through digital strain indicators and switch and balance units shown in Figure 6.11. Complete instrument specifications and limitations are contained in Appendix C.

The vertical load applied by the gearjack through the "footing" was measured with a calibrated proving ring, while the horizontal deflection of the wall was measured by means of tracing the deflection profile on a paper grid.

6.2 Friction Table

6.2.1 Soil

Various sizes of circular disks were found to be most suitable to represent the granular material in the

Figure 6.10 Strain Gauged Connector in Reinforcing Strip

Figure 6.11 Instrumentation Monitoring Setup

two dimensional base friction model. Although the use of cohesive soils in reinforced earth was considered by other researchers (Fang, 1978; Murray, 1977; Boden et al, 1977), it does not readily lend itself to two dimensional model studies as considered here. The stability of reinforced earth structures using cohesive soil as back-fill material is to a large extent a function of the pore-water pressures within the soil. The build-up and dissipation of excess porewater pressures which effect the degree of friction mobilized between the soil and the reinforcing element cannot be simulated with this apparatus. For studies of this type, a much larger three dimensional apparatus with elaborate measuring devices would be required, such as a centrifuge, for example.

The initial modelling of the granular soil was done with steel disks of 6.4 mm, 9.5 mm, 12.7 mm, 15.9 mm and 19.1 mm in diameter and 3 mm in thickness. These were punched out of a sheet. Burrs were removed by placing the disks in a tumble-mill.

This material seemed to work satisfactorily, and behaved as a cohesionless soil mass when the overall area of the model was kept small. However, problems occurred in the form of local buckling or overriding of the disks when the model was increased to a reasonable size for testing. This was due to a combination of the buildup

of stresses and the high unit weight of the disks coupled with the high aspect ratio (d/h) since the disks were relatively thin. Many different ratios of disk sizes were experimented with, but all produced the same inadequate result. Furthermore, since the disks had been punched out, their sides were not uniform, but they tended to have a truncated cone shape and this contributed largely to the "overriding" problem.

It was therefore decided to try and set up a magnetic field in a flat steel plate located directly below the belt. The magnetic flux going through the plate and the disks was to prevent buckling and to make the disks "stick" to the belt. It was hoped that this method could also be used to simulate some cohesion of the soil mass, since the disks would also be attracted to each other.

To create the magnetic field, a soft steel core, 50 mm in diameter and 1.5 m long, was wound with 2400 turns of No. 12 square copper wire. The core was then connected at both ends to the flat steel plate forming the base of the friction table. Alternating current was unsuitable due to the heat generated by rapid reversal of the poles. Therefore, a direct current was used to generate the magnetic field. The D.C. source was obtained from three 12-volt car batteries connected in series.

This produced about 24 amperes with a corresponding magnet strength of 57,600 Amp. turns. It was successful to a certain degree, in that it prevented the disks from overriding one another at the centre of the table where the magnetic flux was the strongest, but it did not solve the problem as the distance from the magnet's core increased. Eventually, the concept of the steel disks in a magnetic field had to be abandoned.

To alleviate the problem of overriding of disks, a lighter and thicker model material was required. Disks cut from a rigid acrylic plexiglass tubing filled with a substance similar to plaster of paris (hydrostone), performed very well. Using tubing of 9.5 mm, 12.7 mm and 15.9 mm outside diameters (O.D.) and having a wall thickness of 1.6 mm the optimum thickness for the disks was found to be 5.8 mm. Furthermore, the ratio of 2:2:1 respectively for the 9.5, 12.7 and 15.9 mm diameter disks was used to simulate representative soil void ratios in the field. The different diameter disks are shown in Figure 6.12. No further problems of overriding occurred with these disks. A total of six thousand disks were cut by hand on a band saw with the same ratio of sizes as explained above. The removal of burrs was accomplished by lining the inside of a standard sieve with emery cloth, and shaking the disks in a mechanical sieve shaker.

Figure 6.12 Components used in Base Friction Tests

6.2.2 Soil Parameters

The kinetic angle of repose for the disks on the belt surface was obtained by running the base friction apparatus with a large area of "soil" on it. These tests began with the disks forming rather steep slope angles for the mass of soil, with progressive failures causing the slope angles to become flatter. Eventually the slope reached a state of equilibrium with respect to further failures. The resulting slope angle was taken as representative of the kinetic angle of repose for the disks on the base friction belt. This was repeated several times, and the average angle with deviations of less than 2° was found to be 41° for the belt speed used. This speed of the belt was kept constant throughout the testing. This value was taken as the internal friction angle, ϕ of the model soil.

The average density for the disks was 1.32 g/cm^3 . This meant that the weight of the individual disks having an O.D. of 15.9, 12.7 and 9.5 mm was 1.55 g, 1.00 g and 0.525 g respectively.

The average void ratio of the two dimensional arrangement of disks was 0.25. It should be noted that the placing of the disks was purely at random, but all efforts were made to ensure that the distribution of disks was well-graded for each test.

6.2.3 Tensile Reinforcing Strips

The reinforcing strips used in the base friction tests were cut from the same material as those used in the steel rod model tests. These were cut to a width of 6 mm (thickness 0.04 mm) and had lengths of 275, 412.5 and 550 mm. They had an ultimate tensile strength of 0.96 kN, that is, they were sufficiently strong not to rupture during any phase of a test. A typical strip was shown in Figure 6.12.

6.2.4 Wall Panels

Commensurate with the steel rod model tests, and with the actual method of reinforced earth construction, individual wall panels were also used for the base friction tests. These consisted of wooden strips 6.4 mm thick, and 12.7 mm wide. Three different lengths of panels were used (75, 100 and 150 mm), that is, identical to the panels in the steel rod model investigation. Figure 6.12 shows one of the wooden panels connected to a reinforcing strip

Since the wall panels rested directly on the moving belt, the contact areas were covered with teflon tape in order to reduce the effects of friction. Teflon tape was also used to reduce friction at the contact areas between panels.

A small hole was drilled in the centre of each panel so that a thin wire could be passed through and thus connect the reinforced earth strip to the panel. This also allowed easy interchanging of different sized panels and strips. This is also visible in Figure 6.12.

CHAPTER VII

MODEL WALL TESTS

7.1 Steel Rod Model.

Model tests regarding the overall behaviour of reinforced earth structures which were conducted on the steel rod model yielded similar results to those obtained on the model tests using the base friction apparatus. The latter tests will be discussed in section 7.2 of the thesis.

Two different modes of failure associated with the pullout of reinforcing ties were recognized. These are, Mode 1, which is the most common reinforced earth configuration where sufficient volume of soil is present behind the reinforced earth block to allow a natural failure surface to form in the unreinforced soil mass; and Mode 2, where the failure plane goes through the reinforced earth block. Mode 2 type failures will occur in reinforced earth retaining walls adjacent to rock cuts, or in bin or crib structures where the opposite walls are not tied together. Figures 7.1 a and b illustrate Mode 1 and Mode 2 types of walls.

The stability of model reinforced earth walls was investigated using the steel rod apparatus by varying two of the components which are used in prototype structures. These are, the length of the reinforcing strips, (L) and

Figure 7.1.a Cross Section of Reinforced Earth Wall;
Mode 1 Configuration

Figure 7.1.1.b Cross section of Reinforced Earth Wall;
Mode 2 Conditions

the vertical spacing of the strips or, the height of the individual wall panels, (S). The width of the reinforcing strips, w was kept constant for all of the tests. Three different lengths of reinforcing strips were selected (275, 412.5 and 550 mm), and were based upon the general design rule which specifies that the length of the reinforcing strips should be equal to approximately 80 percent of the total height of the retaining wall. Also, three different heights of wall panels were used in conjunction with the various strip lengths. These were 75, 100 and 150 mm in height. Therefore, this part of the study was undertaken to expand the knowledge concerning reinforced earth by investigating Mode 1 and Mode 2 failure patterns by using all possible combinations of the variables L and S in conjunction with model structures.

7.1.1 Instrumented Model Reinforced Earth Walls

The first series of tests were conducted using the instrumentation package described in detail in section 6.1.5. This included load measuring cells in the retaining wall's foundation, and strain gauged connectors inserted in the reinforcing strips. The walls in these experiments were constructed to a predetermined height, based upon stability calculations, and then tested to failure by applying a surcharge load through the model footing described in section 5.1.

Figure 7.2 shows the conditions after failure of one of the test walls. The reinforcing strips and strain gauge locations can be seen within the "soil" mass, while the load cells appear as three small openings in the base of the foundation. Failures of this sort were rather spectacular and destructive. They also posed a hazard to the individual who was setting up the experiment, as the mass of steel retained by these walls represented a dangerous amount of potential energy. Therefore, as a safety precaution, a temporary restraining device (seen in Figure 7.4) was added to prevent the total collapse of the structure.

The setup prior to testing is seen in Figure 7.3. The surcharge load was applied in small increments through the footing by the gearjack, which was situated above the retaining wall and which was fastened to the reaction beam (Figure 5.1). As the "soil" mass behaved in an elastic fashion, there was immediate dissipation of some of the surcharge load. Therefore, time was required for the surcharge load to be stabilized, and the mass of "soil" to be in a state of elastic equilibrium, before readings could be taken on the load cells, strain gauges, settlement dial gauges and horizontal deflection profile. There was no interference from strain gauge wires, as they were put through hollow plastic tubing which had the same diameter (6.4 mm) as the largest diameter steel rod, and were incorporated directly into the "soil" mass.

Figure 7.2 Failure of Instrumented Test Wall

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The test shown in Figure 7.3, when surcharge loaded, produced a bearing capacity failure beneath the model footing. As this was undesirable in these experiments, a uniform surcharge load was placed on the top surface of the wall to alleviate this problem. This surcharge load was comprised of canvas bags filled with lead shot. Each bag weighed exactly 44.5 N.

