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REINFORCEMENT OF EARTH STRUCTURES USING SCRAP TIRES

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***FOR
HUGUETTE AND MAURICE
O'SHAUGHNESSY***

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ABSTRACT

Scrap tires are undesired urban waste which are produced at increasing rates every year, particularly in metropolitan areas. They are non-degradable and, due to their shape, quantity and compaction resistance, require large space for storage. Sanitary landfills are increasingly becoming expensive facilities which require engineered design and controlled construction. Thus it is no longer economical to dump scrap tires in landfills. Large amounts of scrap tires are accumulated in open air tire piles, where they are exposed to constant danger of fire. Tire deposits also provide breeding ground for vermin and insects, a problem which assumes greater importance in tropical environments with poor sanitation conditions. The research project investigated the use of scrap tires in engineered fills. This research was aimed at providing geotechnical information on the design and construction procedures related to the use of scrap tires in gravity retaining structures and reinforced fills. If layers of tires, side by side, are filled with soil and tied together to make a mat or a chain, and then placed in successive layers, the resulting structure can be used as a retaining wall or reinforced fill and can provide a practical alternative for the use of this waste. The research also investigated the chemical quality of effluent water emanating from buried tire waste.

A tire is composed of two sidewalls and a tread. It consists of strongly reinforced rubber or polymer with synthetic fibres and metals, and constitutes a material with very high tensile strength and which is able to accommodate large deformations. Its mechanical properties remain available even after its ordinary life as a car wheel element has expired. This research was therefore aimed at making use of the excellent mechanical properties of scrap tires to reinforce embankments and retaining walls. The use of tires as reinforcement is not new and has been successfully used for several years in other countries. The concept is similar to that employed in the use of geogrids for soil reinforcement and well accepted in engineering practice worldwide. Up to the present date, a number of tire reinforced structures have been built in France, England and the

USA at costs which are reported to be significantly lower than those associated with conventional methods. As of 1990, more than 250 structures are reported to be built in France and 12 in Algeria.

The shear resistance of a soil-geotextile interface may be a fraction of the shear resistance of the soil itself. If the reinforcement function is accomplished with tire mats filled with soil, the shearing resistance between the soil and the reinforcement is primarily due to the shear resistance of the soil itself. If the tires are cut to remove the sidewalls, the placement of the soil in the tires is facilitated. A higher degree of compaction is therefore obtained and the shear resistance of the interface between the reinforcement and the soil corresponds to the shear strength of the soil. Earth structures using tires filled with soil have more flexibility than conventional structures and are able to withstand large differential settlements. The flexibility of the resulted material implies that it can be used to support earth fills on compressible ground.

A prototype test embankment was constructed on the private property near Ottawa. A large number of tires were already stored on the site for several years. Also, this site had been operational as a auto recycling facility for a considerable period of time. The test fill incorporated three configurations of reinforced slopes, and three tire reinforced gravity wall sections. In the reinforced slope, the tires were used either as whole tires, or with one sidewall removed. The latter is referred to as the cut tire. In the reinforced fill section, a mat of tires tied together was placed followed by a compacted backfill layer of soil, 0.3m thick, followed by the next layer of the tire mat. Hence in the reinforced slope, the tire layers were separated by a layer of compacted soil. In the construction of the retaining wall section, however, the tires were stacked on top of each other in a staggered manner. The voids were filled with soil and compacted before the next layer was placed. Hence in the retaining walls, a sandwiched layer of soil was not provided.

An important feature of the investigations was a large number of field pull-out tests on different configurations of tire mats. These tests provided valuable data on the interaction

between the different soils and the tire reinforcement. This behaviour is necessary for the design of tire reinforced structures. Also, three large plate loading tests were performed on the surface of the completed test fill to assess the load deformation behaviour of this composite material.

The prototype embankment demonstrated the practical feasibility of using scrap tires as a soil reinforcement technique for both tire reinforced slopes and tire reinforced gravity retaining walls. These structures can be constructed with conventional fill placement equipment. Virtually no damage was observed as the trucks and the lightweight compactors traversed over the tires. Reinforced fills can be constructed with both cohesionless as well as cohesive soils. However, it is recommended that only tires with one side wall removed should be used with cohesive backfills. Cohesive fills do require careful compaction. Also, retaining walls constructed with cohesive backfills can experience large outward lateral deformation.

Pull-out resistance of tire mat reinforcement was governed by the effective internal angle of friction of the soil. Since the tire mat reinforcement geometry was able to fully capitalize on the shear strength provided by the soil, it provides an efficient means of reinforcing the soil. However large displacements were required to fully mobilized the ultimate pull-out resistance. Plate load tests indicated that the tire reinforcement decreased the bearing capacity of the infill soils.

To evaluate any toxic effects of buried used tires on the surrounding groundwater, a drainage system was installed below the embankment and the effluent collected in three wells. Samples were periodically collected and analysed for chemical quality. Additional tests on water quality were performed in laboratory test columns in which tire chips were embedded in sand or clay to provide a conservative estimate

Field monitoring of the effluent indicated that no significant adverse effects on groundwater quality had occurred over a period of 2 years. Some organic compounds can

be leached out of the tire reinforced structure placed above the water table. Laboratory batch tests performed on tire chips embedded in sand provided evidence of an increase in solution of certain metal elements of which some exceed their respective drinking water standards. This increase was attributed to the exposed steel reinforcements found in the tire chips. The amount of organic compounds leached from the tire chips decreased with the number of exposure periods or pore volumes flushed through the soil. This decrease could be attributed to several factors: a limited amount of leachable compounds found at the surface of the tire, the formation of biological or inert inhibitors at the surface, or a compositional change at the tire rubber surface over time.

Finally, recommended construction and design guidelines are provided based on the results of the current investigation.

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LIST OF NOTATIONS

A	=	Area
A_c	=	Cross-sectional area of steel attachment minus estimated corrosion losses
A_{re}	=	Tire reinforcement area
BCR	=	Bearing capacity ratio
c	=	Cohesion in terms of total stress
c'	=	Effective cohesion
c_b	=	Cohesion of backfill
c_f	=	Cohesion of foundation soil
c_u	=	Undrained shear strength
c_v	=	Coefficient of consolidation
C_c	=	Compression index
C_m	=	Modulus of compressibility
C_r	=	Recompression index
CRF	=	Creep reduction factor for polymeric attachments
d	=	Depth, diameter
D_r	=	Relative density
E	=	Modulus of elasticity
e	=	Eccentricity
F^*	=	Pull-out resistance factor
FC	=	Construction damage factor of safety
FD	=	Durability factor of safety
FS	=	Factor of safety
FS_p	=	Factor of safety with respect to pull-out, includes an allowance for deformability
FS_r	=	The targeted minimum safety factor for the reinforced slope
FS_u	=	The factor of safety for an unreinforced slope
G	=	Shear modulus
H	=	Wall or slope height
H'	=	Wall or slope height modified to include uniform of sloping surcharge

K	=	Stress ratio
J	=	Tensile modulus of the tire reinforcement
K_a	=	Active earth coefficient of the retained backfill
K_o	=	Coefficient of earth pressure for at-rest condition
L	=	Length of tire reinforcement
L_d	=	The design embedment length of tire reinforcement, not greater than two passenger tire widths ≈ 1.2 m
L_e	=	Total embedment length of tire reinforcement to resist pull-out
M	=	Moment, mass
M_D	=	Driving moment
N	=	The minimum required tire reinforcement layers
N_c	=	Bearing capacity factor
N_γ	=	Bearing capacity factor
P	=	Available pull-out resistance
P_b	=	Resultant of earth pressure due to the retained backfill
P_d	=	The pull-out capacity used in design, includes deformability restriction (L_d)
P_q	=	Resultant of earth pressure due to the uniform surcharge
P_r	=	The pull-out capacity per unit width of tire reinforcement (per metre)
P_t	=	The available pull-out resistance per tire width (passenger tire width ≈ 0.6 m)
q	=	Surcharge load
q_a	=	Allowable bearing capacity
q_{ult}	=	Ultimate bearing capacity
R	=	Resultant force
R_a	=	Attachment coverage factor, relates the number of attachments per unit width of reinforcement
R_t	=	Tire reinforcement coverage ratio: the equivalent area of tire reinforcement per unit width
S_v	=	Vertical space between the horizontal tire reinforcement layers
T	=	Tension of the tire reinforcement
T_a	=	Allowable tensile strength of the attachment
T_{dc}	=	The design tensile capacity of the tire reinforcement which considers rupture, pull-out, and deformability restrictions
T_{max}	=	Maximum tensile force in the tire reinforcement per unit length
T_r	=	Tensile tire strength per unit width of reinforcement

T_s	=	Sum of the required tensile force per unit width of reinforcement
T_{ult}	=	Ultimate tensile strength of the attachment
V	=	Volume
V_w	=	Volume of frontal soil wedge (for pull-out tests)
w_{opt}	=	Optimum water content
W	=	Vertical force due to weight of the fill
W'	=	Weight of surcharge
x	=	A dimension of coordinate
z	=	Depth below a reference level
α	=	scale correction factor for pull-out resistance
α_b	=	Bond efficiency coefficient
α_{ds}	=	Direct sliding efficiency coefficient
α_f	=	Structural geometric factor for pull-out frictional resistance
α_β	=	Structural geometric factor for pull-out passive resistance
β	=	Slope of soil surface
δ	=	Interface friction angle
Δ	=	Displacement
ε	=	Strain
γ	=	Unit weight
γ_b	=	Unit weight of backfill
γ_{dmax}	=	Maximum dry unit weight
γ_f	=	Unit weight of foundation soil
γ_{re}	=	Unit weight of the reinforced zone, or tire retaining wall
ϕ'	=	Effective angle of internal friction
ϕ'_b	=	Peak effective angle of internal friction of the backfill
ϕ'_f	=	Peak effective angle of internal friction of the foundation soil
ϕ'_{re}	=	Peak effective angle of internal friction of the reinforced backfill
ϕ_u	=	angle of internal friction for undrained conditions
λ	=	Inclination of earth pressure resultant relative to the horizontal
μ	=	Friction coefficient
θ	=	Inclination of the slope or wall face with the horizontal
σ_h	=	Horizontal stress

- σ'_p = Preconsolidation stress
 σ_v = Vertical stress
 τ = Shear resistance
 ν = Poisson ratio
 ω = Face batter of the tire wall

Note: The following subscripts are used to denote a specific soil region:

b, for the backfill material, i.e., the fill material located beyond the reinforced soil section.

f, for the foundation soil.

re, for the fill material used in the reinforcement zone, for filling tires.

CHAPTER 1

INTRODUCTION

1.1 Background

The disposal of scrap tires has become a major environmental problem on an international scale. It is estimated that the United States discards approximately 250 million used tires annually (Williams 1987). The Province of Ontario alone generates per year over 10 million of equivalent passenger tires of which approximately 40 per cent are disposed in landfills or tire stockpiles. Both of these procedures require large disposal areas since whole tires are resistant to compaction. The stockpile method is not desirable since it adds a significant pressure to the environment by providing a good breeding habitat for disease-carrying insects and vermin. A potential fire hazard due to improper storage can result in serious environmental damage and high cleanup cost. Sanitary landfills are now becoming expensive engineered facilities and therefore it is no longer economically feasible to store large volumes of waste tires. Exposed tire piles are also not aesthetically pleasing.

The numerous problems associated with the stockpiling of used tires are a direct result of limited research in the area of recycling, reuse, and innovative disposal. The potential use of scrap tires as reinforcing elements in engineered earth structures is one possibility to reduce this worldwide tire disposal problem. Discarded tires have been reused in the past in their original or shredded form as road subgrades, light weight fills, artificial reefs, breakwaters, rubberized asphalt, and as earth retaining structures (Drescher and Newcomb 1994).

The concepts involved in earth reinforcement have been used by human beings for centuries. The basic principles of earth reinforcement that are of particular advantage in civil engineering are: simplicity in design, ease of construction, and reduction in construction costs. The use of scrap tires connected together and embedded in the soil can provide a satisfactory and simple

material capable of reinforcing earth structures at a low cost. Tires made of rubber sidewalls and treads strongly reinforced with fibers or metals which result in a material with high tensile strength and which is also able to accommodate large deformations. The high tensile strength is mainly associated with the circular tire geometry that is capable of resisting large radial stresses. Its mechanical properties remains largely available even after the tires have been worn out. If sidewalls and treads are to be used as a reinforcement embedded in soil, a material with improved mechanical properties is made available for many civil engineering purposes.

1.2 Objectives

The major objective of this research was to develop engineering design and construction guidelines for retaining walls and reinforced slopes using scrap tires. Even though tire reinforced structures have been successfully constructed in several countries, there is a lack of quantitative data on their engineering properties. Emphasis is placed in this study on quantifying the engineering behaviour of these structures. It was important to identify the various key design parameters, construction procedures and performance criteria. Consequently, a tire reinforced earthfill was constructed which incorporated gravity retaining wall sections as well as reinforced slope. This structure was instrumented to observe its performance.

The study also investigated the interaction properties between tire reinforcement and soil, compressibility and bearing capacity of the tire reinforced embankment, and the influence of backfill material characteristics and type of tire reinforcement used.

Guidelines for the design of retaining walls and reinforced slopes using conventional reinforcing materials such as geosynthetics, are well established. It was therefore of interest to examine if the same, well accepted, approaches could be applied to tire reinforced structures.

An equally important objective was to assess the environmental impact of buried tires on the surrounding ground water quality. Finally, preliminary design and construction guidelines are proposed based upon the results of this study.

1.3 Scope of Work

This study consists of seven phases.

- The first phase was to undertake a comprehensive review of published literature on earth reinforcement with emphasis on the use of tires as fill.
- The second phase of the research consisted of preliminary laboratory tests on ropes, tires, and backfills.
- The third phase was the design and construction of a fully instrumented reinforced earth structure using scrap tires tied together. The soil structures of interest were reinforced slopes and gravity retaining walls.
- In the fourth phase, field experiments were undertaken to monitor the engineering behaviour of the above structure by performing tire pull-out tests, plate loading tests and finally by placing a surcharge over the entire earth structure.
- In the fifth phase of investigation, a parametric study, using a finite element program, was undertaken to examine the effects of various key parameters on tire wall deformations.
- The sixth phase of the study involved the chemical analysis of effluent collected from the above test fill, as well from laboratory lysimeter cells containing tire chips embedded in different soils, in order to identify potential problems related to surface and ground water quality.
- The final phase of the research program was to recommend preliminary design and construction guidelines based upon the results of the current research.

1.4 Outline of the Thesis

This thesis is divided in nine chapters. Chapter 2 presents a review of traditional earth reinforcement practice and also summarises previous experience with tire reinforced structures. It also presents key characteristics of tire reinforcement such as durability, tensile strength, and attachments between tires.

Chapter 3 indicates the key soil and reinforcement parameters required for engineering design of such structures. It also includes results of various material properties, including those for tires and attachments, obtained in this study.

Chapter 4 concentrates on a discussion of tire-soil interaction characteristics. It presents the results and the analyses of numerous tire pull-out tests undertaken in the field for this investigation, as well as evaluation of interface characteristics from laboratory testing.

Chapter 5 provides details of the prototype test embankment, and field instrumentation. The Chapter presents results of an analyses of field monitoring and in-situ plate load tests, and presents a general design methodology.

Chapter 6 presents the results of a parametric study based on finite element analysis which simulated the effects of varying several key parameters on the lateral deformations of tire reinforced walls. A simplified design procedure for estimating tire wall deformation is also presented.

Chapter 7 presents results of water quality monitoring of effluent collected from both field and laboratory samples and presents a discussion of the data.

Chapter 8 provides recommendations for design and construction of reinforced earth structures using scrap tires. A step by step design methodology is also presented.

Finally, Chapter 9 provides the conclusions of all aspects of the investigation, and suggests future research activities.

The Appendix presents two examples of the design of a tire reinforced wall and a tire reinforced embankment.

CHAPTER 2

REVIEW OF EARTH REINFORCEMENT

2.1 Introduction

Reinforced earth structures are widely used in geotechnical engineering. These structures include the construction of retaining walls, steep embankments, natural or cut slopes and foundations. Reinforced soil can also be used for bridge abutments and wing walls, containment structures for water impoundment, dams and dikes and for sea walls (Mitchell and Christopher 1990).

Earth reinforcement construction techniques are not new. Their basic principles are known to have existed in the 5th and 4th millennium BC (Jones 1985). Today, several types of reinforcement material and complementary systems (products) are available for a broad range of civil engineering purposes. Reinforcing materials range from steel strips or bars, steel or polymeric grids, geogrids, geotextile sheets, nails, wood, and used tires which are usually distributed horizontally within the soil. These reinforcing materials must be able to withstand the applied tensile forces, and in some cases, bending and shear stresses as well (Mitchell and Christopher 1990).

Over the last 15 to 20 years, reinforced soil systems have gained wide acceptance for a variety of applications. Growth and expansion of reinforced soil systems can be attributed to many factors: cost, reliability, simple construction techniques, flexibility in adapting to different environmental conditions, the ability to deform without distress, and aesthetics. Soil reinforcement construction techniques are particularly suited for areas where the terrain is steep or unstable, and where ground settlement (ground deformation) is a potential problem. However, some reinforcement techniques are not good for settlement problem areas such as reinforced walls with hard concrete facings.

2.2 Concept of Earth Reinforcement

Soils, in general, are structurally strong in compression but are very weak in tension. The inclusion of strong tensile reinforcing elements to a soil produces a composite material that exhibits strength characteristics of both components. A successful reinforced soil system is able to combine the two elements into a compatible material in terms of surface characteristics or geometry, or a combination of both, so that stress can be properly transferred from one component to the other (Mitchell and Villet 1987).

2.3 Historic Overview

The Ziggurat of the ancient city of Dur-Kurigatzu now known as Agar-Quf and the Great Wall of China are the oldest remaining reinforced earth structures. The Agar-Quf structure was constructed in the first millennium BC and still towers a remarkable 54 m, and was believed to have been over 80 m high originally (Jones, 1985). The tower of Babel is another Ziggurat structure that was completed circa 550 BC (Copplesstone 1963). These structures were built using clay bricks reinforced with woven mats of reed which were laid horizontally over layers of sand and gravel, and were vertically spaced between 0.5 and 2 m. The Great Wall of China was constructed in sections that consisted of a mixture of clay and gravel reinforced with tamarisk branches (Jones 1985).

A timber wharf that has been preserved in the Thames mud for over 1990 years was built by the first-century Roman Army. This structure (Figure 2.1) was originally 1.5 km in length and 2 m in height, and was constructed with oak beams measuring up to 9 m in length that were reinforced horizontally with timber elements embedded in the back fill (Bassett 1981). The construction techniques used in the timber wharf were remarkably similar to present day timber crib methods.

In the past, most reinforced soil structures were used to build river training and dyke works (Ingold 1994). Early examples are the construction of clay fills reinforced with reeds along the

Tigris and the Euphrates in the third millennium BC. The randomly placed reed or branch reinforcement system was replaced by the faggoting technique in which bundles of willow, alder or brushwood were placed in an orderly fashion. This technique was primarily used for land reclamation in Holland, United Kingdom and for the construction of the Mississippi levees in the United States (Haas and Weller 1952). Faggoting was widely used until recently (Doran 1948).

In the 1930s, significant advancement in the concept of soil reinforcement occurred when Coyne introduced his "mur a échell," ladder wall. This retaining wall system consisted of precast concrete facing blocks placed above each other, generally inclined, and tied to a flexible tensile reinforcing steel with an anchor (Figure 2.2). By 1945, Coyne realized that the high internal frictional properties of granular fill provided the necessary bond capacity with the lateral reinforcement members without the need of end anchors.

Modern concepts of earth reinforcement were first idealized by Casagrande and implemented by Vidal in the 1960s. Vidal (1966, 1969) introduced the term "Terre Armé" as a composite material formed of flat reinforcing strips laid horizontally in layers within the soil. The only interaction between the soil and reinforcing member is friction that is generated by gravity alone. The first major retaining wall structures used a steel grid (precursor of today's polymeric geogrids) as reinforcement, and was constructed near Menton in the south of France in 1968. In 1990, over 12,000 structures representing over 4.6 million m² of vertical wall facing with steel strip reinforcement were constructed in 37 different countries (Christopher et al. 1990).

The reuse of scrap automobile tires as a high tensile reinforcing member within a soil mass was one of the ideas inspired by Vidal's "Reinforced Earth." The introduction of Vidal's reinforced earth structure resulted in a better understanding of the fundamental concepts involved and led to the rapid development of new and improved forms of reinforcements. The use of synthetic polymer-based materials (geosynthetics) developed in the early 1970s have become more popular than steel in recent years. Geosynthetic reinforcement materials are divided into two categories: geotextiles and geogrids. The use of polymeric geogrids in earth reinforcements started in the 1981, and since then, more than 300 earth structures have been constructed (Mitchell and

Christopher 1990). The first geotextile reinforced wall was built in France in 1971. Today, many thousands of projects have been completed in North America alone (Elias et al. 1996). Several proprietary and nonproprietary systems have been developed and are currently being successfully used. Many reinforcing techniques are listed in Table 2.1 by proprietary name, reinforcement type and facing arrangement. Arguably, the most popular system are dry-cast modular block systems (Bathurst and Simac 1995).

2.4 Reinforced Earth System

The principle components of an earth reinforcement system are illustrated in Figure 2.3. The essential elements are the reinforcements, the backfill or in-situ soil that interacts with the reinforcement, and the front facing. The reinforcement system can be characterized by the reinforcement geometry and/or material, the stress transfer mechanism, the extensibility of the reinforcement and the method of placement (Mitchell and Christopher 1990). Different reinforcement systems are summarized in Table 2.2.

Reinforcements are usually defined in terms of their material composition and geometry. Reinforcement material is categorized into two main groups: metallic and nonmetallic. Successful reinforcing materials include steel, aluminum, concrete, glass-fibers, wood, rubber and thermoplastics. Three types of reinforcement geometry can be considered: linear unidirectional (strip reinforcement), composite unidirectional (grid strips or bar mats) and planar bi-directional (sheet reinforcement) (Christopher et al. 1990).

The in-situ backfill properties will clearly affect the performance characteristics of the composite soil structure. Granular soils are usually specified for long term conventional reinforced structures in order to meet stress transfers, durability, and drainage requirements.

Facing elements are required for vertical structures to prevent erosion of the exposed soil, to allow compaction of the soil near the face, and to provide a suitable architectural treatment to the structure for aesthetics. A variety of products are available for facing elements, which can be

used for a specific application. These facing materials include precast concrete panels, prefabricated metal sheets and plates, gabions, welded wire mesh, shotcrete (reinforced or not), inclusion of intermediate reinforcements between main reinforcement layers at the face, seeding of exposed soil, and looping of geotextile reinforcements at the face and dry-cast modular block systems (Elias et al. 1996).

The two main stress transfer mechanisms between soil and reinforcement are friction and passive resistance. The different elements of frictional and passive resistance are illustrated in Figure 2.4. Friction resistance occurs along soil-reinforcement and shear stresses are transferred to this interface. Passive soil (bearing) resistance can also be developed for any reinforcing element oriented normal to the direction of relative displacement between the soil and the reinforcement. Friction resistance is predominant for linear and planar reinforcements, such as: strips, rods, and geotextile sheets. Composite reinforcements have many transversal components such as bar, mats, grids, and wire mesh which restrict displacement predominantly by passive resistance or a combination of both frictional and passive resistance.

Reinforcement materials are classified as either extensible or inextensible. Extensible reinforcement is able to deform to a greater extent than the surrounding soil in which they are embedded while inextensible reinforcement cannot. However, extensible reinforcement can be placed at a high enough density to prevent yielding of the soil system, in which case, the total load carried by the extensible reinforcement can be made equal to that of an inextensible system. Typically, polymeric material such as geogrids and geotextiles are considered extensible, whereas steel grids and bars systems are not (Koerner 1994, Wilson-Fahmy et al. 1994). Internal stresses imposed on the reinforcing structure are a function of its overall stiffness, density, geometry and also a function of the strength and deformation characteristics of the surrounding soil.

2.4.1 Available Systems

The two primary types of systems used in North America are the placed reinforcement system and in-situ reinforcement system. In general, placed reinforcements are vertically spaced

between 0.3 and 1.0 m and a granular soil is usually specified for backfill. Typical mechanical properties and geometries for several different reinforcing materials are provided in Figure 2.5. In-situ reinforcement techniques consist of placing, in the existing ground, closely spaced passive inclusions such as steel bars, metal tubes or other metal elements that are able to restrict movement and limit decompression during and after excavation (Juran et al. 1990). This reinforcing procedure is commonly referred to as soil nailing.

2.4.1.1 Strip Reinforcement

Strip reinforcements are flexible linear elements, either metal (steel, aluminum, copper) or plastic, having a width greater than their thickness. These strips are generally placed in horizontal planes between successive lifts of soil backfill. Metal strips are typically plain or ribbed. The use of ribs or grooves increases friction between the soil and reinforcement. The strip reinforcements are connected to the facing elements that usually consist of either precast concrete panels or prefabricated metal members. High strength polyester or polyaramid plastic strips have been used in order to avoid the potential problem of metal corrosion. A polymeric strip reinforcement system is presented in Figure 2.6. The backfill material is specified based on geotechnical and durability considerations.

2.4.1.2 Grid Reinforcement

Grid reinforcement system consists of transverse and longitudinal members arranged in a rectangular form, in which the transverse elements are placed parallel to the face or free edge of the structure. The grids resist outward movement mainly through passive soil resistance on the transverse members and are therefore relatively stiff elements. However, substantial frictional resistance can develop along longitudinal members depending on grid geometry. Longitudinal members may be flexible elements possessing a high modulus of elasticity making them resistant to creep. Grid reinforcements are fabricated from steel or metallic elements in the form of plain or galvanized steel mesh or polymeric tensile resistant elements, known as "Geogrids."

2.4.1.3 Sheet Reinforcement

Sheet reinforcement consists mainly of polyester or polypropylene fibers formed into a continuous sheet. The stress transfer mechanism between the soil and sheet reinforcement is primarily friction. The geotextile sheet can be fabricated from several different processes. These processing techniques include woven, nonwoven needle-punched, nonwoven heat-bonded, nonwoven resin bonded. The reinforced soil structure is constructed in successive lifts of reinforcements and soil backfill. The geotextile sheet is wrapped around the exposed soil at the face to form a stable facing unit, demonstrated in Figure 2.7. The exposed geotextile fabric is covered with gunite, asphalt emulsion, shotcrete, or with soil and vegetation for long term protection against ultraviolet light degradation and vandalism. Concrete panels, gabions, modular blocks or other structural materials can also be used as facings. The geotextile sheet and structural facing units are joined by casting into concrete, friction, nailing, overlapping, or other connecting procedures.

2.4.1.4 Rod Reinforcement

Rod reinforcement employs slender flexible linear elements containing one or more pronounced protrusions or deformities that perform as abutments or anchors in the soil. Anchored Earth reinforcement restricts structural movement mainly through passive resistance. Rod reinforcements are attached to concrete panel facings, as illustrated in Figure 2.8. Anchored Earth systems have not been extensively used (Mitchell and Villet 1987).

2.4.1.5 Fiber Reinforcement

Fiber reinforced soil or Texsol is a composite material consisting of randomly placed tensile resistant strands or filaments (fibers) within a granular soil. Fiber reinforcement is a relatively old technology and is being investigated extensively in France. Fiber materials included plants, reeds, polymeric filaments (polyester) and metallic filaments (metallic needles). Fiber reinforcement has the potential to strengthen the soil structure in three directions. However, a

uniform distribution of fibers within the backfill is difficult and requires a complex mixing process that is expensive. Experimental results indicate that fiber reinforcement increases the bearing capacity and self-healing processes when subjected to erosion (Leflaive 1988). Several Texsol retaining wall structures (more than 50 applications) have been constructed in France, demonstrating the technical feasibility of this method.

2.5 Tire Reinforcement

Tire reinforcement consists of old or scrap tires, either whole or in parts (two sidewalls and a tread) tied together in tiers or in a continuous mat that is able to resist large tensile forces. At the end of 1993, over 250 tire reinforced soil structures had been constructed in France alone. Many other tire structures have been implemented in other countries around, e.g. United States, Brazil, Switzerland, Germany. Tire-soil reinforced structures have numerous civil engineering applications, including retaining walls, reinforced slopes, lightweight fill, energy dissipators, sound barriers, slope or river protection, and many other potential applications. Soil structures using scrap tires can be economical when compared to traditional materials as evident by their early diverse and numerous engineering applications (Long 1993). These structures are economical because of the large supply of old tires and low requirement for any specialized construction equipment or technique for placement.

2.6 Tire Constituents

Tires are fabricated with vulcanized rubber that contains reinforcing textile cords, high strength steel or fabric belts and a high strength steel wire reinforcing bead. The different components of a radial tire are presented in Figure 2.9. The beads consist of rubber covered metal wires or braids that do not easily deform. The tire cord is usually made of braided rayon cord. These are very strong: a cord only a few tenths of a millimeter in diameter (0.6 to 0.8 mm) may have a tensile strength of 400 N. The major tire constituents are shown in Table 2.3.

The most commonly used tire rubber is styrene-butadiene-copolymer (SBR) containing about 25% styrene. Other tire rubbers used are natural rubber (cis-polyisoprene), synthetic cis-polyisoprene and cis-polybutadiene. A typical chemical composition for tire rubber is provided in Table 2.4. Carbon black is added to strengthen the rubber and increase abrasion resistance. The extender oil is a mixture of aromatic hydrocarbons that act to soften the rubber and to increase its workability. Sulphur is employed to harden the rubber by cross-linking the polymer chains within the rubber that prevents high temperature deformation. The zinc oxide, stearic acid and an organo-sulphure accelerator are used to aid in the vulcanization process and also to enhance the physical properties of rubber (Williams et al. 1990). Anti-oxidant and other additives are also added to prevent deterioration of the rubber complex.

2.7 Tires as a Civil Engineering Material

Tires, even worn tires, have many important material characteristics that enable them to be used in civil engineering projects (WYMCC 1977). These material characteristics are:

- (1) Economical (low cost) and Ready Availability:** It is probable that tires may be supplied to the construction site free of charge except for transportation costs.
- (2) Durability:** Tires are essentially non-biodegradable and have an estimated life span of at least 100 years.
- (3) Flexibility and Extendibility:** Embedded tires are able to bend and move with ground displacement without fracturing and are able to deform far beyond the capability of the surrounding soil. Tire flexibility can also generate high interlocking frictional forces as the tire deforms with the surrounding soil.
- (4) High Radial Strength:** Tires due to their circular geometry can resist very large tensile forces.

(5) Friction: The frictional forces developed between tires can be high even without deformation, with a coefficient of friction estimated to be about 0.75. Further, tire deformation could result in higher frictional forces and the production of a positive interlock.

(6) Handling and Transportation: Tires are easily handled by conventional construction equipment and labour. Off-loading can be easily accomplished by one labourer. Tires can be readily transported to the construction site in loads ranging from 600 to 1000 tires per truck depending on truck capacity.

2.8 Tire Reinforcing Elements and Configurations

Scrap tires can be cut into different reinforcing elements that can be arranged in several different configurations and some are shown in Figure 2.10. The different tire reinforcing elements are:

- tire sidewall laid flat;
- tire tread only (both sidewalls have been removed) placed on edge;
- tire tread which is cut and laid flat;
- whole tire (passenger or commercial) laid flat;
- tire with one sidewall removed and laid flat.

These different tire reinforcing elements can be assembled to form reinforcing mats (Figure 2.10). Each distinct tire component can be attached together with several different materials. The construction of the different tire reinforcing configurations should not require any specialized fabrication technique, in order to retain economically viability.

2.9 Attachments

Some form of attachment is required to connect the tires to form a mat. A variety of connectors (attachments) were investigated by Long (1990): synthetic ropes and straps, metal parts, metal hooks. The results indicated that the appropriate connector was related to the type of reinforcing

structure (retaining wall, embankment reinforcement, energy absorber, etc.). However, most tire reinforced structures in France were constructed with polyester strips tied together in a double knot. Essentially, once the knot is in place and embedded in compacted soil, it becomes virtually impossible to undo. Important characteristics of any attachment are:

- they must satisfy a minimum strength requirement;
- the contact area of attachment should be broad enough to prevent puncture of the rubber;
- be durable; they should be compatible with the service life of the structure;
- they must be inexpensive (another “waste material” would be ideal);
- they should be environmentally acceptable.

2.9.1 Strength Parameters

The required strength parameters are:

- the yield strength and modulus of the attachment;
- the long-term allowable tensile strength of the attachment.

The long-term design strength of polymeric attachments must include considerations for potential creep, construction damage, and aging effects. These properties are difficult to determine for a specific project due to the time requirement for testing. Reduction factors are used to account for these factors. Metallic attachments are susceptible to corrosion. The estimated loss in tensile strength by corrosion can be counteracted by increasing its cross-sectional area.

2.9.1.1 Metallic Attachment

The allowable tensile force per attachment, T_a , is determined by:

$$T_a = \sigma_a A_c \quad (2.1)$$

where: σ_a = allowable tensile stress = $0.48 \sigma_y$ for connections

σ_y = yield stress of the metal attachment;

A_c = design cross-sectional area of the attachment minus the expected corrosion losses occurring during the design life.

2.9.1.2 Polymeric Attachment

The determination of the allowable tensile force (T_a) per attachment is more complex for polymeric materials such as synthetic rope. These materials are affected by creep, construction damage, aging, temperature, and confining stress. In addition, the material characteristics from the same base polymer may vary. Long-term behaviour requires extensive testing. In the absence of sufficient test data, T_a can be estimated from the following simplified expression (modified from, Elias et al. 1996):

$$T_a = \frac{T_y (C R F)}{F D \bullet F C \bullet F S} \leq T_s \quad (2.2)$$

where: T_y = ultimate or yield tensile strength.

T_s = long term tension capacity of the polymeric material at the design strain (usually less than 5%).

FD = Durability factor of safety. Dependent on several factors such as biodegradation, thermal oxidation, chemical attack, environmental stress cracking and can range from 1.1 to 2.0. (2.0 is recommended when no specific durability information is available). For more information on attachment durability see Section 2.9.2.2.

FC = Construction damage factor of safety and range from 1.05 to 3.0. For flexible polymeric attachments, construction damage would be minimal.

FS = Overall factor of safety for uncertainties in the structure including tire properties and geometry, backfill properties, and externally applied loads. For any permanent vertical structure (retaining walls), a minimum value of 1.5 should be applied. For reinforced slopes, a minimum value is 1.0, since the required factor of safety is accounted in the stability analysis

CRF = Creep Reduction Factor ($CRF = T_c/T_y$, where T_c is the creep limit strength resulting from creep tests). If the CRF value is not available, (Elias et al. 1996). recommends the following reductions for geosynthetic reinforcements:

<u>Polymer Type</u>	<u>Creep Reduction Factor</u>
Polyester	0.4 to 0.5
Polypropylene	0.2 to 0.25
Polyethylene	0.2 to 0.4

2.9.2 Durability Consideration

The required service life of permanent reinforced soil structures may exceed 75 years. Therefore, the durability of the attachments is an important factor in the design of such structures. Metallic attachments should be made of galvanized mild steel that is corrosion resistant. Metal bars and strips can be chemically treated with an epoxy coating that further protects surfaces from oxidation processes. Epoxy and PVC coating may provide corrosion protection despite their susceptibility to construction damage. Mitchell and Christopher (1990) stated that resistance of steel elements to corrosion or deterioration is greatly influenced by the soil's resistivity, pH, chloride, and sulphate content. A highly corrosive environment consists of high content of dissolved salts in an acidic or alkaline pH condition. Electrical currents produced between different metals, stray currents, infiltration of chlorides and other salts during the service life, stress level, oxygenation and changes to the ground water quality can in part or all accelerate corrosion. Therefore, corrosion rates will depend on the environmental conditions of the site including type of backfill, and the presence of salinity.

Polymeric attachments (polyester straps and strips, polyethylene and polypropylene rope) are not susceptible to corrosion, but may degrade due to physicochemical activity in the soil such as

hydrolysis, oxidation, biodegradation, and environmental stress cracking. Furthermore, polymeric reinforcing materials are vulnerable to construction damage.

2.9.2.1 Metallic Attachments

DiMaggio (1988) studied the rate of corrosion for galvanized mild steel bars or strips buried in different soil environments commonly used in reinforced structures. Results indicated that deterioration can be estimated and accounted for by using a metal reinforcement of greater thickness. However, most galvanized steel reinforced structures have used a backfill material with low corrosive potential. The prediction of corrosion rates is difficult and uncertain. To avoid corrosion problems, some projects have used stainless steel reinforcements.

The design attachment cross-section, A_c , accounting for strength loss due to corrosion, is defined as: $A_c = A_n - A_s$, where A_n is the original attachment cross-sectional area, and A_s is the sacrificial area lost to corrosion. The minimum allowable material stress should correspond to the estimated service life of the structure. For example, permanent supporting structures such as railroads and roadways may have a service life of up to 100 years. The design of the system should also consider any potential environmental changes that may occur in the surrounding soil, for instance, the application of deicing salts.

It should be noted that the FHWA (U.S. Federal Highway Administration) based on Elias et al. (1996) proposed corrosion rates are for mildly corrosive soils having controlled electrochemical properties limits and are therefore, suitable for conventional design.

The FHWA proposed the following corrosion rates for mildly corrosive soil:

- For zinc, 15 μm per year and per side, for the first two years; thereafter 4 μm per year.
- For residual carbon steel, 12 μm per year and per side.

2.9.2.1.1 Epoxy and PVC Coatings

Fusion bonded epoxy and PVC coatings are increasingly used to limit the effects of corrosion on steel reinforcements. To provide suitable protection, the coating must be correctly applied and be of appropriate thickness (AASHTO M-28d for strips). The recommended minimum thickness (epoxy coating) for permanent structures is 457 μm . Elias et al. (1996) recommends a 1.5 mm of sacrificial steel when protective coatings are used. Nevertheless, coatings should provide protection for the entire service life of the structure. Epoxy and PVC coatings could be susceptible to construction damage. The recommended maximum backfill aggregate size should be limited to 19 mm (DiMaggio 1988).

2.9.2.2 Polymeric Attachments

Polymeric attachments are not susceptible to corrosion attack. They have varying resistance to chemical attack, sea water, and biological activity. Deterioration predominantly results from mechanical damage, strength loss due to creep, and especially, degradation from exposure to ultraviolet light. The polymer formulation and resin additive of the attachment should be compatible with the backfill material. Any long term environmental changes in soil system over the life span of the structure should be considered. Environmental considerations should include: pH, chloride content, organic, oxidation agents, the presence of any chemical solvents, fossil fuels, industrial waste and any other potential factor. Tire connections are usually buried, and therefore ultraviolet light degradation is usually not a concern.

The resistance to chemical attack, hydrolysis, oxidation, biodegradation, and environmental stress cracking differs for each polymeric material and is dependent on polymer quality and additives used. Because of the varying polymer quality, Elias (1997) provides a general approach to evaluating polymeric materials (geosynthetics) durability and making lifetime predictions.

Damage during handling and construction are minimal for flexible attachments such as ropes and straps. The joining of the two free ends, either by tying a knot or using clips, may be subjected to

some degree of construction damage. A 10 percent minimum reduction in strength should be used (safety factor FC equal to 1.1). If geosynthetic sheets are used to bind the tires together by friction, compaction damage could significantly reduce the effectiveness of the composite reinforcement. Leclercq et al. (1990) stated that compaction alone can reduce the initial tensile strength by 30 percent. Christopher et al. (1990) recommended a 50 percent reduction related to construction damages. **If the durability of a certain attachment is questionable, field trials should be performed to assess the potential for construction damage.**

2.10 Facings

Some facing treatment is always required to protect the exposed face of a reinforced soil from erosion and/or deterioration. They also provide an aesthetic treatment to the exposed face. A list of different potential facing materials is provided in Table 2.5. The type of facing element used will control settlement tolerances. Separate panel facings provide the flexibility to absorb differential movement in both vertical and horizontal directions without cracking which could occur in a more rigid facing structure (Christopher et al. 1990). Since facings are required for all reinforced structures, its cost does not negate the economic benefits derived from using scrap tire reinforcement. However, there are limits to acceptable differential settlements, as in the AASHTO (1996) guidelines.

2.10.1 Tire Facing

Tires have some times been used for facing elements as in the case of a gravity retaining structure or with inclusion of intermediate reinforcements between each tire facing element. Gourc and Matichard (1994) reported the construction of a combined tire facing and geotextile sheet reinforced structure on the main road (RN90) between Albertville and Moutiers, as part of the preparatory works for the 1992 Winter Olympic in Albertville. In view of the topography, the structure had to be built 7 m high with up-slope gradient of 60° to the horizontal, as illustrated in Figure 2.11. The purpose of the structure was to provide road protection from falling rocks. The

stability requirements are similar to cellular facing type structures. The internal stability was provided by the horizontal layers of woven polypropylene geotextile reinforcement while the local stability was ensured by using good quality scrap tires from heavy vehicles. The permanent use of tires in exposed environment should be carefully assessed.

2.11 Tire Applications in Earthfills

2.11.1 United States Experience

One of the first practical applications using discarded tires was the repair of a hill side fill slipout along Calif-236 north of Santa-Cruz in the mid-1970s (Forsyth and Egan 1976). After the removal of debris and the placement of a drainage system, the California Department of Transportation (CalTrans) rebuilt the road embankment by reinforcing the soil with tire sidewall mats that were vertically spaced at 0.6 m (Figure 2.12). The individual tire sidewalls were joined together by steel clips to form a continuous mat that was then extended beyond the embankment face by 100-150 mm in order to provide erosion protection. The use of tire sidewall mats permitted the construction of a side slope of 0.5:1, rather than the conventional 1.5:1, which resulted in a saving of some 70000 m³ of expensive fill (Hausmann 1990).

According to Drescher and Newcomb (1994), several other tire projects have been undertaken in California. One project used scrap truck tires to control shoulder erosion of an embankment on Route 32, in Tehama County. Here, whole truck tires were interconnected with 2.7 mm steel reinforcing bars, to form a continuous mat. The reinforcing mats were secured to the embankment by salvage anchor posts and then covered with approximately 0.7 m of compacted permeable fill.

Another project involved the placement of discarded truck tires in a low velocity drainage channel to limit slope erosion. The truck tires were placed seven to eight tires high against the banks, interconnected by lacing through each tire with no. 8 gauge wire and tying each row to an end post. The tire mat was covered with 0.45 m of compacted sandy loam.

Automobile tire piles and tied tire walls were also used on State Route 111, near Palm Springs, to provide temporary barriers against wind blown sand.

The Minnesota Department of Natural Resources, in Saint-Louis County, Minnesota, undertook a feasibility study to determine if roads constructed over highly compressible soils (soft ground) which were reinforced by used automobile tires, could improve the subgrade performance (Drescher and Newcomb 1994). The study consisted of a roadway divided into eight 120 m test sections, constructed over a weak subgrade, namely peat, which ranged from 1.5 to 5.1 m in depth. Each section consisted of a tire mat arranged over a geotextile and a geotextile placed at each end of the test section, in order to provide a control test area. The tire mat and geotextile were fixed at the base of the shoulders. To construct the tire mats, holes were prepunched in each tire, and through which a nylon toggle strap was inserted between two tires and tied together. The reinforcement was covered with 0.3 m of silty sand. The reported settlements were less than would be expected from a conventional soil embankment without reinforcement.

The different methods used to tie the tires together to produce mats in each of these projects were highly labor-intensive. The American experience showed that construction of reinforced earth structures using scrap tires can provide an alternative solution to many geotechnical problems.

2.11.2 United Kingdom Experience

The first project in England using scrap tires was the construction of an experimental gravity wall at the Mechanical Engineering Services at Lofthouse in West Yorkshire (WYMCC 1977). The building of the gravity tire wall (cribwall type of construction) coincided with the construction of the tire reinforced embankment slope near Santa-Cruz, California, in the mid 1970s. The experimental cribwall was built in order to provide extra valuable space for future office expansion or car parking. The construction of the retaining wall used whole tires from cars or light commercial vehicles ranging from R-13 to R-15 (radius of the tire rim in inches) with corresponding tire widths varying from 125 to 200 mm. The height of the tire wall varied up to

3.7 m and incorporated a curve. The maximum length of the structure, with 4500 tires, was 45 m with an average tire layer thickness of 0.15 m and created a useable area of 100 m², approximately.

The investigators observed that effective interlock at the face of the tire wall was difficult for slopes steeper than 1 to 1. To stabilize the face during construction, steel link bars of 12 mm in diameter were used for every 15 tires (not all tires were interconnected). The cost of the experimental tire wall was estimated to be approximately one quarter the cost of a similar conventional retaining wall.

Dalton and Hoban (1982) report on the construction of a tire wall on the west-bound exit of the M62 at Junction 26, near the Bradford passes close to the Hunsworth dyeworks. The tire wall was proposed by the Engineering Development Section of West Yorkshire MCC, as an alternative to the traditional gabion wall solution. The initial proposal saved the County of Yorkshire approximately £10,000 in construction costs as compared to more conventional solutions. The reinforced soil structure was an anchored or tied-back tire wall, as illustrated in Figure 2.13. A schematic representation of the constructed anchored tire wall is shown in Figure 2.14. The face of the wall was constructed by placing tire treads to tread to form a single line (Figure 2.13). Alternate layers of face tires, an approximate vertical distance of 0.3 m, were connected to a secured anchored tire by 1000 kg Paraweb webbing. The anchor tires were positioned 3 m back and centered horizontally from the front line of tires. The paraweb was threaded through all face tires and secured to the anchor tires at regular intervals. A granular backfill was used. The paraweb and tire anchors were able to prevent local failure of the tire wall face by providing sufficient tensile strength. They also provided enough lateral restraint against wedge type failure and slip circle failure within the block.

2.11.3 French Experience

The first research in France on soil reinforcements using old tires was commissioned in 1976 and resulted in the submission of a report to the Délégation Général à la Recherche Scientifique et

Technique. The Laboratoire Central des Ponts et Chaussées conducted studies on reinforcement in the form of whole tires, sidewalls, or treads placed on edge or cut and laid flat (Long 1990).

The first experimental reinforced tire wall was 5 m high and 10 m long. It was constructed in 1982 at Nancy in the Langres region (Figures 2.15 and 2.16). The wall consisted of several reinforcing tire tread mats tied to flexible precast concrete facing units. Each mat was vertically spaced no more than 0.5 m apart and back filled with a granular material. The individual tire treads were placed edgewise and tied together with polyester straps.

Since then, French engineers have constructed more than 250 structures with tire reinforcement in France, and 12 in Algeria up to 1993 (Long 1993). They have also classified this reinforcing technique as "Pneusol," a registered trade mark. One quarter of the French projects have used scrap tires for slope remediation (Figure 2.17) and for erosion protection (Figure 2.18). Other applications have included construction of retaining walls, providing a light weight ground fill that has a unit weight between 6 and 8 kN/m³, decreasing active pressures on structures (Figure 2.19), energy absorption barriers for rockslides and snow avalanches, sound barriers (Figure 2.20), reducing load distribution above buried culverts by arching, and improving ground serviceability for military vehicles. Tires could also be used as a structural media in a number of ways: creating artificial islands and reefs, land terracing, crash barriers, bridge abutments, reinforcing soil foundations, sea defenses, stabilizing soil heaps, and many other potential applications.

Long (1993) reported on the construction of a gravity retaining wall using commercial truck tires to repair a failed slope near Dommiers. The retaining structure consisted of placing whole tires flat in a orthorhombic configuration in order to maximize interlocking forces. Each tire layer was covered with a nonwoven geotextile in order to insure proper bonding between each tire. The retaining wall was constructed with a 1 to 1 slope. This type of tire structure is very cost effective, especially in cases where the subgrade is weak or deformable. Also, the placement of tire in the form of mats limits the effect of frost heave.

Some of the larger French structures reported by Long (1990) are:

- The retaining wall at Fertrupt (1984) having a total length of 54 m, a height of 5 m and a flexible concrete facing.
- Energy absorption structure at La Grave (1984) to prevent avalanches or rock slides, length: 120 m, thickness: 1 m.
- Reinforced steep slope at Kruth-Marstein (1984) to support a road on unstable ground, length: 80 m, height: 4 m.
- Reduction of active earth pressure at the Mende Wall (1986), length: 54 m, height: 5 m, and thickness: 4 m.
- A light ground fill at the Romilly-sur-Seine motocross track (1989), length: 1200 m, width: 4 m, and thickness: 1 m.

It is important to note, however, that the French literature provides very limited quantitative information, especially on the mechanical behaviour and deformation characteristics of these tire structures, and design criteria are not well defined.

2.12 Shredded Tires (Tire Chips)

In the United States, research has been undertaken on the use of shredded tires as a form of lightweight fill in roads and retaining wall construction (Giesler et al. 1989, Eldin and Senouci 1992, Drescher and Newcomb 1994). Several test structures have been built in Minnesota and Wisconsin. In most of these projects, the test fills comprised of three or four of the following layers (Figure 2.21):

1. asphalt concrete layer or gravel;

2. granular base;
3. shredded tire fill;
4. a rigid subgrade, or the placement of a geotextile over a weak and compressible subgrade.

The installation of the geotextile also reduced the risk of embankment shear failure. Other test fills tested include: a multi-layer system of alternating tire chips and granular soil layers, tire-soil mixtures with varying tire chip sizes and percentage, and a soil system that contains a layer of wood chips. Published results (Giesler et al. 1989, Eldin and Senouci 1992, Drescher and Newcomb 1994, Humphrey and Eaton 1995) indicate the following:

- shredded tires produce a lightweight fill that reduces settlements, up to 50%, and increase the stability of the road embankment;
- shredded tires increase the shear strength of soils;
- the tire chip layers do not produce excess pore water pressure since they are free draining;
- tire shredding produces tire pieces (chips) ranging in size from <50.8 mm to 610 mm, and having an uncompacted bulk unit weight varying from 2.35 to 4.70 kN/m³;
- the coefficient of lateral pressure (K) ranges from 0.49 at low vertical stress to 0.82 at high vertical stress;
- the road embankment performance is influenced by the tire chip size, method of placement, and soil-cap thickness;
- the use of shredded tires does not present any major handling and placement problems in road construction;

- the compressibility and rebound characteristics are much higher than traditional granular fills and somewhat problematic which could lead to premature fatigue failure of hard surface pavement;
- the shredded tire mass appears to behave as an anisotropic material;
- early environmental tests indicate little or no likelihood of any detrimental effects of the tire chips on the surrounding ground water;
- shredded tires have good thermal properties;
- preliminary research indicates that tire chips have good adsorbing properties for volatile organics.

Recent research in the use of shredded tires as a lightweight construction material appears promising and the number of applications will certainly grow. Further study is required in order to properly assess the engineering properties and key controlling parameters. These studies should lead to the development of engineering and environmental guidelines for the proper use of shredded tires in soil structures. However, the additional process of shredding the tires when compared to whole tire applications, may increase the overall cost of the structure. An important disadvantage is that there is considerable consumption of energy in the shredding process to produce tire chips. Several cases of exothermic reaction in tire chip fills have been reported in the USA (Humphrey 1996).

2.13 Durability of Tires

Tires are composed of polymeric materials, mainly vulcanized rubber, and therefore are usually not susceptible to corrosion. However, initial mechanical properties of the tire material may be altered due to physico-chemical aging, UV radiation, creep, and aggressions at the setting (construction damage mainly due to compaction). Physico-chemical aging occurs as a result of

the chemical nature of polyester and polyamides that can be hydrolyzed. Hydrolysis of polyester material is a function of temperature, pH, and stress level. However, hydrolysis may never be observed within the service life of the reinforced soil structure (Leclercq et al. 1990). Macromolecular chains of polymers are susceptible to ultraviolet light (UV) radiation breakdown. This phenomenon can be neglected in most cases because the reinforcing elements are sheltered from UV radiation (covered by soil). Creep is the property of structural elements that leads to deformation with time at constant load. The amount of creep is dependent on the nature of the polymer used.

AB-Malek and Stevenson (1986) studied the physical condition of vulcanized natural rubber submerged in 24 m of sea water for a period of 42 years. Their investigation revealed that no serious deterioration of the rubber had occurred. After 42 years of submersion, the maximum amount of water adsorbed was 4.7% and had no adverse effect on strength properties. The limited amount of water absorption was attributed to the formation of a thin surface layer (0.05 mm) of an iron base material. There was no visual breakdown of the tire by marine organisms. The thin rubber layer prevented corrosion of the mild steel tire reinforcement, even in water which was highly oxygenated.

The principle factors affecting the durability of tires are:

(1) U.V. Radiation: UV radiation breakdown of tire rubber can be neglected in most cases, where the tires are buried and thus sheltered from UV radiation.

(2) Heat: drying of the rubber results in superficial cracking of the rubber. This problem would be minimized for buried tires, since the average temperature at depth would vary little, between 3 and 4 °C. Near the surface, the temperature would vary according to seasonal conditions. A maximum temperature of 15 °C would be expected during the summer months. This temperature would be much less than that due to road usage.

(3) Ozone: rubber is degraded by ozone. However, ozone concentration in the soil is negligible.

(4) Soil Acidity: strong acid ($\text{pH} \sim 1$) will destroy rubber. However, reported pH values for acidic groundwater usually range between 4 and 5. At these pH levels, deterioration of the rubber is minimal. However, this parameter could be important if waste material is used as backfill, such as industrial waste or acid mine tailings.

(5) Construction Damage: damage during construction and compaction have not been reported. Tires are able to withstand high stress levels imposed during compaction by being able to deform to the same extent as the surrounding soil. If tires are tied together to form a tire mat, the interconnecting elements such as steel bars, polyester straps and ropes could be subjected to greater construction damage than the surrounding tires. The amount of damage would be a function of the tensile strength and flexibility of the connection. If ropes are tied together by a knot, it may become undone by the passing of heavy construction vehicles.

Cotton fabrics will rot but rayon and nylon fabrics are highly resistant to any form of attack. The high tensile steel in the beads and belts is well protected by the rubber coating. Therefore, even worn tires represent a very high quality reinforcing material that is very resistant to corrosion and deterioration.

2.13.1 Resistance to Fire

Tires are a combustible material. However, they are also difficult to ignite. The WYMCC (1977) investigated the potential fire hazard of a structure constructed with tires. A 1 m high circular mound of tires filled with soil was built. The filling of the tires was achieved by placing the soil over the tire without compaction. Attempts to ignite the tire mound were carried out by using several different procedures. A strong wind prevailed during testing. Although the tire structures did not represent an actual reinforced tire wall, several important properties were observed. These were:

- To set a tire structure on fire, a continuous and intense application of heat is required (a bonfire of dry timber). The use of gasoline or the application of a gas blow torch is insufficient.
- The application of a gas blow torch produced only slight charring of the rubber, and compared to more conventional building materials, considerable less damage was produced.
- Burning of the tires would be restricted to exposed tires only and would not penetrate into the fill.
- The possibility of fire can be eliminated by covering the exposed tire facing with incombustible materials such as bricks, concrete blocks, shotcrete, metal panels, gabions or soil. For a properly constructed tire wall in which the tires have been filled with soil and compacted, ignition of the tires facing could be very difficult. However, further research is required to substantiate this possibility.

2.14 Tensile Strength of Tire Parts

Tires are composed of various parts (sidewalls, treads) that are reinforced with steel and cord. Tensile strength tests were performed on passenger-car tires in France. A statistical study was undertaken on the test results. The principle findings reported by Long (1990) are:

- the mean tensile strength of tire treads is 56 kN with a standard deviation of 24 kN; the probability that the tensile strength of all treads exceeds 26 kN is 90%; that it exceeds 36 kN is only 80%;
- as for sidewalls, there is no particular difference between the two sidewalls of a given tire. The sidewall strength varied from 17 kN for the least reinforced to 52 kN for the most heavily reinforced, with a mean of 25 kN and a standard deviation of 10 kN.

The stress-strain properties and the corresponding tensile modulus were not reported.

2.15 Strength and Behaviour of Soils Reinforced With Tires

Tire reinforcement improves the mechanical properties of a given soil, either anisotropically or isotropically, depending on the tire reinforcing scheme. Anisotropic behaviour occurs when the tire reinforcement in the form of mats sandwiched between layers of soils used. Isotropic behaviour (uniform properties in all directions) usually applies to large tire structures in which the individual tire elements are continuously linked and mixed uniformly with soil such as whole tire mats employed in gravity retaining walls, artificial islands, bridge pier protection.

2.15.1 Shear Strength Characteristics

2.15.1.1 General

The interaction between the reinforcement and the soil results in a composite material of higher shear strength than that of an unreinforced soil. This increase in shear strength has been explained in two ways. Schlosser and Vidal (1969) postulated that the inclusion of tensile reinforcing elements and the subsequent stress transfer between soil and reinforcement results in a composite material possessing an apparent cohesion. In the second concept, deformations in the direction of the reinforcement are restrained which produces a rotation of the principal strain and stress directions. The soil composite is considered anisotropic (Bassett and Last 1978).

The reinforced soil system can either fail by pullout of the reinforcement usually under low confining stresses or by breakage of the reinforcement under high confining pressures. Schlosser and Long (1972) demonstrated this behaviour with triaxial compression tests on sand reinforced with thin aluminum sheets placed horizontally, as illustrated in Figure 2.22. The two modes of failure are also shown in Figure 2.22 as two distinct zones. The Mohr diagram in Figure 2.23 demonstrates the increase in shear strength produced by the apparent anisotropic cohesion concept. The parameter c'_R is the apparent cohesion generated by the reinforcement and σ'_R is

the increase in the major principal stress at failure. The internal angle of friction remains unchanged and is equal to that of the unreinforced soil.

2.15.1.2 Soil Reinforced with Tires

Full scale laboratory tests on the strength characteristics of tire reinforced soil are non-existent because of the physical size of the tires. Laboratory simulation of embedded tires by using latex rubber rings, or cut rubber pipes carried out by some researchers should be treated with extreme caution. The only satisfactory way to assess the tire-soil interface strength is either to conduct full scale tests in the laboratory (clearly not feasible) or to conduct pull-out tests in the field to failure.

Long and Pouget (1980) conducted laboratory tests on cylindrical specimens of Fontainebleau sand reinforced with aluminum foil discs and latex rubber. Sand specimens reinforced with aluminum foil discs displayed considerable cohesion whereas the latex rubber reinforcement showed no cohesion and even a slightly smaller internal friction angle (Figure 2.24). The apparent cohesion was proportional to the degree of reinforcement. Based upon these results, Long (1990) postulated that the strength behaviour of a granular soil reinforced with flat tire elements (sidewalls, flat treads) in which the primary soil/tire interaction was essentially frictional. The resulting strength behaviour was extrapolated between these two reinforcing cases. Therefore, the Mohr strength envelope of a tire reinforced soil would lie between that of an aluminum or steel reinforced soil and a rubber reinforced soil.

Similar results were reported from triaxial tests conducted on shredded tires finer than 76 mm in a silty clay soil. The tire soil mixture demonstrated an increase in cohesion of about 21 kPa and a slight decrease in the angle of internal friction (one degree) when compared to an unreinforced soil sample (Drescher and Newcomb 1994). The test also demonstrated an increase in strain at failure. The failure of such a composite material results primarily from the failure of the soil.

Long (1990) also presented a stress-strain curve for sands reinforced with flattened tire elements (cut treads laid flat) (Figure 2.25). This hypothetical stress-strain curve corresponds to a material

having a low initial modulus and greater strength at failure when compared to an equivalent unreinforced sand. The postulated strength behaviour, however, does not reflect the most common case in which whole tires or treads are placed on edge. The behaviour of reinforced soils with whole tires is highly complex and is dependent on many factors, such as:

- tire mat configuration;
- tire mat attachment scheme;
- tire parts used;
- the effect of soil-tire friction;
- tire anchoring effects;
- type of soil used;
- soil strength characteristics;
- geometry of the reinforced soil structure.

Under low confining stresses, the composite system will usually fail by pull-out of the reinforcing tire mat. At higher confining stresses, failure will occur by breakage of either the individual tire elements or the attachments, whichever is the weakest. Quantitative data on shear strength of tire reinforced soils is lacking.

2.15.2 Pull-Out Resistance of Tires

Full-scale tensile tests on various tire parts including sidewall laid flat, treads on edge, treads laid flat were conducted by the Laboratoire Central des Ponts et Chaussées (Long 1990). The principal components of the pull-out test set-up are presented in Figure 2.26. The testing program involved placing the various tire parts in either a linear configuration or mat form embedded in soil. Six embankments were constructed and varied in height from 1 m to 2m. Table 2.6 summarize some of the results obtained during testing. Typical pull-out force-displacement curves for the three elementary tire units are presented in Figure 2.27. A synthesis of the major findings is presented below.

In general, the pull-out force-displacement curves indicated a steep gradient near the origin at the point of origin and had a subsequent well defined linear component (Figure 2.27). The following secant tensile moduli (defined by ASTM D 4595-86 as the ratio of change in force per unit width to a change in strain between two points on a force per unit width strain curve) were reported:

- 11 to 90 kN/m for treads laid flat with an average reported value of 40 kN/m;
- 17 to 115 kN/m for sidewalls with an average reported value of 43 kN/m;
- 12 to 105 kN/m for treads only placed on edge with an average reported value of 30 kN/m.

The maximum load required to pull-out treads placed on edge and treads laid flat, embedded at a depth of 1 m, is approximately the same for a single tire element, roughly 30 kN (Figure 2.27); the maximum pull-out resistance for sidewalls was reduced to 20 kN. The corresponding displacement for sidewalls and flattened treads is around 0.09 m, while the treads placed on edge gave a much greater deformation of 0.18 m at failure.

The deformation characteristics of tire reinforcement were dependent on which part of the tire was utilized. The reported maximum reinforcement deformation was 0.10 m for flattened tread, 0.20 m for sidewall, and 0.30 m for treads placed on edge. Sidewall and treads placed on edge elongated in the direction of pull by transforming from a circular shape to a pronounced oval form. After rear displacement of a tire was observed, the deformed shape of the entire reinforcing tire element remained essentially constant.

In the case of linear reinforcement (two or three elementary tire elements tied behind one another, see Figure 2.10), a progressive failure was observed. The first tire element would fail, followed by the second and finally by the third element. After the failure of a tire element, it would slide along the soil while engaging the next reinforcing element. This characteristic behaviour increased the amount of displacement required to fully mobilize the pull-out resistance. The

maximum pull-out resistance would also increase with the number of tire elements used. The maximum reported force necessary for pull-out including the corresponding displacements are given in Table 2.6, in terms of each type of tire reinforcement employed. However, as the vertical surcharge is increased, there is rupture of either the attachment or the tire element at the point of junction.

The pull-out tests on reinforcing tire mats resulted in rupture of the attachment in most cases, and therefore, a complete soil-tire reinforcement behaviour was not reported. However, tire reinforcing mats demonstrated higher anchorage capability. The majority of the tests involved the application of the pull-out force at one particular point; no attempt was made to distribute the force, even for tire mats composed of two to three frontal elements.

The tire pull-out resistance is a function of several soil-tire interaction mechanisms:

- soil-tire interface friction;
- passive soil resistance against any frontal elements;
- soil to soil friction in the annulus along the two horizontal shear surfaces;

The direct mobilization of soil to soil friction by the reinforcement makes it possible to use earth fill of inferior quality. However, pull-out resistance of tire reinforcements embedded in cohesive soils have not been reported. The above mentioned tests were all performed in good quality sand (Long 1990). In addition, no attempt was made to model the soil-tire interaction behaviour.

2.15.3 Bearing Capacity

A full scale bearing capacity test was conducted by the Laboratoire Central des Ponts et Chaussées (Long 1993) on a massive “pneusol” structure. This structure was composed of 4 layers of commercial tires ranging from 1.1 to 1.2 m in diameter with corresponding thickness between

0.20 and 0.25 m. Each tire layer was covered with 0.10 m thick sand layer, except the top tire layer. This was overlaid first with a geotextile sheet and then covered with 0.50 m of sand. The composite tire-soil structure had a depth of 1.8 m and covered an area of approximately 140 m². This structure was confined within an excavated area. The loading plate was circular with a surface area of 2 m² (1.6 m in diameter) and a thickness of 0.20 m. It was composed of reinforced concrete with a steel circular plate 1 m in diameter and 30 mm thick placed in the center at the top with another 10 mm thick steel plate covering the entire bottom to ensure a smooth contact with the foundation soil. The corresponding pressure required to push the plate to a depth of 15% of its width, based upon the corrected settlement curve, was defined as the bearing capacity (Long 1993). Two tests were performed and the corresponding bearing capacities, based upon the above deformation criteria, were 247 and 262 kPa, respectively. However, these values are about half of those observed from tests performed on sand alone, employing the same testing method and loading apparatus. The apparent decrease in bearing capacity may be attributed to the compression of the void space within the tires, although this was not stated.

Table 2.1. Summary of reinforcement and face panel details for selected MSE wall systems (Elias et al. 1996).

<u>System Name</u>	<u>Reinforcement Detail</u>	<u>Typical Face Panel Detail¹</u>
Reinforced Earth The Reinforced Earth Company 2010 Corporate Ridge McLean, VA 22102	Galvanized Ribbed Steel Strips: 4 mm thick, 50 mm wide. Epoxy-coated strips also available.	Facing panels are cruciform shaped precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
VSL Retained Earth VSL Corporation, 2840 Plaza Place Raleigh, NC 27612	Rectangular grid of W11 or W20 plain steel bars, 610 x 150 mm grid. Each mesh may have 4, 5 or 6 longitudinal bars. Epoxy-coated meshes also available.	Hexagonal and square precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
Mechanically Stabilized Embankment Dept. of Transportation, Division of Engineering Services 5900 Folsom Blvd. P.O. Box 19128 Sacramento, CA 95819	Rectangular grid, nine 9.5 mm diameter plain steel bars on 610 x 150 mm grid. Two bar mats per panel (connected to the panel at four points).	Precast concrete; rectangular 3.81 m long, 610 mm high, 200 mm thick.
Georgia Stabilized Embankment Dept. of Transportation, State of Georgia No. 2 Capitol Square Atlanta, GA 30334-1002	Rectangular grid of five 9.5 mm plain steel bars on 610 x 150 mm grid 4 bar mats per panel	Precast concrete panel; rectangular 1.83 m wide, 1.22 m high, 200 mm thick with offsets for interlocking.
Hilfiker Retaining Wall Hilfiker Retaining Walls, P.O. Drawer L Eureka, CA 95501	Welded steel wire mesh, grid 50 x 150 mm of W4.5 x W3.5, W9.5 x W4, W9.5 x W4, and W12 x W5 in 2.43 m wide mats.	Welded steel wire mesh, wrap around with additional backing mat 6.35 mm wire screen at the soil face (with geotextile or shotcrete, if desired).
Reinforced Soil Embankment Hilfiker Retaining Walls, P.O. Drawer L Eureka, CA 95501	15 cm x 61 cm welded wire mesh: W9.5 to W20 - 8.8 to 12.8 mm diameter.	Precast concrete unit 3.8 m long, 610 mm high.
ISOGRID Neel Co. 6520 Deepford Street Springfield, VA 22150	Rectangular grid of W11 x W11 5 bars per grid	Diamond shaped precast concrete units, 1.5 by 2.5 m, 140 mm thick.
GENESIS Tensor Earth Technologies, Inc. 5775-B Glenridge Drive, Ste 400 Lakeside Center Atlanta, GA 30328	HDPE Geogrid	Keystone [®] Standard unit (200 mm high by 40 mm long face, 600 mm nominal depth); OR Keystone International Compact [®] (200 mm high by 450 mm long face, 300 mm nominal depth).
PYRAMID The Reinforced Earth [®] Company 2010 Corporate Ridge McLean, VA 22102	Galvanized WWM, size varies with design requirements or Grid of PVC coated, Polyester yarn (Matrex Geogrid)	Pyramid [®] unit (200 mm high by 400 mm long face, 250 mm nominal depth)
Maccaferri Terramesh System Maccaferri Gabions, Inc. 43A Governor Lane Blvd. Williamsport, MD 21795	Continuous sheets of galvanized double twisted woven wire mesh with PVC coating.	Rock filled gabion baskets laced to reinforcement.
Strengthened Earth Gifford-Hill & Co. 2515 McKinney Ave. Dallas, Texas 75201	Rectangular grid, W7, W9.5 and W14, transverse bars at 230 and 450 mm.	Precast concrete units, rectangular or wing shaped 1.82 m x 2.13 m x 140 mm.

¹Additional facing types are possible with most systems.

Table 2.2. Comparison of reinforcement soil systems (Mitchell and Christopher 1990).

IMPROVED EMBANKMENT TYPE APPLICATION		REINFORCEMENT TYPE	ALLOWABLE SLOPE ANGLE 30° 60° 90°	RECOMMENDED SOIL TYPE Clay Silty Sand Gravel .002 .02 .2 2mm	STRESS TRANSFER MECHANISM		REINFORCEMENT MATERIAL		EXTENSIBILITY	PROPRIETARY SYSTEM / PRODUCT NAMES
					Surface friction	Passive Resistance	Metall	Non-Metall		
	STRIP	Smooth	I	I	•		•		I	Reinforced Earth
		Ribbed	I	I	•		•		I	Reinforced Earth
	GRID		I	I		•	•		•	Paratek
			I	I		•	•		•	VSL MSE, GAE, MSE, and Welded Wire Wall
			I	I		•	•		•	Masconfort Gabion
	SHEET		I	I	•		•		I	Tensar, Mirafi and Tenax Geogrids
	BENT ROD		I	I		•	•		I	Geotextiles
	PIER		I	I	•		•		I	Anchored Earth
IN SITU GROUND IMPROVEMENT APPLICATION	FLEXIBLE, SMALL DIAMETER NAILS		I	IN SITU SOILS	•		•		•	
	RIGID, LARGE DIAMETER PILES		I	IN SITU SOILS		•	•		•	

*Based on stress transfer between soil reinforcement. Other Criteria may preclude use of soils for specific applications.

Table 2.3. Tire constituents by weight (Humpstone et al. 1972)

Tire Constituents	Weight (kg)	Weight (%)
Fabric	1.41	10
Bead Wire	0.46	4
Rubber Compound	9.80	86
Total	11.67	100

Table 2.4. Typical composition of tire rubber (Williams et al. 1990)

Component	Weight (kg)	Weight (%)
Rubber Polymer (SBR)	6.088	62.1
Carbon Black	3.039	31.0
Extender Oil	0.186	1.9
Zinc Oxide	0.186	1.9
Stearic Acid	0.118	1.2
Sulphur	0.107	1.1
Accelerator	0.069	0.7
Total	$\cong$ 9.80	99.9

Table 2.5. Potential facing materials for vertical structures using tire reinforcement (modified after Jones 1988).

Materials	Potential Advantages	Potential Disadvantages	Other Considerations
Aluminum	Durable Easily transported and handled Very versatile for erection purposes Can be cast	Requires experience in extrusion technique for manufacturer Aesthetics (surface finish may distort) May encourage electrolytic action	Little used to date
Steel, galvanized	Relatively inexpensive Easily transported, rate of production high Easily shaped Good for industrial structures	Durability Aesthetics	Normally used with industrial environment
Steel, stainless	Durable Easily transport, rate of production high Easily shaped	Special surface treatments increases cost significantly Can be expensive Thin sections lead to great flexibility	
Steel, weather resistant	Relatively inexpensive Interesting architectural finish	Overlapping thin plates corrode	
Segmental precast concrete	Relatively inexpensive Industrial technique well established Durability, Good finish Flexibility to absorb differential movement	Difficult to reinforce Potential staining problem from reinforcement Fixing to tire may be difficult Shape of unit is critical to production technique	Used in tire reinforced retaining walls
Cast-in -place concrete, shotcrete	Durability, Good finish Relatively inexpensive Particularly suited for large construction where cost of formwork is low Individual units able to adsorb differential movement	Care required over durability, air entrainment may be necessary Shape of unit critical to rate of production and cost May be difficult to have even finish	common
Concrete prestressed	Readily available, if adopted from existing construction elements. e.g. Double Tee Very durable Easily transported and erected Good aesthetics	Best suited to small and medium-height structures	Most durable form of facing

Table 2.5. Continued.

Materials	Potential Advantages	Potential Disadvantages	Other Considerations
Brick or masonry	Good material common to industry Good aesthetics Very durable	Produces a stiff facing unsuitable for soft foundations or where differential movements are likely Not particularly suited to tall structures	Very suitable for small structures
Fabric/textile/geogrids	Very lightweight Very flexible Good for temporary structures Good for military structures	Aesthetics Durability, susceptible to UV, fire, vandals and rodents	Covering face with soil or vegetation may eliminate problems
Glass-reinforced plastic	Durable and strong, very resistant to impact Finish is good Very light and easily shaped Rate of unit output high	Colour affected by ultra-violet light Susceptible to damage from intense heat	Used in Maintenance
Plastics (PVC, ABS)	Very light-weight, strong Finish good Easily shaped Rate of unit output high and easily transported	Melts at relatively low temperatures Fairly new materials, life expectancy unknown Susceptible to UV light Creep characteristics unknown Durability	Used in Maintenance
Gabions	Low cost Easy installation Flexible design Good drainage (depending on soil) Provides increase stability	Aesthetics, Uneven Surface Exposed backfill material Erosion of backfill may occur Shorter life span due to corrosion of wire mesh susceptible to vandalism	
Timber (planks)	Readily available Particularly suited for short life or temporary structures Suitable for third world countries	Aesthetics Durability Susceptible to termites	
Tires	Readily available Decrease overall cost of the structure Easily transported Strong Lightweight Easily handled by one man	Aesthetics Durability, susceptible to UV degradation, vandalism	A potential environmental solution

Table 2.6. Some tire pull-out test results (Long 1990).

	Tread on Edge						Sidewall			
										
Number of elements										
Maximum force (kN)	>44.3	>68.0	>44.4	>40.9	>38.2	>56.0	>43.0	54	>57.5	>33
Frontal displacements (mm)	>430	>260	>240	>200	>240	>135	>360	460	>385	>255
Traction for 100 mm displacement (kN)	26	45	21.5	26.2	21.2	49.5	18	25	22.5	20

	Tread Flattened			Tread on Edge			Sidewall		
									
Number of elements									
Maximum force (kN)	31	49	68	33.5	32	41	19.5	30	>33.5
Frontal displacements (mm)	120	290	440	170	320	620	70	400	>290
Traction for 100 mm displacement (kN)	30	39	36	25	19	16	19	20	25

Test results are for tire reinforcements under 1 m high surcharge

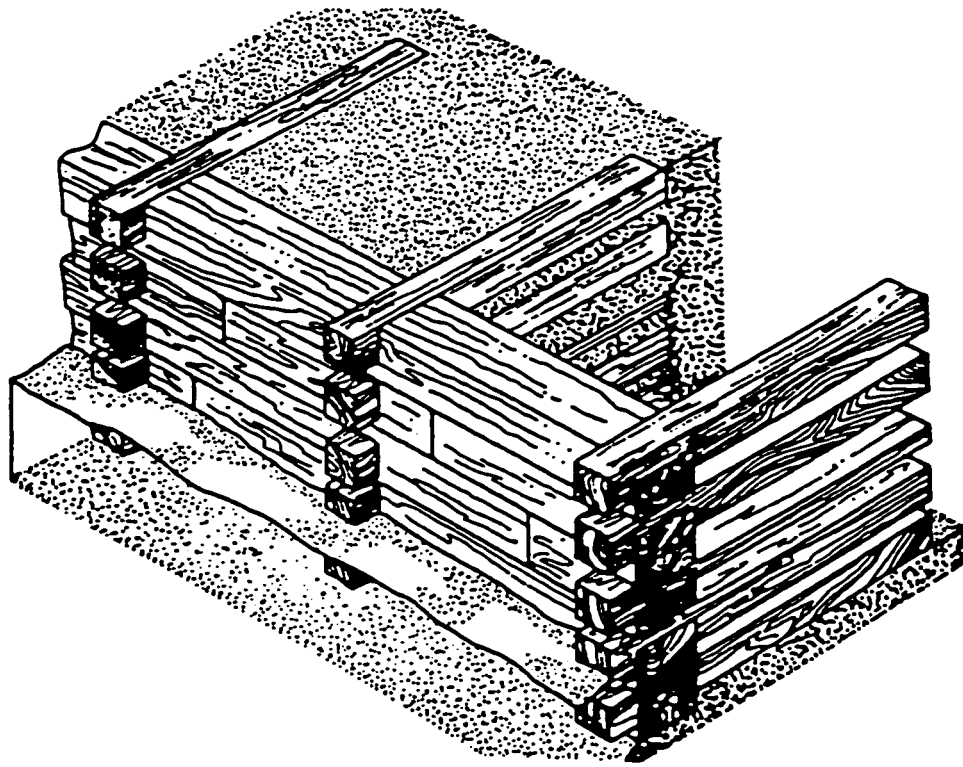


Figure 2.1. Timber wharf constructed by the first-century Roman Army (Jones 1985).