Figure 7.4 shows a model reinforced earth structure prior to testing, and the various components used in the test. The displacement of the model footing at the top of the wall is measured by two dial gauges. Thus, the complete load-settlement behaviour of the structure was obtained. The canvas bags mentioned previously are placed around the footing to prevent the displacement of the "soil" from around its base. The temporary restraint device mentioned earlier can be seen just to the left of the face of the wall. Also, the device used to trace the horizontal deflection profile of the structure is seen in the background. Finally, the front face of the wall (parallel to the length of the steel rods) was marked using a 50 mm-square chalk-line grid to aid in visualization of the distortions and movements of the "soil" mass.

7.1.2 Critical Height Tests

The instrumentation setup was time consuming, and

Figure 7.3 Model Reinforced
Earth Wall Prior
to Testing

Figure 7.4 Model Reinforced Earth
Wall with Modifications
Prior to Testing

contained some obvious shortcomings. The most critical of these is that the strain gauged connector strip was far too bulky in relation to the simulated "soil" mass and the size of the model structure. As a result of this, it is possible that a small passive wedge could form around the strain gauged reinforcing strip, and might affect results. In addition, it was decided that no further information could be obtained through continued testing using this setup. Therefore, a complete series of tests to determine the height to failure (H_{crit}) of model walls without any instrumentation or surcharge loading were conducted.

The construction technique used in these model walls was a duplication of the actual construction procedure used in prototype structures. That is, the bottom panel was temporarily supported as the rods (backfill) was placed behind it. When the level of the "fill" reached the required height for the spacing (S) of the strips, the surface was leveled and a strip was fastened to the wall panel. Backfilling then continued until the level of the "soil" reached the top of the panel. The friction forces between the "soil" and the reinforcing strip were then sufficient to hold the panel and strip in place. The stacking of the steel rods was facilitated by an adjustable panel, 150 mm high and long enough to span the entire width of the frame of the apparatus. This panel was fastened at the back of

the frame and was used to ensure even packing of the steel rods, and to provide a means of temporarily supporting the topmost panel as the wall construction proceeded. This temporary support was provided by a steel bracket which was made of steel bars welded together and which was clamped to the adjustable back panel. This stage construction process was repeated until the entire structure collapsed.

Figure 7.5 shows a typical wall nearing completion. The adjustable backing strip, the support bracket for the topmost wall panel and the reinforcing strips and their connection with the individual wall panels are visible in the picture. Placing of the steel rods was completely random. Also, both the wall panels and reinforcing strips were placed using a carpenter's level to ensure that the wall was vertical and the reinforcing strips were horizontal. This ensured the uniformity of each structure.

7.1.3 Recording of Test Results

The walls were built until failure occurred, and the entire process was recorded using a Sony black and white video camera. This entire setup, including the camera, the TV monitor and the recording machine is shown in Figure 7.6. Since the movement of the wall at collapse was too fast to be visualized with the naked eye, the actual failure event was captured on high-speed 35 mm film. These exposures

Figure 7.5 Model Reinforced Earth
Wall During Construction

Figure 7.6 Video Camera, and
Recording Equipment

were obtained by replaying the wall's collapse and by taking pictures directly off the screen of the video monitor using slow shutter speeds of between $1/8$ and $1/2$ second. Using this technique, pictures were obtained in which the portion of the "soil" mass undergoing failure appeared as a blurred region, while the portion which was stationary remained clear. Figure 7.7 is a typical example.

7.1.4 Test Results

A total of 21 tests including those with instrumentation and those considering the various wall geometries were performed on the steel rod apparatus. Table 7.1 is a summary of the test results, including the height of the wall, the spacing and lengths of the strips and the normalized H/S and L/S ratios for each test.

Typical failure patterns for Mode 1 and Mode 2 configurations are shown in Figures 7.8 and 7.9. The failure surface for the Mode 1 type structures initiates at a shallow angle, θ_1 , within the zone of reinforcing, and then changes abruptly to a much steeper angle, θ_2 , in the backfill material behind the reinforced zone. The trend of the failure surface within the reinforced earth block is a result of a zone of decreased friction angles, which is below that of the internal friction of the soil, and is found adjacent to the reinforcing strip. Furthermore, this

Figure 7.7 Failure of Mode 1 type Model
Reinforced Earth Wall

Table 7.1 Summary of Steel Rod Model Test Results

Test #	H (mm)	L (mm)	S (mm)	Mode	H/S	L/S	Comments
1	800	550	200	--	4.0	2.75	- one-piece retaining wall
2	800	550	200	--	4.0	2.75	- one-piece retaining wall with strain gauged reinforcing strips
3	800	550	200	--	4.0	2.75	- load cells in foundation; strain gauged strips; settlement gauges; vert. loading
4	800	550	200	--	4.0	2.75	- uniform surcharge loading - same instrumentation as above
5	800	550	200	--	4.0	2.75	- static failure occurred with no external forces
6	750	550	150	--	5.0	3.67	- double sided crib-type wall
7	750	550	150	--	5.0	3.67	- double sided crib-type wall tested for overturning
8	350	550	150	2	---	----	- failure due to compaction
9	450	275	150	2	3.0	1.83	- failed
10	600	275	100	2	6.0	2.75	- failed
11	900	550	150	2	6.0	3.67	- stable
12	750	550	150	1	5.0	3.67	- stable

Steel Rod Model Test (Cont'd)

Test #	H (mm)	L (mm)	S (mm)	Mode	H/S	L/S	Comments
13	712.5	275	75	2	9.5	3.67	- failed
14	525	275	75	1	7.0	3.67	- failed
15	505	275	100	1	5.05	2.75	- failed
16	412.5	275	150	1	2.75	1.83	- Failed
17	435	275	150	1	2.9	1.83	- failed, repeat of test #16
18	740	412.5	150	2	4.93	2.75	- failed
19	700	412.5	100	2	7.0	4.125	- stable
20	605	412.5	150	1	4.03	2.75	- failed
21	635	412.5	100	1	6.35	4.125	- failed

Figure 7.8 Typical Pattern for
Mode I type Failures
L= 275 mm, H= 500 mm,
S= 100 mm

Figure 7.9 Typical Failure Pattern for
Mode 2 type Failures
L=275 mm, H=712 mm, S=75 mm

Figure 7.10 Failure Surface for Mode 1
Configuration, L=275 mm,
H=525 mm, S=75 mm.

phenomena can be explained by the movement of the bottom panel. If the magnitude of the maximum horizontal force at the bottom panel is large enough to cause horizontal movement of the panel and the reinforcing strip (tie pull-out), then the failure will occur on a parallel plane close to or on the reinforcing strip surface. Hence, following the path of least resistance, or occurring in the zone which possesses the minimum internal friction coefficient. This zone, which surrounds the reinforcing strip is transitional, having a minimum angle of internal friction at the soil/strip interface, and which increases with increasing distance from the reinforcing tie, until at some distance it reaches the ultimate internal friction angle of the soil. Other examples illustrating this failure pattern are shown in Figures 7.10 and 7.11.

The failure trend for Mode 2 configurations is influenced by the edge of the frame in the model tests, and by the soil/rock boundary in prototype structures. This type of failure has a rotational tendency which, as in the Mode 1 situation, results in the failure surface acting almost parallel to the bottom reinforcing strip. Typical examples of the Mode 2 failure patterns occurring in the model walls are shown in Figures 7.12 (a) and (b).

In both the Mode 1 and Mode 2 model tests it was observed that the bottom panel underwent a rotational

Figure 7.11 Failure Occuring in Mode I Structure
L=412.5 mm, H=605 mm, S=150 mm

(a)

(b)

Figure 7.12 Failures in Mode 2 Configurations
(a) $L=275$ mm, $H=600$ mm, $S=100$ mm
(b) $L=412.5$ mm, $H=740$ mm, $S=150$ mm

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movement, while the central portion of the reinforced zone was pushed out laterally. This is illustrated in Figures 7.13 (a) and (b) and in Figure 7.14 (Mode 2). In these cases, the middle portion of the model reinforced earth structure remains relatively un-deformed, while some minor bending of the reinforcing strips farthest away from the wall is also shown.

7.2 Base Friction Apparatus

For the model wall tests with the base friction apparatus, the same two modes of failure were recognized as in the steel rod model tests. These are Mode 1, which is the actual reinforced earth configuration where sufficient volume of soil is present behind the strips to allow a natural failure plane to form in the unreinforced soil mass; and Mode 2 where the failure plane goes through the reinforced earth block.

The selection of the length of the reinforcing strips was based on the general design rule which specifies the length of the strips is equal to 80 percent of height of the retaining wall. Therefore, using this as a guide, and taking into consideration the physical limitations of the model apparatus, strip lengths of 275, 412.5 and 550 mm were selected. This would correspond to wall heights of 344, 516 and 687 mm respectively. The 412.5 mm length was

(a)

(b)

Figure 7.13 Rotation of Bottom Panel in
Model Configurations

(a) $L=412.5$ mm, $H=605$ mm, $S=150$ mm

(b) $L=412.5$ mm, $H=635$ mm, $S=100$ mm

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Figure 7.14 Rotation of Bottom Panel in
Mode 2 Configurations
L=275 mm, H=600 mm, S=100 mm

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arbitrarily chosen as it is midway between the short and long strip. The testing program therefore covered a full range of wall heights presently being built in actual structures. The stability of reinforced earth systems presently in use could therefore be evaluated. Of the many variable factors which will affect the stability of such a structure only three quantities were varied: the length of the strips, (L), (275, 412.5 and 550 mm); the vertical spacing of the strips or height of the individual wall panels, (S), (75, 100, 150 mm); and the height of the retaining wall, (H), (400, 450, 700 and 750 mm). Both the length of the strips and height of the wall panels were identical to the steel rod model tests, in order that a comparison of the shape of the failure planes could be made. The width of the strips have no significance in a two-dimensional model such as this.