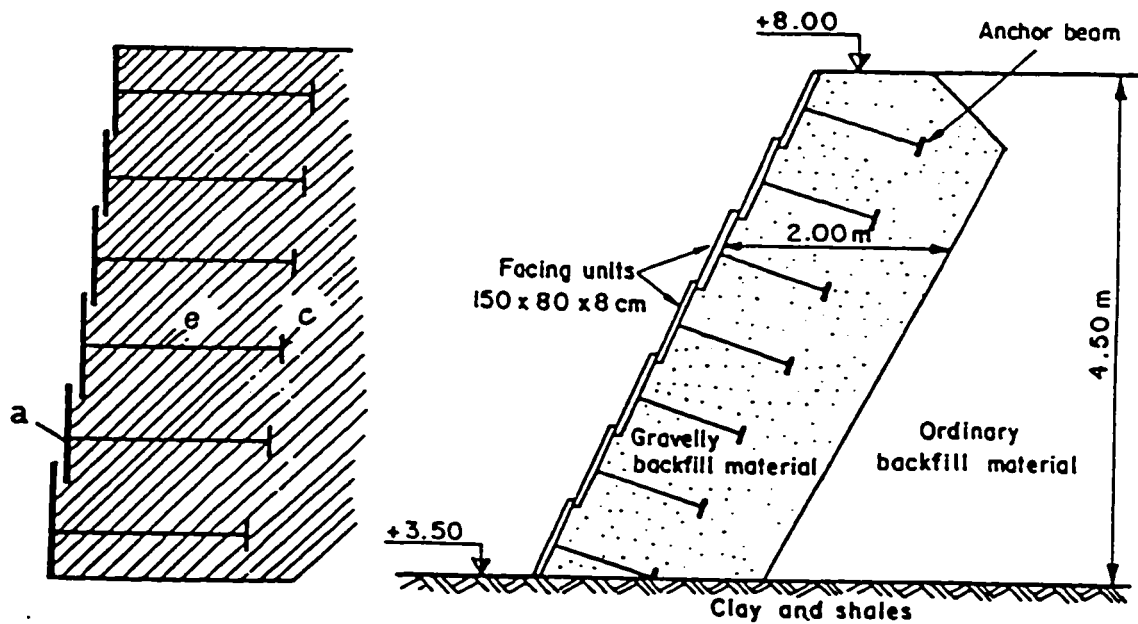


Figure 2.2. Coyne ladder wall system (Coyne 1945).

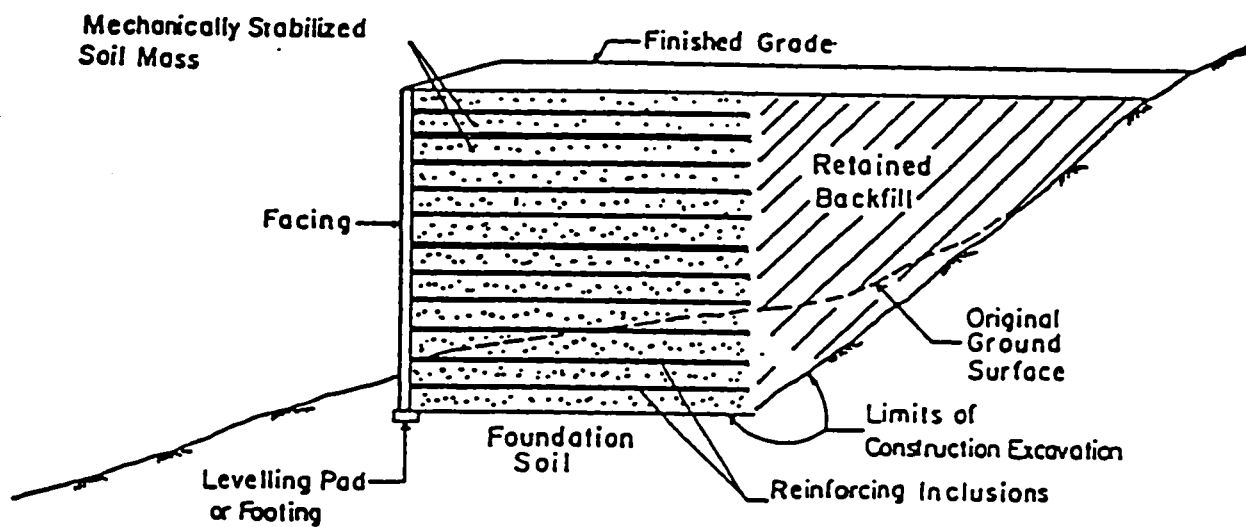
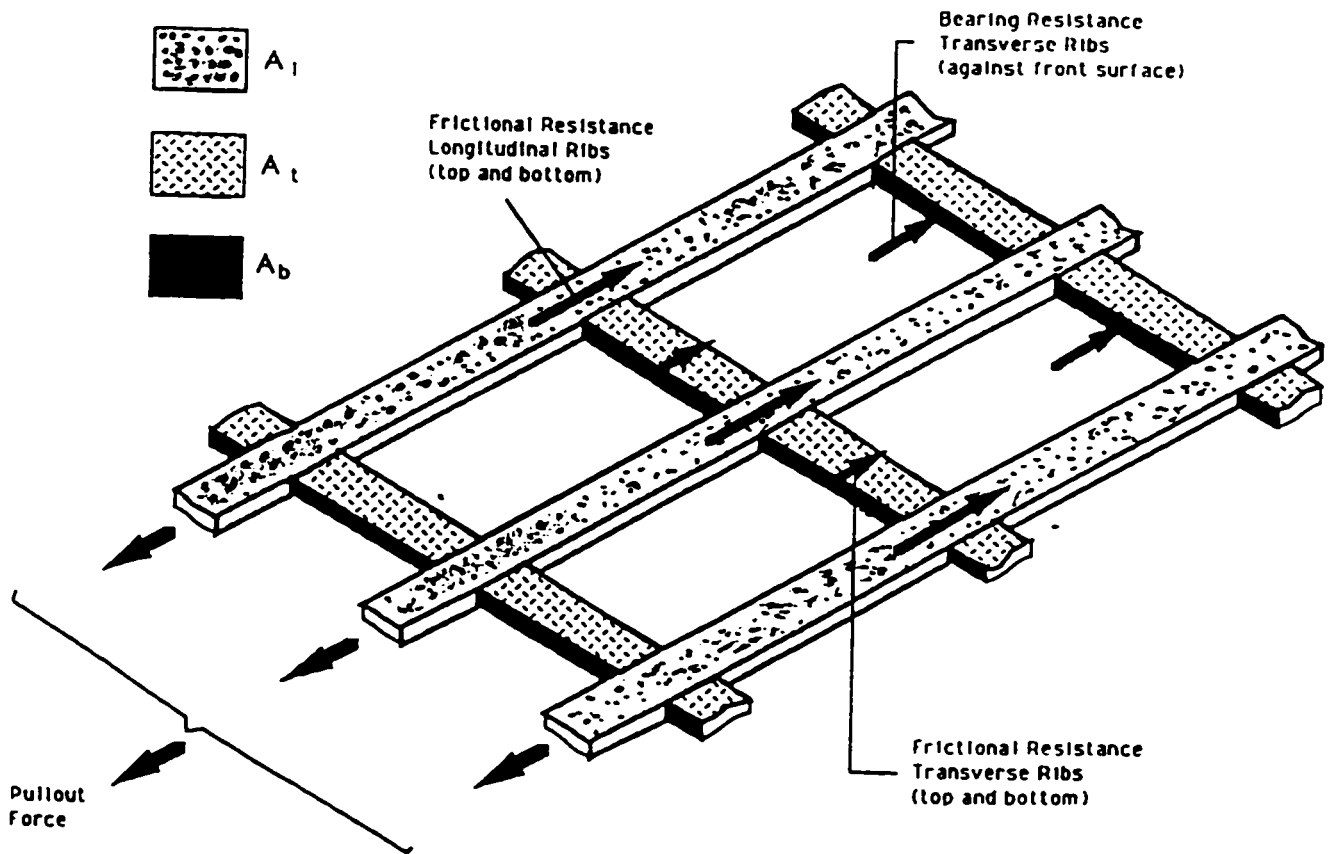


Figure 2.3. Principle components of a placed reinforced soil structure and its geotechnical environment (Mitchell and Christopher 1990).



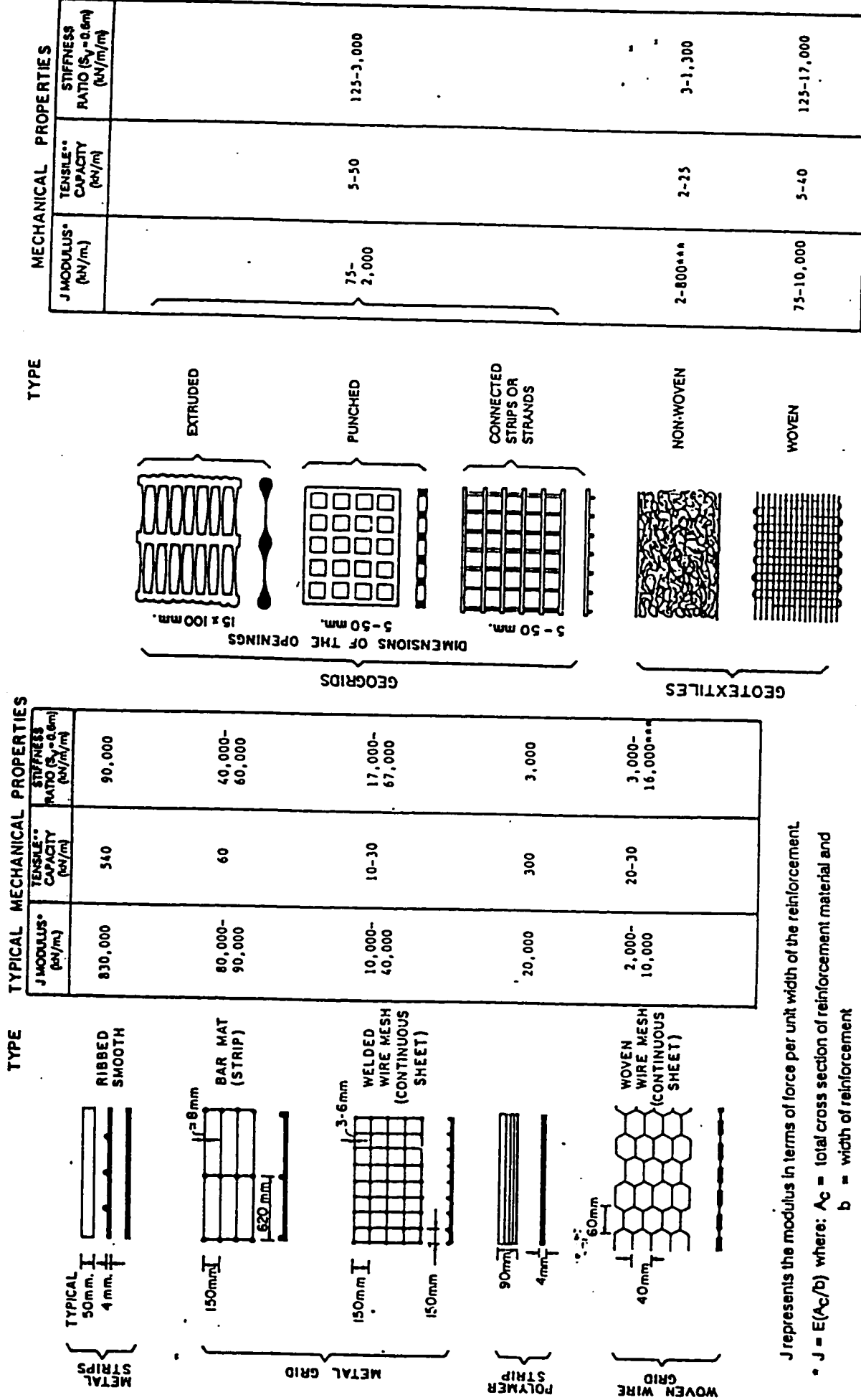
A_l = area covered by longitudinal ribs: frictional resistance

A_t = area covered by transverse ribs: frictional resistance

A_b = bearing area of transverse ribs: passive resistance

Figure 2.4. Components of resistance to pull-out force: stress transfer mechanisms (Wilson-Fahmy et al. 1994)

Figure 2.5. Mechanical properties of several different reinforcement systems (Mitchell and Christopher 1990)



*** CONFINED

J represents the modulus in terms of force per unit width of the reinforcement

* $J = E(A_c/b)$ where: A_c = total cross section of reinforcement material and

b = width of reinforcement

E = modulus of material

** Allowable values with no reduction for durability considerations

*** Confined

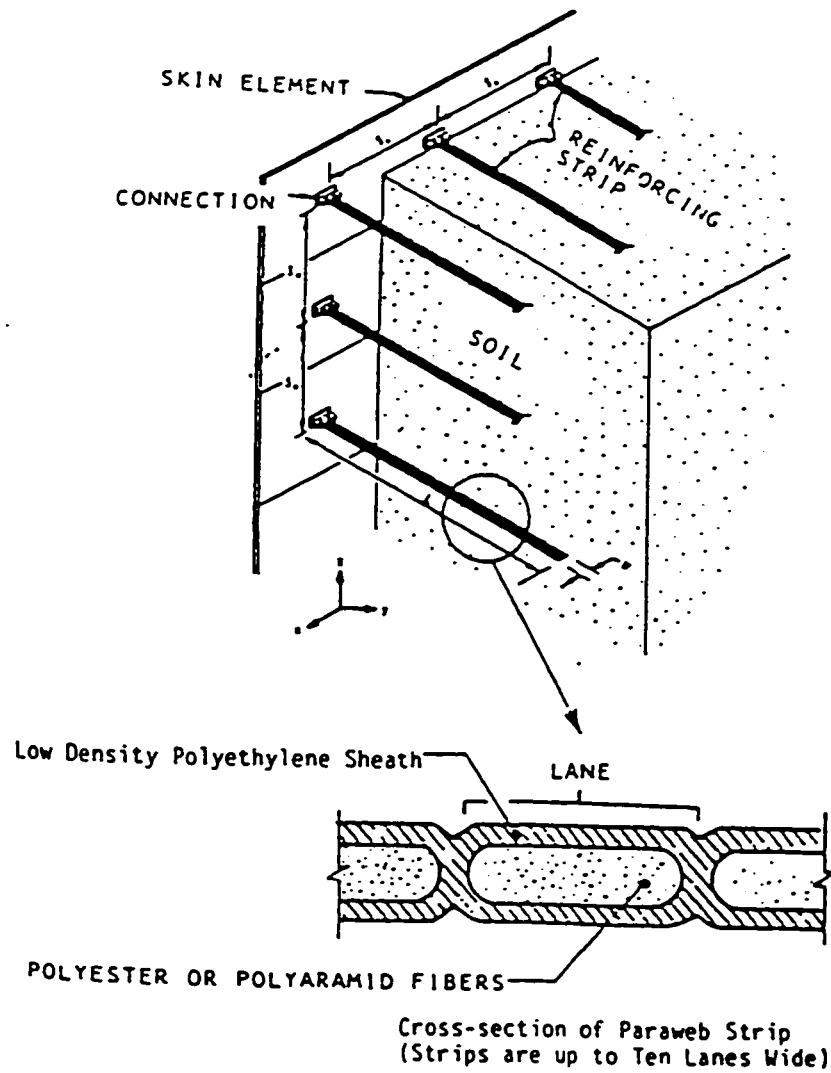


Figure 2.6. A schematic diagram of a polymeric strip reinforcement structure (Mitchell and Villet 1987).

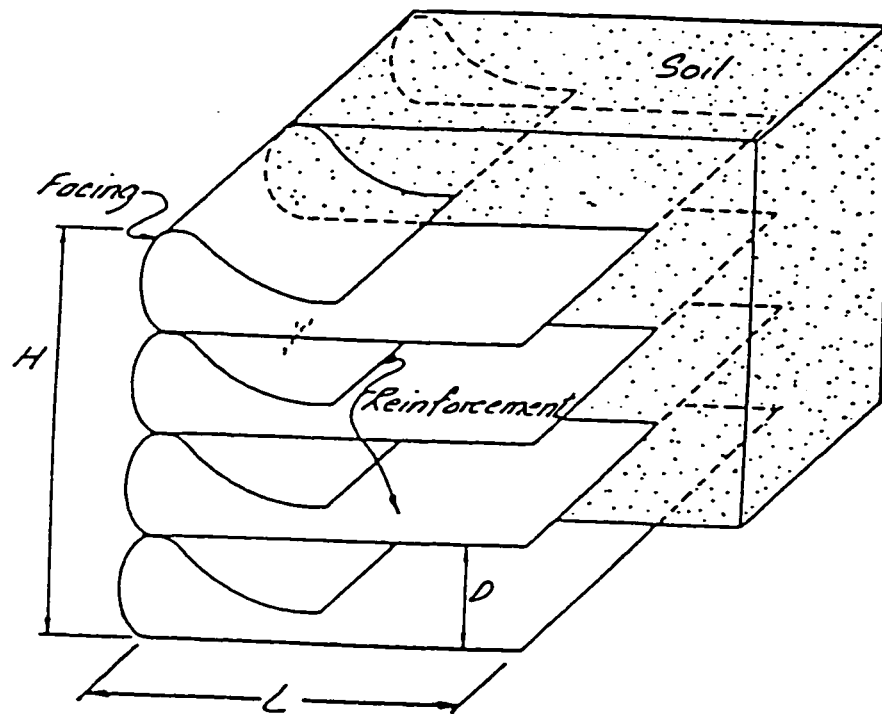


Figure 2.7. Reinforced soil system using wrap-around geotextile sheet reinforcement.

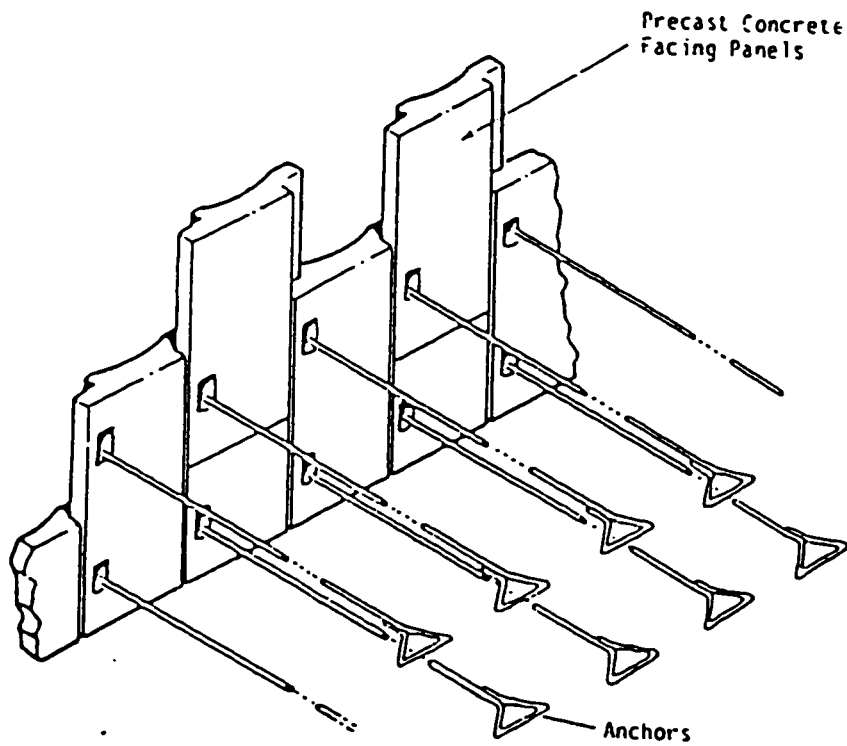
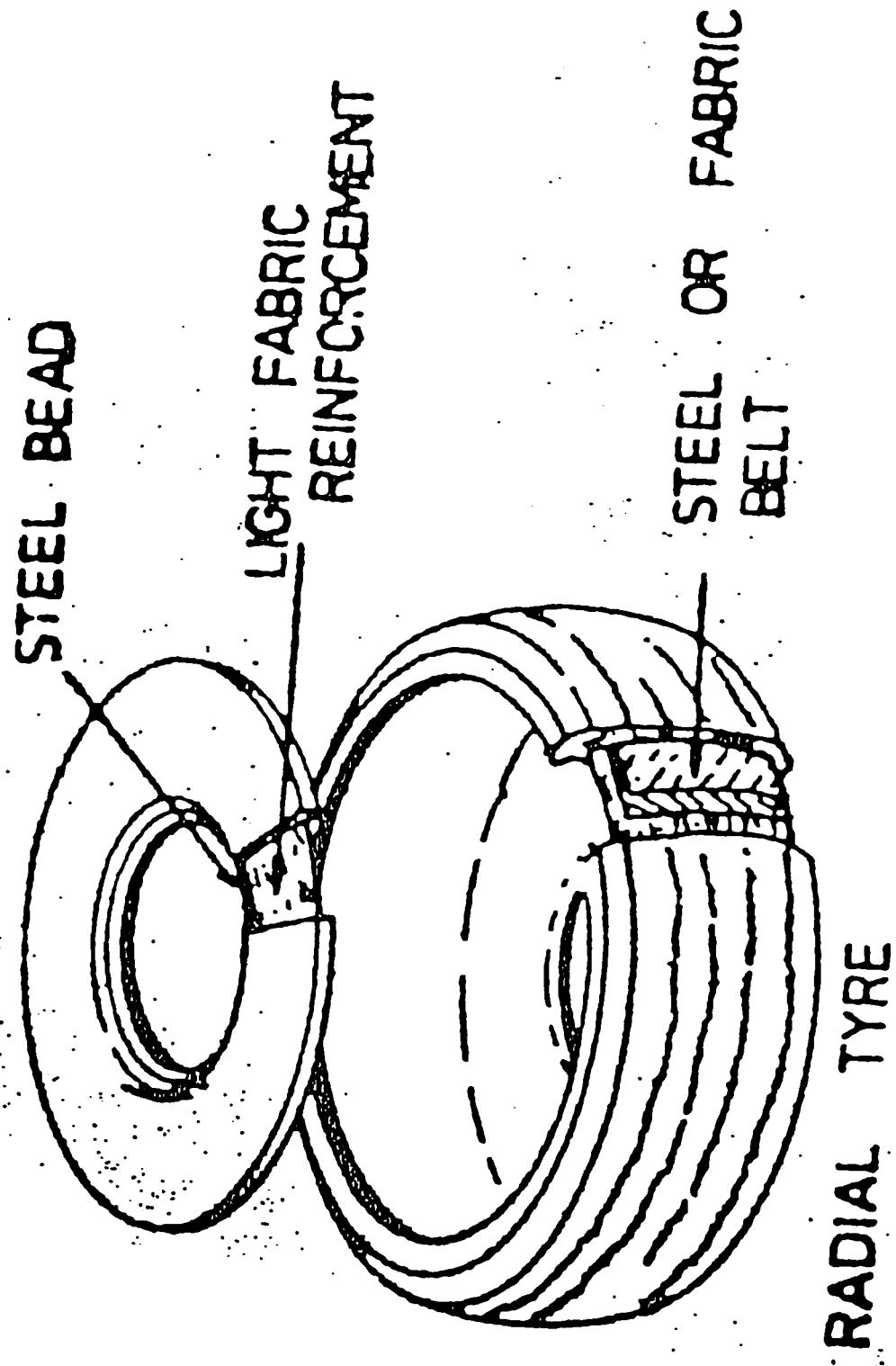


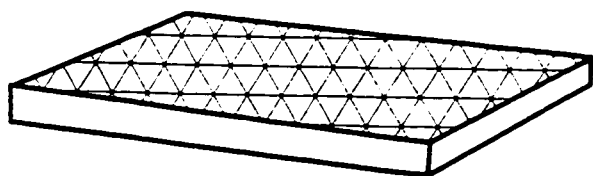
Figure 2.8. Schematic diagram of an Anchored Earth retaining wall system (Murray and Irwin 1981)

Figure 2.9. The different components of a radial tire (Dalton and Hoban 1982).





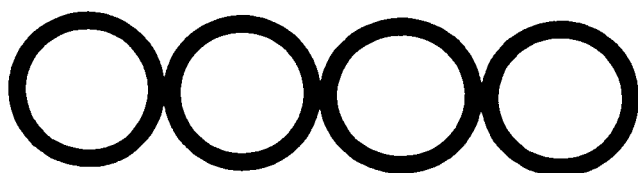
Sidewall Laid Flat



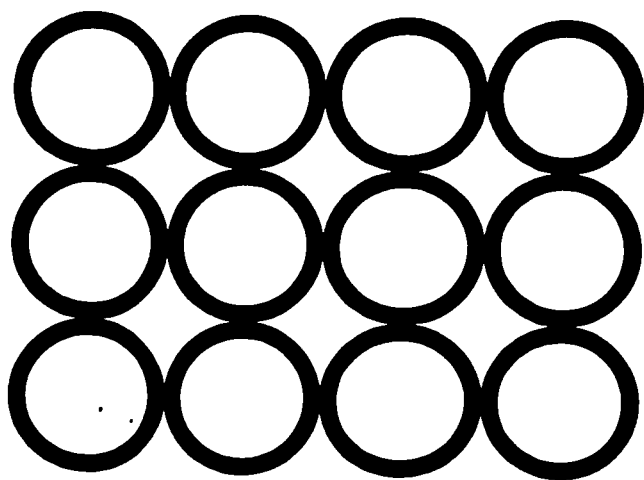
Tread Laid Flat



Tread Placed on Edge



Linear Configuration



Mat Configuration

Figure 2.10. Different tire reinforcement elements and possible configurations.

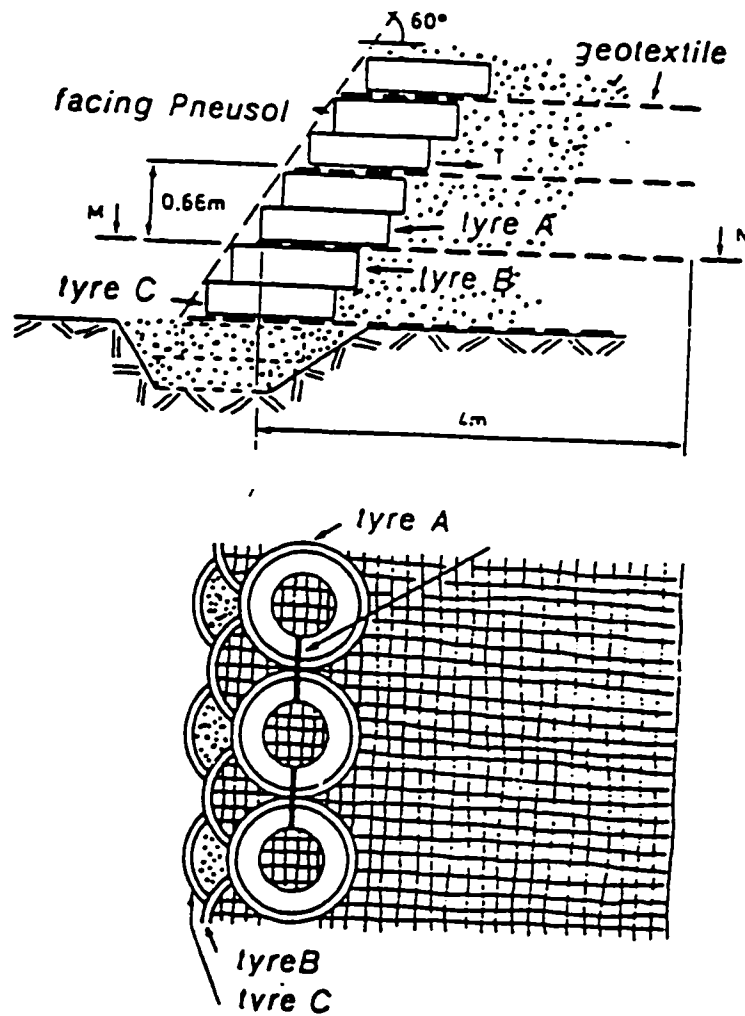


Figure 2.11. Combined tire facing and geotextile sheet reinforced structure constructed on road RN90 between Albertville and Moutiers (Gourc and Matichard 1994).

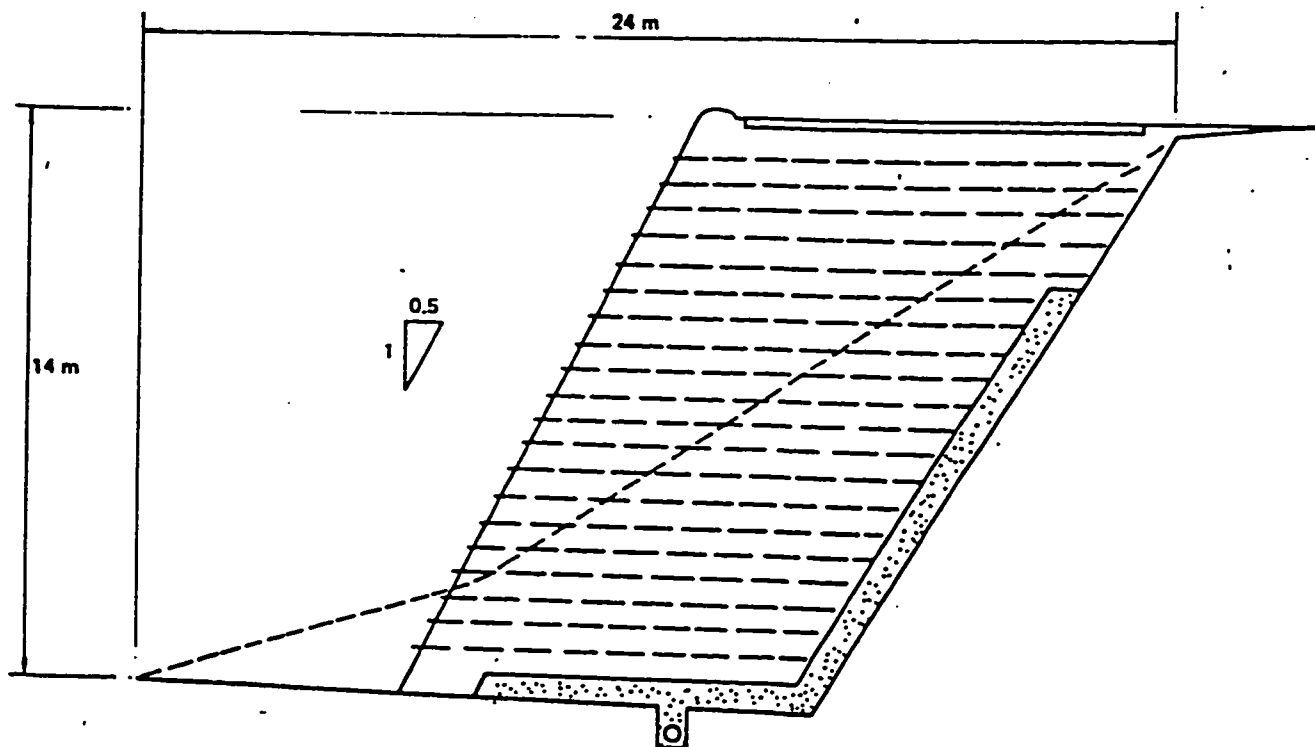


Figure 2.12. Sidewall tire reinforced embankment slope, near Santa-Cruz (Jones 1985).

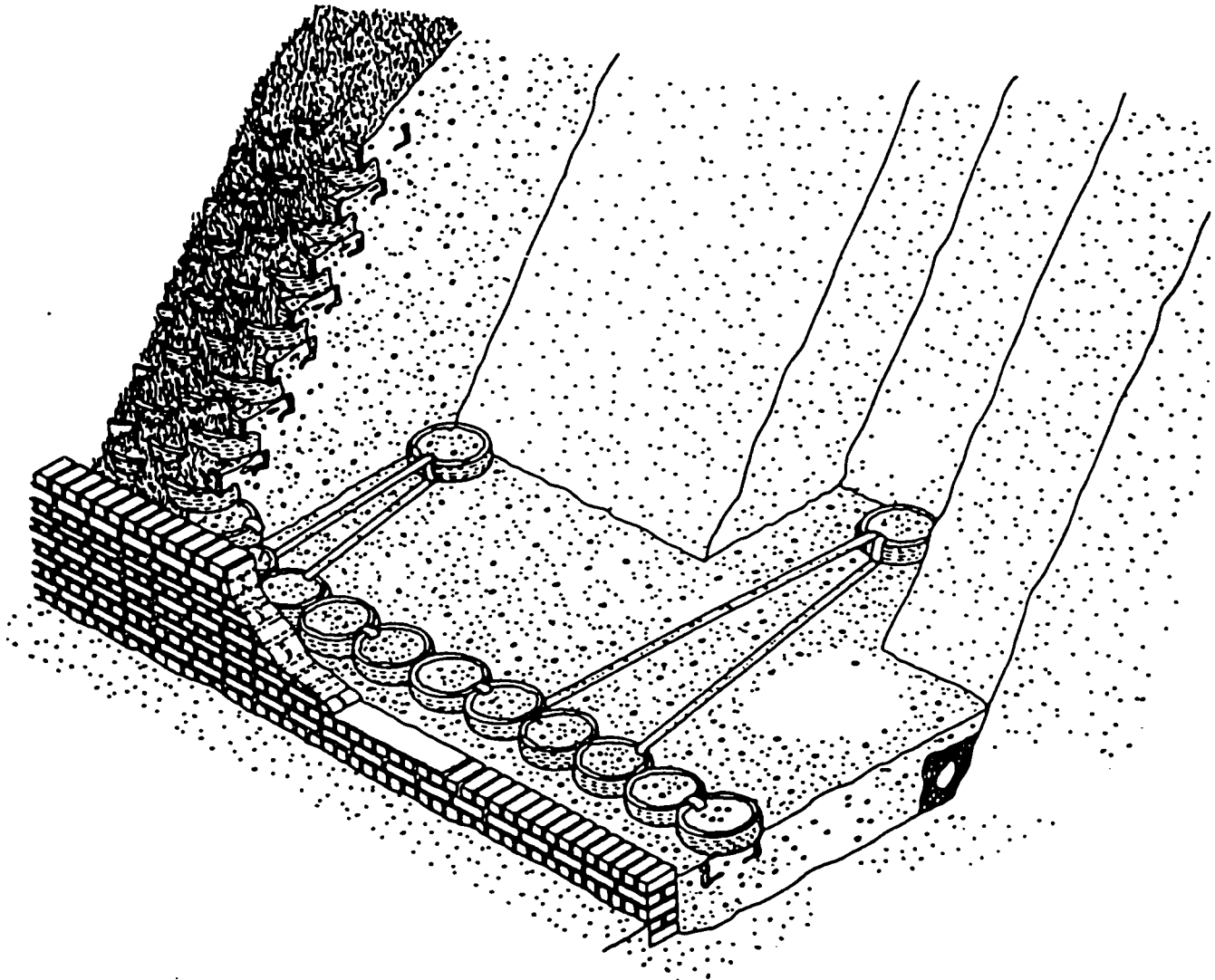
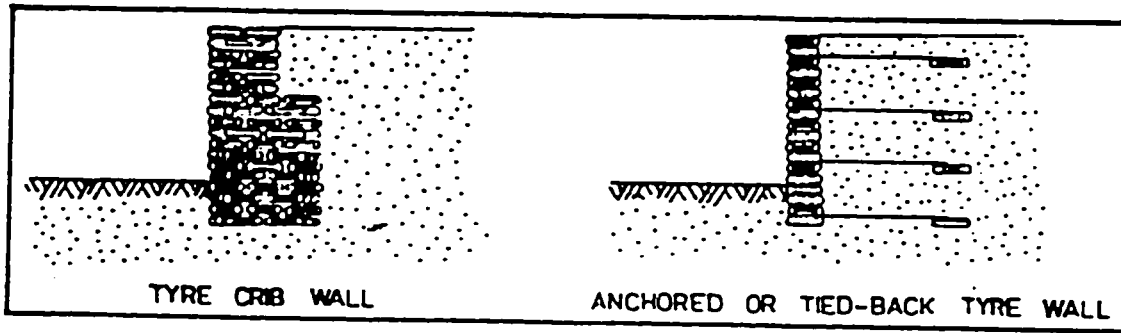


Figure 2.13. Anchored or tied-back tire wall and tire crib wall used in English tire projects (Dalton and Hoban 1982)

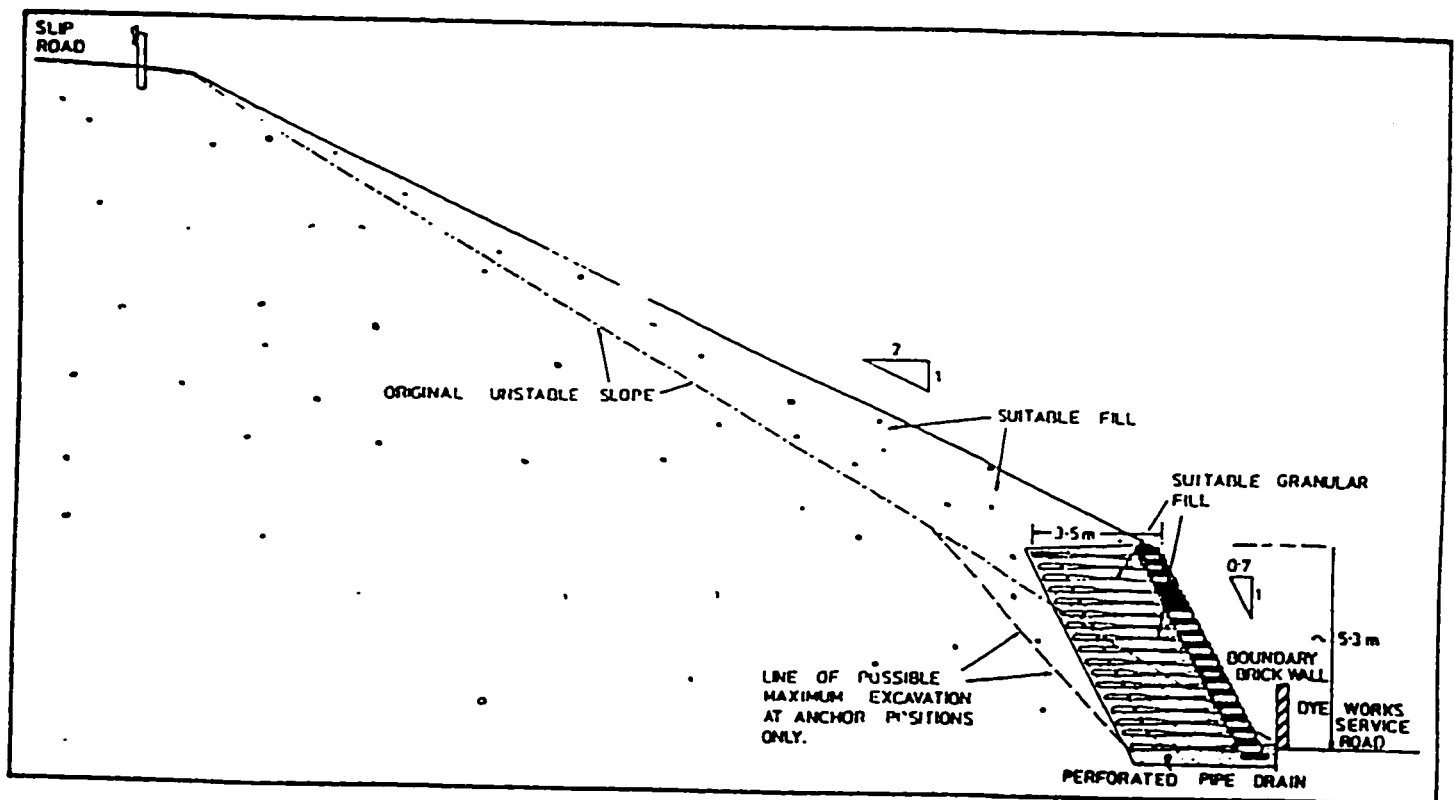
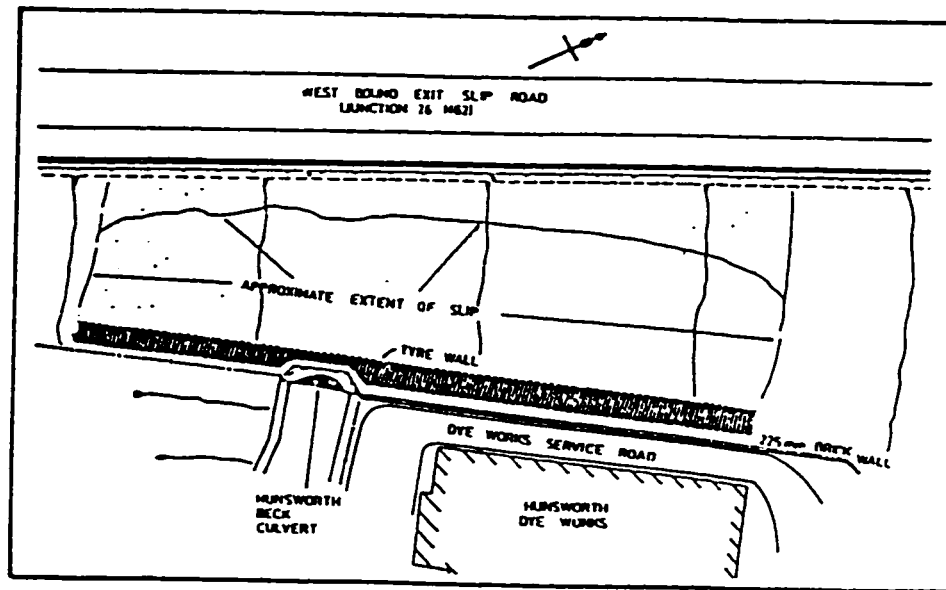


Figure 2.14. A schematic representation of the tire anchored wall at West Yorkshire, England (Dalton and Hoban 1982)



Figure 2.15. Experimental tire wall at Nancy in the Langres region (Long 1990).

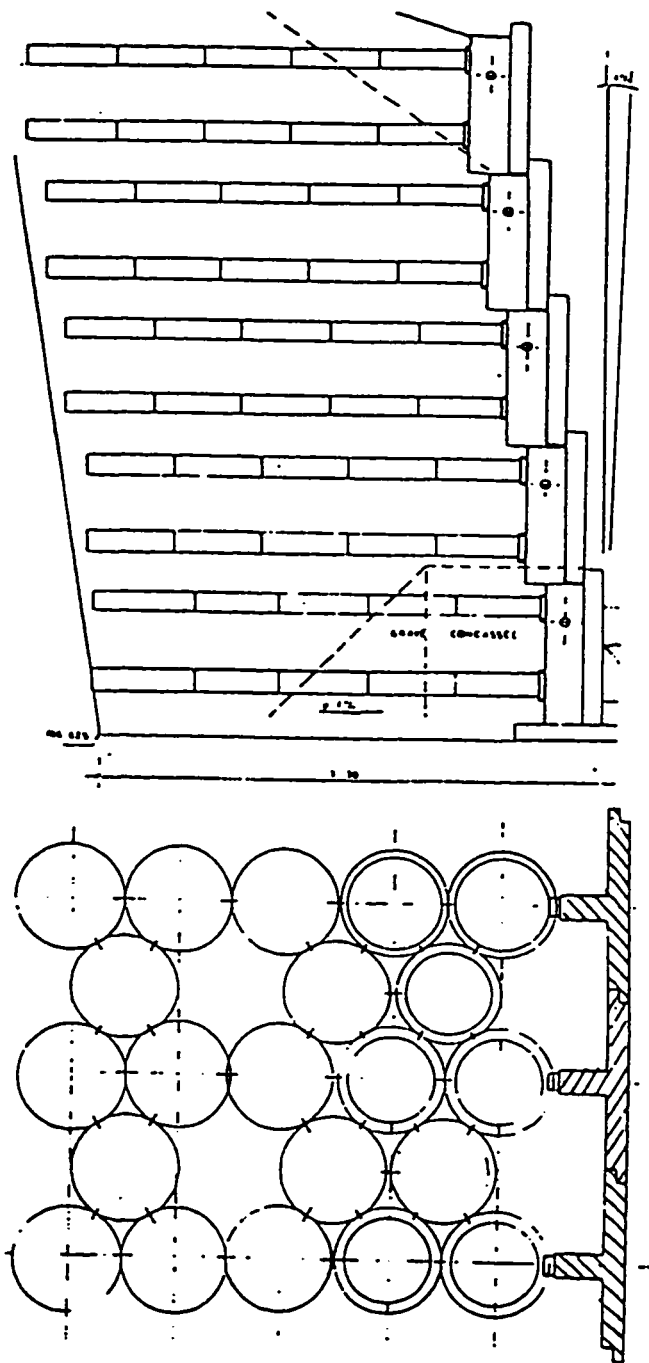
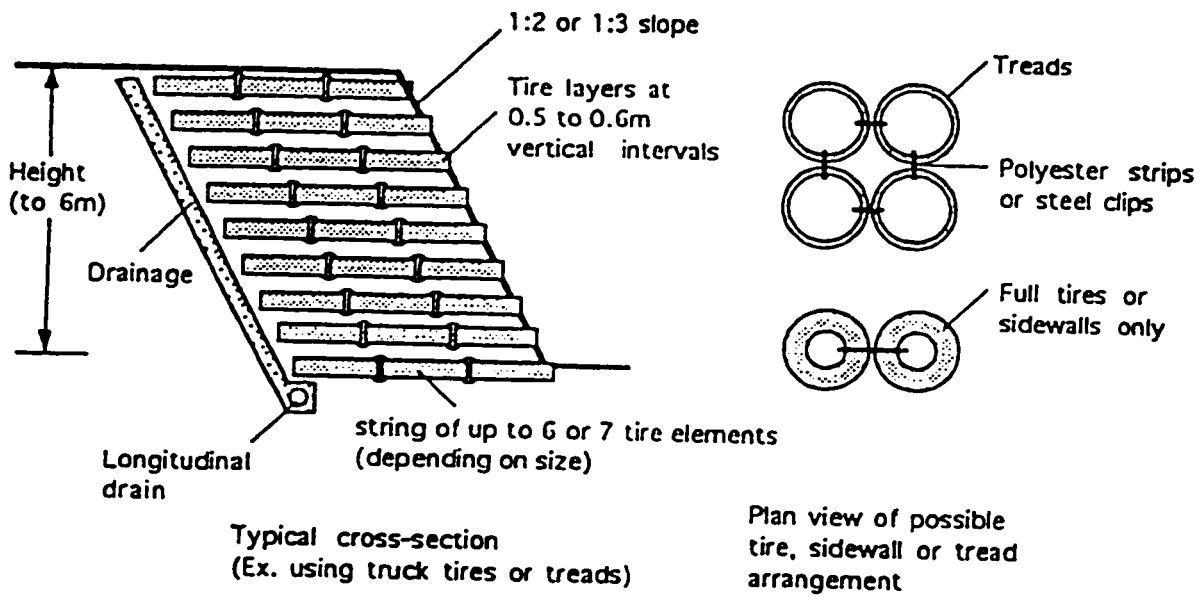


Figure 2.16. Experimental tire wall at Nancy in the Langres region showing the tire reinforcement and concrete facing (Long 1990).



(A)



(B)

Figure 2.17. Slope remediation using tire reinforcements: (A) typical cross-section showing all the essential elements (Hausmann 1990), (B) oblique frontal view showing reinforcing tires (Long 1990).



Figure 2.18. Slope protection from erosion forces (Long 1990).



Figure 2.19. Reduction of active earth pressures on a retaining structure (Long 1990).



Figure 2.20. Construction of a sound barrier in Switzerland (Long 1990).

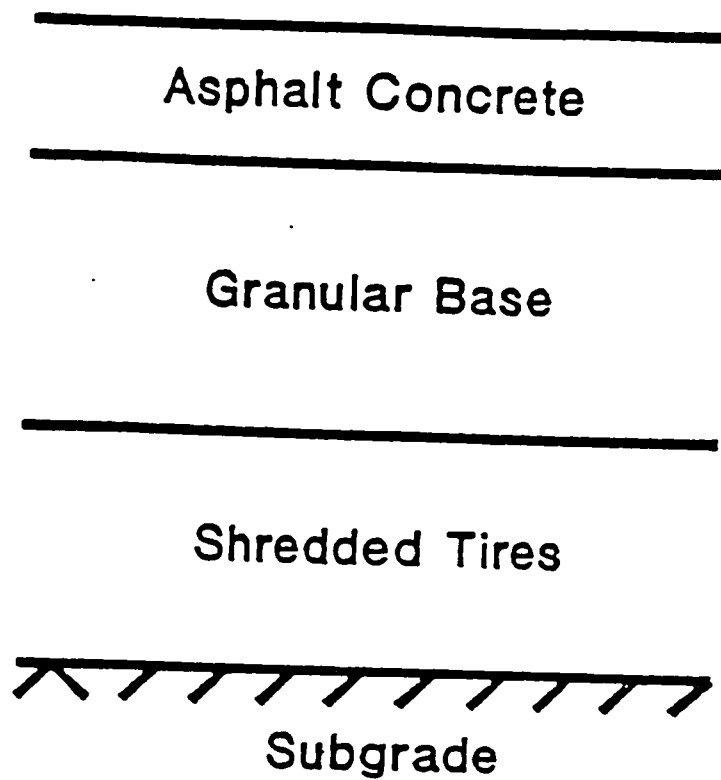


Figure 2.21. Typical four-layer pavement system using shredded tires as a lightweight fill
(Drescher and Newcomb 1994).

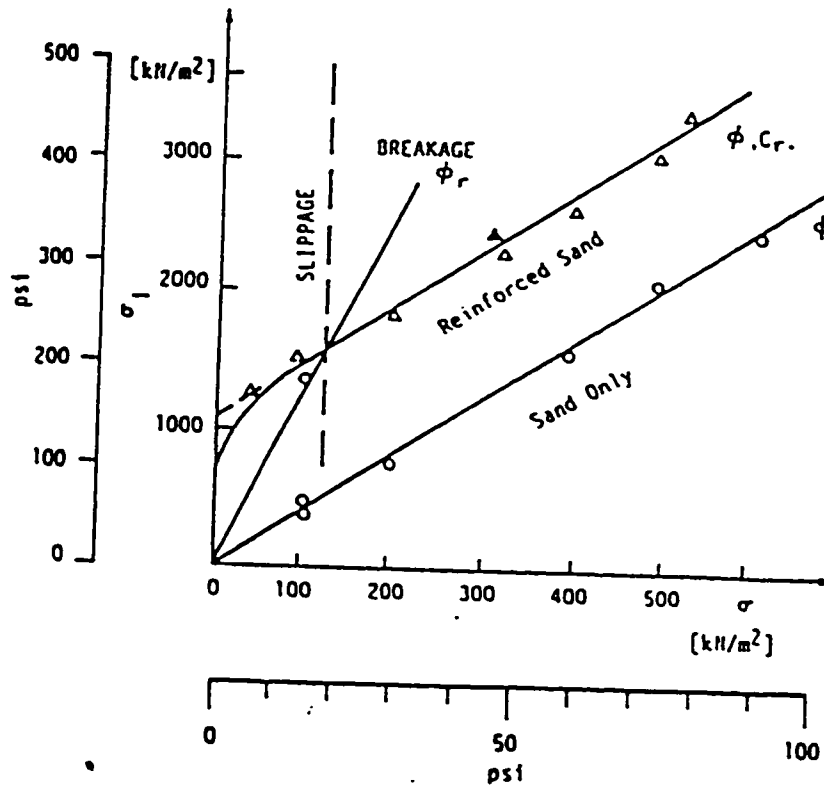


Figure 2.22. Strength envelopes for sand and reinforced sand (Schlosser and Long 1972).

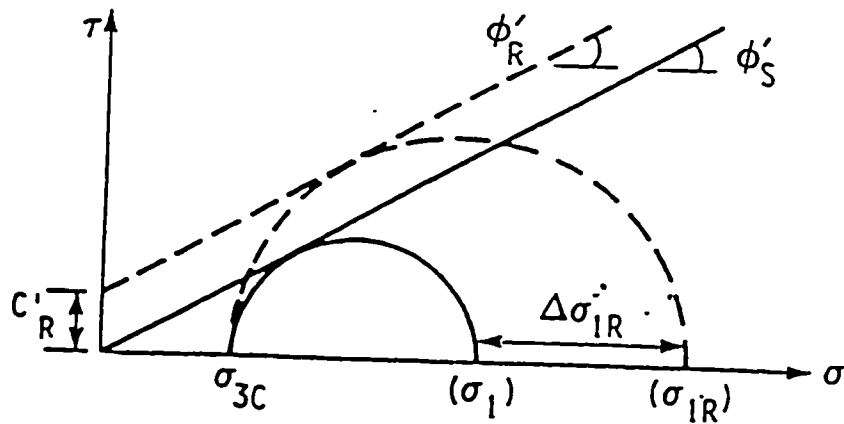


Figure 2.23. Apparent cohesion concept interpretation of strength increase due to reinforcement (Mitchell and Villet 1987).

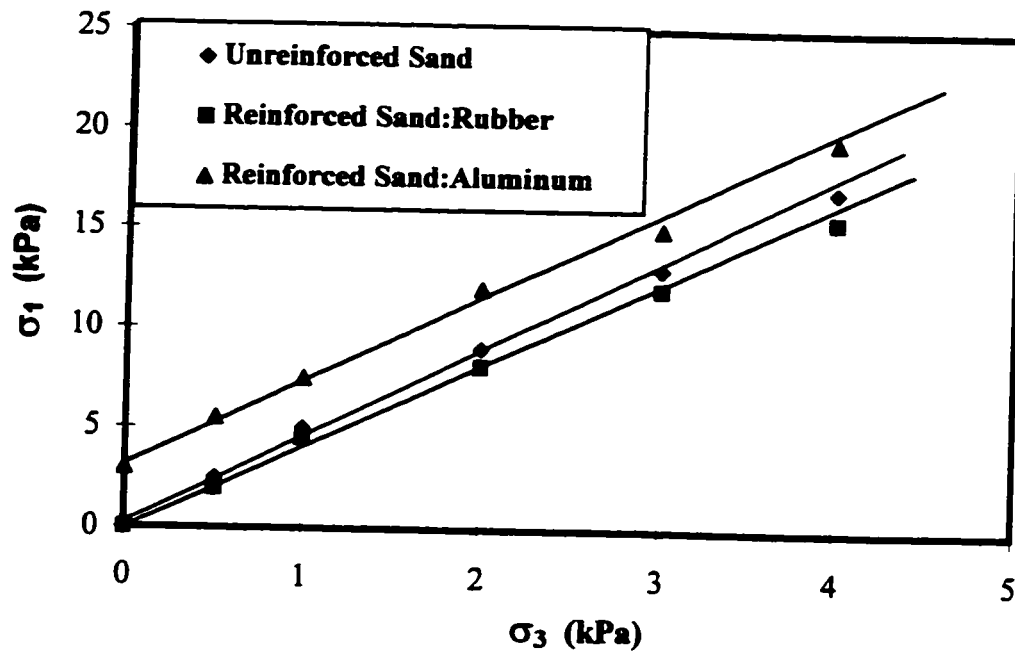


Figure 2.24. Failure envelopes for natural sand, reinforced with aluminum foil and latex rubber (Long 1990).

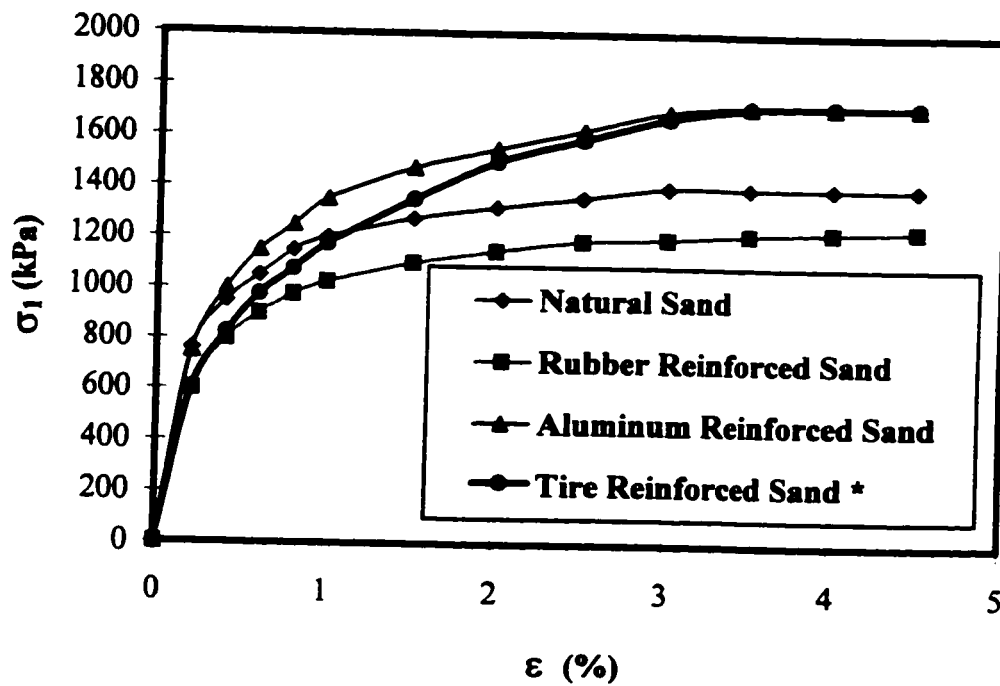


Figure 2.25. Stress-strain curves for natural sand, reinforced with aluminum, latex rubber, and tire (Long 1990). * Tire reinforced sand curve is hypothetical.

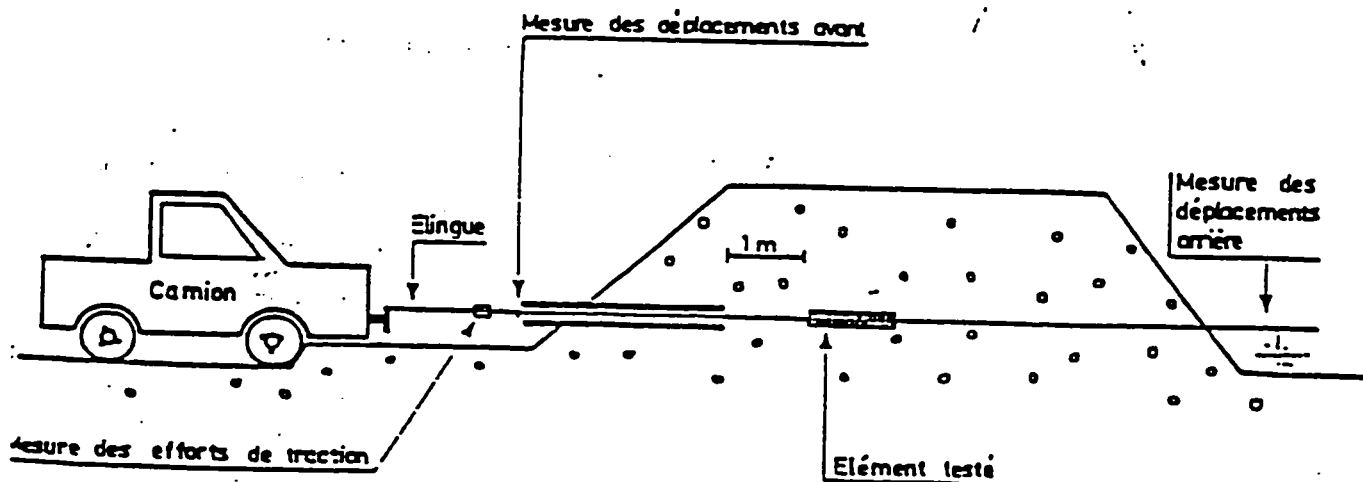


Figure 2.26. The principal components of the pull-out test setup (Long 1993).

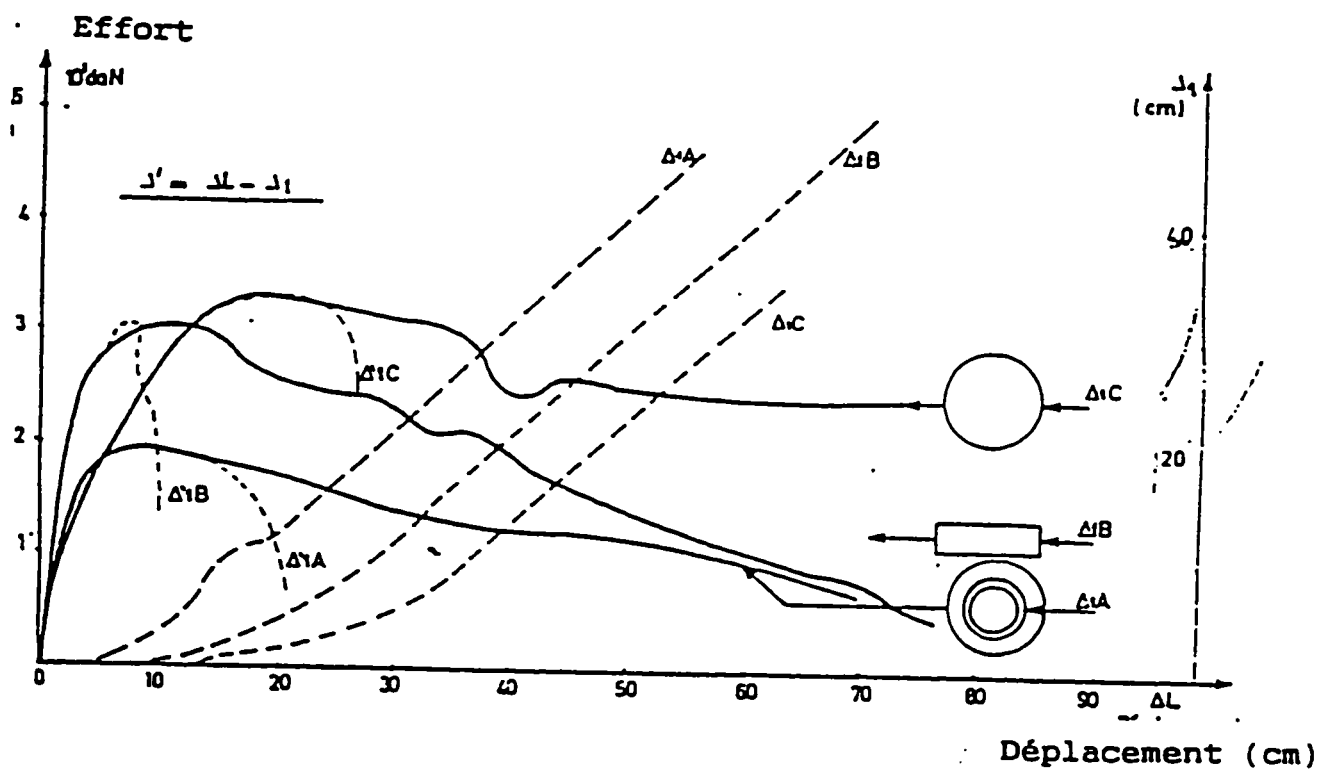


Figure 2.27. Typical pull-out force-displacement curves for the three elementary tire reinforcing units (Long 1993).

CHAPTER 3

MATERIAL PROPERTIES OF SOIL AND TIRE REINFORCEMENT

3.1 Introduction

The introduction of tire reinforcement in a soil, and the consequent lateral restraint alters the stress and strain characteristics of the soil when compared to an equivalent soil system without reinforcement. The presence of reinforcing elements results in a soil that displays improved stiffness and shear strength. The performance and behaviour of a reinforced soil structure is influenced by several factors. These factors include the type of reinforcement, geometrical distribution of reinforcement, soil classification, soil stress state, construction procedures, any external loading, environmental conditions and durability (Jones 1985; Mitchell and Villet 1987).

Important material properties of the reinforcement are the form (shape), surface attributes, dimensions, strength, and stiffness. Proper reinforcement distribution is the key criterion for a cost effective design by correctly assessing the location, orientation, length and spacing of the reinforcement. It is also important to classify the soil used in the reinforced structure. The type of soil used may determine the form, distribution, and amount of reinforcement needed, and therefore will influence the overall cost of the earth structure. Soil type may affect the durability of certain reinforcements and tire attachments. The behaviour of the structure will also depend on the in-situ state of soil; defined by density, overburden pressure, state of stress, degree of saturation. The degree of compaction and the system used during construction will determine the soil state and the amount of material damage inflicted on the reinforcement. Reinforcement durability is important when assessing long term performance of the earth structure. Durability is a function of both the corrosion resistance of the reinforcement (including the attachment scheme) and the amount of damage sustained during construction.

This Chapter presents the results of tests carried out to determine the properties of sandy and cohesive soils used as fill and the tire reinforcement. The tire soil interaction behaviour as determined by pull-out tests are however discussed in detail in Chapter 4. Reinforcement distribution, soil state, construction techniques and durability performance will be covered in Chapter 5.

3.2 Soils

3.2.1 Background

Soil is plentiful and cheap and is therefore, in most cases, an ideal construction material. Reinforcement can theoretically improve the shear properties of any soil. However, in practice, most vertical earth slopes are constructed using good cohesionless soils as backfill. Marginal or poor quality materials are sometimes used in low embankments. In general, indigenous and waste materials are more economical although their performance may be inferior. The backfill material used must be able to adequately transfer stress to the tire reinforcement. The mineral composition of the soil and the pre water chemistry may influence the durability of the reinforcing system.

Long term conventional structures are usually designed with well-graded cohesionless soil (granular backfill) or a high quality cohesive frictional soil. The advantage of a good quality backfill, usually a cohesionless soil, are stability, good workability, high permeability, insensitivity to frost, and durability of the reinforcing material (Jones 1985). The possible high cost of a good quality imported backfill might off-set its many benefits. Cohesive soils usually have inferior characteristics (lower strength, gradation, high plasticity, corrosive nature) which may result in a structure that is more massive, heavily reinforced, deformable and with higher maintenance costs. The possible corrosive nature of these soils may lead to long term durability problems.

However, it is important to note that the use of reinforcement such as tires can make the use of poor quality soils feasible and therefore may provides an attractive construction option.

3.2.2 Cohesionless Soil

Cohesionless soil or a coarse-grained soil is usually a material of good quality, well graded, non-corrosive, and possessing a high effective angle of internal friction ($\phi' \geq 30^\circ$). The Canadian Foundation Engineering Manual (2nd Edition) defines a coarse grained soil as having more than 50% of the dry weight larger than a particle size of 0.075 mm. Examples include crushed rock, sand or gravel.

The key soil index properties that are required include:

- density or unit weight, ρ , γ ,
- grain size distribution,
- uniformity coefficient, C_u ,
- angle of internal friction corresponding to an appropriate effective stress range, ϕ' ,
- Compressibility characteristics.

Index property tests are performed to classify and categorize the engineering behaviour of the soil at a given construction site. Shear strength parameters are needed for internal, external and bearing stability analysis of the reinforced earth structure.

3.2.3 Cohesive Soil

Cohesive soil or fine-grained soil is a soil having more than 50% of the dry weight smaller than a particle size of 0.075 mm. Examples are silts, clays and waste materials such as mine tailings and pulverized fuel ash. Cohesive soils are used as a fill material for vertically faced reinforced walls and embankment structures as long as the reinforcement is able to improve the mechanical properties of the indigenous or marginal material. Jones (1988) and Mitchell and Villet (1987)

stated several reasons why cohesive soils may be unsuitable for steep reinforced earth structures. They are:

1. short term stability: bonding between the soil and reinforcement is poor and is further reduced if positive pore pressures are developed;
2. higher degree of plasticity: high plasticity of the backfill could lead to greater creep deformation, especially when the soil is wet;
3. high percentage of fines: the backfill results in a soil that is no longer free draining and therefore, can suffer potential problems from high water pressure;
4. the lower effective internal friction angle results in higher internal horizontal earth pressure which the facing and reinforcement must resist;
5. some fine grained soils are more chemically aggressive and, therefore, accelerate metal corrosion and deterioration of the reinforcement.

The required material and index properties for cohesive soil are:

- density or unit weight, ρ , γ ,
- grain size distribution,
- liquid limit, w_L ,
- plasticity index, I_p ,
- angle of internal friction under drained condition (effective), ϕ' ,
- cohesion under undrained and drained conditions, c_u and c' ,
- consolidation parameters, C_c , C_{cr} , c_v .

Consolidation parameters are also required for compressible cohesive soils located below the foundation of the reinforced soil structure. Settlement analysis of the underlying soil is important for assessing the long term performance of the reinforced earth structure. Reference to any standard textbook in geotechnical engineering is recommended for a detailed discussion of the soil parameters mentioned above.

3.2.4 Durability Parameters

As stated in Section 2.9.2, reinforcements and attachments buried in low quality backfill (waste materials) may be susceptible to corrosion or deterioration. The soil properties required to assess durability may include:

- soil pH,
- chloride content, Cl^- ,
- total sulphate content, SO_3 ,
- resistivity, P_a ,
- redox potential, E_r .

These parameters provide the necessary characteristics for proper design against corrosion and degradation, and assist in the proper selection of tire attachment or reinforcing elements with adequate durability.

Corrosion rates for metallic attachments presented in Chapter 2 are for “moderately to mildly corrosive environments”. The resistivity defines the aggressiveness of a soil and is characterized by the following qualitative relationship:

<u>Aggressiveness</u>	<u>Resistivity ohm-cm</u>
Very corrosive	< 700
Corrosive	700 - 2000
Moderately corrosive	2000 - 5000
Mildly corrosive	5000 - 10000
Non-corrosive	> 10000

Acidic soils (pH less than 4.5) and alkaline soils (pH greater than 9) are able to corrode carbon steel at a high rate. Corrosion protection by galvanization is significantly reduced in highly acidic or alkaline soils. A water content between 60 and 70 percent saturation generates maximum corrosion rates. High concentrations of soluble salts particularly chlorides and sulphate disrupt the formation of protective layers such as zinc coating over carbon steel. Recommended electrochemical soil properties suitable for metallic attachments are provided below (Elias et al. 1996):

<u>Properties</u>	<u>Criteria</u>
Resistivity	> 3000 ohm-cm
pH	> 4.5 - < 9.5
Chlorides	< 100 ppm
Sulphates	< 200 ppm
Redox potential ¹	> 0.35 V

¹(after Jones 1988)

Polyolefins are resistant to corrosion and strong alkaline environments, while polyesters are not. However, polymeric material will slowly deteriorate when exposed to strong oxidizing acids. In addition, they are susceptible to physico-chemical activity in the soil such as hydrolysis, oxidation, biodegradation, environmental stress cracking, and are vulnerable to construction damage.

3.2.5 Soils Used in the Prototype Embankment

The test embankment consisted of three sections. Two sections consisted of a good quality backfill, a cohesionless sand imported to the construction site. The third section was composed of various silty clay cuttings and random clay fill collected over time from several construction sites in the Ottawa area. This waste material had been used by the test site owner as a soil barrier around his property for several years. All soil tests were performed on samples compacted to

their respective densities determined from standard proctor tests. A complete description of the test embankment will be presented in Chapter 5.

3.2.5.1 Imported Sand

A summary of index and strength properties of imported sand with the corresponding testing methodology is given in Table 3.1. The relative density is 2.67. Standard proctor tests indicated a maximum dry density of 1845 kg/m^3 at a optimum water content of 10.5% and a unit weight of 20 kN/m^3 (Figure 3.1). The particle size distribution is presented in Figure 3.2. The sand is uniform and free draining, less than 5% of fines, with a trace of gravel. The shear stress and shear displacement from laboratory direct shear tests on compacted sand are given in Figure 3.3 with the corresponding vertical movement of some samples are plotted in Figure 3.4. The stress-strain behaviour is typical of a dense sand. The failure envelope is presented in Figure 3.5 and shows an effective internal friction angle of 42° , for an average dry density of 1790 kg/m^3 .

3.2.5.2 On-Site Cohesive Soil

The index and strength properties of the on-site cohesive backfill soil are given in Table 3.1 together with the corresponding testing method used for their determination. The relative density of the material is 2.66. The compaction curve produced by a Standard Proctor effort is presented in Figure 3.6. Based upon compaction tests, the soil has a unit weight of 19 kN/m^3 and a maximum dry density of 1508 kg/m^3 at an optimum water content of 29%. Particle size analyses were performed on three soil samples from different locations around the site (Figure 3.7). The soil varies from a sandy silt with some gravel to a clayey silt with trace gravel (Unified Soil Classification System). An average particle size distribution can be used to classify the soil as a sandy silt, some gravel, trace of clay. The liquid limit is greater than 50 (52.8) indicating a high degree of plasticity. The plasticity chart characterizes the behaviour of the soil as either a sandy silt or sandy silty clay of high compressibility. Consolidated undrained direct shear tests (CU) were performed on soil samples compacted at their natural water content. The cohesive soil

samples were sheared at a high displacement rate of 0.6 mm/minute, as to minimise pore water dissipation. The corresponding shear stress and horizontal shear displacement is presented in Figure 3.8 and also included in Figure 3.9 is the relationship between the vertical displacement and shear displacement. The consolidated undrained strength parameters (Figure 3.12) are an apparent angle of internal friction of 19° and an apparent cohesion of 68.4 kPa. The latter parameters are in terms of total stresses since pore water pressure cannot be measured in direct shear tests. Consolidated drained tests (CD) were also performed on compacted cohesive soil. The shear displacement rate was calculated based upon consolidation results as to insure complete pore water dissipation. A shear displacement rate of 0.00064 mm/minute was used. The resulting shear stress-displacement relationships are presented in Figures 3.10 and 3.11. The compacted backfill material has an effective angle of internal friction of 32° (Figure 3.13). One-dimensional consolidation tests were also performed on compacted soil samples at their natural moisture content to determine compression characteristics. The change in void ratio (e) with applied vertical effective stress is shown in Figure 3.14. The initial void ratios were standardized for different tests for comparison purposes. The on-site cohesive backfill has a compression index of 0.108 and a recompression index of 0.031. The vertical coefficient of consolidation (c_v) varies from 2.4×10^{-7} to 1.05×10^{-6} m²/s depending on the applied vertical effective stress, as shown in Figure 3.15.

3.3 Reinforcing Materials (tires tied with rope)

Internal failure of a reinforced soil structures can occur in several ways, such as pull-out of the reinforcement due to an improper adherence length behind the failure surface, sliding along the interface between the soil and reinforcement, or tensile failure of the reinforcement along a plane of maximum tensile force. In addition, deformations are an important consideration when using extensible reinforcement. The reinforcement material properties will influence the load-displacement behaviour of the structure. The required reinforcement properties are form, strength, stiffness, durability, and soil reinforcement interaction (discussed in detail in the next chapter). Durability of the different reinforcing elements have been discussed in the preceding chapter.

In this study, scrap tires were tied together with polypropylene rope to form a tire mat. These tire mats were used to construct the reinforced soil structures. Two types of soil structures were studied: retaining walls and steep embankments. In both cases, assessing the material properties of the attachment (polypropylene rope) and the used tires was required.

3.3.1 Form

The different tire elements are usually tied together to form a reinforcing mat. This reinforcing mat will vary in thickness depending on which tire elements are used. A tire mat composed of sidewall only, a relatively thin reinforcement, would possess similar geometric characteristics as geosynthetic sheets and grids. Using whole passenger tire or a passenger tire with one sidewall removed (referred to as a cut tire), as in this study, the thickness would vary from 150 to 200 mm. As a result, the soil material placed within the tire mat would be confined, and therefore, the adhesion developed between soil and reinforcement is primarily governed by the shear strength of the soil.