In order to compare the performance of each test quantitatively considering the various wall geometries, the measured time to failure was recorded. Failure was arbitrarily defined as the movement of 10 percent of the wall height either in settlement (s) or by horizontal deflection (δ). These limits of deformation were marked physically on the test surface with strings suspended above the model. The strings were not in contact with the belt or the disks. These string lines are clearly visible

on the photographs which were taken throughout the tests.

For the Mode 1 tests, there was sufficient extent of soil behind the reinforcing strips for an active failure plane to form in the unreinforced soil. In the Mode 2 tests the entire soil mass was reinforced, that is, the reinforcing strips terminated at the rigid boundary of the friction table. The set up for both Modes can be seen in Figures 7.17 and 7.18.

A belt speed of 142 mm/minute (lineal) was selected as being most suitable. This speed, therefore, was used throughout the testing program. A more comprehensive investigation would have included the varying of belt speeds (and hence, gravity) to determine the stability of the various configurations over a wider range of implied gravitational forces. This would require a much more extensive testing program, and thus is beyond the scope of this study.

The main objective was to assess the stability of the reinforced soil structures and to determine the most prominent failure plane for each. Since the breaking of the reinforcing strips was not investigated as a possible mode of failure, the failure planes determined by these tests apply only to the case of tie pullout, and may differ somewhat when the mode of failure is by tie rupture.

7.2.1 Construction of Walls

Construction proceeded in the same manner as in

the actual case of a reinforced earth structure. That is, "soil" was built up to the level of the first layer of reinforcing strips, (in this case, only one strip) and then the wall panel and strip were put into place. A small weight was placed on the panel in order to keep it perfectly positioned while the disks were placed against the wall and adjacent to the strip. The process was then repeated until the wall was constructed to the predetermined height.

Figure 7.15 illustrates the method of construction of a wall. The guide strings defining the failure criteria mentioned in the previous section were then positioned, and the model was ready for testing.

7.2.2 Recording of Test Results

All tests were recorded in their entirety by a Sony black and white video camera. A scaffold was erected around the base friction apparatus (see Figure 7.16) in order to position the camera directly above the model. A 12.5 mm wide-angle lens was used on the camera which was mounted at a height of 1.74 m above the surface of the table. This provided a field of view sufficient to cover the complete testing area. In addition, many stages of the tests were photographed with a 35 mm camera (see Figures 7.17 to 7.19). The still photographs and the video movies provided a continuous record of the deformation behaviour.

Figure 7.15 Construction of a Model Reinforced
Earth Wall on the Base Friction Table

Figure 7.16 , Base Friction Apparatus and
Camera Setup

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(b)

(a)

Figure 7.17 Test Setup for Mode 1 Configuration of Base Friction Table. (a) Before Testing, (b) After Failure

(a)

(b)

Figure 7.18 Test Setup for the Mode 2 Configuration on Base Friction Table. (a) Before Testing, (b) After Failure

(a) Start of Test

(b) After Failure

Figure 7.19 Tilting of the Top and Bottom Panels

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of the walls throughout the tests.

7.2.3 Test Results

A total of 38 tests considering the various wall geometries were performed with the base friction table. A complete summary of the test results is given in Table 7.2. The video tapes which were originally recorded on 19 mm wide - 60 minute cassettes were transferred to 12.7 mm wide - long playing tapes of the type used in home television video systems. This facilitated the viewing of the tapes in a fast forward or reverse direction which greatly aided the viewing procedure that was used in the analysis of results. This system was used in conjunction with a small television screen. Direct measurements of deformation were made by placing a transparent sheet on the television screen, and recording all pertinent features of the test with a felt tipped pen. Certain disks at strategic points within the soil mass were marked with a dot in order that their path of movement could be followed throughout a test. Furthermore, at the end of a test, the position of the strips, wall panels, and ground surface were also drawn on the transparency which had been placed over the screen. This process yielded a significant number of displacement vectors in order that the failure plane(s) could be determined. In some tests the failure

Table 7.2 Summary of Friction Table Test Results

Test # (Appendix D)	H (mm)	L (mm)	S (mm)	Mode	Time to Failure (Secs)	Remarks
1	600	275	150	2	35	on video & 35mm; failure criteria - $h=10\%$ $s=10\%$
2	600	275	150	1	40	video & 35mm - same failure criteria
3	600	275	100	1	78	video & 35mm
4	700	275	100	1	55	video - one strip popped up at top
5	400	275	100	1	72	top failure line in wrong place
6	400	275	100	2	78	- failure by settlement, top strip popped out
7	600	275	100	2	67	- settlement failure
8	700	275	100	2	35-settlement(s)- 42-deflection(δ)	- rotational failure
9	450	550	150	1	264	- more area provided behind reinforcing - disks painted black in 2 lines - bars on belt spaced at 5cm - failure - horizontal deflection
10	400	550	100	1	210	- very stable - no real failure
11	400	3-412.5 1-275 (top)	100	RE slope	335	- surcharge on RE slope - 2:1 - no panels (bottom strip popped up) - surcharge increased $\frac{1}{2}$ way thru test

Friction Table Test (Cont'd)

Test # (Appendix D)	H (mm)	L (mm)	S (mm)	Mode	Time to Failure (Secs)	Remarks
12	400	N/A	N/A	2:1 slope	145	- easily failed - same dimensions as above
13	450	275	150	2	s=50 $\delta=64$	
14	750	275	150	2	s=35 $\delta=44$	
15	450	275	150	1	$\delta=64$	- did not fail in settlement
16	750	275	150	1	$\delta=45$	- failure plane visible
17	750	275	75	1	$\delta=71$	- top strip popped up
18	450	275	75	1	$\delta=93$	- did not fail in settlement
19	450	275	75	2	s=60 $\delta=110$	
20	750	275	75	2	s=45 $\delta=58$	
21	450	412.5	150	1	$\delta=61$	- did not fail in settlement
22	750	412.5	150	1	δ & s = 75	- δ & s coincided
23	750	550	150	1	s=255 $\delta=272$	- largest test, very stable, therefore with 550 strips cannot decrease S

Friction Table Test (Cont'd)

Test # (Appendix D)	H (mm)	L (mm)	S (mm)	Mode	Time to Failure (Secs)	Remarks
24	700	550	100	1	s=360 δ =450	- stable, even with strips popping up - #7 at start, #6 towards the end
25	700	550	100	2	s=90 δ =did not fail	- large deformations, did not fail in δ - test stopped after 2 min 40 secs
26	400	550	100	2	test stopped after 308. sec- did not fail (either s or δ)	
27	400	412.5	100	1	δ =189	- did not fail in settlement
28	700	412.5	100	1	s=150 δ =168	- $\bar{W}$ roughened foundation base and grid
29	750	412.5	75	1	s=120 δ =146	- same as above, tip strip popped up
30	450	412.5	75	1	δ =275 did not fail in s	- $\bar{W}$ rough foundation and grid
31	750	412.5	75	2	s=120 δ =288	- top strip popped up at end; rough foundation and grid
32	450	412.5	75	2	s=345 test ended at 480	- did not fail in δ (after 8 min)
33	400	412.5	100	2	s=300 δ =311	- $\bar{W}$ rough foundation and grid

Friction Table Test (Cont'd)

Test # (Appendix D)	H (mm)	L (mm)	S (mm)	Mode	Time to Failure (Secs)	Remarks
34	700	412.5	100	2	s=90 $\delta=214$	- W rough foundation and grid
35	750	412.5	150	2	s=70 $\delta=113$	- W rough foundation and grid excellent rotational failure
36	450	412.5	150	2	δ & s = 150	- rough foundation and grid
37	450	550	150	2	s=240 $\delta=360$	- rough foundation and grid
38	750	550	150	2	s=150 $\delta=371$	- rough foundation and grid

plane(s) could be drawn in objectively by reviewing the test progress in a fast forward speed. Appendix D contains all the figures illustrating the failure planes and relative displacement vectors plotted for each test. Typical test results for Mode 1 and Mode 2 failures are illustrated in Figures 7.20 and 7.21.

An examination of the failure planes indicates that for both modes of failure the failure plane initiates at a shallow angle, θ_1 , within the reinforced block, and then bends up sharply to a much steeper angle, θ_2 , in the backfill behind the reinforced earth material. This phenomena was previously realized by Romstad et al (1978) in a finite element analysis incorporating soil-strip slippage and nonlinear soil behaviour.

The shape of the failure surface within the reinforced earth block is governed by the friction developed between the soil and the reinforcing strips. Therefore the failure planes tend to occur at small horizontal angles within the reinforced soil block, and in some instances the slip surfaces even tend to be parallel to the reinforcing strips. In other words the failure surface tends to follow the path of least resistance with respect to shear. Where the slip surfaces are quasi parallel to the reinforcing strip, the shear resistance along the soil/strip boundary is less than the resistance through the unreinforced soil

TEST 22 $H = 750 \text{ mm}$, $L = 412.5 \text{ mm}$, $S = 150 \text{ mm}$, MODE I

Figure 7.20 Typical Mode I Failure Pattern

TEST 31 $H = 750 \text{ mm}$, $L = 412.5 \text{ mm}$,
 $S = 75 \text{ mm}$, MODE 2

Figure 7.21 Typical Mode 2 Failure Pattern

mass. This explains why the shear surface close to the wall follows approximately the direction of the reinforcing. When the failure surface is beyond the zone of reinforcing it then deviates to the Rankine active failure wedge angle of $45 + \phi/2$ degrees to the horizontal. It is evident that the reinforcing ties lower the angle θ_1 , which should be in the order of 65 degrees for an unreinforced active Rankine failure condition.