3.3.2 Attachment Properties (polypropylene rope)

3.3.2.1 Methodology

A polypropylene rope of 9.4 mm (3/8 in.) in diameter was used to tie together the different tire elements to form a reinforcing mat. Polypropylene is very resistant to chemical and biological attack and is commercially available. A list of the physical properties of polypropylene is provided in Table 3.2. The rope was tested in a Tinius Olsen Testing Machine in the laboratory for determination of the tensile strength and the modulus of elasticity. The use of several different knots were examined in order to determine which type of knot was most effective, while remaining simple to tie. A square knot was selected. The appropriate length of rope was placed between two eye bolts in the testing machine; the two loose ends were tied together by means of the selected knot, and pulled apart. Displacements were measured by a scale placed between the two testing beds of the machine. Testing proceeded until failure occurred.

However, several wraps of the rope around the tires were required to provide adequate attachment strength. Therefore, the rope was looped two and three times around the eye bolts, tied, and tested to failure. A typical test set-up is given in Figure 3.16. This allowed the evaluation of the overall strength characteristics of the rope in terms of the number of wraps used in the test embankment and pull-out tests.

3.3.2.2 Results and Discussion

The stress-strain relationship for polypropylene rope in terms of the number of wraps around the two eye bolts is given in Figure 3.17. Averaging the results produced an almost identical stress-strain relationship for rope wrapped more than once. The linear portion of each average stress-strain curve was used to determine the modulus of elasticity. The elastic modulus ranged from 240 MPa for a single loop to 320 MPa for two to three loops. The initial slack in the attachment scheme combined with the tightening of the knot at the beginning of testing resulted in some degree of slippage in the system. This slippage was responsible for the apparent non-linearity observed at the initial stages. This non-linearity was more predominant in the case of a single wrap.

A list of the different material properties for a 9.4 mm diameter polypropylene rope as a function of the number of loops around the eye bolts is presented in Table 3.3. The ultimate tensile strength varied from 11.9 kN for a single loop to 29.4 kN for three loops. The estimated tensile strength per rope length (the number of wraps time two) decreased with the number of wraps, from 6 kN to 5 kN per rope length. This behaviour is the result of an uneven stress distribution (each wrap does not carry the same load) and therefore, the ultimate tensile strength is not directly proportional to the number of wraps, based upon a single rope element. This behaviour should be considered in design when establishing the allowable tensile force of the reinforcement. In this research, the weakest part in the tire mat construction is the attachment itself, since the ultimate tensile strength of tires will exceed 26 kN in 90 percent of the cases (see Section 2.14). A sudden loss of strength due to failure of the attachment could have serious consequences since the improvement in shear strength is directly proportional the maximum

force generated within the reinforcing tire mat. A safety factor must be employed, to prevent this mode of failure. A desirable failure mechanism would be the loss of adherence between the soil and the tire mat reinforcement, in which case a redistribution of shear stress is possible without a failure of the structure. Strains at failure ranged from 21% to 26% and was dependent on the number of wraps used. The stiffness of the attachment represented by the elastic modulus will influences the deformity of the reinforced soil structure. An attachment scheme using two or three wraps of rope around a tire would increase moderately the stiffness of the reinforcing system.

3.3.3 Used Tires Material Properties

3.3.3.1 Methodology

The material properties of used tires were also tested in the Tinius Olsen Testing Machine. A random selection of used tires was made from a stockpile, located at the construction site. A list of the different tire brands used to determine the material properties is provided in Table 3.4; these tires were also used to evaluate the chemical leaching characteristics of buried used tires. Tires from this stockpile were also used to construct the test embankment. Whole tires were tested first; a second series of tests were conducted on the same group of tires, following the removal of one sidewall (a cut tire) which was replaced inside the tire. The tire was placed between the beds of the testing machine, tied to two eye-bolts by 3 wraps of 9.525 mm diameter polypropylene rope. A schematic representation of the set-up to determine the material properties of the tire reinforcement is presented in Figure 3.18, also included is the location of the various measuring points. The square knot selected for embankment construction was used. The tire element was tested in tension until either the maximum displacement allowed by the machine was reached or 30% of the ultimate tensile strength of the rope was reached. To evaluate the tensile modulus (J) of both the tire and composite system (rope and tire), deformation of the inner radius of the tire and between the two testing beds was measured by a scale (measuring tape).

Additional tests were carried out on whole tires to determine the effect of confinement on tire elongation. This case better represents the field condition where some confinement is provided to the tires and this restricts lateral deformation. A calibrated stiff spring with a stiffness constant of 185 kN/m was placed within the inner radius of the tire, perpendicular to the applied tensile force (see Figure 3.18). The spring was set-in a box system that allowed the beads of the tire to transfer radial stresses to the spring. The amount of stress transferred was measured by a liquid transducer located inside the spring (Figure 3.19). Consequently, displacements were also measured. To examine the effects of confinement on the tensile modulus, vertical displacement of the inner radius of the tires and the flexibility of the testing beds was also monitored.

3.3.3.2 Results and Discussion

Tire reinforcement strength and stiffness properties should be expressed as a tensile force per tire element or unit width, rather than by stress. The variation in cross-sectional area and moment of inertia preclude reliable estimates of stress. A series of photographs show the progressive elongation of the Motormaster XTS P215/75 R15 without confinement, as a whole tire (Figure 3.20) and after removal of one sidewall, i.e. a cut tire (Figure 3.21). The corresponding tensile force-strain relationships are shown in Figure 3.22. The elongation behaviour of an unconfined tire, whole or cut, is governed by two factors: collapse of the inner rim and stiffness of the attachment (elastic modulus of the rope). The initial collapse of the inner rim produces 80% or more of the measured strains for both whole and cut tires. The remaining strains are generated by the elongation of the polypropylene rope. As the inner rim collapsed, the stiffness of the tire element (tensile modulus: J) increased from 7 kN per tire to 85 kN per tire (slope of the curve), with a sharp increase occurring around a tensile force of about 2.5 kN. After this point, the contribution from the rope to the overall vertical strain increased dramatically, up to approximately 85%. Generally, all tires tested without lateral confinement demonstrated the same deformation trend and characteristics. However, whole tires did demonstrate a slight increase in stiffness compared to cut tires.

In a reinforced soil structure, the tire mat reinforcements would be buried within the soil mass. Infilling of the tires would provide resistance against lateral deformation, and therefore would increase the overall stiffness of the reinforcement. As mentioned earlier, tests were also carried out in which a spring with a stiffness constant of 185 kN/m was placed within the inner rim of the whole tire. The stiffness of the spring has a great influence on the amount of vertical elongation generated, and therefore must be taken into account when analyzing the results. A series of photographs showing the progressive elongation of the Motormaster XTS P215/75 R15 with confinement is presented in Figure 3.23. Overall and inner rim vertical strains generated as a function of the applied tensile force are given in Figures 3.24 and 3.25, respectively. Again, all tires tested with lateral confinement generally demonstrated the same deformation trend and characteristics. The variation in total vertical strains was no larger than 10%. However, this value is probably large because the existence of slack between the rim and spring system which had to be taken up before fully engaging the liquid transducer. This aspect could explain the large strains produced at the initial stages of testing. Also, slippage of the spring system occurred several times, especially at higher levels of tension. Both figures demonstrate an increase in the reinforcement stiffness with increasing tensile force. The vertical deformation of the inner rim was directly proportional to the amount of strain developed in the spring (Figure 3.26), or the percentage of the tensile force transferred to the spring (Figure 3.27). For the overall vertical deformation, strains produced by the elongation of the rope must be included with those generated at the inner rim. At the initial stages, the deformation of the rope contributed up to approximately 75% of the overall elongation of the system. This contribution increased with increasing tension, up to approximately 85%. The tensile modulus (J) varied from 32 kN per tire to 130 kN per tire. It should be noted that these values are higher than those associated with unconfined tires.

The stiffness of the tire reinforcement (i.e. including the attachment) would increase until the confining pressure would not allow any further deformation to occur. If the tire element cannot deform, the maximum tensile modulus would be given by the modulus of the attachment (Figure 3.28). For example, several tire elements tied together by three wraps of polypropylene rope, the maximum tensile modulus (J) is approximately equal to 135 kN per tire. The lower bound for

the tensile modulus is defined by the collapse of the inner rim, approximately equal to 7 kN per tire (Figure 3.28). Therefore, the tensile modulus of the reinforcement lies between these two limits, and is a function of the confining pressure imposed by the surrounding soil. The confining pressure is influenced by such factors as the state of stress, soil density, overburden pressure, dilatancy, compaction, stress-strain properties of the soil, construction method and sequence used.

Table 3.1. Index and strength properties of the backfill materials.

Property	Testing Procedure	Sand	Clayey/Sandy Silt
Relative Density, D_r	ASTM D-854-92	2.67	2.66
Unit Weight, γ	ASTM D-698-91 (Standard Proctor)	20 kN/m ³	19 kN/m ³
Maximum Dry Density, δ_d		1845 kg/m ³	1508 kg/m ³
Optimum Water Content, w		10.50%	29%
Particle Size Distribution ^A Gravel Sand Silt Clay uniformity coefficient, C_u	ASTM D-422-92	5% 93% 2% 2.8	12% 24% 55% 9% 25
Plastic Limit, w_p Liquid Limit, w_L Plasticity Index, I_p	ASTM D-4318-87		24.4 ^A 52.8 28.4
Unified Soil Classification System	ASTM D-2487-92	SP	CH MH
Strength Parameters c_u ϕ_u ϕ'	ASTM D-3080-90 Direct Shear		68.4 kN/m ² 19° 32°
Compression Parameters C_{cr} (consolidation) C_c (consolidation)	ASTM D-2435-90		0.031 0.108

^A Average Value

Table 3.2 Physical properties of polypropylene.

Properties	ASTM Test Method	Polypropylene
Specific Gravity	D792	0.900 - 0.915
Tensile Strength (kPa)	D638, D651	2320 - 3850
Elongation (%)	D638	200 - 700
Modulus of Elasticity (kPa)	D747	94500 - 145500
Compressive Index (kPa)	D695	6200 - 7300
Thermal Conductivity (see footnote 1)	C177	2.8
Thermal Expansion (10^{-5} per $^{\circ}\text{C}$)	D696	6 - 8.5
Resistance to Heat ($^{\circ}\text{C}$ continuous)		120 - 160
Heat Distortion Temp. ($^{\circ}\text{C}$)	D648	98 - 104
Volume resistivity (see footnote 2)	D257	6.5×10^{16}
Water Absorption (24 hr., 3.125 mm thickness, %)	D570	<0.01
Burning Rate	D635	Slow
Effect of Sunlight		Requires Black
Effect of Weak Acids	D543	Very Resistant
Effect of Strong Acids	D543	Attacked Slowly by Oxidizing Acids
Effect of Weak Alkalies	D543	None
Effect of Strong Alkalies	D543	Very Resistant
Effect of Organic Solvents	D543	Resistant Below 80 oC

1 10^4 cal. per sec. per cm^2

2 ohm-cm (50% relative humidity and 23 $^{\circ}\text{C}$)

Table 3.3. Material properties of polypropylene rope.

Number of Loops Around the Eye Bolt	T_u (Yield) (kN)	σ_u (Yield) (MPa)	ϵ_{yield} (%)	Elastic Modulus (MPa)
1	11.9	83.2	26.4	242
2	21.0	73.7	21.4	318
3	29.4	68.7	20.9	319

Table 3.4. List of tire brands used to evaluate the material properties and the chemical leaching characteristics of used tires.

Brand Name and Type	Tread Plies	Sidewall Plies	Inner Rim Diameter (mm)
Aurora Steel Radial 825 P185/80 R13 905 Tubeless Radial M+S	2 Steel Belts 2 polyester	2 polyester	325
Michelin All Seasons P155/80 R13 x A4 Tubeless Radial M+S	2 steel Belts 1 polyester	1 Polyester	325
Sport IV R785 P155/80 R13 Tubeless Radial M+S	2 Steel Belts 1 Polyester	1 Polyester	325
Yokohama All Season 370 P195/70 R13 Tubeless Radial	2 Steel Belts 2 Polyester	2 Polyester	325
Sears Road Handler II P155/80 R13 Tubeless Radial M+S	2 Steel Cord 1 Polyester Cord	1 Polyester	325
Motomaster XR P185/80 R13 Steel Belted Radial	2 Steel Belts 1 Polyester	1 Polyester	325
Aurora Super Run R817 P205/70 HR14 Tubeless Radial M+S	2 Steel Belts 2 Polyester 1 Nylon	2 Polyester	350
AmeriWay Radial P195/75 R14 Tubeless Radial M/S	2 Steel Belts 1 Polyester	1 Polyester	350
Goodyear Vector P185/70 R14 Tubeless Radial M+S	2 Steel Cord 1 Polyester Cord	1 Polyester	350

Table 3.4. Continued

Brand Name and Type	Tread Plies	Sidewall Plies	Inner Rim Diameter (mm)
Motomaster SL All Seasons Radials P195/70 R14	2 Steel Belts 2 polyester	1 polyester	350
Uniroyal Tiger Paw A/S P185/75 R14 Tubeless Radial M+S	2 steel Belts 1 polyester	1 Polyester	350
Winfield Weather Mate All Seasons Radials 781 P185/75 R14 89S Tubeless Radial M+S	2 Steel Belts 1 Polyester	1 Polyester	350
Motomaster XTS All Seasons Radials P215/75 R15 Tubeless Radial M+S	2 Steel Belts 2 Polyester 2 Nylon	2 Polyester	375

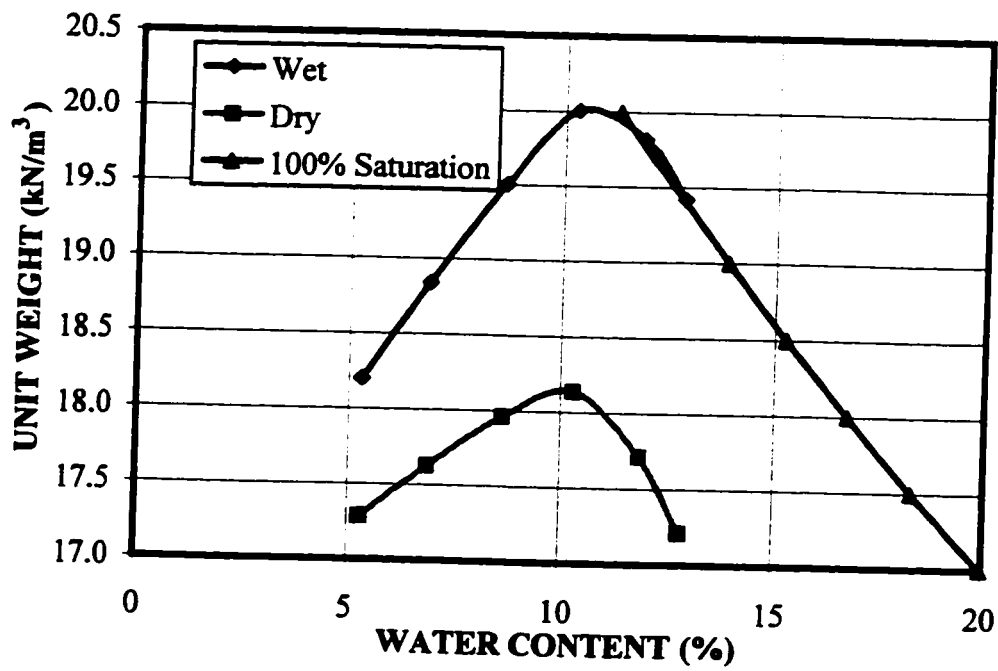


Figure 3.1. Compaction curve for imported sand.

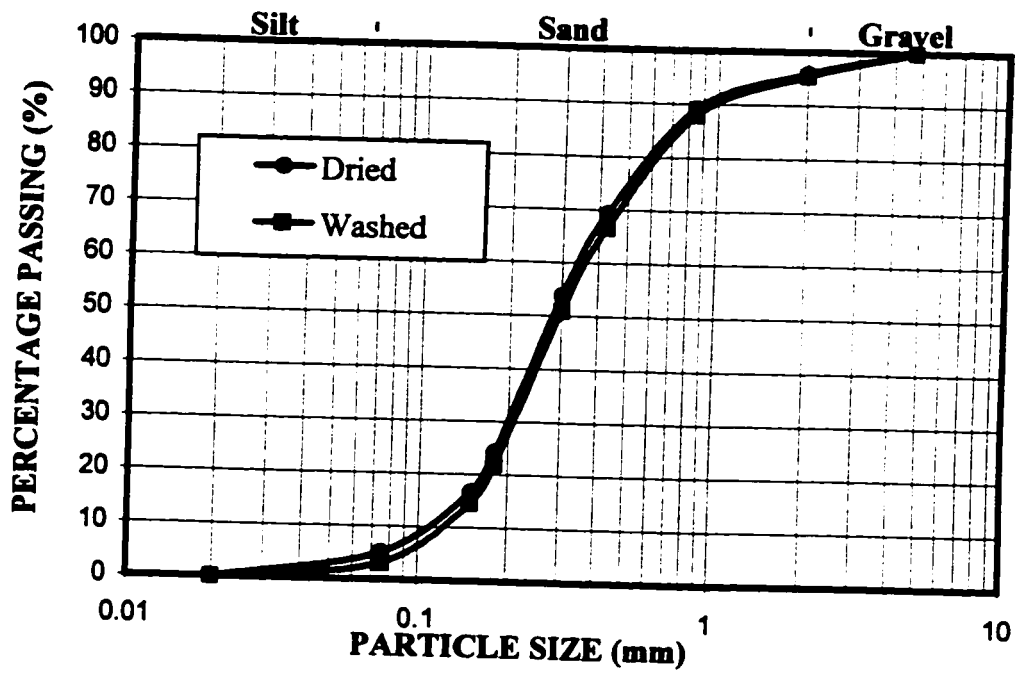


Figure 3.2. Particle size distribution of imported sand.

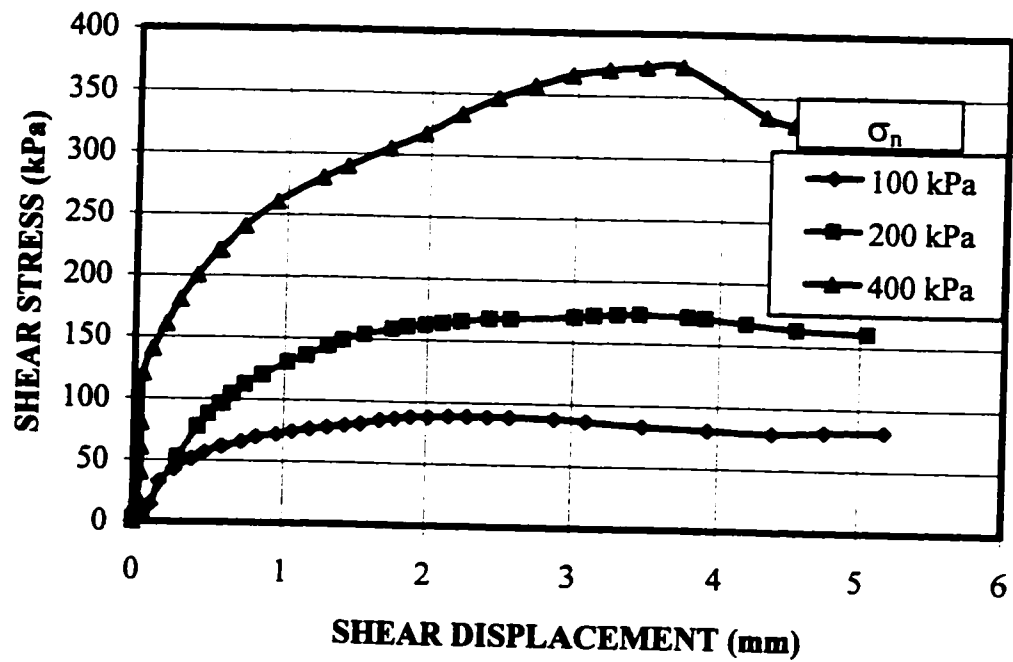


Figure 3.3. Shear stress and shear displacement relationships for imported sand.

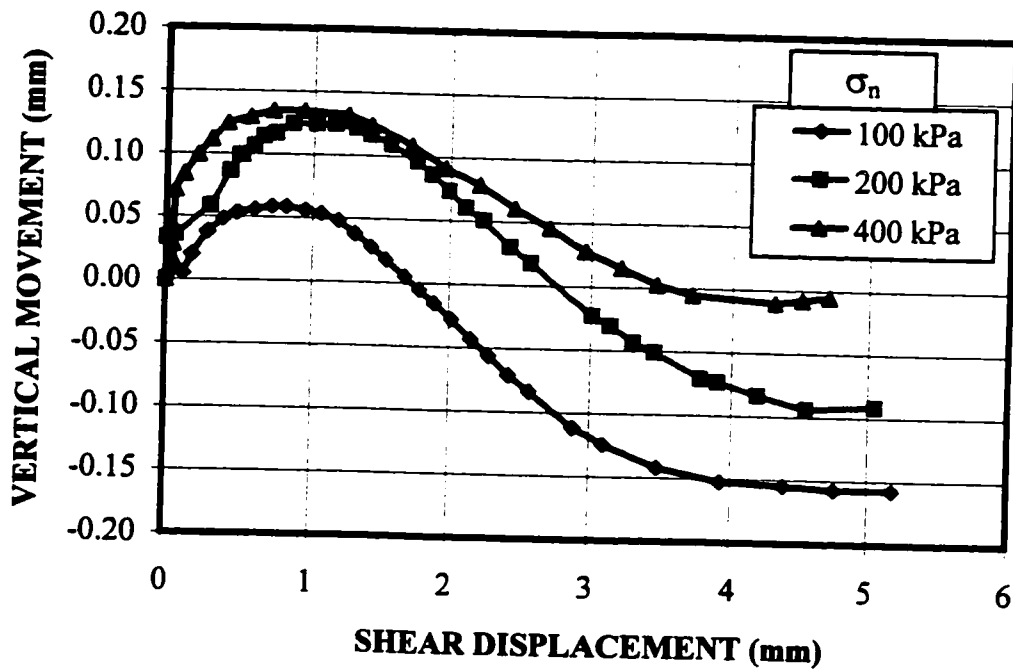


Figure 3.4. Relationship between vertical movement and shear displacement for imported sand in a calibrated shear box.

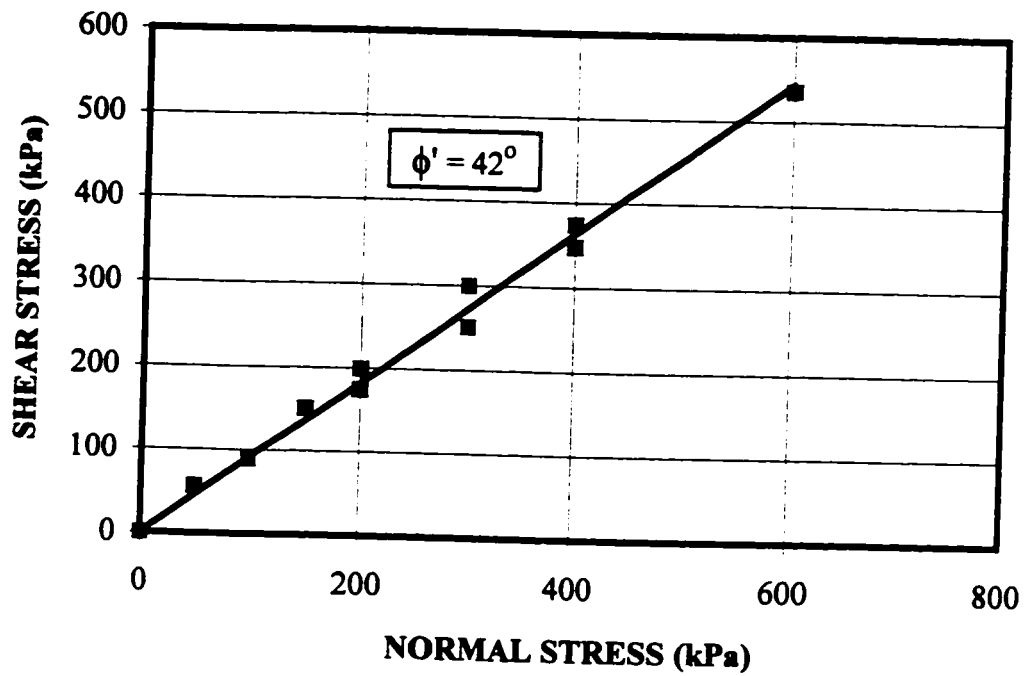


Figure 3.5. Failure envelope for dry imported sand.

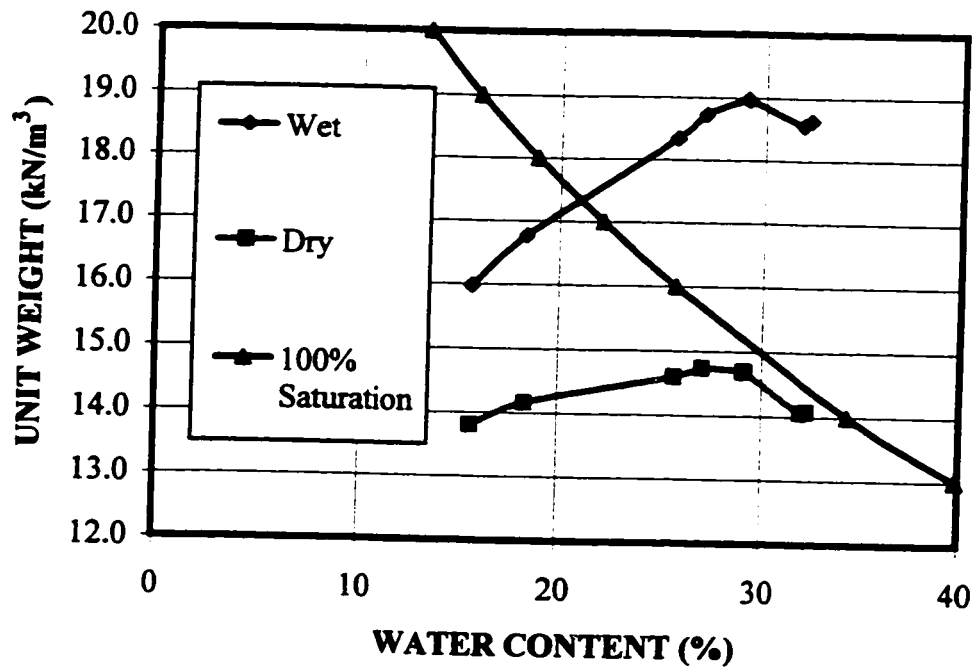


Figure 3.6. Compaction curve for the cohesive soil indigenous to the test site.

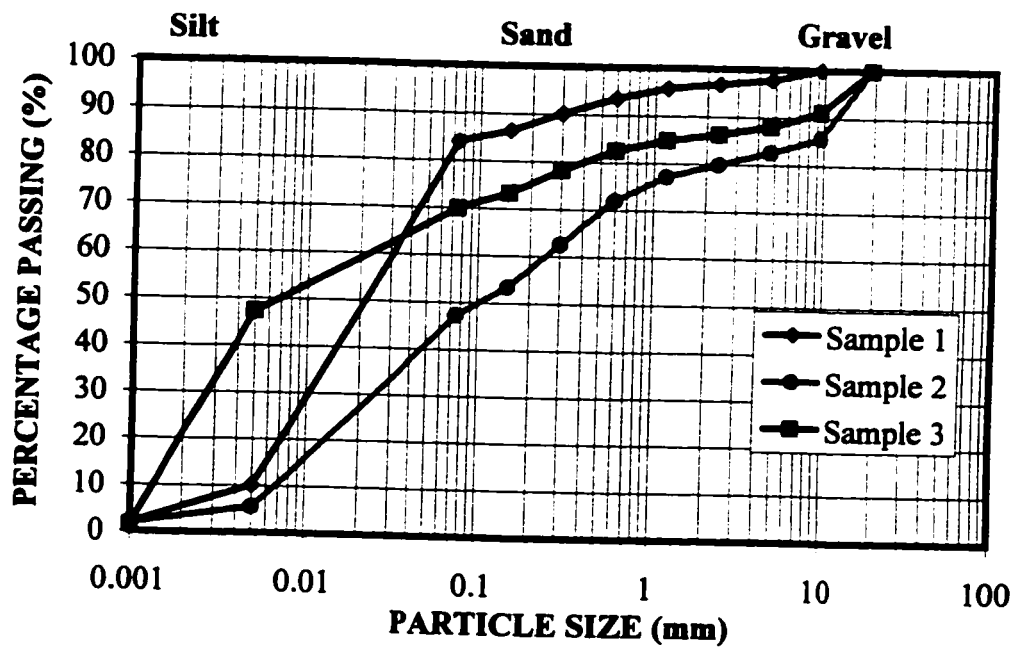


Figure 3.7. Particle size distribution of indigenous cohesive soil.

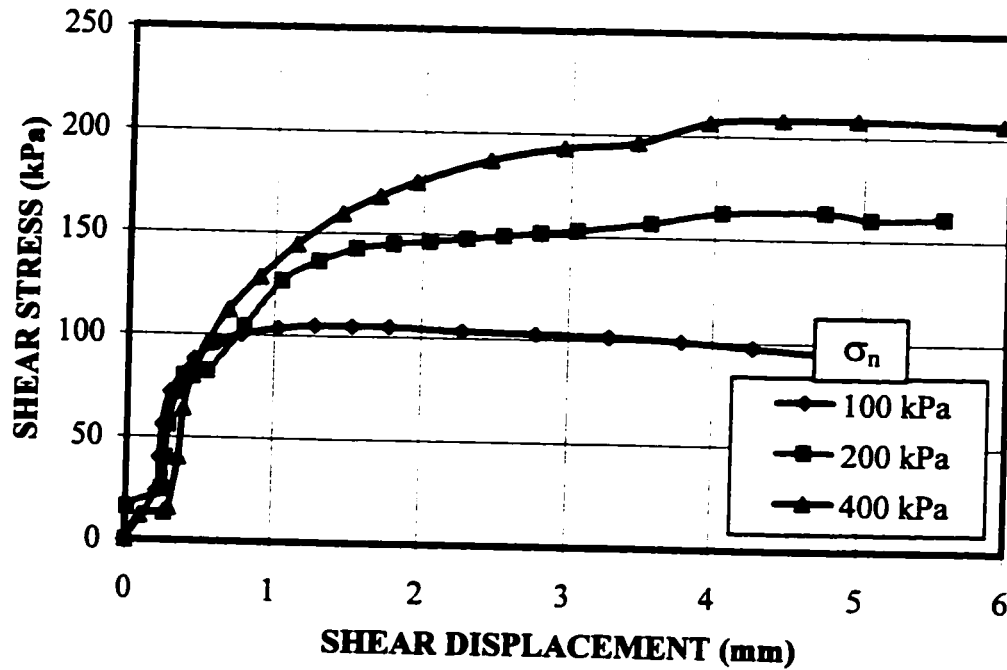


Figure 3.8. Shear stress and horizontal displacement relationships from C.U. test performed on the cohesive soil.

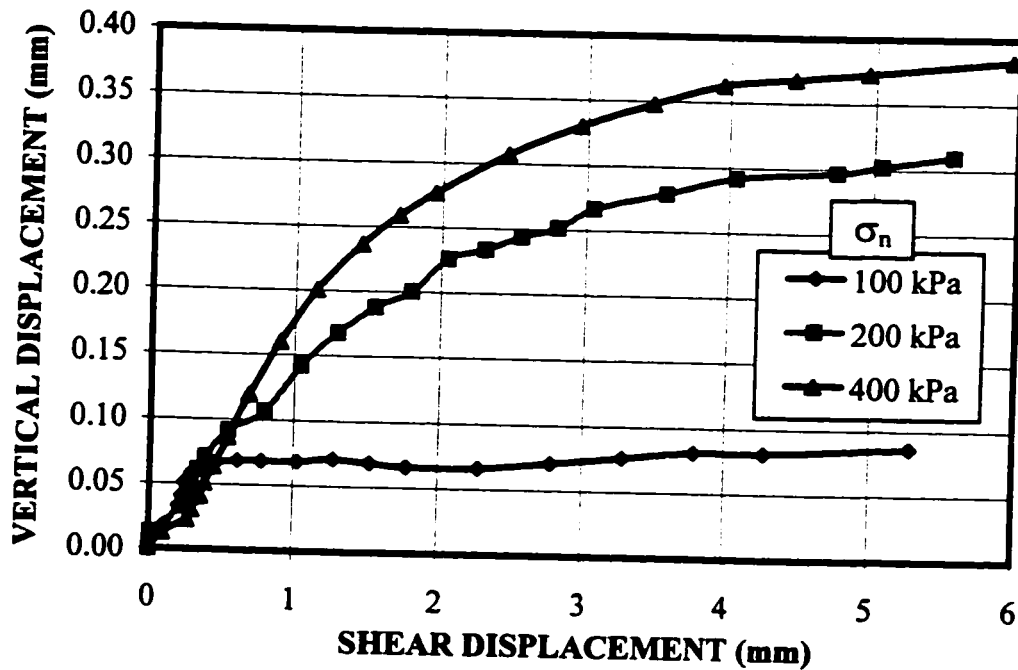


Figure 3.9. Vertical movement versus shear displacement from C.U. test performed on the cohesive soil in a calibrated shear box.

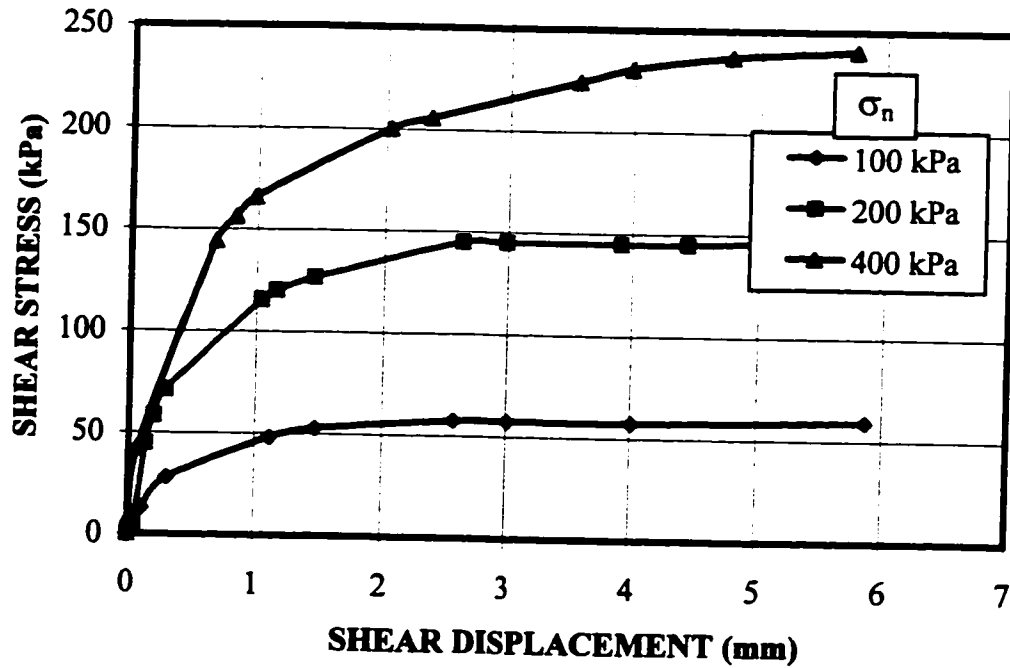


Figure 3.10. Shear stress and horizontal displacement relationships from C.D. test performed on the cohesive soil.

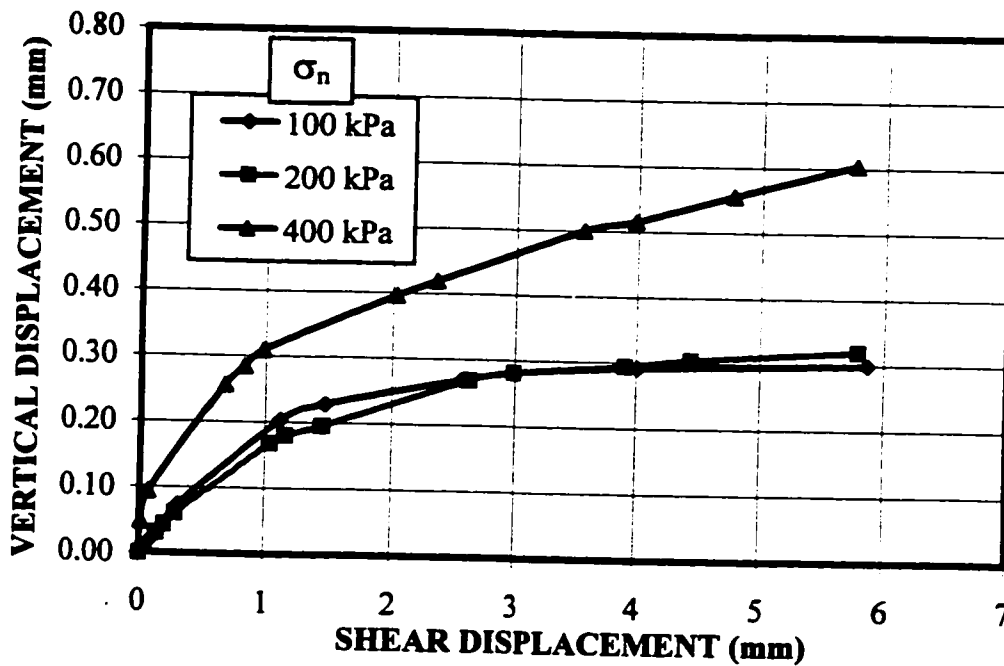


Figure 3.11. Vertical movement versus shear displacement from C.D. test performed on the cohesive soil in a calibrated shear box.

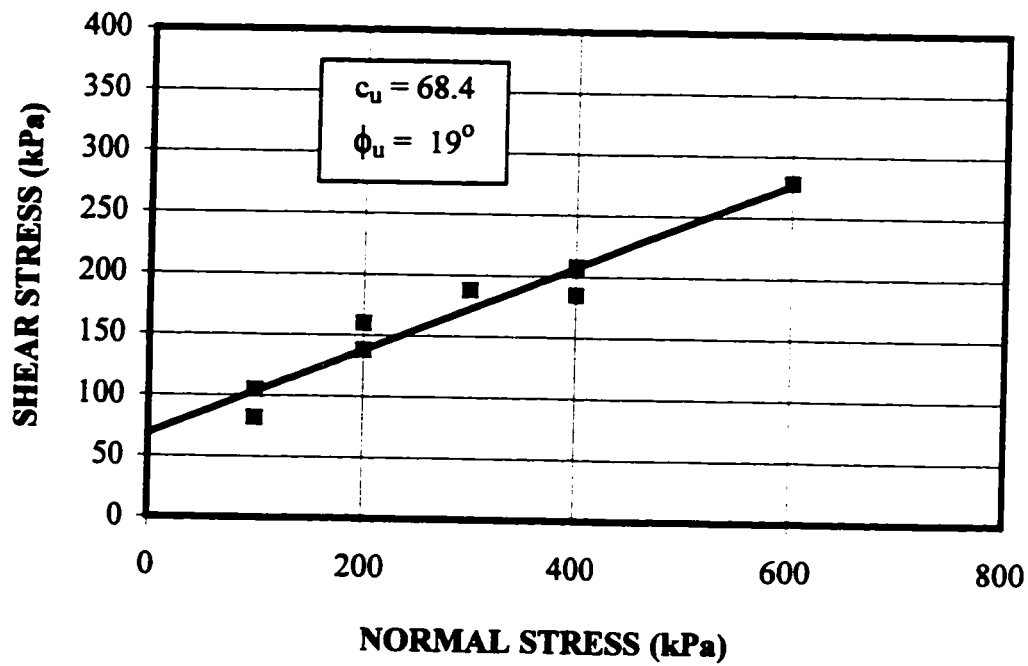


Figure 3.12. Failure envelope for the cohesive soil based upon the C.U. test.

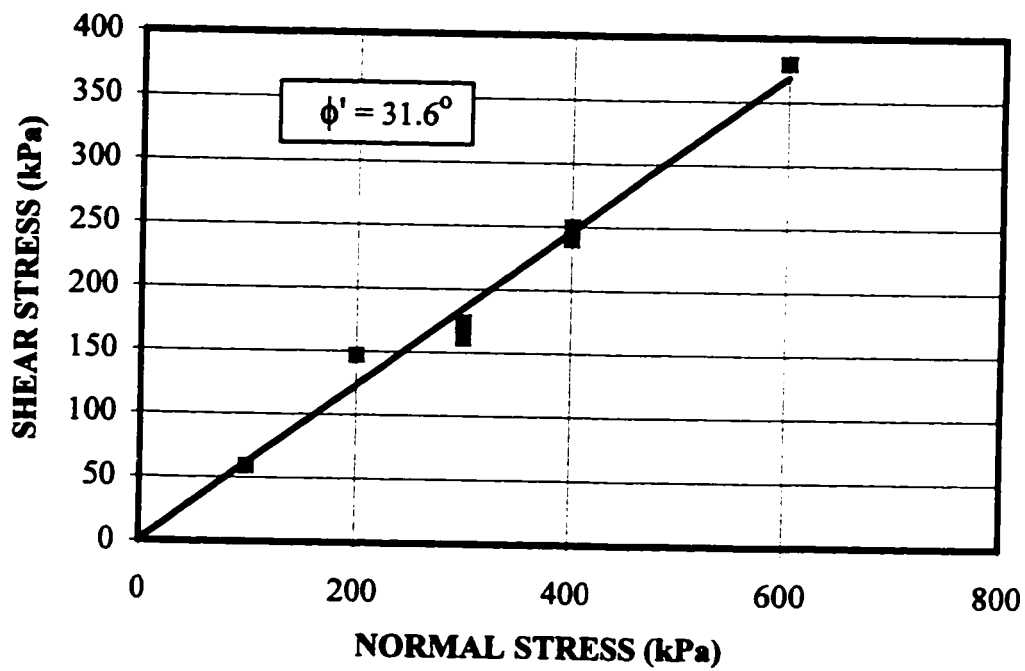


Figure 3.13. Failure envelope for the cohesive soil based upon the C.D. test.

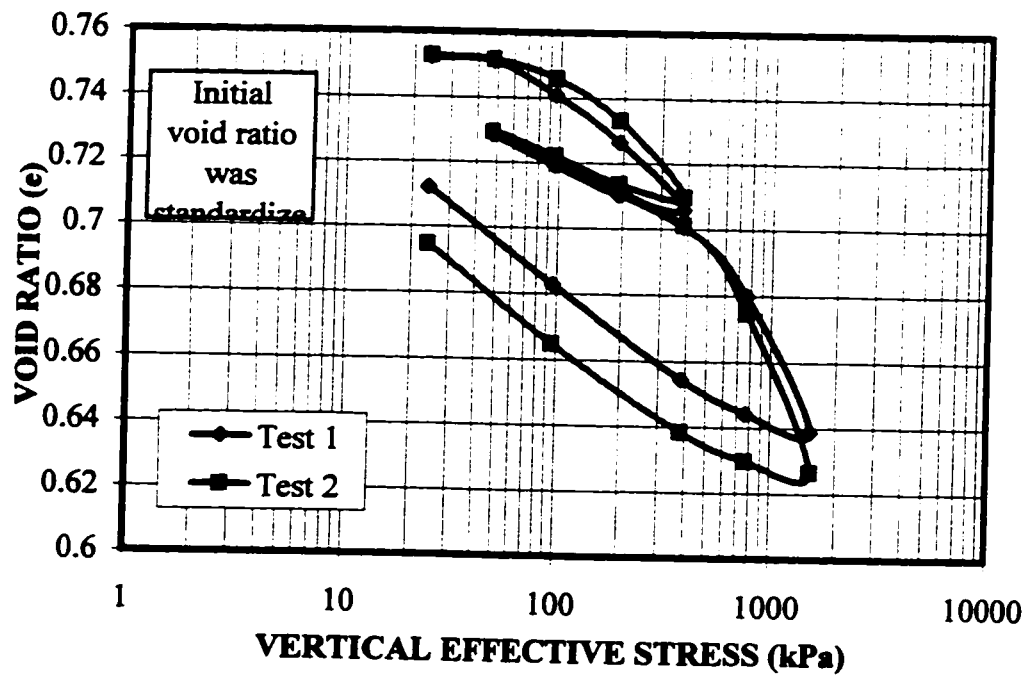


Figure 3.14. Void ratio and vertical effective stress relationship.

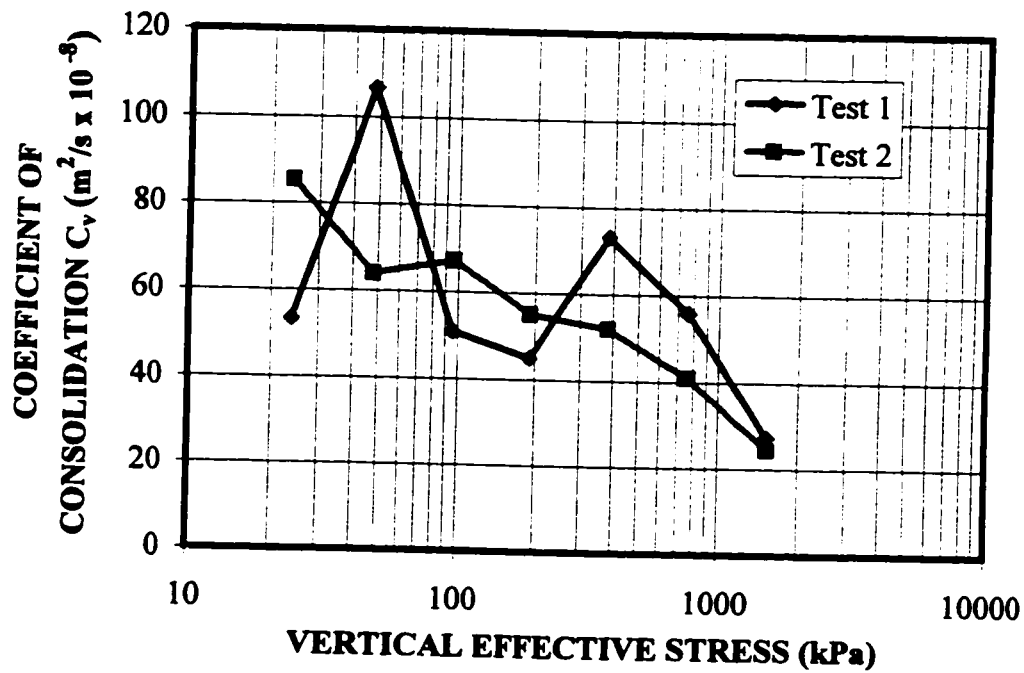


Figure 3.15. The coefficient of consolidation as a function of the applied vertical effective stress.



Figure 3.16. A photograph showing a typical set-up for testing the material properties of the polypropylene rope, three loops displayed.

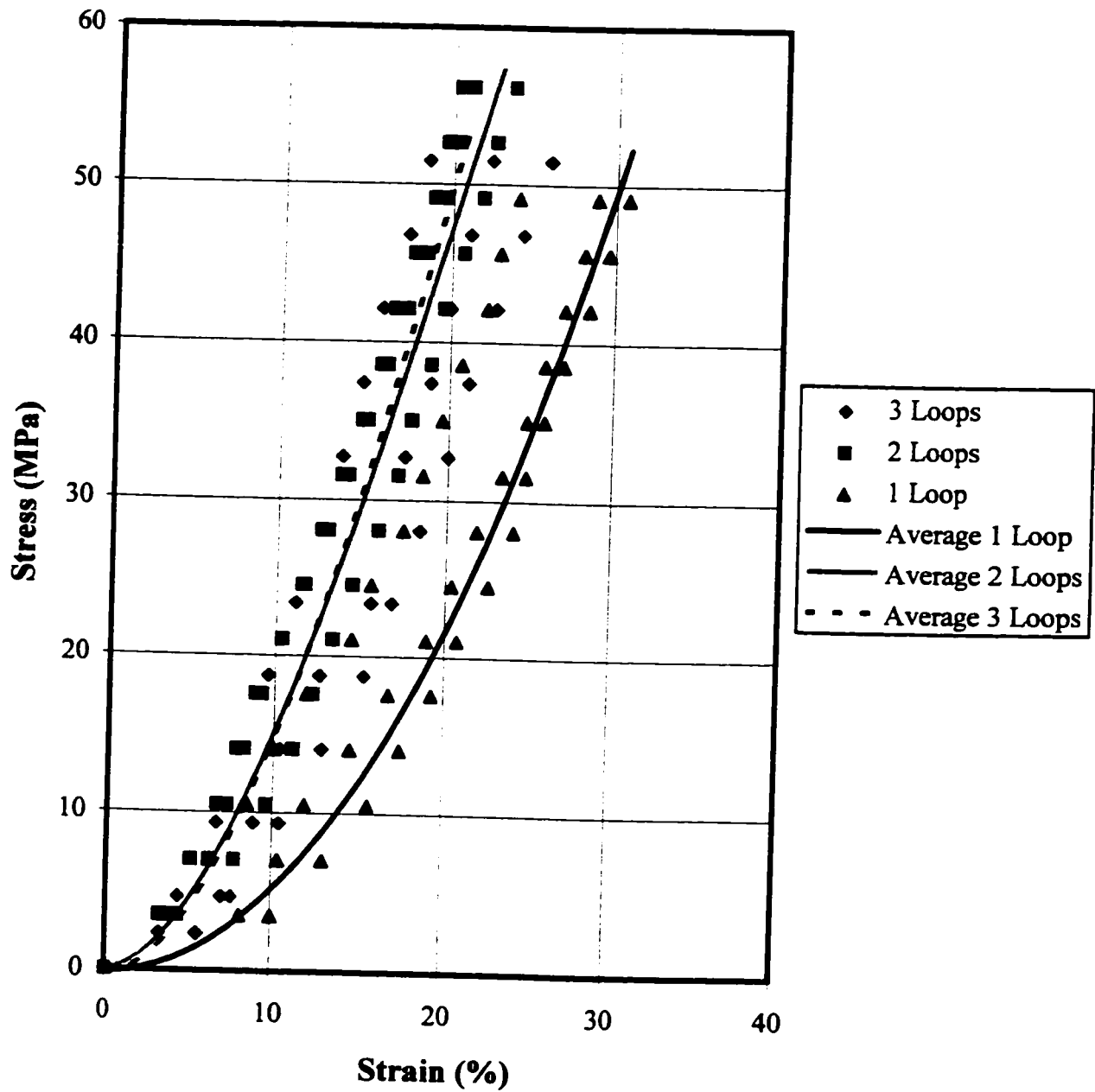
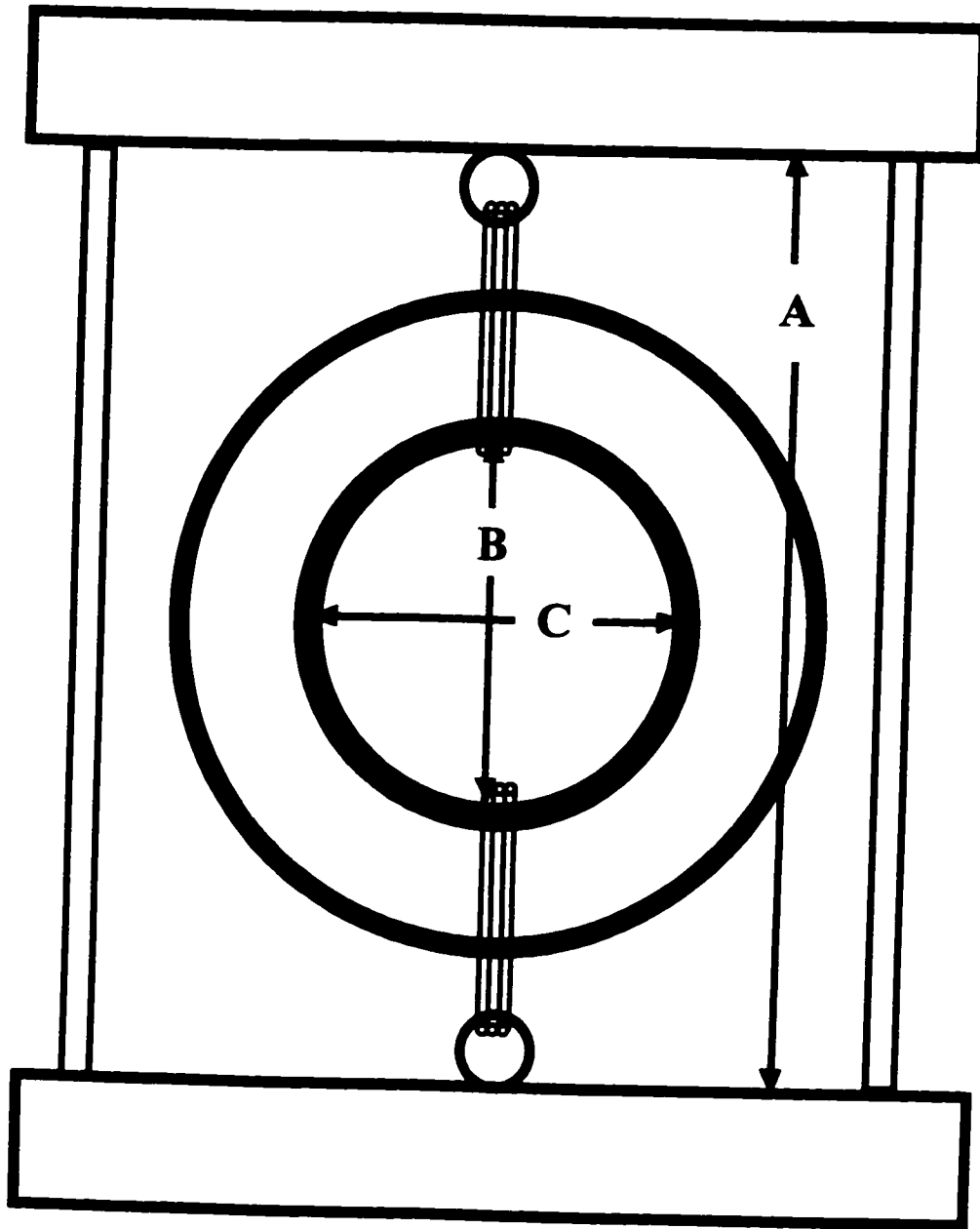


Figure 3.17. Stress-strain behaviour of polypropylene rope in terms of the number of loops used.



A: Overall Vertical Strain

B: Inner Rim Vertical Strain

C: Inner Rim Horizontal Strain and Location of Spring

Figure 3.18. A schematic representation of the testing set-up used to determine the material properties of used tires.

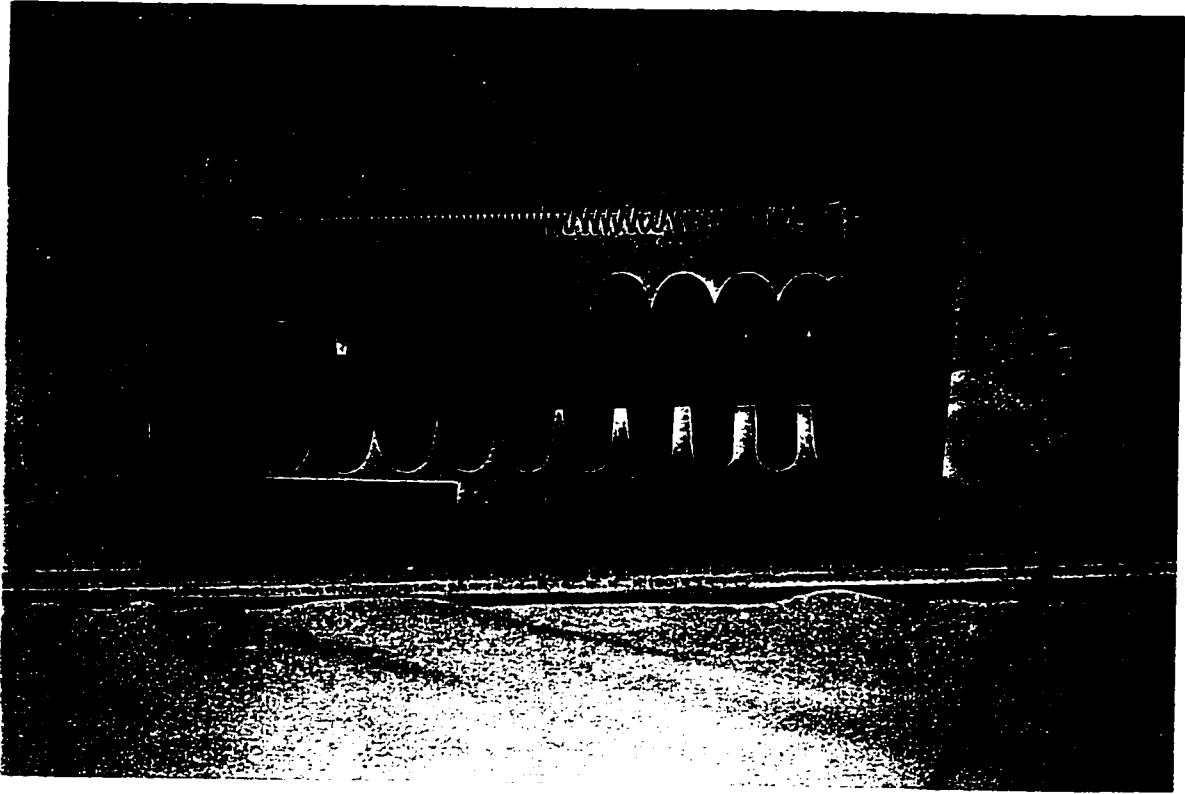


Figure 3.19. A photograph showing the spring system and the liquid transducer which was placed within the inner rim of the tire, perpendicular the applied tensile force.

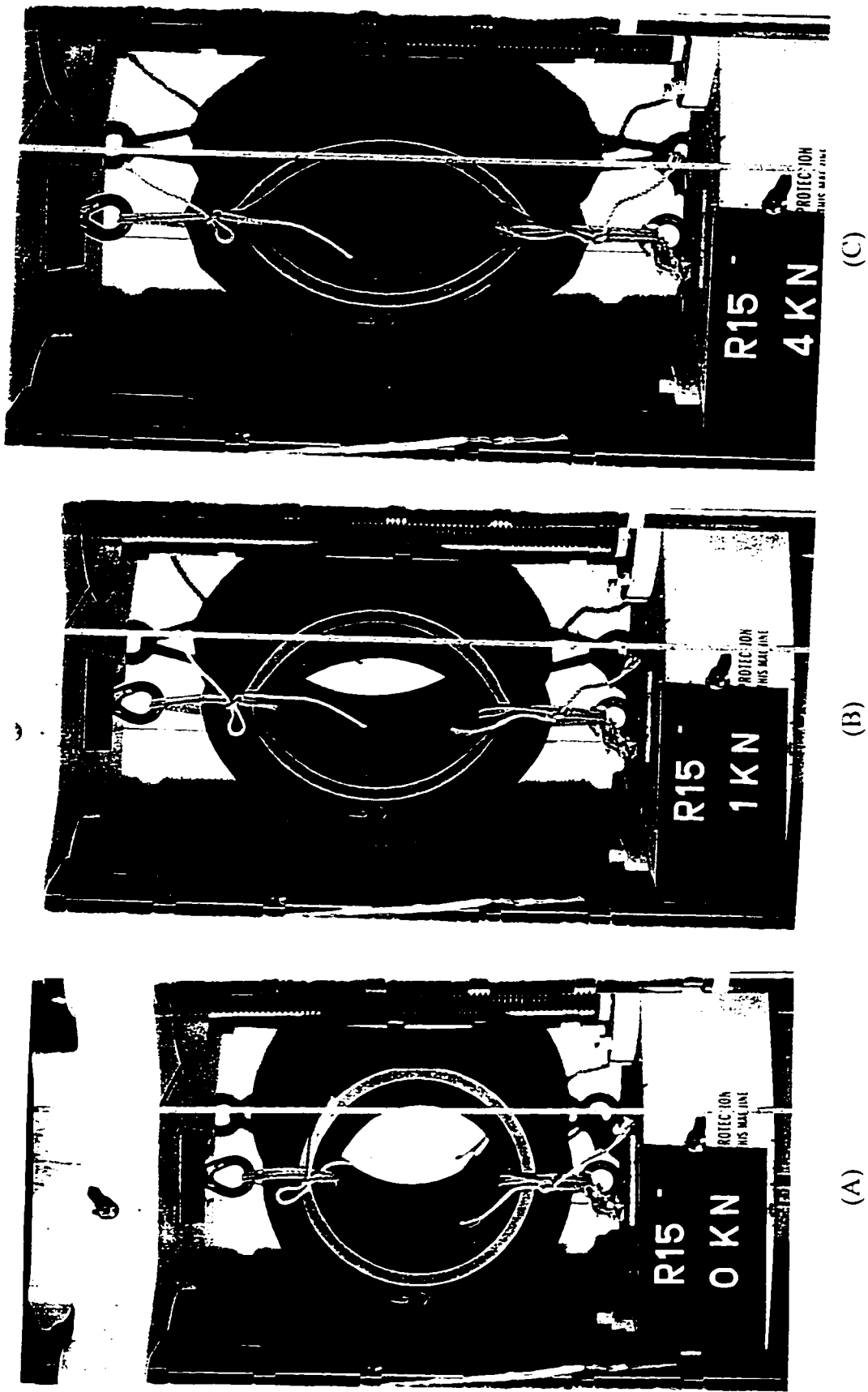
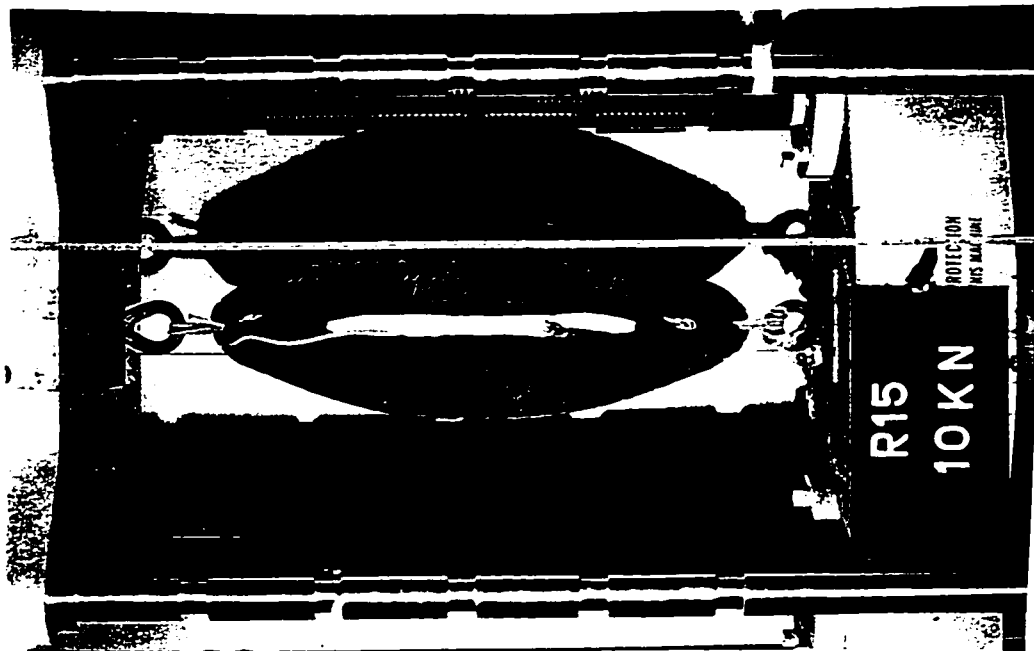
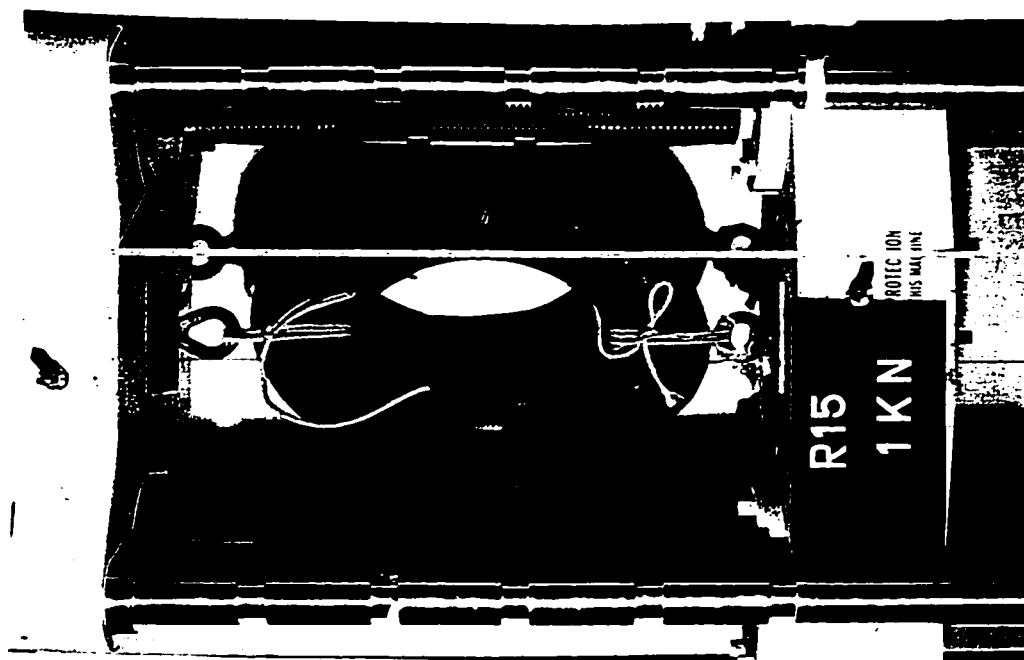


Figure 3.20. A series of photographs showing the change in elongation of the Motormaster XTS P215/75 R15 tire with applied tensile force: (A) 0 kN, (B) 1 kN, (C) 4 kN.



(B)



(A)

Figure 3.21. Two photographs showing the change in elongation of the Motormaster XTS P215/75 R15 tire with one sidewall removed and replaced inside the tire as a function of applied tensile force: (A) 1 kN, (B) 10 kN.

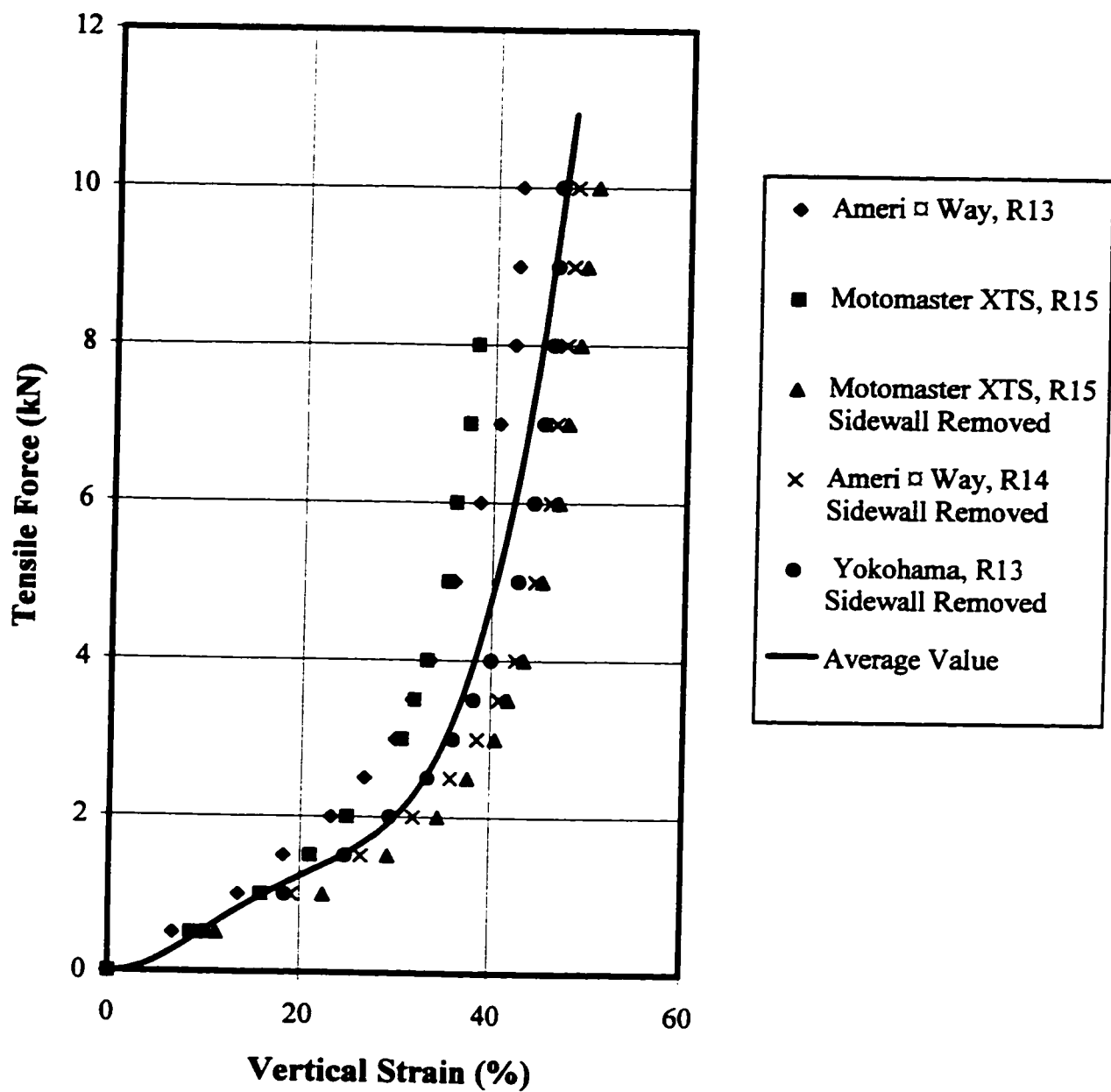


Figure 3.22. The relationship between the applied tensile force and vertical strain within the tire without any lateral restraint for several different tire brands.

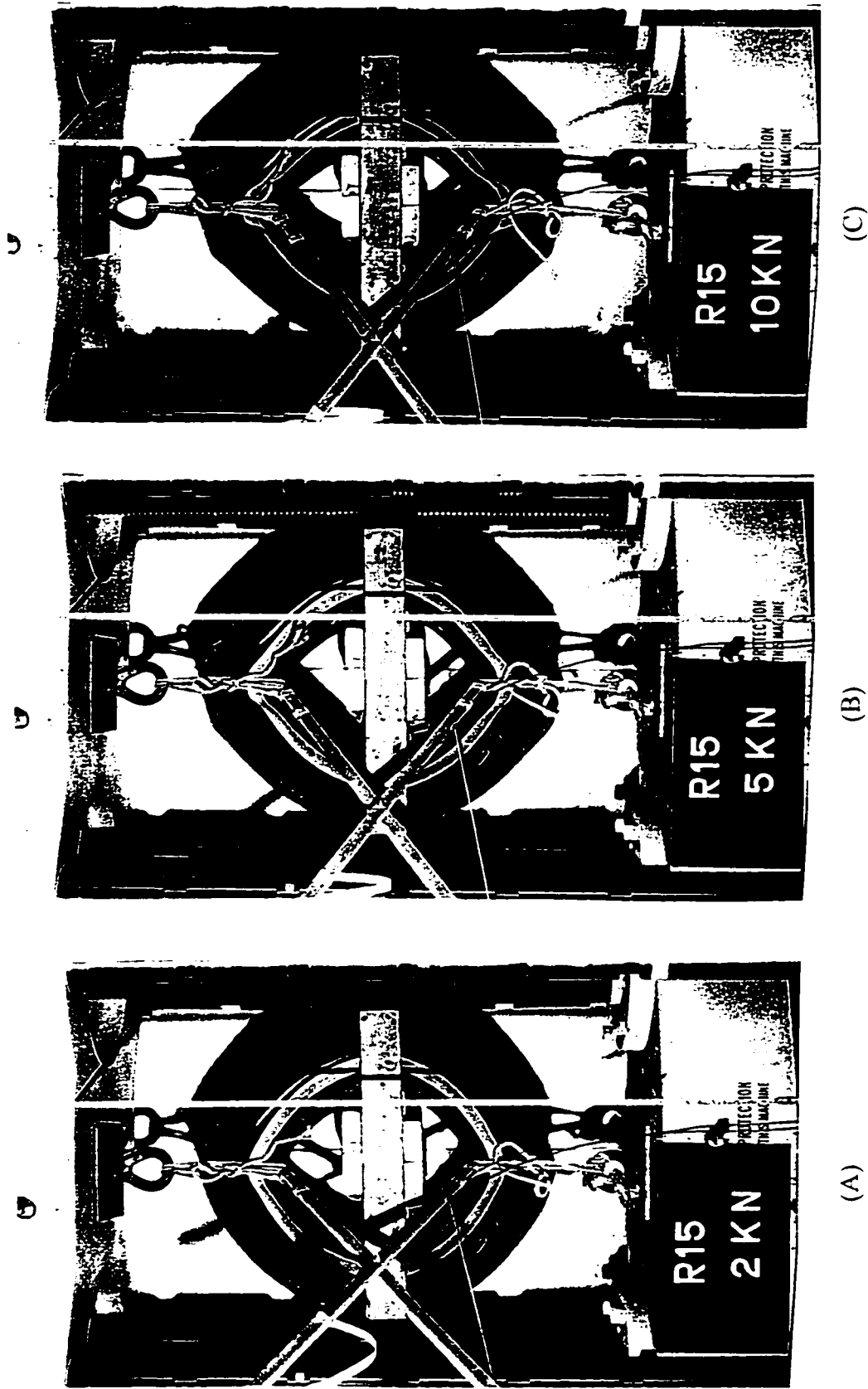


Figure 3.23. A series of photographs showing the change in elongation of the Motormaster XTS P215/75 R15 tire with lateral confinement as a function of the applied tensile force: (A) 2 kN, (B) 5 kN, (C) 10 kN.

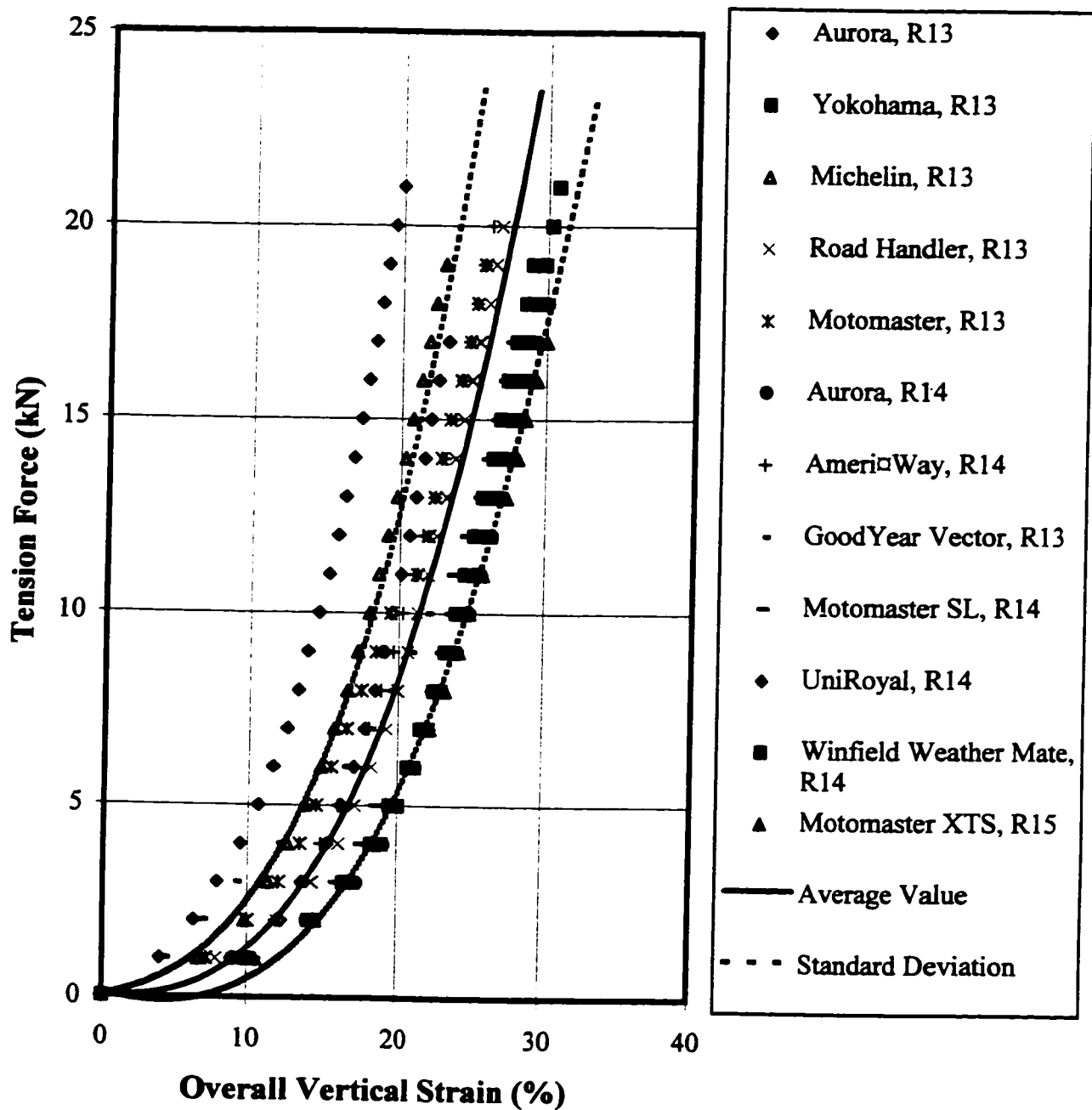


Figure 3.24. The overall tensile force-strain relationship for several different brand name tires which were laterally constrained.