The shape of the failure surface can be approximated by two straight intersecting lines. The first line within the reinforced block is at an angle of 14 to 16 degrees from the horizontal, while the second line initiating beyond the reinforcing strip has an angle of inclination of 60 to 65 degrees, corresponding to the slope of a failure plane in an unreinforced soil mass.

Generally, the failure planes for the Mode 2 tests tended to be somewhat steeper than those of Mode 1. However, this is to be expected, since the aspect ratio of height to width (h/w), where h is the total height of the wall and w the total width of the structure, is higher. Nevertheless, the slopes of the failure planes for both test series were remarkably consistent regardless of wall height and spacing of reinforcing.

In almost all cases, the near horizontal portion of the failure plane intersected the wall slightly above

one obtains a comparison between the predicted and experimental value. Appendix E contains the detailed calculation for the above example. Although Romstad et al's method is very tedious, it gives remarkable results as to the comparison of the critical H/L ratio at failure along with the failure wedge angles θ_1 and θ_2 . For this particular example, using the actual H/L ratio and the measured θ_1 and θ_2 values from the model test in conjunction with Romstad et al's theories, one obtains results which are more or less identical.

The results obtained from the base friction study allow a more direct application of the theoretical model. For actual cases, Romstad et al proposed Equation 3.39 to be used by assuming various values of θ_1 and range θ_2 for each chosen θ_1 , seeking a minimum value of H/L . However, the friction table study afforded the opportunity of observing directly the angles θ_1 and θ_2 , and applying them to the expression along with the H/L values; thus verifying the assumptions.

Table 8.1 contains the values of the failure plane angles θ_1 and θ_2 for tests of the Mode 1 configuration on the base friction apparatus. The average values obtained for θ_1 and θ_2 are 15 and 73 degrees respectively. Therefore, using this as a guide, for a given geometry that pertains to a reinforced earth structure one should be

CHAPTER VIII

ANALYSIS OF RESULTS

8.1 Steel Rod Model

8.1.1 Instrumented Tests

The results of reinforced earth investigations, with instrumentation, which were performed on the steel rod apparatus were analysed. The typical response of model reinforced earth walls with regard to the rate and magnitude of strain in the reinforcing strips, and the horizontal deflection (δ) of the wall to the effects of vertical strip loads is illustrated in Figure 8.1. The strip load, q , applied by the model footing is expressed as a ratio of the total ultimate load, q_{ult} . For each increment of vertical load, the strain recorded in each of the four reinforcing strips, expressed as a ratio of the initial (static) strain (S/S_0), and the horizontal deflection of the wall, which is expressed as a percentage of the total wall height (δ/H) were recorded. Figure 8.1 indicates that five percent δ/H and 90 percent of the total vertical load is indicative of failure in the model structure. At this point, the δ/H curve becomes steeper and the S/S_0 curves tend to become horizontal, thus indicating that any further increase in vertical load causes no further increase

Figure 8.1 Effects of Surcharge Load on the Strain in Reinforcing Strips and the Horizontal Yielding of the wall for the Steel Rod Apparatus. Test 4

in the strain in the reinforcing strips. In other words, the strips are merely yielding (pulling out). Furthermore, this phenomena occurs in all of the reinforcing strips, but is more prevalent in strips 3 and 4 which are the bottom-most reinforcing ties. Also, the δ/H curve indicates that at 5 percent horizontal deflection any further increase in load causes an abrupt change in the deflection of the wall.

Figure 8.2 demonstrates the formation of a failure wedge which occurs in the zone of reinforcing and within the backfill "soil" for an instrumented test with a surcharge load. At present there are no known theories in the field of soil mechanics which can effectively be used to analyse this occurrence.

8.1.2 Coulomb Wedge Analysis

A similar example to Figure 8.2 is shown in Figure 8.3. However, failure occurred in this model reinforced earth structure (Test 5) without the application of any surcharge or other external forces. The entire sequence of failure was captured on video film, and the shape of the failure wedge, which occurs entirely within the backfill material, is outlined with tape as shown in the 35 mm picture. Subsequently, an equilibrium analysis was performed using the dimensions of the wedge; assuming that Coulomb conditions prevail. Applying Equation 3.29, which was

Figure 8.2 Failure of Steel Rod Model Test with
Instrumentation and Surcharge Load

Figure 8.3 Shape of Coulomb Failure Wedge in
Steel Rod Model (No Surcharge Load)

proposed by Smith and Brahsby (ref. 97), on the Coulomb failure wedge and substituting $\theta = 28^\circ$ (measured directly off the model); $\phi = 33^\circ$; and the critical ratio, $H/L = 1.27$, to the equilibrium conditions of the model yield a result which does not accurately depict the true critical H/L ratio. Therefore, the theories postulated by Smith and Wroth (1978) were also examined. Using Equation 3.30 and the parameters $\theta = 28^\circ$ and $\phi = 33^\circ$ the height of action Y is found to be 0.153 m from the base. Furthermore, applying this value, along with the value of Q (Equation 3.27) to the condition of equilibrium for the model reinforced earth structure yield an estimate of the critical H/L ratio that differs from the actual H/L value by only 8 percent. The value for the height of action, Y , had to be determined using Equation 3.30, instead of performing a similar analysis using load cell results, as the load cell data for the steel rod model tests was inconclusive due to the high overall height of the model test.

8.1.3 Critical Height Ratios

The results of the analysis of the critical height ratios of the reinforcing parameters for tests conducted on the steel rod model apparatus are shown in Figure 8.4. The normalized failure ratios, L/S and H/S are plotted for both Mode 1 and Mode 2 conditions. In comparing the results,

NORMALIZED FAILURE RATIOS FOR TWO-DIMENSIONAL REINFORCED EARTH SYSTEMS

Figure 8.4

the stability for the two-dimensional reinforced earth model is the same up to H/S values of three, and L/S values of two. A further increase in the height of the structure results in increased stability for Mode 2 types of failures compared with those for the Mode 1 situation. In other words, the Mode 2 configuration is more stable than the Mode 1 setup beyond L/S values of two and H/S values of three.

8.2 Base Friction Apparatus

An examination of the films of the retaining wall tests revealed that the Mode 1 failures can be analysed by the classical wedge analysis, while the Mode 2 failures are best represented by a rotational type of failure. This latter failure pattern was also realized by Hausman and Lee (1978) from experiments using panel walls of several metres in height. Similar observations were made by Al-Hussaini and Perry (1976) using a 3.3 m high reinforced earth wall subjected to surcharge loading. Both cases showed that the walls followed a predominantly rotational deformation instead of a translational one. However, due to the nature of their tests, it is not possible to determine which mode of failure would be prevalent or what the shape of the failure surface would be.

8.2.1 Logarithmic Spiral

The rotational failure observed in the Mode 2 situation is best analysed by assuming that failure takes place along a portion of a logarithmic spiral (section 3.9). This logarithmic slip plane is brought about due to the rotational failure of these types of structures as evidenced by the films taken during the tests. In the logarithmic spiral method the resultant force makes an angle of ϕ with the tangent to the spiral and its line of action passes through the centre of the spiral. The logarithmic spiral which was given in chapter three as

$$r = r_0 \exp(\theta \tan \phi) \quad (3.48)$$

can be fitted as closely as possible to the observed failure plane. Test 35 is used here as a typical example. The log spiral extends to a point where the slope of the spiral makes an angle of $45 + \phi/2$ (Figure 8.5). After determining the centroid of the composite failure area, the active earth pressure, determined using Rankine's earth pressure theory, and the weight of the sliding wedge can be calculated. From these two forces acting through the centroid, the resultant to the weight of the sliding soil mass and the active earth pressure with respect to the straight portion of the rupture surface can be determined. The wall is stable if the resultant R passes along a line of action

$H = 750 \text{ mm}$

$L = 412.5 \text{ mm}$

$S = 150 \text{ mm}$

Figure 8.5 TYPICAL FAILURE OF
MODE 2 STRUCTURE

that falls to the left of the centre of the logarithmic spiral (Broms, 1978). As can be seen in Figure 8.5, the extension of the resultant passing through the centre of gravity is slightly to the left of the centre of the logarithmic spiral and therefore the structure is statically stable. Again it should be kept in mind that in the base friction apparatus all structures will eventually fail if given sufficient time and applying the proper belt speed. Once a structure has failed and the failure mode has been determined, it can be analysed statically.