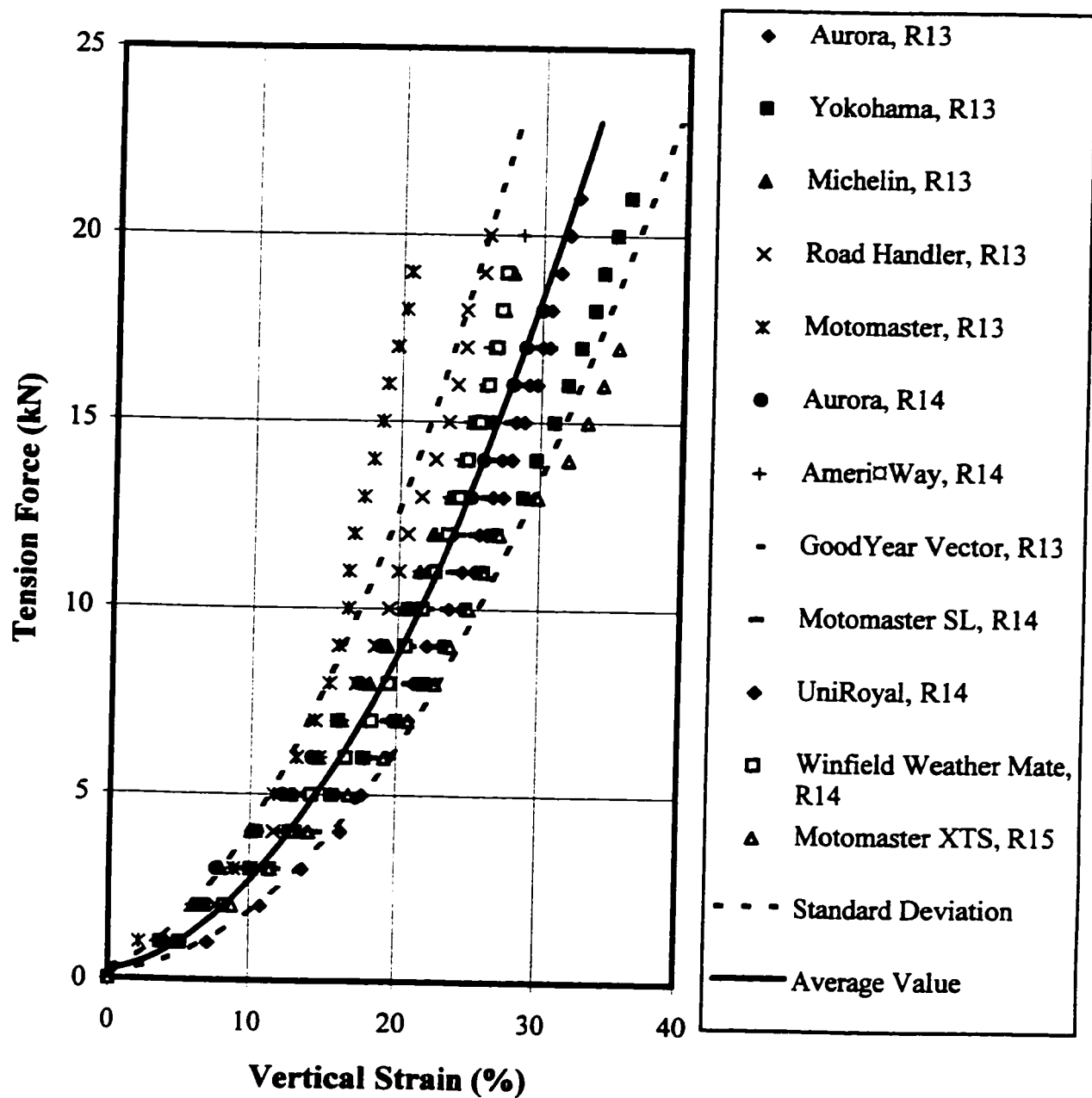


Figure 3.25. The relationship between the applied tensile force and vertical strain within the tire for several different tire brands.

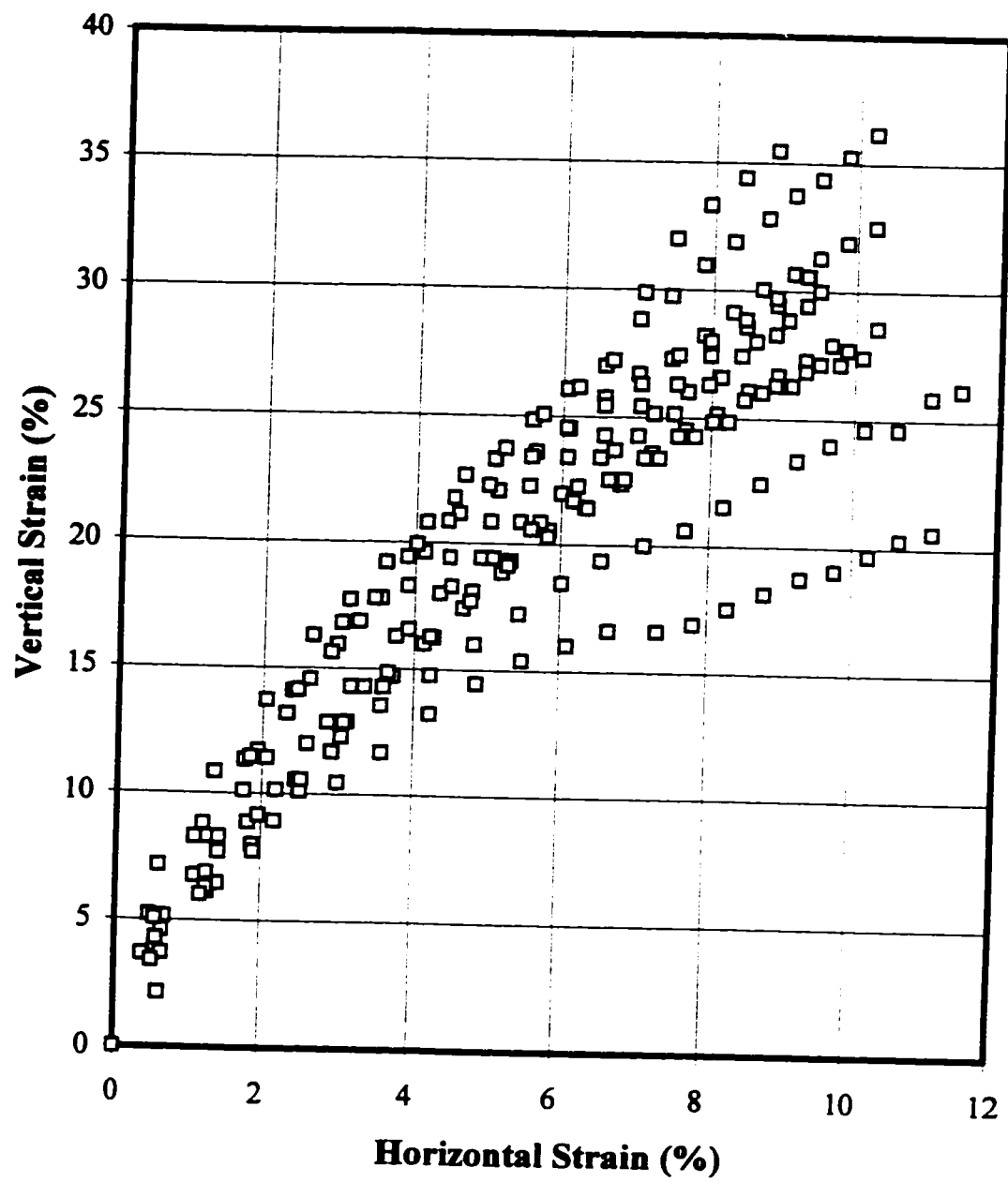


Figure 3.26. The vertical and horizontal strain relationship: whole tires only.

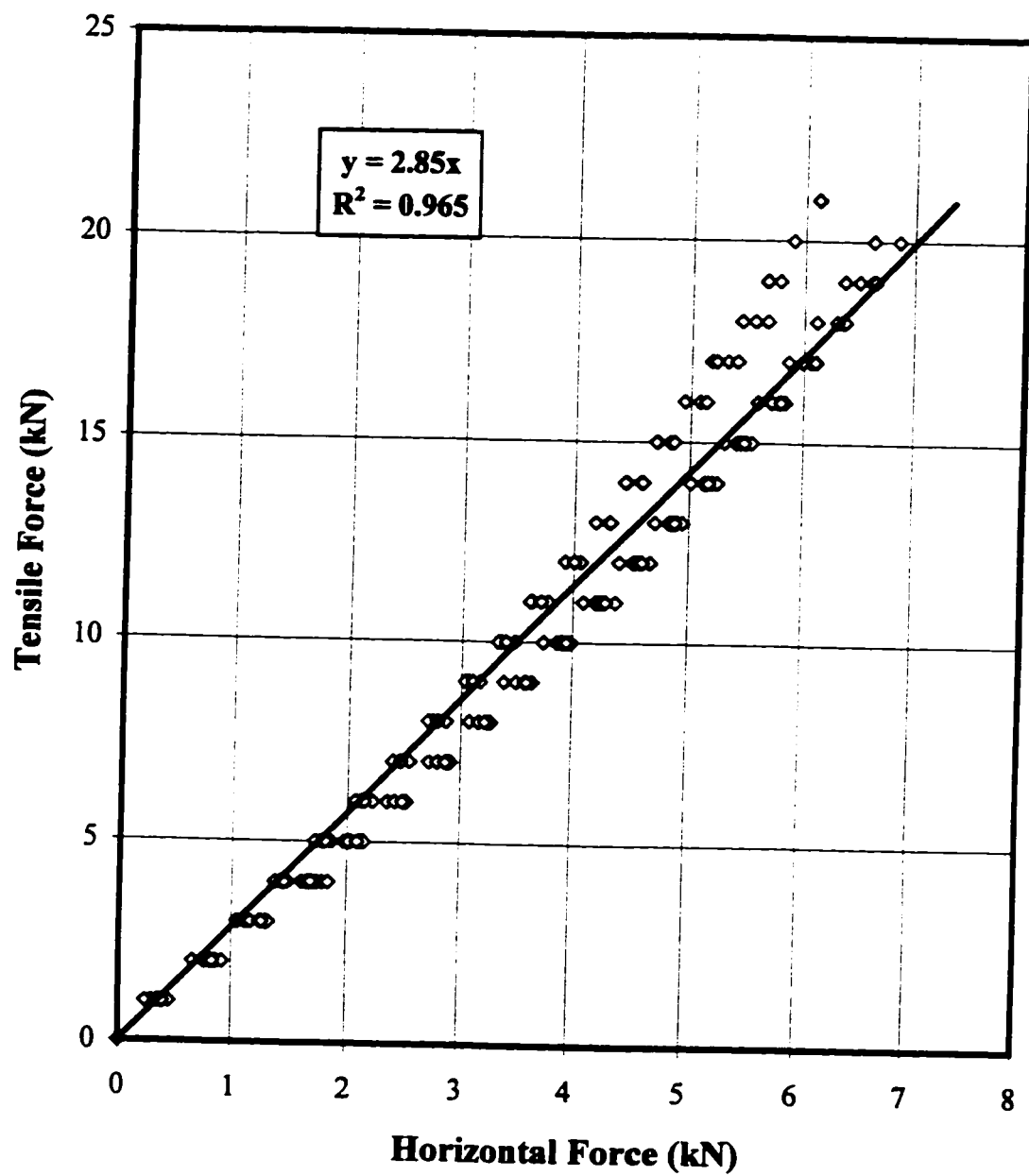


Figure 3.27. The relationship between the applied tensile force and the measured horizontal force.

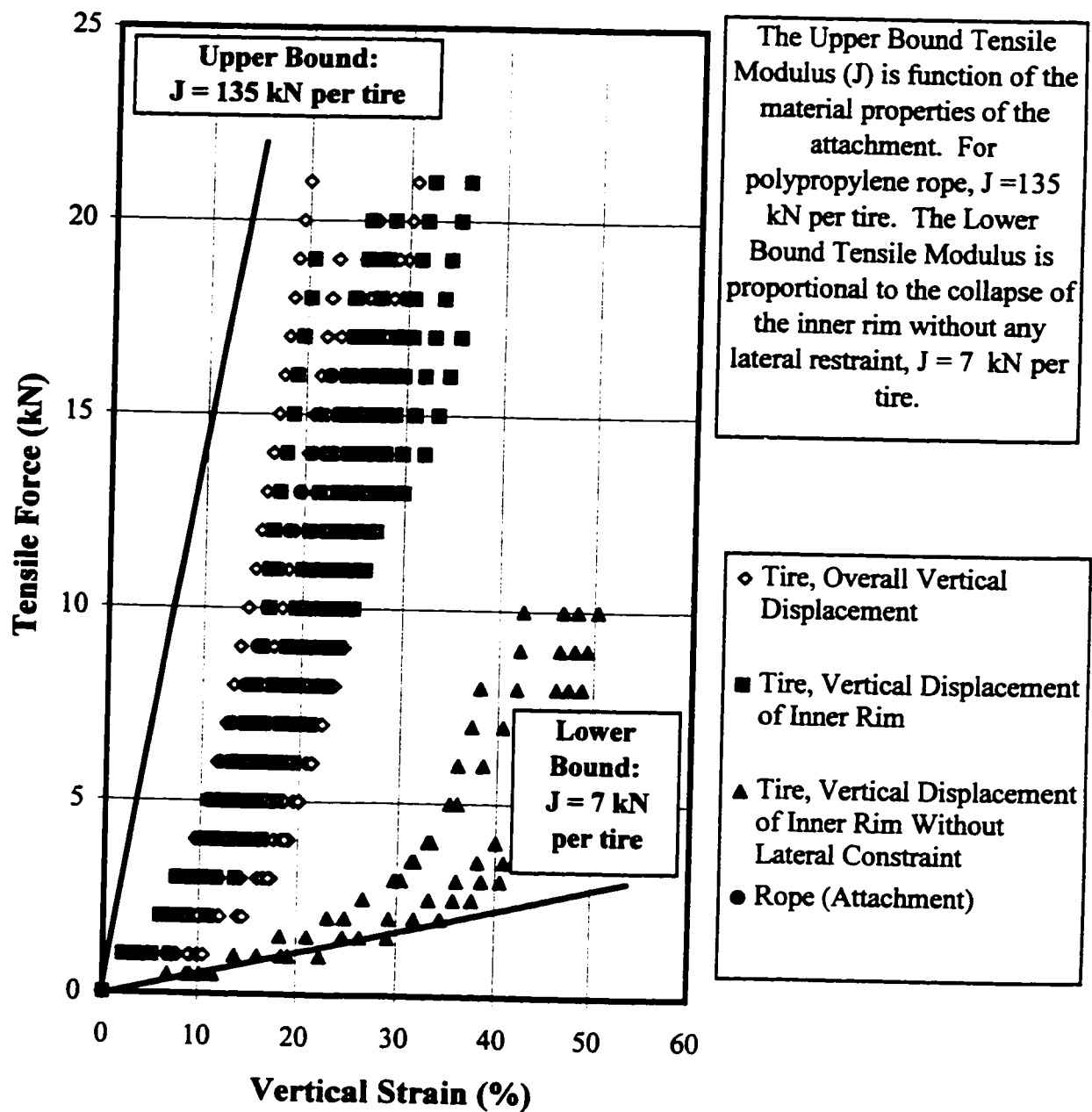


Figure 3.28. The possible range of values for the tensile modulus of tire reinforcement.

CHAPTER 4

INTERACTION PROPERTIES

4.1 Introduction

One of the major technical components of the investigations was to assess the strength behaviour of the tire-soil composite material by means of full scale pull-out tests. Complementary data was also obtained from a limited number of laboratory tests. This Chapter describes these investigations and summarises the results.

The effects of forces acting upon a deformable body can be analyzed with respect to three conditions: equilibrium, compatibility, and stress-strain relations. Theory of plasticity is often used in soil mechanics to model soil behaviour, especially at large strain. This theory assumes that sufficient strains have developed at every point in the zone of plastic deformation to mobilize the ultimate strength. This theory simplifies the analysis to one of equilibrium. Limit equilibrium analysis of reinforced soil structures characterizes two mechanisms or modes of interactive failure (Figure 4.1) which are: preferential direct sliding of a mass soil over a layer of reinforcement (as part of a composite failure mechanism) or pull-out of the reinforcement due to insufficient bond length behind the failure plane. An understanding of the relevant parameters governing this interaction is essential for a proper analysis and for design of reinforced earthworks.

During the course of the present study, the interface characteristics were assessed by using two techniques, namely, a laboratory direct shear box apparatus, and full scale pull-out tests in the field on selected tire assemblages.

4.2 Mechanisms of Interaction

Jewell et al. (1984) identified three main modes of interaction between the soil and the reinforcement (Figure 4.2). These are:

- soil shearing on plane surfaces of the reinforcement (interface friction) which are parallel to the direction of the relative movement of the soil;
- soil bearing on surfaces (passive resistance) which are normal to the direction of the relative movement of the soil;
- soil shearing over soil within the apertures of the reinforcement (internal resistance of the soil).

4.3 Resistance to Direct Sliding

Direct sliding failure occurs when a shearing force causes a soil to shear at its interface with the reinforcement. For soil structures reinforced with tire mats, two mechanisms of interaction resist direct sliding: soil shearing over sidewall surfaces and soil shearing over soil contained in the tire apertures.

4.3.1 Interface Friction

The mechanism of interface friction between a plane reinforcing surface and a soil is schematically represented in Figure 4.3. Adhesion between the reinforcement and the soil transfers the applied shear stress to the reinforcement. Friction is solely relied upon to develop the sliding resistance from planar structural elements such as sheets, bars, or strips. For grids and anchors, the sliding resistance is increased with the addition of bearing elements resulting in a more positive interlock between the reinforcement and the soil (discussed later). The sliding resistance generated is dependent on the interface characteristics of the soil and reinforcement

and the applied normal stress. The stress-strain characteristics of the soil are non-linear and are stress level dependent. Consequently, selected design parameters are usually determined experimentally, e.g., by pull-out tests, by direct shear tests between soil and reinforcement, instrumented models and measurements on full-scale structures (Mitchell and Villet 1987).

The shear stress along the soil-reinforcement interface is given by:

$$\tau = \mu \sigma_n \quad (4.1)$$

where σ_n is the normal stress exerted on the reinforcement; μ is the coefficient of friction between the soil and reinforcing material. The friction coefficient, μ , is defined by the angle of interface friction (δ), also called skin friction, which is usually determined from modified direct shear tests. The shear test is performed on a plane surface of the reinforcement mounted flush with respect to the mid-plane of the apparatus permitting the soil in the upper half to be sheared across it. The direct shear test is simple and easy to operate, however it has several limitations. Yoshimi and Kishida (1981) listed four main disadvantages. First, the shearing stresses and strains are not equally distributed over the reinforcement surface which induces a progressive failure of the soil specimen. Second, it is difficult to measure any change in the effective normal stress under constant volume testing conditions. Third, the generation of vertical friction along the upper shear box walls interferes with the free volume change of the soil specimen under constant normal stress conditions. Four, the stress-displacement relationship is hampered because of the inability of the direct shear apparatus to make independent measurements of the shear strains produced by the soil and slippage along the interface.

The ratio of interface friction to soil friction (δ/ϕ) for soils ranging from sands to silts acting on various construction surface materials was performed by Potyondy (1961), and typically ranged between 0.5 and 0.8. That is

$$\mu = \tan \delta \approx (0.5 \text{ to } 0.8) \tan \phi \quad (4.2)$$

where δ is the interface friction angle between a soil and a smooth surface; and ϕ is the internal friction angle of the soil. Yoshimi and Kishida (1981) evaluated the interface friction between sand and metal surfaces by a ring torsion apparatus. The coefficient of skin friction varied from 0.2 to 0.8 and was governed by surface roughness of the steel, irrespective of the sand density. The shear deformation of a sand mass was found to be unaffected by surface roughness (Uesugi and Kishida 1986). Direct shear tests between various geotextiles and different cohesionless soils demonstrated that many geotextiles could mobilize a high percentage of the soil's friction, between 80 90% (Koerner 1994). Coefficients of interface friction between tire rubber and soil have not been previously reported.

Jewell et al. (1984) provided the following expression to estimate the efficiency coefficient of resistance to direct sliding from two different contributions in a composite material:

$$\alpha_{ds} = 1 - f \left(1 - \frac{\tan \delta}{\tan \phi} \right) \quad (4.3)$$

where α_{ds} is the efficiency coefficient of resistance to direct sliding; ϕ is the angle of friction for soil in direct shear, δ is the interface friction for soil on plane reinforcement surfaces; and f is a structural geometric factor related to the fraction of the surface area of the reinforcement to the total area of the sliding plane. In a unit area of tire reinforcement, sidewalls contribute approximately 25% of the area, then the f factor is approximately equal to 0.25.

4.3.2 Testing Procedure

The interface friction or adherence between the tire rubber and the soil were obtained from modified shear box tests. A schematic representation of the 100 mm x 100 mm tire-soil interface shear box is given in Figure 4.4. A solid piece of tire sidewall rubber was secured to the bottom half on the box. The soil of interest was placed in the top half and compacted to the required density by dropping a small square rod from a constant height. Compaction for the cohesive soil was achieved by dropping the small square rod over 3 layers of soil. A normal load was applied

to the test specimen. The shear load, and horizontal and vertical deformations were monitored during the test. The test was repeated for several different normal loads. Both undrained and drained tests were performed. The results were plotted on a Mohr diagram, from which the interface friction angle δ was determined. Interface tests were performed on both backfill materials. For the on-site cohesive fill material, both friction and adhesion were evaluated from the modified shear box test.

4.3.3 Results and Discussion

The shear stress and sliding displacement relationships between tire rubber and dry sand is given in Figure 4.5. The interface behaviour of the cohesive soil was evaluated under drained and undrained conditions; the resulting relationships are presented in Figures 4.6 and 4.7, respectively. Consolidated undrained interface shear tests were performed at a high displacement rate of 0.6 mm/minute to minimise pore water dissipation. A shear displacement rate 0.00064 mm/minute was used for consolidated drained tests. This rate was determined from consolidation tests results (see Chapter 3). The interface friction coefficients (μ), plotted as the shear stress ratio (τ/σ_n), for the different soils and loading conditions are shown in Figure 4.8. The relationship between μ and δ can be stated by $\mu = \tan \delta$. The dry sand gave the highest interface friction coefficient, 0.58, ($\delta = 30^\circ$) while the cohesive backfill under undrained conditions had the lowest value of 0.40 ($\delta = 22^\circ$). The coefficient of interface friction of the sand falls within the range reported for other materials such as concrete, wood, steel, and geotextiles in the literature (Potyondy 1961; Yoshimi and Kishida 1981; Uesugi and Kishida 1986). Sands and gravels should provide better resistance to a direct sliding failure than finer grain soils. Under pressure, coarser soil particles may penetrate the rubber and deform or “roughen” the surface which improves the shear interaction (increases the sliding resistance). The interface friction of cohesive soils are highly influenced by the roughness of the surface and the moisture content (Potyondy 1961). An undrained test with cohesive soil represents the loading conditions predominantly found during the construction stage. For a reinforced structure constructed with cohesive soils, resistance to direct sliding would be at a minimum during construction,

representing the critical condition since the construction pore pressures do not dissipate rapidly. External loads such as heavy equipment used during construction, or the placement of a surcharge can also contribute to failure.

A summary of interface friction results is provided in Table 4.1 and in Figure 4.9. The interface friction or the skin friction (δ) was lower than the shearing strength of the respective soils, in all cases studied. The ratios of skin friction (δ/ϕ) are similar to those reported by Potyondy (1961) for conventional construction materials (steel, wood, and concrete) and Koerner (1994) for geotextiles. It may be more desirable to express the interface friction in terms of the Mohr-Coulomb failure criterion, since the interface shearing interaction is dependent on several factors, including, soil type and testing conditions. The interface friction can be expressed as (Potyondy 1961):

$$\tau_{\text{interface}} = f_c c + \sigma_n \tan(f_\phi \phi) \quad (4.4)$$

where $f_c = c_a/c$, and f_ϕ is equal to δ/ϕ , where c and ϕ are the soil shear strength parameters, and c_a is the interface adhesion. The f_c and f_ϕ parameters for the two soils and drainage conditions, determined in the current study are given below.

Soil Type	f_c	f_ϕ
Dry Sand		0.71
Cohesive backfill (CU)	0.11	1.16
Cohesive backfill (CD)		0.81

The interface behaviour between tire rubber and soil is a function of several factors. These factors include the friction characteristics of the soil, soil density, type of soil, gradation, angularity of the grains, moisture content, effective overburden stress, loading conditions (state of stress), and the amount of fines in the soil.

The efficiency coefficient of resistance to direct sliding (α_{ds}) [Equation 4.3] ranged from 0.82 to 0.95 (Table 4.1). These high values indicate that sliding resistance of a tire mat reinforced structure is predominantly governed by the shear strength characteristics of the soil used in its construction. The tire mat reinforcement geometry is able to fully capitalize on the shear strength provided by the soil. This characteristic of tire mat reinforcement can allow the use of lower quality backfill, further decreasing the cost of the structure.

4.4 Pull-Out Resistance

It is important to understand the mechanism of interface behaviour for reinforcement placed in a soil fill. This Section attempts to provide a basic understanding of this mechanism. A discussion on behaviour of traditional reinforcements is first provided in general terms. The interaction parameters related specifically to tire reinforcement are subsequently presented.

Interaction between reinforcement and soil during pull-out involves three essential load transfer mechanisms: frictional resistance (interface) along planar surfaces, passive resistance generated on transversal elements, and shearing resistance of interlocking soil particles located in reinforcement apertures. Pull-out resistance is highly dependent on soil type, reinforcement properties and testing procedure used. Important soil parameters are shear strength characteristics, dilatancy properties, relative density, the overburden pressure and the amount of fines. Reinforcement properties influencing pull-out behaviour are geometry or form of the reinforcement, extensibility, creep characteristics, and orientation. Parameters regarding testing are the loading system, dimensions of the specimen, boundary conditions and testing procedure.

4.4.1 Frictional Resistance

Based upon Equation 4.1, knowing the vertical stress σ_v , the evaluation of frictional resistance would be a simple calculation. Unfortunately, the normal effective stress is altered by the soil-reinforcement interaction. Dense granular soils tend to dilate during shearing. The reinforcement partly restricts this change in volume, and consequently, the local confining stress increases (Figure 4.10). The actual increase of the normal stress is unknown. The tendency of a soil to dilate decreases with increasing confining stress. Also, most reinforcements have protruding elements in order to increase their adhesive properties; these elements enlarge the shearing zone of the soil. For these reasons, a reliable coefficient of friction is usually estimated from direct measurements. These values are often referred to as the apparent or effective friction coefficients, μ^* . The apparent friction coefficient, μ^* , can be defined by the following ratio from pull-out tests:

$$\mu^* = \frac{\tau}{\sigma_v} = \frac{T}{2bL\sigma_v} \quad (4.5)$$

where τ is the average shear stress along the reinforcement, σ_v is the overburden stress, T is the applied pull-out force, b is the width of the reinforcement, and L is the length of the reinforcement. Reported values of μ^* range from 0.5 for smooth reinforcement buried at depth to values much greater than 1 for rough or ribbed reinforcement at low confining stress (small depths of overburden) (Mitchell and Villet 1987). However, these reported effective friction coefficient values are valid for conventional reinforcing materials and may not reflect the case of soils reinforced with tires.

The influence of surface characteristics of steel strip reinforcement is demonstrated in Figure 4.11. The presence of ribs increases the shearing zone of the soil, and which coupled with the local increase in stress due to soil dilatancy, increases the apparent friction coefficient. For smooth reinforcement, relatively small displacements are required to mobilise maximum bonding strength. Greater displacements are required to fully develop maximum resistance for

reinforcements with roughened surfaces. The variation of the apparent friction coefficient with overburden stress for both smooth and ribbed reinforcement is presented in Figure 4.12. Clearly, the decrease in μ^* with increasing vertical depth is more pronounced for reinforcements with roughened surfaces. Under high confining stress, the presence of ribs enables the reinforcement to mobilize the shearing resistance of the soil and therefore the apparent friction coefficient approaches the value of $\tan \phi$ (internal friction angle of the soil). For smooth reinforcement, the apparent friction coefficient value converges towards $\tan \mu$, the soil-reinforcement interface angle at large displacement.

4.4.2 Passive Earth Resistance

The passive resistance of the soil is mobilized along reinforcement surfaces (large ribs, discs and or transversal members: bars mats) normal to the pull-out force direction, or when the inclusion has some bending stiffness and is sheared or bent along the zone of potential failure. Load transfer by passive soil resistance is illustrated in Figure 4.13. The maximum pull-out resistance developed on transversal elements is similar to the bearing capacity of a deep foundation. The bearing members are considered deeply embedded since the thickness of the reinforcement is small compared to overburden height (z/t is large, where z is the burial depth and t is the thickness of the reinforcement). The passive resistance or effective bearing resistance (σ'_b) can be expressed as a function of the vertical stress in the form

$$\sigma'_b = F\gamma'\sigma'_v \quad (4.6)$$

Where $F\gamma'$ is a bearing factor dependent upon soil strength characteristics and dilatancy properties and to a lesser extent on the reinforcement surface roughness and the initial stress state of the soil (Mitchell and Villet 1987). These two last factors have little influence at large depth and are usually neglected.

The bearing factors ($F\gamma'$) have been evaluated by several analytical procedures and the results are presented in Figure 4.14 as a function of soil friction angle. Rowe and Davis (1982) reported the

behaviour of anchor plates in sand using a finite element analysis. The two curves, shown in Figure 4.14, were generated for the case of no dilation (constant volume) and for soil dilating during shear according to the stress dilatancy flow rule. The lower bound curve represents punching failure mode of the soil and was obtained by slip line solution derived by Jewell et al. (1984) to give:

$$F\gamma' = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \exp\left(\frac{\pi}{2} \tan \phi\right) \quad (4.7)$$

The upper bound curve is based upon the classical Prandtl bearing capacity theory with the assumption made by Jewell et al. (1984) that $\sigma'_h = \sigma'_v = \sigma'_n$ which results in the following expression:

$$F\gamma' = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \exp(\pi \tan \phi) \quad (4.8)$$

Jewell et al. (1984) summarized the results of several investigations on the bearing resistance of anchors and grids and compared them with theoretical values as shown in Figure 4.14. Oversen and Stroman (1972), Neely et al. (1973), and Das and Seely (1975) investigated the pull-out resistance of vertical anchors. Pull-out tests on short grids embedded in sand were undertaken by Hueckel and Kwasniewski (1961). Chang et al. (1977) and Peterson (1980) performed relatively large-scale pull-out tests on grids. Jewell (1980) conducted large scale direct shear tests on a reinforcement grid inclined to a central plane. Although the various results show some spread and variability, they are bounded by the upper and lower theoretical predictions and are therefore reasonable well defined by these Rowe and Davis's curves. Hence, a reasonable assessment of the passive resistance of a transverse element for conventional earth reinforcement can be predicted from Figure 4.14. However, bearing resistance of frontal tire elements would not be similar to deeply embedded plates, since confining pressures are greatly reduced.

4.4.3 Combined Frictional and Passive Resistance

Generally, the overall pull-out capacity of most reinforcements is a combination of both friction and passive resistance, with the exception of geotextiles used as sheet reinforcement and smooth strip or rod reinforcements. Friction and passive resistance are not necessarily completely additive. The relative contributions of each resistance to the overall pull-out capacity is dependent not only on the maximum values of both frictional and passive force that can be mobilized, but also on the relative soil to reinforcement displacement required to fully mobilize each component. Therefore, the respective individual contribution to the pull-out capacity is strongly dependent upon the relative displacement. Bacot (1981) and Schlosser et al. (1983) investigated the relative importance of the two parameters by performing pull-out tests on bar mats. Morbois and Long (1984) measured the respective contribution of both friction and passive resistance on the pull out capacity of rods equipped with circular transversal anchor plates (Figure 4.15). The relative soil to reinforcement displacement necessary to activate maximum frictional resistance is small (~5mm) as compared to the required movement for total passive resistance, greater than 25 mm. Schlosser (1990) stated that friction is always completely mobilized before the passive resistance, and consequently, the most important soil reinforcement interaction parameter is friction.

Christopher et al. (1990) proposed the following general equation to estimate the ultimate pull-out capacity per unit width of reinforcement, P_r :

$$P_r = F^* \times \alpha \times \sigma'_v \times L_e \times C \quad (4.9)$$

in which $L_e \times C$ is the total area of the reinforcement in the resistant zone behind the potential failure surface, L_e is the embedment or adherence length of the resisting zone behind the failure surface, C is the reinforcement effective unit perimeter: 2 for strips, grids, and sheets; π for nails; F^* is the pull-out resistance or friction-bearing interaction factor, α is a scale effect correction factor and σ'_v is the effective vertical stress at the soil-reinforcement interface. The pull-out

resistance factor F^* is usually evaluated from pull-out tests. It may also be estimated from the general equation:

$$F^* = \text{Passive resistance} + \text{Frictional resistance}$$

$$F^* = F_q \times \alpha_\beta + K \times \mu^* \times \alpha_f \quad (4.10)$$

in which F_q is the embedment or surcharge bearing capacity factor, α_β is a structural geometric factor for passive resistance, K is a ratio of the actual effective normal stress to the effective overburden stress, dependent on the reinforcement geometry, μ^* is the apparent friction coefficient and α_f is the structural geometric factor for frictional resistance. The non-linearity of the $P_r - L_e$ relationship is reflected in the scale effect correction factor α and is primarily dependent on the extensibility of the reinforcement. It is approximately equal to 1 for inextensible reinforcement and can be significantly less than 1 for extensible reinforcement (Mitchell and Christopher 1990).

4.4.4 Pull-Out Resistance of Tire Mat Reinforcement

Soil structure constructed with tire mat reinforcement would be similar to conventional reinforced structures using a planar reinforcement. Therefore, interaction between soil and tire mat reinforcement is predominantly governed by friction. Frictional resistance is a combination of interface friction between soil and tire sidewall and friction between interlocking soil particles found in the reinforcement apertures, i.e. the tire annulus. Because of the lack of transversal elements during pull-out, any passive resistance developed would be minimal. An average tire width for a passenger vehicle tire is approximately 0.60m. An estimate of the ultimate pull-out capacity per unit width of tire reinforcement (P_r) can be made by modifying Equation 4.9.

$$P_r = F^* \times R_t \times \sigma'_v \times L_e \times C \quad (4.11)$$

where:

R_t = tire reinforcement coverage ratio: the equivalent area of tire reinforcement per unit width (per metre) and is equal to 5/6 for passenger tires (the unit width divided by 2 passenger tire widths: $1/(2 \times 0.6 \text{ m})$);

C = the tire mat reinforcement unit perimeter and is equal to 2 ;

F^* = the apparent friction coefficient;

the remaining parameters have already been defined.

The apparent friction coefficient F^* is best evaluated from pull-out tests performed in the specified backfill used to construct the soil structure. Alternatively, F^* can be defined by a theoretical relationship:

$$F^* = \alpha_b \times \tan \phi' \quad (4.12)$$

where:

ϕ' = the effective interface friction angle of the backfill;

α_b = the bond efficiency coefficient, a correction factor for reinforcement geometry and any scale effects.

For planar or sheet reinforcement, the bond efficiency coefficient (α_b) and the resistance to direct sliding efficiency coefficient (α_{ds}) are identical. However, for extensible reinforcement such as the case of tire mats, the non-uniform mobilization of bond along the length of the reinforcement could result in a lower value for the bond efficiency coefficient than the one estimated from Equation 4.3, especially in a cohesive soil.

Rearranging Equation 4.11, the ultimate pull-out capacity per unit width of tire reinforcement, under drained conditions ($c' = 0$), can now be expressed as:

$$P_r = (5/3) \times \alpha_b \times \tan \phi' \times \sigma'_v \times L_e \quad (4.13)$$

Under undrained conditions (a cohesive backfill), the ultimate pull-out capacity can be expressed as:

$$P_r = (5/3) \times \alpha_b (c_u + \sigma_v \tan \phi_u) \times L_e \quad (4.14)$$

In practice, pull-out tests should be performed to measure the actual bond resistance and to verify that it does not surpass the value derived from analysis. For design purposes, the lesser of the two values should be adopted.