8.2.2 Wedge Analysis

A typical wedge shaped failure block for Mode 1 wall for Test 16 is shown in Figure 8.6. Here, the failure plane is initiated in the reinforced earth block at an angle of 24 degrees from the horizontal and extends to a point just past the reinforcing where this failure surface then changes to a slope angle of about 83 degrees from the horizontal. An analysis of such a failure pattern was carried out by Romstad et al (1978) based on their finite element results as discussed in detail in section 3.8. The analysis proceeds by dividing the area into a component rectangle and two triangles and determining the maximum resultant normal and tangential shear forces at collapse. The assumption that the curved failure plane can be replaced

TEST 16

H = 750

L = 275

S = 150

MODE 1

Figure 8.6

Typical Failure of

Mode 1 Structure

by two straight lines will result in a slight difference in the angles of slopes between the actual and assumed failure surfaces. However, this should not affect the results significantly, since in both cases the total length of the failure plane, which will greatly influence the stability of the structure, were similar. In this instance, the difference in length between the observed failure plane and that which was used in the theoretical analysis is approximately 3 percent. Using Test 16 as a typical example, and applying Romstad et al's analysis to the observed geometry using

$$\begin{aligned}
 & [(\sin \theta_2 - \cos \theta_2 \tan \phi) \cot \theta_2 - rK(\sin \theta_1 - \sin \theta_2 - (\cos \theta_1 - \cos \theta_2) \\
 & \tan \phi) + K(\cos \theta_1 + \sin \theta_1 \tan \phi - (\cos \theta_2 + \sin \theta_2 \tan \phi))](H/L - \tan \theta_1)^2 \\
 & + 2(\sin \theta_1 - \cos \theta_1 \tan \phi)(H/L - \tan \theta_1) + (\sin \theta_1 - \cos \theta_1 \tan \phi) \tan \theta_1 \\
 & - \frac{2\sigma_{Ti} \max}{\gamma L^2} (\sin \theta_1 \tan \phi + \cos \theta_1) - \frac{2}{\gamma L^2} (c_1 L_1 + c_2 L_3) = 0 \quad (3.39)
 \end{aligned}$$

for the values of

$$\theta_1 = 24^\circ$$

$$\theta_2 = 72^\circ$$

$$\phi = 41^\circ$$

$$K = \frac{1 - \sin 41}{1 + \sin 41} = 0.208$$

$$c_1, c_2 = 0 \text{ for cohesionless soil}$$

one obtains a comparison between the predicted and experimental value. Appendix E contains the detailed calculation for the above example. Although Romstad et al's method is very tedious, it gives remarkable results as to the comparison of the critical H/L ratio at failure along with the failure wedge angles θ_1 and θ_2 . For this particular example, using the actual H/L ratio and the measured θ_1 and θ_2 values from the model test in conjunction with Romstad et al's theories, one obtains results which are more or less identical.

The results obtained from the base friction study allow a more direct application of the theoretical model. For actual cases, Romstad et al proposed Equation 3.39 to be used by assuming various values of θ_1 and range θ_2 for each chosen θ_1 seeking a minimum value of H/L . However, the friction table study afforded the opportunity of observing directly the angles θ_1 and θ_2 , and applying them to the expression along with the H/L values; thus verifying the assumptions.

Table 8.1 contains the values of the failure plane angles θ_1 and θ_2 for tests of the Mode 1 configuration on the base friction apparatus. The average values obtained for θ_1 and θ_2 are 15 and 73 degrees respectively. Therefore, using this as a guide, for a given geometry that pertains to a reinforced earth structure one should be

Table 8.1 Values of Failure Plane Angles
 θ_1 and θ_2 for Base Friction Tests,
 Model

Test #	θ_1 (degrees from horiz.)	θ_2 (degrees from horiz.)
2	19	52
3	19	63
4	13	55
5	12	54
9	27	51
10*	45	65
15	12	72
16	24	83
17	10	88
18	10	84
22	12	87
23	10	80
24	7	68
27	9	80
28	8	87
29	8	80
30	9	83

* Failure due to pullout of reinforcing (no definite FP)

able to estimate the shape of the failure surface and hence calculate the factor of safety for the structure based upon the wedge theory.

8.2.3 Effects of Variations in Retaining Wall Geometry

An analyses of the results of the model reinforced earth investigations which were conducted using the base friction apparatus show several trends. Figure 8.7 is a graph of the normalized ratios L/S (Length/Spacing) of the reinforcing strips plotted against time-to-failure. Time-to-failure was considered to be an indication of the stability of a wall. In the base friction apparatus all structures can eventually be made to fail by increasing the belt speed and allowing sufficient time. In other words, for a given belt speed, the amount of deformation to failure is time dependent and, therefore, the more stable an arrangement, the longer is the time required to failure.

The tests using the base friction apparatus were divided into a series of low and of high walls for both modes of failure. Low walls are those that are less than 500 mm and the high walls are greater than 500 mm in height. Figure 8.7 shows that the Mode 2 configuration is slightly more stable than the Mode 1 walls. That is, for a given time to failure and a given strip spacing the Mode 2 wall will need shorter strips for the same stability. The

FIGURE 8.7 FAILURE RATIOS VS TIME
FOR FRICTION TABLE TESTS

greatest difference in L/S ratios is between the Mode 2 short and the Mode 1 high walls. Generally all curves become more or less horizontal after a failure time greater than 100 seconds, corresponding to a mean L/S ratio of about four. This means that for L/S ratios greater than four, the structures are relatively stable, and if below that value ~~the structures~~, can be considered unstable. This criterion for stability was also checked for the steel rod tests.

8.3 Comparison of Test Results

A direct comparison of the shape of the failure surface, as determined by the steel rod model and the base friction apparatus, for the Mode 2 configuration was possible in several instances. For base friction Tests 7, 8, 13 and 35, the height of the structure was identical to the critical height for those tests performed on the steel rod apparatus. Since the length and spacing of the ties were also the same, by superimposing a projection of a 35 mm slide showing the failure occurring in the steel rod model onto a drawing of the failure pattern in the base friction tests, a direct comparison of the failure shape could be made. The results of this comparison are shown in Figures 8.8, 8.9, 8.10 and 8.11. It can be seen that the shape of the failure surface determined in these tests

Figure 8.8 Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2.

TEST 7

$H = 600$ mm

$L = 275$ mm

$S = 100$ mm

MODE 2

Figure 8.9 Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2.

TEST 8

$H = 700 \text{ mm}$

$L = 275 \text{ mm}$

$S = 100 \text{ mm}$

MODE 2

Figure 8.10 Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2.

TEST 13

H = 450 MM, L = 275 MM,

S = 150 MM, MODE 2

TEST 35 H = 750 mm, L = 412.5 mm,
S_z = 150 mm, MODE 2

Figure 8.11 Comparison of the shape of the failure surface between the Steel Rod Model and the Base Friction Apparatus for Mode 2.

compares quite favourably. Such a comparison was not possible for the Mode 1 situation, since the critical height at failure on the steel rod model tests did not exactly coincide with the height of structures which were tested on the base friction apparatus. However, Figures 7.7, 7.8, 7.10 and 7.11 clearly illustrate the shape of the failure surface for the Mode 1 situation determined by the steel rod apparatus. These results are excellent comparisons with the failure planes in the base friction tests, and in the theoretical analysis proposed by Romstad et al (1978).

Another occurrence at failure which is common to the results obtained from both apparatus is that the bottom wall panel undergoes a rotation. This is due to the failure surface which intersects the wall part-way up the bottom panel. Examples of this phenomena occurring in both the steel rod apparatus and in the base friction model can be seen in Figures 7.13, 7.14, 7.17 and 7.19.

Finally, a comparison of the L/S ratios for both models reveals that the separation between Mode 1 and Mode 2 structures in terms of overall stability occurs at L/S values between two and three.

S

CHAPTER IX

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

An extensive review of literature pertaining to the behaviour of reinforced earth structures showed that there was a paucity of information regarding the shape of the failure surface in these structures. Furthermore, little information was available concerning the various modes of failure that these structures would undergo. Such information is of paramount importance in order to safely and economically design reinforced earth retaining walls. In this study, two purely two-dimensional physical models were utilized to determine the shape of the failure planes in model reinforced earth structures having varied wall geometries. In addition, the investigation also includes the recognition of two of the modes of failure that are possible in reinforced earth systems.

An analysis of the results and comparisons with existing theories were presented and discussed. Furthermore, a comparison of the results obtained from the two physical models used in this study were also examined.

9.2 Conclusions

Based upon the results obtained from this study,

the following conclusions can be drawn:

(1) The two-dimensional physical models used in this study were capable of depicting the shapes and modes of failure of model reinforced earth structures.

(2) The shape of the failure plane which occurs in reinforced earth retaining structures is greatly different from that which the Rankine theory predicts. Therefore, using the design criteria of other researchers who assume that the Rankine failure plane has the same shape as an unreinforced soil mass may lead to difficulties.

(3) Only the steel rod model can be used to predict the critical H/L ratio for reinforced earth walls.

(4) The theories proposed by Romstad et al (1978) and by Smith and Wroth (1978) can be used successfully to analyse the stability of Mode 1 type structures. These are particularly applicable to model studies because the critical H/L ratios vary with wall height and with the unit weight of the soil. Furthermore, there is less empiricism associated with these analyses, and they are more flexible and allow for greater variations in the shape of the failure surface.

(5) The logarithmic spiral method is applicable to predict the stability for the Mode 2 type structures.

(6) Theories proposed by Lee, Al-Hussaini and Perry, etc. are not suitable for predicting the critical

height of model structures. Their theories regarding the stability of reinforced soil structures are independent of structure heights.

(7) Theories which stipulate that the stability of reinforced earth structures is governed by the length of the reinforcing tie which extends beyond the theoretical failure plane are somewhat in doubt. It has been demonstrated that it is possible to construct walls in which the failure plane will extend into the soil behind the reinforced zone. Therefore, the stability of these structures will be governed more by the total length of this failure surface.

(8) The base friction principle is suitable for predicting the behaviour of cohesionless soil masses.

(9) The results obtained from tests using the steel rod model are reproducible on the base friction apparatus.

A

9.3 Recommendations for Future Work

This study has attempted to investigate the various modes of failure and the shape of the failure surfaces in reinforced earth retaining structures. To date, only a limited number of experiments have been performed using two-dimensional apparatus to investigate the behaviour of reinforced earth structures; and further research is necessary to clearly establish the trends observed in this

study. Based on the test results, it is recommended that this initial program of research be continued along these lines, with the following suggestions for future study in this area:

(1) The use of the steel rod apparatus and the base friction model to examine the effects on the shape of the failure plane when the reinforced earth strips fail by breaking.

(2) The effects on the failure plane when surcharge loads are applied to the top of the retaining wall.

(3) That the base friction apparatus be used with a highly compressible substance in the foundation to simulate large settlements in reinforced earth structures.

(4) The effects on the stability of model reinforced earth structures by inclining of the tensile reinforcing strips instead of using them horizontally.

(5) Further verification of the work by Romstad et al (1978) using large three-dimensional test apparatus, such as at the University of Ottawa, to more accurately simulate field conditions. Also, the incorporation of Romstad et al's theories on a computer program to facilitate the rapid calculation of H/L ratios by varying the failure plane angles θ_1 and θ_2 , and

(6) Tests to corroborate the ratios of wall height, and the length and spacing of reinforcing strips in actual field conditions.

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APPENDIX A

SUMMARY OF RESEARCH CONCERNED WITH REINFORCED EARTH

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Chang, Hannon & Forsyth (ref. 22)	F, L	sand, gravel, decomposed granite	flexible, & rigid	box 45.7 x 91.4 x 137.2 cm. & actual RE wall	pull out	steel strips & bar mesh	Bar mesh has 6 times pull out resistance of steel strips
McKittrick (1979) (ref. 73)	T, F	N/A	N/A	N/A	pull out	smooth & ribbed steel strips	some design criteria
McKittrick (1979) (ref. 74)	T	N/A	variable	N/A	N/A	N/A	technical considerations
Lee, Adams & Vagneron (1973)	T, L	Ottawa No. 90 sand	rigid	76 x 120 x 61 cm. wooden box	retaining wall collapses	aluminum foil strips	major discussion on design F.P. determinations
Murray, Carder & Krawczyk (1979)		uniformly graded sand ($D_{60}/D_{10} = 2.3$)	rigid	3m high wall	pull out tests under both static & dynamic ic-loading	steel & fibre reinforced plastic strips	sand fill also subjected to vibration

* L = laboratory F = field T = theoretical

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Kennedy, Laba & Mossaad (1980)	T, L	coarse, dry Erie sand $C_u = 3.7$	rigid	plexiglass box 56 x 76 x 52.5 cm.	R.E. retaining wall subjected to strip loads	steel strips	estimate of force distribution transmitted to wall facing
Smith (1977) (Ref. 100)	T	N/A	N/A	N/A	N/A	N/A	basic theoretical approach
Bolton, Choudhury & Pang (1977) (ref. 10)	L, T	well graded coarse to fine glacial sand	rigid	20 cm. high walls x 40 cm. long	centrifugal modelling	steel welding rod, aluminum foil, & stainless steel strips	some limit state discussion
Jones (1977) (ref. 55)	T	N/A	N/A	N/A	N/A	N/A	economic considerations
Murray (1977)	T, L	not given	not given	not given	small scale static & centrifuge testing	not given	use of limit equilibrium analysis
Andrawes & McGown (1977)	L	Leighton Buzzard sand $C_u = 1.13$	rigid	unit cell 15.2 x 10.2 x 10.2 cm. & box 61 x 26 x 7.6 cm.	plane strain & compression & footing tests	steel, aluminum strips & fabric	measurement of displacement vectors

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Smith & Wroth (1977)	L	dry Leighton Buzzard sand	rigid	20 cm. wide glass-walled tank	wall failure tests	aluminum foil	F.P. determination by radiographs
Boden, Irwin & Pocock (1977)	F, L	dry sand & clay		1m x 2m x 1m	static measurements	fibre reinforced plastic stain-instrumented less steel, plastics, plastic coated steel & concrete	tests very well
Finlay (1977)	F	burnt oil shale - "blaes"	variable	106m long x 1.8 - 7.2m high	full scale	stainless steel strips 6.5m long	meas. of vert. & horiz. stresses & movements
Mallinder (1977)	L, F	not given	variable	4m x 12m x 7m long wall	- accelerated long term strength - biological degradation - model wall tests	fibre reinforced forced plastics (Fibretain)	strain gauged strips
King (1977)	T	N/A	N/A	N/A	N/A	metal	corrosion of R E strips

Summary of Research Con'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Lee & Edwards (1977)	T	N/A	N/A	N/A	N/A	static steels	corrosion related
McKeand & Sissons (1977)	T	N/A	N/A	N/A	tensile strength punching etc.	textiles	
Cross (1977)	F	heavy clay (aggressive) uniformly graded sand (non-aggressive)	variable	3m high	corrosion	steel & aluminum	corrosion related
Bell, Stilley & Vandre (1975)	L, F	fine quartz sand & clean sand	rigid	60.96 x 91.44 x 45.72 cm. high box & 3.0m x 10.7m long	surcharge tests to failure	fabric	Rankine analysis used
Sharma & Wandschneider (1980)	F	fine granular material	variable	10.6m high x 260m long	actual retaining wall	steel strips	large settlements

Summary of Research Conf'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Romstad, Herrmann & Shen (1974)	T	N/A	N/A	N/A	N/A	N/A	orthotropic & nonlinear elastic analysis
Naylor & Richards (1973)	T	N/A	rigid	10m high x 15m deep	theoretical assessment	N/A	slipping is significant
Lee (1978) (ref. 65)	T	N/A	N/A	N/A	N/A	N/A	describes areas requiring further study
Holtz (1978)	T	N/A	N/A	N/A	N/A	N/A	general applications
Al-Hussaini & Johnson (1972)	F, T	concrete sand	variable	3.66m high wall	uniform sur-charge loading to failure	galvanized steel	finite element analysis
Al-Hussaini & Perry (1972) (ref. 3)	F	clean concrete sand ($C_u = 2.2$)	variable	wall, 3.66m high x 4.88m long x 3.1m wide	uniform sur-charge loading to failure	galvanized steel ties	strain gauge instrumented ties

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Bacot, Ilfis, Lareal, Paumier & Sanglerat (1978)	L, F	sand, C $\approx$ 0	rigid & variable	50m x 20m fill	-shear box -pull out tests -active pressure apparatus	-steel strips -aluminum foil	determination of soil-reinforcement friction coeff
Baquetin (1972)	F	--	variable	full scale	performance of actual structures	--	
Bassett & Last (1972)	L	dry Leighton Buzzard sand	rigid	91.44cm x 91.44cm x 2.03cm wide	simple shear apparatus & test tank	steel rods & drinking straws	bearing capacity consideration
Binquet (1972)	T	dry Ottawa No. 90 sand	rigid	76 x 120 x 61 cm. wooden box	comparison of theory with test data	aluminum foil	uses Rankine Theory
Bolton, Choudhury & Pang (1972) (ref. 19)	L	dry sand	flexible & rigid	20cm. high x 40 cm. x 15 - 20cm. deep	centrifugal model	steel & copper coated steel & aluminum foil	
Broms (1972)	T	N/A	N/A	N/A	N/A	fabric	design methods for fabric reinforced walls

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Darbin, Mailloux & Montuelle (1972)	T, F	shales, clays, sands	N/A	N/A	rate of corrosion	steels	corrosion of R.E. strips
Dash (1973) (ref. 34.)	F	sand & gravel	variable	wall, 260m long x 3 - 13.8m high	actual R.E. wall	steel	use of parabolic shaped failure plane
Forsyth (1972)	L, F	clean fills	rigid & variable	122m long x 2 tiers 6.1m high & 91.5m long x 16.8m high	pull out resistance & actual R.E. walls	rebar mats, steel straps, solid plates & car tires	study of mat arrangement
Hausmann & Lee (1972)	L	medium to fine quartz sand	rigid	76cm. wide x 61cm. high x 76cm. deep plywood	stability tests	mylar strips	rigid as opposed to flexible wall rotating about its base
Herrmann (1972)	T	N/A	rigid	2.4m high x 2.4 deep hypothetical R E wall	N/A	N/A	2 dimensional F.E. analysis

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Hoshiya (1972)	L	dry sand	rigid	30cm. x 50cm. x 41.5 cm. box with glass sides	strength, pull out & deformation	brass strips	follow Lee's principles
Iwasaki & Watanabe (1972)	F	sand	variable	30cm. high x 50cm. wide embankment & 3m high x 5m wide embankment	withdrawal, vibration & artificial rainfall tests & corrosion	nets, metal strips	reinforced railway embankment
Juran & Schlosser (1978)	T, L	dry Fontainebleau sand ($C_u = 1.87$)	rigid	73cm. high plexiglass tank	critical height	aluminum strips	various failure modes
Juran, Schlosser, Long & Legeay (1978)	F	red schist	variable	5.6m high x 15 m long	actual bridge abutment	stainless steel	well instrumented
King & Nabilzadeh (1972)	F	cohesive & non-cohesive soils	N/A	--	corrosion test	steels	

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
McKittrick (1978) (ref. 71)	--	--	--	--	-- pull out re- sistance -- corrosion	smooth & ribbed steels	fill material criteria
Naylor (1978)	T	cohesionless	rigid & soft	10m high x 8m deep hypothetical R E wall	strip slip analysis	strips	finite element study
Rea & Mitchell (1978)	L	Monterey No. 0 quartz sand (fine) ($C_u = 1.45$)	flexible - rigid	box 0.915m square	circular plate loading	paper grid cells	bearing capacity & cyclic loading
Richardson (1978)	T	--	--	--	harmonic shake table tests	--	earthquake re- sistance & design
Romstad, Al- Yassin, Herr- mann & Shen (1978)	T	N/A	N/A	N/A	N/A	N/A	predicted failure heights & F.P. by F.E. analysis
Salomone, Holtz & Kovacs (1978)	T	N/A	N/A	N/A	N/A	N/A	prediction of tensile forces in reinforcement

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Schlosser & Elias (1972)	F, L	Fontainebleau sand & glass balls powdered clay	variable & rigid	70cm. high x 250 cm. wide	pull out tests & actual R E structures	steel strips (ribbed & smooth)	mobilization & magnitude of friction
Smith & Wroth (1972)	L	dry Leighton Buzzard sand	rigid	glass tank 83cm. x 50 cm. x 20 cm. wide	critical height to failure	aluminum foil	radiographs for determining failure plane
Wolfe, Lee, Rea & Yourman (1972)	L	clean, fine sand	rigid	0.6m high x 6.1m square shaft, stable	vertical acceleration tests	smooth mylar strips & fibre-glass screen strips	vertical seismic motions on R E walls
McKittrick (1972) (ref. 72)	T, F	--	variable	--	pull out, shear tests, corrosion etc.	mostly steel, ribbed & smooth	discussion of basic theory & overview of actual structures
Verma & Char (1972)	L	Ennore sand	figid	--	triaxial tests on reinforced sand	plates, fibers, mild steel & aluminum	

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Saran, Talwar & Dash (1978)	L	Ranipur sand ($C_u = 1.7$)	rigid	standard triaxial & box 80cm. x 40cm. x 8cm. wide	triaxial compression & plane strain footing tests	aluminum foil rings & strips	load-settlement characteristics of R E below footing
Al-Hussaini & Perry (1978) (ref. L)	F, T	concrete sand ($C_u = 2.0$)	variable	wall 4.88m long x 3.05m wide x 3.66m high	field measurements	nylon-coated fabric strips	finite element analysis comparison with field results
Soydemir & Espinosa (1978)	L	sand ($C_u = 1.68$) & Ottawa sand ($C_u = 1.45$)	N/A	60mm x 60 mm	direct shear tests	aluminum foil	comp. of Rankine & Coulomb theories
Boyd, Verge & Reid (1978)	T	N/A	N/A	N/A	N/A	N/A	discussion of impt. facets of R E
Leece (1978)	F	sand	variable	3,453 m ² wall	actual R E structure	steel strips	practical assessment of R E system

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Dash (1978) (ref. 35)	F, L	Tartown sand & gravel	variable	260m long x 3.05 to 7.6m high	R E wall with instrumentation & pull out testing	metal strips	meas. of peak & residual pull out resistance
Kawakami & Abe (1978)	L	Sai-River sand	rigid	wall 1m high x 1.2m long	vertical load- ing tests	synthetic resin net	concrete block retaining walls
Holtz & Broms (1978)	L	cylindrical aluminum rods, 4mm & 6mm in diameter x 10mm long	rigid	44cm. x 39cm x 10cm. 2-dimensional model	wall behaviour measurements	polyester fabric	-rigid wall -sand glued to the rods
Habibagahi & Fouladakhsh (1978)	L	Ottawa sand	rigid	box, 91.4cm. wide x 76.2cm. high x 101.6cm. long	wall behaviour measurements	mats of poly- thylene tubing & galvanized steel	rigid wall hinged at the bottom
Hausmann (1978)	L	Ottawa sand (med.-fine)	rigid	76cm. square x 61cm. deep box	static and dynamic tests	nylon strips	rigid rotating wall

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Fang (1978)	L	silty clay & clean sand ($C_u = 3.8$)	rigid	38.1cm. x 45.7cm. x 7.6 - 10.2cm. high	seismic loading	bamboo strips or mats both horiz. & vert.	embankment slopes of 1:1 to 1:5
Walter (1978)	L	3 soils, with varying moisture content & densities	rigid	steel box, 41.2cm. long x 27.8cm. deep	uniform sur-charge load with pull out tests	steel, ribbed & smooth	comparison of ribbed & smooth strips
Brown & Poulos (1978)	T	granular	rigid	--	plane strain analysis	N/A	finite element analysis of reinforced embankments
Popescu (1979)	T	N/A	rigid	N/A	N/A		internal & external stability
Binquet & Lee (1975)	L, T	Ottawa No. 90 sand ($C_u = 1.5$)	rigid, flexible & compressible	flex-box, 150cm. long x 51cm. wide x 33cm. deep	bearing capacity using strip footing loaded vertically	aluminum foil strips	increase in strength of 2 - 4 times

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Smith & Bransby	L, T	dry Leighton Buzzard sand	rigid	20cm. wide x 86cm. long x 50 cm. high glass walled tank	critical height	aluminum foil	failure by overturning - used radio-graphs
Akinmuisuru & Akinbolade (1961)	L	dry sand	rigid	wooden box 1.0m square x 0.7m deep	bearing capacity	rope fibre strips (iko)	square footing
Banerjee (1975)	T	N/A	rigid	N/A	N/A	N/A	complete design analysis
Chang, Forsyth & Beaton (1974)	F	granular fill	variable	max ht. = 16.8m x 161m long	actual R E structure	steel strips	comparison of field instrumentation & analytical results
Schlösser & Long (1974)	T	N/A	N/A	N/A	actual, R E structures	N/A	comparison of theory with field results

Summary of Research Cont'd

Article	Type*	Material	Foundation	Geometry	Nature of Test	Type of Reinforcing	Remarks
Price (1975)	T	N/A	N/A	N/A	N/A	N/A	complete design analysis
Yoshimi & Kishida (1971)	L	dry sand	N/A	29.8cm. diameter	friction between sand & strips	steel, brass & aluminum	ring torsion apparatus
Brown & Poulos (1971)	T	N/A	rigid & compressible	N/A	bearing capacity	N/A	finite element analysis
Narain, Saran & Talwar (1971)	L	poorly graded fine sand ($C_u \approx 1.85$)	rigid	wall, 1.5m high x 2.78m long x 2.0m wide	pull out of R E strips to failure	steel strips	analysis of failure due to slippage
Schlosser & Juran (1979)	T	N/A	N/A	N/A	N/A	N/A	friction & failure modes
Felio (1971)	L	fine quartz sand	rigid	3.65m x 1.84m x 1.84m	pull out resistance	steel strips & steel mesh	meas. of stresses on strips
Tinubu (1976)	L	fine sand	rigid	--	penetration resistance & triaxial compression tests	textile fabric	

APPENDIX B

DESIGN AND CONSTRUCTION OF THE BASE FRICTION APPARATUS

The design and construction of the base friction table was carried out without the aid of any previously made design drawings or helpful descriptive notes. Therefore, this brief description should assist any person wishing to construct such a device.

Due to the time involved in setting up and in performing these tests, the position of the apparatus should be such that it does not involve unnecessary hardships on the person carrying out the experiment. Therefore, the apparatus was designed in order that the model area (belt) was the same height as a standard table top.

The legs of the base friction table were made of 51 mm square tube aluminum sections. A 26 mm thick base was welded to the bottom of each leg. This was drilled and tapped so that tiltable leveling pads could be screwed into the bottom of each leg, allowing the table to be leveled irregardless of the roughness of the floor.

The frame was made of aluminum angles, 51 mm x 51 mm x 4.8 mm thick and 1.6 m long for the width of the table and 1.0 m long for the depth. These were connected

together by welding a 12.7 mm thick facing plate on the end of one of the sections, drilling and tapping it to accommodate four bolts, then drilling corresponding holes in the legs and through the side of the other angle section for the bolts. This meant that complete dismantling was possible with the removal of just a few bolts. This facilitated the task of the removal of the rollers, which was necessary when the belt had to be changed. It also meant that the apparatus could be easily transported.

The surface of the table was made of 19 mm thick plywood with a 1.6 mm thick sheet of galvanized steel glued to the top with epoxy cement. This ensured an absolutely smooth and level surface for the belt to run over. When it was positioned on top of the table, the surface of the galvanized plate was at an elevation slightly above that of the top of the rollers, to ensure that there was no sagging of the belt.

The belt was custom made by the 3M Corporation. It was 1.32 m wide and 2.1 m in circumference, and consisted of No. 220 grit emery on a resin reinforced cloth base. This material is unaffected by liquids, so that modelling of cohesive (moist) soil is possible.

The rollers were made from 102 mm O.D. steel tubing having a wall thickness of 3.2 mm. These were designed to fit inside the working area of the table, and

provide both drive and tension for the belt. The front roller provided the drive, and thus was fitted with "collar and bushing" type friction bearings. A 12.7 mm steel shaft long enough to span the entire width of the table and protrude sufficiently at one end was inserted so that a driving gear could be fitted to it. A special collar at the end of the roller was fitted with a set screw that was clamped to the drive shaft.

The rear roller was also responsible for tensioning the belt. It was therefore, fitted into a machined slot in the flanges of the frame and allowed the whole roller assembly to move independently. The tensioning devices were made from large steel washers with a large enough piece cut out of them that they could simply be hooked over the axle at each end of the roller. A length of threaded rod was welded to each washer and inserted through the back of the frame. Thus, when a nut was tightened on the rod the whole roller could be pulled towards the back frame, with each end moving independently. The bearings for this roller were commercially bought roller bearings.

Crowning of the rollers was not necessary since belt wandering was minimal, especially at low speeds. However, as a precaution the ends of the rollers were fitted with a special cap that had a larger outside

diameter than the rollers. This acted as a barrier to prevent the belt from slipping off the end of the rollers. Torque was effectively transmitted from the rollers to the belt and slippage did not occur so that no frictional material was required on the surface of the rollers.

The drive was supplied by an electric motor rated at $1\frac{1}{3}$ Hp and 1725 R.P.M. This was connected to two sets of reduction gears (50:1 and 10:1 ratio); and a variable speed control. A chain was connected to the sprocket (14 teeth) on the 10:1 reduction gearbox, and to the drive roller sprocket (36 teeth).. With this setup, the belt speed could be controlled from 0.068 R.P.M. up to several hundred R.P.M. simply by turning a control knob. The motor was fastened to a separate frame which rested on the floor. This system was fully adjustable and ensured that no vibrations would be transferred from the motor to the table.

Although it was not found necessary for this particular study, the table was designed to be tilted, so that the actual gravity component could be added or subtracted to the base friction component (Baumgartner, 1980). This was accomplished by adding two adjustable screw-type jacks fastened to the end of the table that allowed the apparatus to be tilted to an angle of about 30° .

With the exception of the construction of the

rollers and bearings, where machined tolerances were small, the work and construction of all the equipment was carried out by the author. Figure B.1 shows the base friction apparatus in its working position.

Figure B.1 Base Friction Apparatus

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APPENDIX C

INSTRUMENTATION SPECIFICATIONS

(A) Load Cell Specifications

Interface, Inc.,
7401 East Butherus Drive,
Scottsdale, Arizona

Model 1210-AF Precision Universal Load Cell

Load Range	10,000 lb ($\pm 44,500N$)
Input resistance	350.2 ohms
Output resistance	350.2 ohms
Recommended excitation	10 VDC or VAC
Maximum excitation	20 VDC or VAC
Maximum error band, computed on a best straight line through zero basis including non-linearity hysteresis and repeatability	$\pm 0.015\%$ FS (full scale)
Compensated temperature range	$-10^{\circ}C$ to $46^{\circ}C$
Thermal zero shift	0.0002% FS/ $^{\circ}C$
Zero Balance	0.18% FS
Tension Output	4.196 MV/V

(B) Strain Gauge Specifications

Micro-Measurements
Romulus, Michigan

Gauge Type EA-06-250BG-120, Lot No. R-A38AD146
Item No. S9158(R.S.)

Resistance	120 ohms $\pm 0.15\%$
Gauge factor @ $24^{\circ}C$	$2.03 \pm 0.5\%$
K_t	$\pm 0.2\%$
Temperature range	$-75^{\circ}C$ to $175^{\circ}C$
Strain limits	$\approx 5\%$ of gauge length

(C) Digital Strain Indicator

Vishay/Ellis - 20,

APPENDIX D

FAILURE PLANE DETERMINATIONS FOR BASE FRICTION TESTS

TEST 1

H= 600 mm, L=275 mm,

S= 150 mm, MODE 2

TEST 2

H = 600 MM, L = 275 MM

S = 150 MM, MODE 1

TEST 3

H = 600 mm, L = 275 mm

S = 100 mm, MODE 1

TEST 4

H = 700 mm, L = 275 mm

S = 100 mm, MODE 1

TEST 5

H = 400 MM, L = 275 MM

S = 100 MM, MODE 1

TEST 6

H = 400 mm

L = 275 mm

S = 100 mm

MODE 2

TEST 7

H = 600 MM

L = 275 MM

S = 100 MM

MODE 2

TEST 8

H = 700 mm

L = 275 mm

S = 100 mm

MODE 2

TEST 9 H = 450 mm, L = 550 mm, S = 150 mm, NODE 1

TEST-10 $H = 400 \text{ mm}$, $L = 550 \text{ mm}$, $S = 100 \text{ mm}$, MODE 1

TEST 11 H = 400 MM, L = 412,5 (BOTTOM 3), S = 100 MM,
275 (TOP)

SURCHARGE ON 2:1 REINFORCED SLOPE (NO WALL)

V

TEST 12 H = 400 mm, SURCHARGE ON 2:1 SLOPE (NO REINFORCING)

TEST 13 H = 450 MM, L = 275 MM,

S = 150 MM, MODE 2

TEST 14

$H = 750$ mm; $L = 275$ mm,

$S = 150$ mm, MODE 2

TEST 15, H = 450 MM, L = 275 MM,
S = 150 MM, MODE 1

TEST 16

H = 750 MM

L = 275 MM

S = 150 MM

MODE 1

TEST 17

H = 750 MM

L = 275 MM

S = 75 MM

MODE 1

TEST 18 H = 450 MM, L = 275 MM,
S = 75 MM, NODE 1

TEST 19 H = 450 mm, L = 275 mm;

S = 75 mm, MODE 2

TEST 20

H = 750 mm, L = 275 mm,

S = 75 mm, MODE 2

TEST 21 H = 450 MM, L = 412.5 MM, S = 150 MM, MODE 1

TEST WAS NOT VISIBLE

TEST 22 $H = 750 \text{ mm}$, $L = 412.5 \text{ mm}$, $S = 150 \text{ mm}$, MODE 1

TEST 23

H = 750 MM

L = 550 MM

S = 150 MM

MODE 1

FP

TEST 24

H = 700 MM

L = 550 MM

S = 100 MM

MODE 1

FP

JESI 25

H = 700 MM

L = 550 MM

S = 100 MM

MODE 2

TEST 26 H = 400 MM, L = 550 MM
S = 100 MM, MODE 2

TEST 27

H = 400 mm, L = 412,5 mm,

S = 100 MM, MODE 1

IESI 28

H = 700 mm

L = 412,5 mm

S = 100 mm

MODE 1

FP

TEST 29

H = 750 MM

L = 412.5 MM

S = 75 MM

MODE 1

FP

TEST 30 H = 450 MM, L = 412.5 MM, S = 75 MM, MODE 1

TEST 31

H = 750 mm, L = 412.5 mm,

S = 75 mm, MODE 2

TEST 32 H = 450 MM, L = 412,5 MM,

S = 75 MM, MODE 2

TEST 33

H = 400 MM, L = 412.5 MM,

S = 100 MM, MODE 2

TEST 34

H = 700 mm, L = 412.5 mm

S = 100 mm, MODE 2

TEST 35 $H = 750 \text{ mm}$, $L = 412.5 \text{ mm}$,
 $S = 150 \text{ mm}$, MODE 2

END

0|8|0|6.8|3

FIN

TEST 3Z

H = 450 mm, L = 550 mm

S = 150 mm, MODE 2

TEST 38 . H = 750 mm, L = 550 mm,
S = 150 mm, MODE 2

APPENDIX E

WEDGE ANALYSIS FOR BASE FRICTION TEST

Calculation of H/L ratio for Test 16

$$\theta_1 = 24^\circ$$

$$\theta_2 = 72^\circ$$

$$\phi = 41^\circ$$

$$K = 0.208$$

$$r = \frac{\cot \theta_2}{K} + \frac{\cos \theta_2 + \sin \theta_2 \tan \phi}{\sin \theta_2 - \cos \theta_2 \tan \phi}$$

$$= \frac{\cot 72}{0.208} + \frac{\cos 72 + \sin 72 \tan 41}{\sin 72 - \cos 72 \tan 41} = 3.22$$

$$\frac{a_1 x_n}{\gamma L^2} = C_1 \frac{H}{L} \tan \theta_1 = 2.98 \frac{750}{275} \tan 24 = 3.62$$

$$\frac{a_2}{a_1} = C_2 \frac{L}{H} = 0.004 \frac{750}{275} = 0.01$$

$$C_1 = \frac{2bsfL}{s_x s_y} = \frac{2(5.8)(0.14)(275)}{(1)(150)} = 2.98 \text{ (assuming friction angle to be } \approx 8^\circ, \text{ valid under low stress conditions)}$$

$$C_2 = \frac{A_s f_y}{2b_s f_y L^2} = \frac{(2.36)(250\text{MPa})}{2(5.8)(0.14)(1.32)(275)^2} = 0.004$$

$$\frac{x_y}{x_n} = \frac{1 + \frac{H/L}{\tan \theta_1} - \sqrt{\left(1 + \frac{H/L}{\tan \theta_1}\right)^2 - \frac{4H/L}{\tan \theta_1} \left(1 - \frac{C_2}{H/L}\right)}}{2} = 0.809$$

$$\frac{\sum T_i \max}{\gamma L^2} = \left[\frac{a_1 x_n}{\gamma L^2} \right] \left[\frac{1}{2} + \left(\frac{a_2}{a_1} - 1 \right) \frac{x_y}{x_n} + \left(\frac{L}{H} \right) (\tan \theta_1) \left(-\frac{1}{6} + \frac{1}{2} \left(\frac{x_y}{x_n} \right)^2 - \frac{1}{3} \right. \right.$$

$$\left. \left(\frac{x_y}{x_n} \right)^3 + \frac{1}{2} \left(\frac{x_y}{x_n} \right)^2 \right] = 0.167$$

Therefore,

$$\begin{aligned} & [(\sin 72 - \cos 72 \tan 41) \cot 72 - 3.22(0.208)(\sin 24 - \sin 72 - \\ & (\cos 24 - \cos 72) \tan 41) + 0.208(\cos 24 + \sin 24 \tan 41 - (\cos 72 \\ & + \sin 72 \tan 41) (H/L - \tan 24)^2 + 2(\sin 24 - \cos 24 \tan 41) (H/L \\ & - \tan 24) + (\sin 24 - \cos 24 \tan 41) \tan 24 - 2(0.167)(\sin 24 \tan 41 \\ & + \cos 24) - \frac{2}{\gamma L^2} (c_1 L_2 + c_2 L_3)] = 0 \end{aligned}$$

Thus,

$$\begin{aligned} & [0.222 + 0.7165 + 0.0273] (H/L - \tan 24)^2 - 0.775 (H/L - \tan 24) \\ & - 0.1724 - 0.423 = 0 \end{aligned}$$

Therefore,

$$0.9658(H/L - \tan 24)^2 - 0.775(H/L - \tan 24) - 0.5954 = 0$$

the equation is now of the form $ax^2 + bx + c = 0$, and its solution can be obtained by using

$$\frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

Thus,

$$\frac{0.775 + \sqrt{(-0.775)^2 - 4(0.9658)(-0.5954)}}{2(0.9658)} = H/L - \tan 24 = 1.283$$

and -1.10

the latter figure is not applicable to this case, since it indicates that $\tan \theta_1 > H/L$ or, that the failure plane will extend through the reinforced earth block only.

So,

$$H/L \tan 24 = 1.283$$

$$H/L = \frac{1.283}{\tan 24} = 2.88$$

furthermore, for test 16 the actual $H/L = \frac{750}{275} = 2.73$ and the error is less than 6%.

END

0|8|0|6|8|3

FIN