4.5 Experimental Behaviour of Tire Reinforcement in Pull-Out

Full scale pull-out tests were performed on different tire assemblages embedded in the two backfill materials. Factors that influence the pull-out resistance of tires were also investigated. Of particular interest were:

- type of tire element used, whole or cut tire;
- reinforcing configuration, single unit, linear or mat reinforcement;
- the number of tire elements used;
- type of soil;
- location of reinforcement in the test berm;
- the effects of grouping the tire together;
- vertical confining stress (embedment depth).

Randomly selected tires from the construction stockpile were used for the pull-out tests.

4.5.1 Test Description

For convenience, the pull-out tests were carried out separately from the experimental test fill. Each reinforcing tire configuration tested was placed on a compacted soil bed and covered with the appropriate material and compacted to the required density using a vibrating roller. The two soil types were the same as those used in the test embankment, i.e., imported sand and on-site

cohesive soil. The tires were tied together with 3 turns of the 9.525 mm diameter polypropylene rope, to produce the different tire reinforcing configurations. A schematic diagram of the principal components of the pull-out test is presented in Figure 4.16. The front tires of each reinforcing element were attached to the pull-out apparatus by a system of chains. To ensure an equal transfer of force to each frontal tire element, a whiffletree (harness) system was employed, as illustrated in Figure 4.17. The pull-out apparatus comprised of a hydraulic cylinder and load cell mounted on a steel frame with sleds. The pull-out apparatus was anchored by a steel cable to a heavy excavator. Tension was applied by an Enerpac RR-5020 hydraulic cylinder which had a maximum tensile capacity of 160 kN, and with a corresponding traction rate of 0.70 mm/s.

The pull-out force was measured by a Servo System load cell with a capacity of 220 kN and a standard error of ± 0.8 kN. The test commenced with the hydraulic jack fully extended. Monitoring of the retraction of the hydraulic cylinder was used to measure frontal displacement. It was marked and set against a scale (ruler); readings were taken from afar by a theodolite and were within ± 1 mm.

To measure rear displacement and elongation of the tire element, a stiff nickel-steel wire, 0.80 mm diameter, was attached to the last tire(s). The free end was directed towards the rear of the berm, and where it was read against a scale (Figure 4.17). Two holes were drilled in the tread section of the rear tire at opposite ends; the wire was fed through both holes and attached to the frontal section of that particular tire element. The wire protruding from the rear of the tire was encased in a plastic tubing, to allow free movement of the wire within the soil. The measured movement of the rear of the tire assembly was directly read from the tip of the wire against the scale to ± 1 mm precision.

4.5.2 Pull-Out Test Configurations

The different tire configurations tested are presented Figure 4.18. Most of these configurations were used for whole and cut tires and in both soil materials. The different tires elements were arranged in several ways:

- as a single unit;
- a linear alignment;
- square mat grouping;
- rectangular mat grouping;
- a diamond or triangular form;

The height of the compacted soil cover varied from 0.5 to 1.0 m. The location of the front tire element was between 0.5 to 1 m from the edge of the berm, and was recorded for each test. For analysis of data, allowance was made for the frictional resistance developed by the soil wedge in front of the tire assembly.

4.5.3 Results and Discussion

4.5.3.1 Load-Displacement Behaviour

Figure 4.19 shows a view of the principal components of the pull-out test. A summary of the pull-out test results is given in Table 4.2. The maximum effort to pull-out a 4x4 cut tire mat configuration embedded in 1 m of sand and placed 1 m behind the edge of the testing berm was 72.5 kN, with a corresponding frontal displacement of 0.651 m. The ultimate pull-out resistance of a 4x4 full tires mat configuration in sand, under identical conditions, was slightly less, around 70 kN with a corresponding frontal displacement of 0.816 m. Similar results were observed for other tire mat configurations between cut and full tires in sand. The two sidewalls in full tires tend to slightly lower the ultimate pull-out resistance. This behaviour was expected, since the density of whole tire reinforcement is slightly less than those of cut tires, due the presence of voids within the tires especially under low confining pressures. The cut tire reinforcement can produce higher frictional resistance by providing a better contact between the soil and the tire reinforcement. The amount of frontal displacement required to achieve maximum resistance varied with the number of tire elements and type of configuration used. The pull-out force per tire width for cut tires (Figure 4.20), placed 1 m behind the edge of the testing berm and under a confining pressure of 14 kPa, varied from 10.6 kN for a single tire element to 26 kN for a 1x4

linear configuration. Tire mat configurations generated less resistance, averaging around 18 kN per tire width for reinforcing length of three or more tires. Linear reinforcement configurations may generate additional frictional resistance along the sides of the tire (tread portion) and some passive resistance can also be produced between successive tire elements in this case. The load-displacement tests for full tires in sand (Figure 4.21) and cut tires in cohesive backfill (Figure 4.22) produced similar results.

For full tires in sand, placed 1 m behind the front edge of the testing berm and under a confining pressure of 14 kPa, the pull-out force per tire width varied from 8.7 kN for a single tire unit to 34.8 kN for a 1x4 linear configuration. Again, the tire mats generated less resistance, averaging about 18 kN per tire width for reinforcing lengths of three or more tires.

For cut tires in cohesive backfill, placed 1 m behind the front edge of the testing berm and under a confining pressure of 10 kPa, a 1x3 linear configuration produced an ultimate pull-out resistance of 16.2 kN. Tire mats generated on average a pull-out force per tire width of 12.5 kN, with the exception of the 4x4 configuration which produced a resistance of 15.5 kN per tire width. It is interesting to note that under higher confining pressure (20 kPa), failure of the attachment scheme occurred for a reinforcing length of three and four tires. Failure of the rope occurred in the 1x3 linear alignment and the 1x4 linear alignment. In the first case, the rope failure occurred at a pull-out force of 57.1 kN with a corresponding displacement of 0.657 m, while for the 1x4 arrangement, the pull-out force increased to 66.8 kN at a displacement of 0.906 m. The laboratory tests on rope strength indicated an ultimate rope capacity of approx. 30 kN for three wraps. The field results indicate that $P_r - L_e$ relationship is non-linear resulting from a non-uniform shear displacement distribution.

Failure of the rope in some tests indicates that the limiting factor in the pull-out resistance of tire reinforcement is the tensile strength of the attachment, especially under high confining stress. Therefore, it is important to select the appropriate attachment scheme but also to correctly evaluate its tensile capacity.

Frontal and rear displacements for the three major testing schemes (3x3 cut and full tire mat arrangement in sand; 4x4 cut tire in cohesive backfill) as a function of applied pull-out force are given in Figure 4.23 to 4.25, respectively. A progressive failure was observed in the tires, similar to those reported by Long (1993). The front row of tires are fully mobilized first, then the force is transmitted to the second row, resulting in the elongation of the rope attachment (Figure 4.26), and finally the third row of tires is engaged after full mobilization of the second row of tires. After failure of the last row of tires, the entire tire mat reinforcement begins to slide along the base of the testing berm carrying the overburden soil with it. This characteristic behaviour increased the amount of displacement required to fully mobilize the ultimate pull-out resistance. The ultimate pull-out force was proportional to the number of tire elements used.

The pull-out behaviour of tire reinforcement can be divided into two zones, extension of the reinforcement which occurs first, followed by sliding of the reinforcement. Photographs showing failure of a 3x3 cut tire reinforcement in sand are presented in Figure 4.27. Tire elements elongated in the direction of pull by transforming from a circular shape to a pronounced oval form (Figure 4.28). After failure of the rear tire element which induced sliding of the reinforcement, the deformed shape of the entire reinforcing tire elements remained unchanged. Pull-out tests performed on tire reinforcements while varying the vertical overburden pressure demonstrated similar load-deformation curve (Figure 4.29). The increase in pull-out resistance was not directly proportional to the increase in the confining pressure. The pull-out resistance at lower confining stress was influenced by the ability of the dense sand to dilate. This behaviour is similar to that reported for conventional reinforcements placed in dense sands (Jones 1985, Mitchell and Villet 1987, Christopher et al. 1990). The degree of confinement of the different tire elements within the soil mass influences the load-deformation behaviour. Tire reinforcement placed closer to the front edge of the test embankment demonstrated greater deformation for an equivalent pull-out force (Figure 4.29). The decrease in pull-out resistance and the corresponding increase in frontal displacement results from both a decrease in frontal wedge resistance (the soil mass placed in front of the first row of tires) and the extrusion of the frontal row of tire from the test embankment (Figure 4.30). As the frontal tire elements extruded from the soil, they were no longer confined and were more easily deformed.

Based on field observations, the amount of pull-out resistance developed during testing was the sum of the shearing resistance developed in both the frontal soil wedge and the tire mat reinforcement, as shown in Figure 4.31. The frontal soil wedge acted as a single mass shearing along its base. Once the individual tire, or row of tires were engaged, they would start to move along the base, pushing the frontal soil wedge. The tire reinforcement would then carry all the soil mass above it, as shown in Figures 4.27 and 4.30. This pull-out behaviour was associated with the low confining pressures at which the tests were conducted. The tire reinforcements were not entirely “pulled out” of the soil mass. It is stipulated that under higher confining pressures, it would be very difficult for the tire mat to carry the overburden soil mass. They would be extruded from the soil, similar to conventional reinforcement, as long as the attachment strength was not exceeded.

Due to the large displacements required to fully mobilize the ultimate pull-out resistance, it is recommended that two tire lengths (the bond length) should be used in design. The use of additional tire elements would not be beneficial in generating additional the pull-out capacity. However, as stated earlier, the strength of the attachment will generally govern the pull-out capacity of the tire reinforcement at depth.

It is of interest to compare the P_r values from pull out tests in tires with results of typical pull out tests on geotextiles and extruded geogrids. Test data on gravelly sand obtained by Bonczkiewicz et. al. (1990) suggests a P_r value of approximately 20 kN/m for non-woven geotextiles, and a P_r value of 60 kN/m for extruded geogrids. The equivalent value for tire mat reinforcement in the type of soils tested was approximately 35 kN/m at peak strength.

4.5.3.2 Apparent Friction Coefficient

Based on the pull-out model presented above, the apparent friction coefficient (F^*) can be estimated by:

$$F^* = \frac{P}{(\gamma_f V_w + A_{re} \sigma_v)} \quad (4.15)$$

Where:

P = the measured pull-out force.

γ_f = the unit weight of the soil.

V_w = the volume of the frontal wedge.

A_{rc} = the tire reinforcement area (one surface only since the tire mat is being dragged out along the bottom interface).

The apparent friction coefficient for cut tires in sand ranged from 0.78 to slightly above 1, using the above equation (Figure 4.32). The frictional resistance developed in the tire reinforcement is primarily a function of the shear strength that can be mobilized within the soil. An apparent friction coefficient of 0.9 represents the peak strength of the sand ($\tan \phi'$). Maximum F^* values for both cut and full tire reinforcement (Figure 4.33) are slightly above or below $\tan \phi'$. The lowest friction values for both cut and full tire reinforcement placed in the sand were reported for the 4x4 tire reinforcement. The 4x4 arrangement was the largest tire mat reinforcement tested. Due to the large displacement required to fully mobilized the peak pull-out resistance, the frontal wedge of soil lost a substantial amount of mass as it was pushed forward. Also, being the longest tire mat reinforcement, it may be more affected by the residual friction angle of the sand. The average apparent friction coefficients for mat reinforcement placed in sand are 0.9 for cut tires, and 0.87 for whole tires. Therefore, in a coarse-grained backfill material, tire mat reinforcement geometry, for both cut and whole tires, is able to fully capitalize the shear strength provided by the soil.

Tests performed on cut tires placed in the cohesive backfill (Figure 4.34) reported a similar behaviour. The apparent friction coefficient varied from 0.92 to 1.2. The pull-out test were performed relatively quickly, and therefore, undrained conditions were assumed to be predominant. The amount of cohesion mobilized during pull-out is questionable. This aspect is further discussed in the estimation of the bond efficiency coefficient.

The apparent friction coefficients were plotted against frontal displacement strain which is defined as the relative frontal movement as a percentage of the length of the tire reinforcement. Hence for a tire mat with a length of three tires, the reinforcement would be 1.8m (3 x 0.6m). In most cases, the peak pull-out resistance was developed at an approximate frontal strain of 30%. Also, most tests showed a strain softening type behaviour after reaching peak resistance. Strain softening was most predominant in the cohesive backfill.

4.5.3.3 Bond Efficiency Coefficient

Bond efficiency coefficients (α_b) for tire reinforcements placed in sand were estimated by:

$$\alpha_b = \frac{(P_{\max} - \gamma_f V_w \tan \phi')}{(A_{re} \sigma_v \tan \phi')} \quad (4.16)$$

The influence of the frontal wedge resistance is subtracted from the pull-out term in Equations 4.16 and 4.17. Therefore, the bond efficiency represents the frictional resistance developed by the tire mat reinforcements. The bond efficiency coefficients for both types of tire reinforcements (cut and whole) in sand are compared to the estimated direct sliding efficiency coefficient (α_{ds}), based upon Equation 4.3 in Figures 4.35 and 4.36. The bond efficiency coefficient estimated by Equation 4.16 for sand are slightly above the α_{ds} line. They tend to fall below the direct sliding efficiency coefficient line for reinforcing lengths greater than 1.8 m (3 tire length). The bond efficiency coefficient is also influenced by the ability of the soil to dilate during pull-out as shown in Figure 4.37. It is clearly observed that under a lower overburden stress, i.e. higher tendency to dilate during shear, a higher bond efficiency coefficient is developed. The influence of soil dilation or overburden pressure on pull-resistance diminishes greatly for embedment depths greater than 1 m. Therefore, Equation 4.3 can be used to provide an estimate of the bond efficiency coefficient (α_b) for granular backfills or for cohesive backfills under drained conditions, where cohesion c' is neglected.

Bond interaction between the cohesive backfill and cut tires is presented in Figure 4.37. The bond efficiency coefficient is estimated by:

$$\alpha_b = \frac{(P_{\max} - (\gamma_f V_w \tan \phi_u))}{((c_u + \sigma_v \tan \phi_u) A_{re})} \quad (4.17)$$

The pull-out tests were performed under undrained conditions and consequently the undrained cohesion has been included in the resistance developed by the tire reinforcement but excluded from the frontal wedge. The bond efficiency coefficients (α_b) are well below the α_{ds} line and essentially range between 0.25 and 0.4 with the exception of tire reinforcements subjected to the larger overburden stress of 20 kPa. The higher values of α_b are associated with the development of cohesion within the frontal wedge which is not considered in Equation 4.17. The average α_b value is slightly less than 0.3. Watts and Brady (1990) reported similar findings for pull-out tests performed on geogrids embedded in clayey backfill. The cohesive component was reduced by a factor ranging between 0.3 and 0.4. Since tire reinforcement are highly deformable during pull-out, it can be deduced that the pull-out force drops rapidly to zero or near zero values towards the rear tire elements. As a result, the peak adhesion (the controlling factor under low confining pressure) is mobilized only over a short length of the tire reinforcement. Consequently, the estimation of bond efficiency coefficient based upon Equation 4.17 may render a low factor. Since no other pull-out tests of tire reinforcement embedded in a cohesive soil have been reported, a α_b value of 0.3 should be used in Equation 4.14. However, in the long term, drained conditions may be more critical.

The results clearly indicate that the bond efficiency coefficient in soils is governed by shear strength of the backfill, loading conditions, and overburden pressure (soil dilation). It should also be noted that in practice, a reasonable estimate of the bond efficiency coefficient for sands is between 0.8 (full tires) to 0.9 (cut tires). The bond efficiency coefficient for the undrained loading case in clays drops to approximately 0.3 for reinforcement length greater than one tire length. This value would increase as the pore pressure dissipates, and an effective angle of

friction is mobilized. The value of pull out resistance that can be mobilized for fills in clays would depend on the shear strength parameters of these clay in undrained and drained conditions. The appropriate shear strength parameters should be applied in Equations 4.13 and 4.14 to determine P_r .

4.5.3.4 Tire Reinforcement Deformation

The load-deformation relationship of the various tire mat arrangements can be conveniently expressed in the following polynomial form:

$$P_t = a_1 \epsilon^4 + a_2 \epsilon^3 + a_3 \epsilon^2 + a_4 \epsilon + a_5 \quad (4.18)$$

where P_t is the pull-out force per tire width; ϵ is the pull-out frontal strain, defined as the relative frontal movement divided by the initial length of tire reinforcement; and $a_1, a_2, a_3, \dots$ are the polynomial constants, which were evaluated using the least-square fitting technique. The secant tensile modulus can be determined at any stress level by differentiating (Equation 4.18) with respect to the strain ϵ . A typical example of this curve fitting and the resulting secant tensile modulus approach is given in Figure 4.39. The secant tensile modulus at 5% strain for cut tires in sand (Table 4.3) ranged from 37.5 to 98.1 kN per tire width; for full tires in sand, it ranged from 34.2 to 97.6 kN per tire width; and finally, for cut tires in cohesive backfill, it varied from 41.8 to 70.3 kN per tire width. These tensile moduli fall within the predicted limits (Figure 3.29) established from tension tests performed on ropes and tires.

A generalized pull-out deformation relationship for each of the three testing schemes (i.e. cut and full tires in sand, and cut tires in cohesive soil) can be established by averaging the different polynomial constants. Figure 4.40 presents these general relationships normalized with respect to the overburden pressure. The normalisation has been carried out by dividing the pull-out force per tire width, P_t divided by the overburden pressure, having a dimension of m^{-1} . In design of most conventional earth reinforced structures, the amount of deformation is usually limited to 5%

strain. Strains exceeding this value render an engineering structure unserviceable. Based upon Figure 4.40, the ratio between the pull-out resistance mobilized at 5% strain and peak pull-out resistance is approximately 0.40. It is recommended, at the present stage, that the ultimate pull-out resistance estimated from either Equations 4.11, 4.13 or 4.14 should be reduced by a factor of 2.5 (a pull-out factor of safety, $FS_p = 2.5$). It is interesting to note that all curves have a similar trend independent of backfill material, loading conditions, and reinforcement type. The large displacements required to fully mobilize the peak pull-out capacity could render the straining characteristics of the backfill material inconsequential to the overall deformation profile. Figure 4.40 shows that the peak strain within the tire reinforcement would be around 30% for any soil type; the corresponding pull-out resistance will be proportional to the shear strength of the soil used and the drainage conditions. However, pull-out tests were not performed under confining pressures greater than 20 kPa. The influence of higher confining pressures on the tire reinforcement deformation characteristics remains unknown and deserves further investigation.

Table 4.1 Summary of interface friction parameters including the direct sliding efficiency coefficient.

Soil Type	c (kPa)	c _a (kPa)	φ Degrees	μ Interface Friction Coefficient	δ Skin Friction Degrees	c _a /c	δ/φ	Tan δ/Tan φ	α _{ds}
Dry Sand			42	0.58	30		0.71	0.64	0.91
Cohesive backfill (CU)	68.4	7.6	19	0.40	22	0.11	1.16	1.17	0.82
Cohesive backfill (CD)			32	0.49	26		0.81	0.78	0.95

Table 4.2. Pull-out tests results.

Soil	Tire Element	Overburden Stress σ_v (kPa)	Frontal Placement (m)	Tire Mat Configuration	Maximum Pull-out Resistance (kN)	Frontal Displacement (m)
Sand	Cut Tire	14	1.0	1 x 1	10.6	0.171
				1 x 2	20.2	0.372
				1 x 3	25.2	0.414
				1 x 4	26.1	0.696
				2 x 1	29.0	0.103
				2 x 2	26.3	0.340
				2 x 3	39.6	0.530
				2 x 3	33.5	0.540
				3 x 3	56.5	0.602
				4 x 4	72.5	0.651
				Small Diamond	23.2	0.480
				Triangle	34.5	0.717
Sand	Cut Tire	14	0.5	1 x 1	2.5	0.067
				1 x 3	18.5	0.599
				3 x 3	36.2	0.442
Sand	Cut Tire	7	1.0	1 x 1	10.3	0.179
				1 x 3	16.2	0.473
				3 x 3	41.1	0.449
Sand	Full Tire	14	1.0	1 x 1	8.7	0.196
				1 x 2	15.5	0.478
				1 x 3	21.9	0.623
				1 x 4	34.8	1.064
				2 x 1	19.7	0.193
				2 x 3	41.3	0.591
				2 x 3	35.6	0.587
				3 x 1	29.4	0.184
				3 x 2	37.9	0.235
				3 x 3	56.7	0.464
				4 x 4	69.9	0.816
				Small Diamond	19.7	0.494
				Large Diamond	57.3 ^a	1.087
				Triangle	34.7	0.695

^a Rope attachment failed

Table 4.2. Pull-out tests results (continued)

Soil	Tire Element	Overburden Stress σ_v (kPa)	Frontal Placement (m)	Tire Mat Configuration	Maximum Pull-out Resistance (kN)	Frontal Displacement (m)
Cohesive Backfill	Cut Tire	20	1.0	1 x 1	27.5	0.242
				1 x 2	43.8	0.416
				1 x 3	57.1 ^a	0.657
				1 x 4	66.8 ^a	0.906
				2 x 1	46.5	0.263
Cohesive Backfill	Cut Tire	10	1.0	1 x 3	14.6	0.425
				2 x 2	25.1	0.406
				2 x 3	25.3	0.562
				3 x 3	38.7	0.555
				4 x 4	61.8	0.832
				Triangle	31.4	0.897
Cohesive Backfill	Cut Tire	10	0.5	1 x 3	13.0	0.617
				1 x 4	24.5	1.212
				2 x 2	16.4	0.29
				3 x 3	27.8	0.438
Cohesive Backfill	Full Tire	10	1.0	1 x 2	17.1	0.485
				2 x 3	27.0	0.689

^a Rope attachment failed

Table 4.3. 4th degree polynomial trendlines for pull-out tests performed on cut tire in sand with the corresponding secant tensile modulus at 5% strain.

Overburden Stress σ_v (kPa)	Frontal Placement (m)	Tire Mat Configuration	Polynomial Trendline 4 th Degree	R ² Value	Secant Tensile Modulus at 5% Strain (kN per Tire Width)
14	1.0	1 x 1	$16.5\epsilon^4 + 92.7\epsilon^3 - 158.3\epsilon^2 + 68.0\epsilon + 0.92$	0.952	52.2
		1 x 2	$-517.5\epsilon^4 + 779.5\epsilon^3 - 411.9\epsilon^2 + 106.6\epsilon + 0.94$	0.953	65.2
		1 x 3	$-46.6\epsilon^4 + 159.9\epsilon^3 - 199.2\epsilon^2 + 107.0\epsilon + 2.21$	0.954	87.1
		1 x 4	$-3.9\epsilon^4 + 26.9\epsilon^3 - 67.8\epsilon^2 + 64.2\epsilon + 5.0$	0.913	57.4
		2 x 1	$-1280.8\epsilon^4 + 2206.4\epsilon^3 - 1157.5\epsilon^2 + 214.5\epsilon + 0.79$	0.979	98.1
		2 x 2	$-3068.7\epsilon^4 + 2177.4\epsilon^3 - 587.3\epsilon^2 + 97.8\epsilon + 1.45$	0.919	37.5
		2 x 3	$-1459\epsilon^4 + 1522.8\epsilon^3 - 610.3\epsilon^2 + 132.6\epsilon + 4.86$	0.923	70.8
		2 x 3	$-9148.7\epsilon^4 + 7419.2\epsilon^3 - 2184.2\epsilon^2 + 281.9\epsilon - 0.18$	0.947	58.9
		3 x 3	$47.3\epsilon^4 + 87.1\epsilon^3 - 213.5\epsilon^2 + 110.2\epsilon + 1$	0.989	88.9
		4 x 4	$-1905.3\epsilon^4 + 1758.8\epsilon^3 - 655.3\epsilon^2 + 136.67\epsilon + 3.98$	0.970	70.2
14	0.5	3 x 3	$1244.4\epsilon^4 - 1130.5\epsilon^3 + 215.9\epsilon^2 + 34.3\epsilon + 1.21$	0.964	56.5
7	1.0	3 x 3	$-1024.6\epsilon^4 + 1283.1\epsilon^3 - 624.5\epsilon^2 + 134\epsilon + 2.63$	0.969	71.0

Table 4.4. 4th degree polynomial trendlines for pull-out tests performed on full tire in sand with the corresponding secant tensile modulus at 5% strain.

Overburden Stress σ_v (kPa)	Frontal Placement (m)	Tire Mat Configuration	Polynomial Trendline 4 th Degree	R ² Value	Secant Tensile Modulus at 5% Strain (kN per Tire Width)
14	1.0	1 x 1	$78.9\text{E}^4 - 85.5\text{E}^3 - 13.1\text{E}^2 + 30.9\text{E} + 0.70$	0.871	29.6
		1 x 2	$185\text{E}^4 - 180.1\text{E}^3 - 51.1\text{E}^2 + 72\text{E} + 0.50$	0.959	67.0
		1 x 3	$50.1\text{E}^4 + 17.2\text{E}^3 - 161.3\text{E}^2 + 84.7\text{E} + 5.54$	0.700	68.6
		1 x 4	$-1333.3\text{E}^4 + 1373.1\text{E}^3 - 548.4\text{E}^2 + 153.1\text{E} + 2.66$	0.952	97.6
		2 x 1	$129.5\text{E}^4 - 163\text{E}^3 + 18\text{E}^2 + 32.3\text{E} + 0.74$	0.947	34.2
		2 x 3	$-1269.1\text{E}^4 + 1471.4\text{E}^3 - 648.3\text{E}^2 + 140.2\text{E} + 4.78$	0.888	74.7
		2 x 3	$-2734.3\text{E}^4 + 2299.6\text{E}^3 - 735.9\text{E}^2 + 137.4\text{E} + 1.99$	0.974	62.4
		3 x 1	$79.9\text{E}^4 - 75.4\text{E}^3 - 33.2\text{E}^2 + 41.9\text{E} + 0.79$	0.964	38.6
		3 x 2	$-1428.2\text{E}^4 + 1486.1\text{E}^3 - 614.4\text{E}^2 + 115.7\text{E} + 3.67$	0.879	53.5
		3 x 3	$-26220\text{E}^4 + 16369\text{E}^3 - 3492.2\text{E}^2 + 320.9\text{E} + 2.90$	0.945	71.9
		4 x 4	$-1277.4\text{E}^4 + 1106.4\text{E}^3 - 409.9\text{E}^2 + 113.4\text{E} - 0.57$	0.993	71.8

Table 4.5. 4th degree polynomial trendlines for pull-out tests performed on cut tire in the cohesive backfill with the corresponding secant tensile modulus at 5% strain.

Overburden Stress σ_v (kPa)	Frontal Placement (m)	Tire Mat Configuration	Polynomial Trendline 4 th Degree	R ² Value	Secant Tensile Modulus at 5% Strain (kN per Tire Width)
10	1.0	1 x 3	$-991.5\text{E}^4 + 1042.3\text{E}^3 - 455.6\text{E}^2 + 112.8\text{E} + 2.22$	0.975	66.5
		2 x 2	$1157.2\text{E}^4 - 1496.1\text{E}^3 + 533.1\text{E}^2 - 23.2\text{E} + 1.67$	0.968	53.0
		2 x 3	$-1013.8\text{E}^4 + 796.7\text{E}^3 - 336.9\text{E}^2 + 97.2\text{E} + 0.56$	0.996	63.0
		3 x 3	$-742.1\text{E}^4 + 844.4\text{E}^3 - 422.5\text{E}^2 + 101.2\text{E} + 2.84$	0.934	58.6
		4 x 4	$-235.6\text{E}^4 + 270\text{E}^3 - 231.9\text{E}^2 + 93.6\text{E} + 0.17$	0.992	70.3
	0.5	1 x 3	$-44.1\text{E}^4 - 13.4\text{E}^3 - 80.4\text{E}^2 + 59.2\text{E} + 2.53$	0.954	51.1
		1 x 4	$-606.9\text{E}^4 + 497.9\text{E}^3 - 158.1\text{E}^2 + 76.8\text{E} - 0.87$	0.995	60.7
		2 x 2	$627.6\text{E}^4 - 561.9\text{E}^3 + 56.8\text{E}^2 + 35.8\text{E} + 1.71$	0.943	41.8
		3 x 3	$499.9\text{E}^4 - 361.8\text{E}^3 - 25.5\text{E}^2 + 48.3\text{E} + 2.08$	0.959	46.0

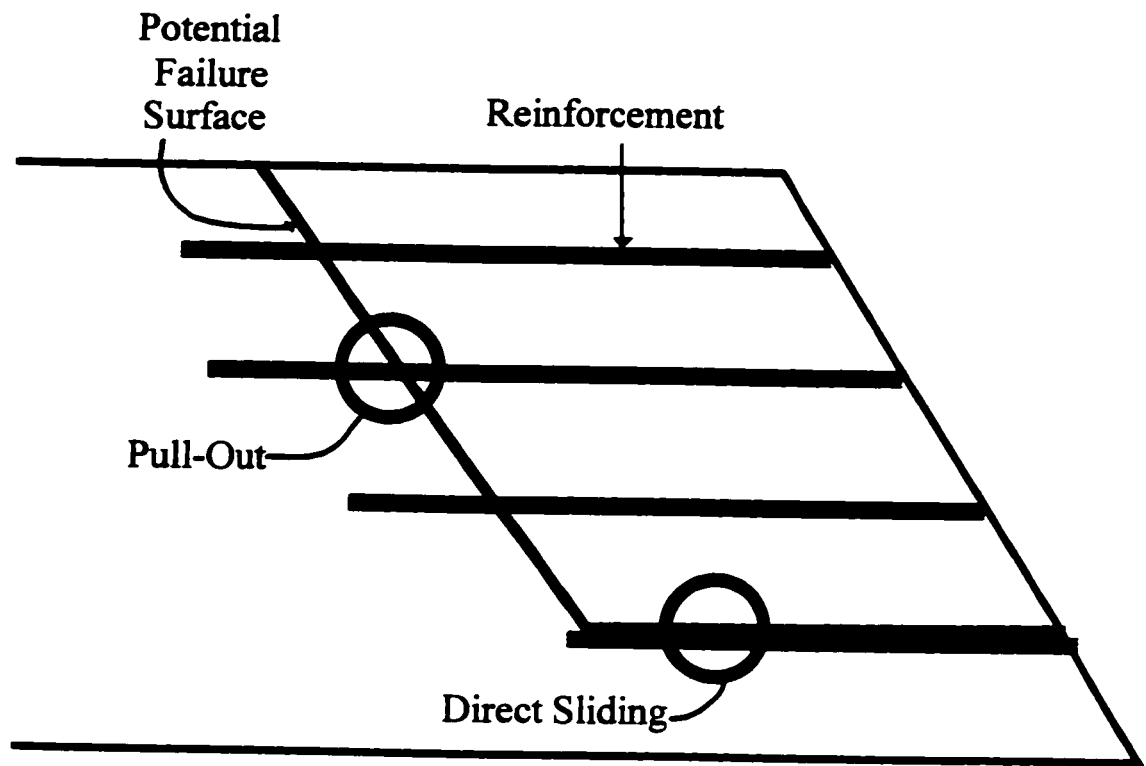


Figure 4.1. The two possible interactive failure mechanisms between reinforcement and soil.

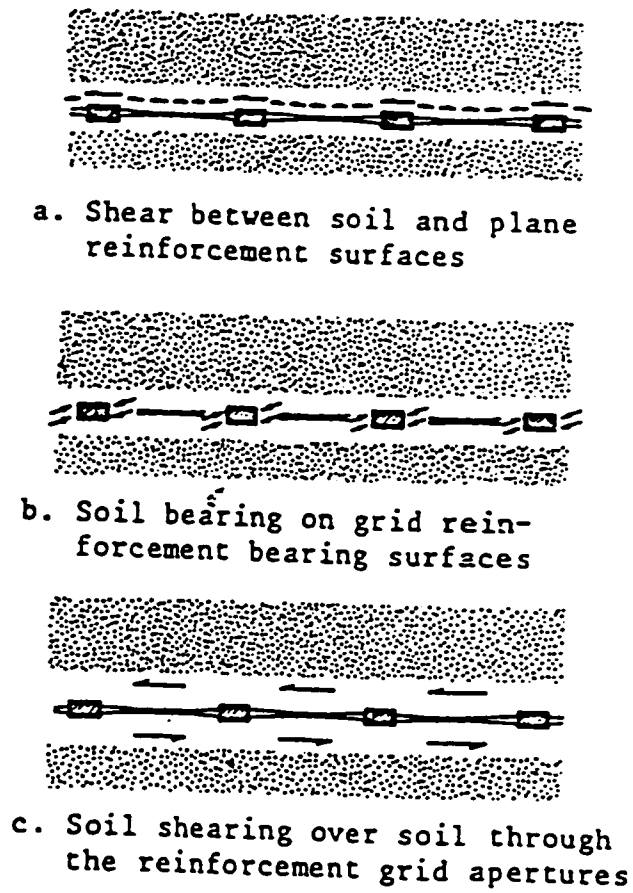


Figure 4.2. Three possible interactions between soil and reinforcement (after Jewell et al. 1984).

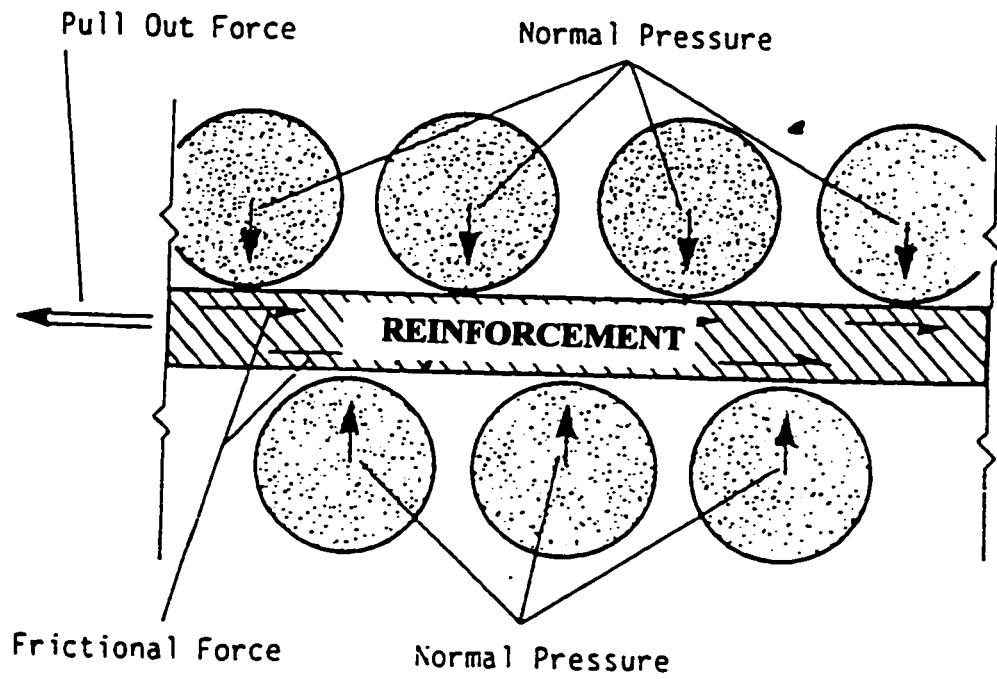
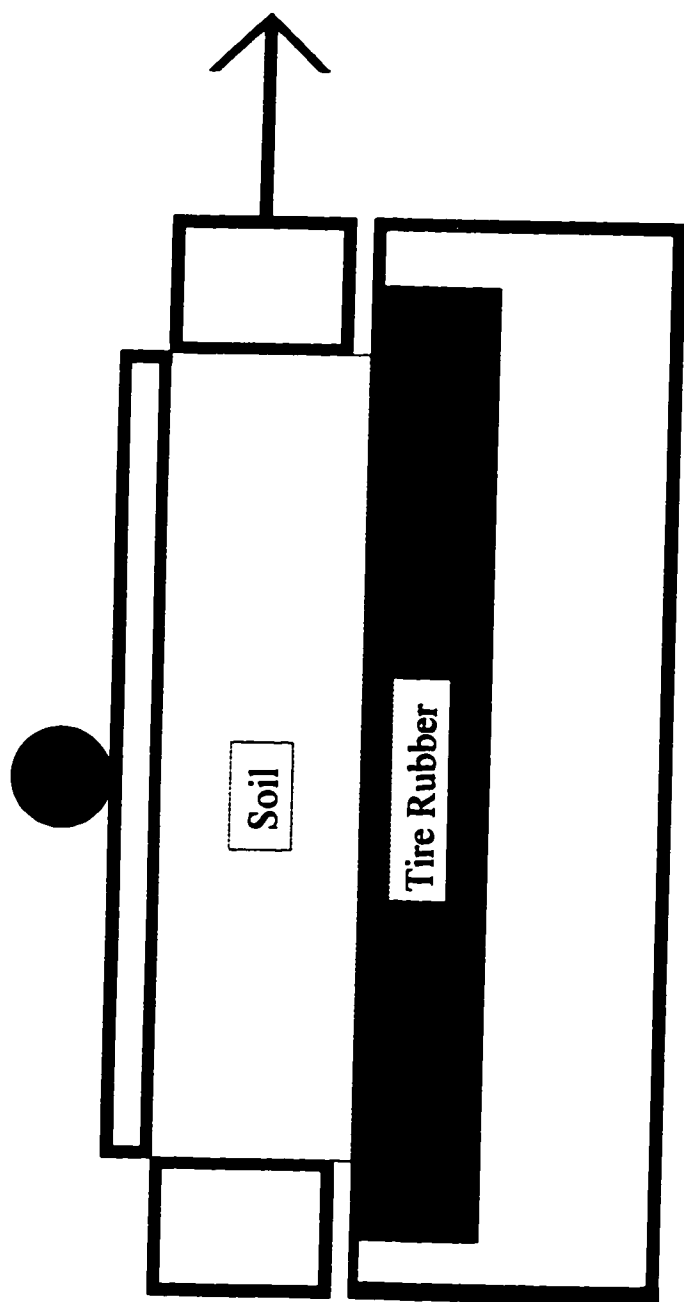


Figure 4.3. Friction transfer between soil and reinforcement (Mitchell and Villet 1987).



Dimensions of shear box: 100 mm x 100 mm

Figure 4.4. Tire-soil interface shear box.

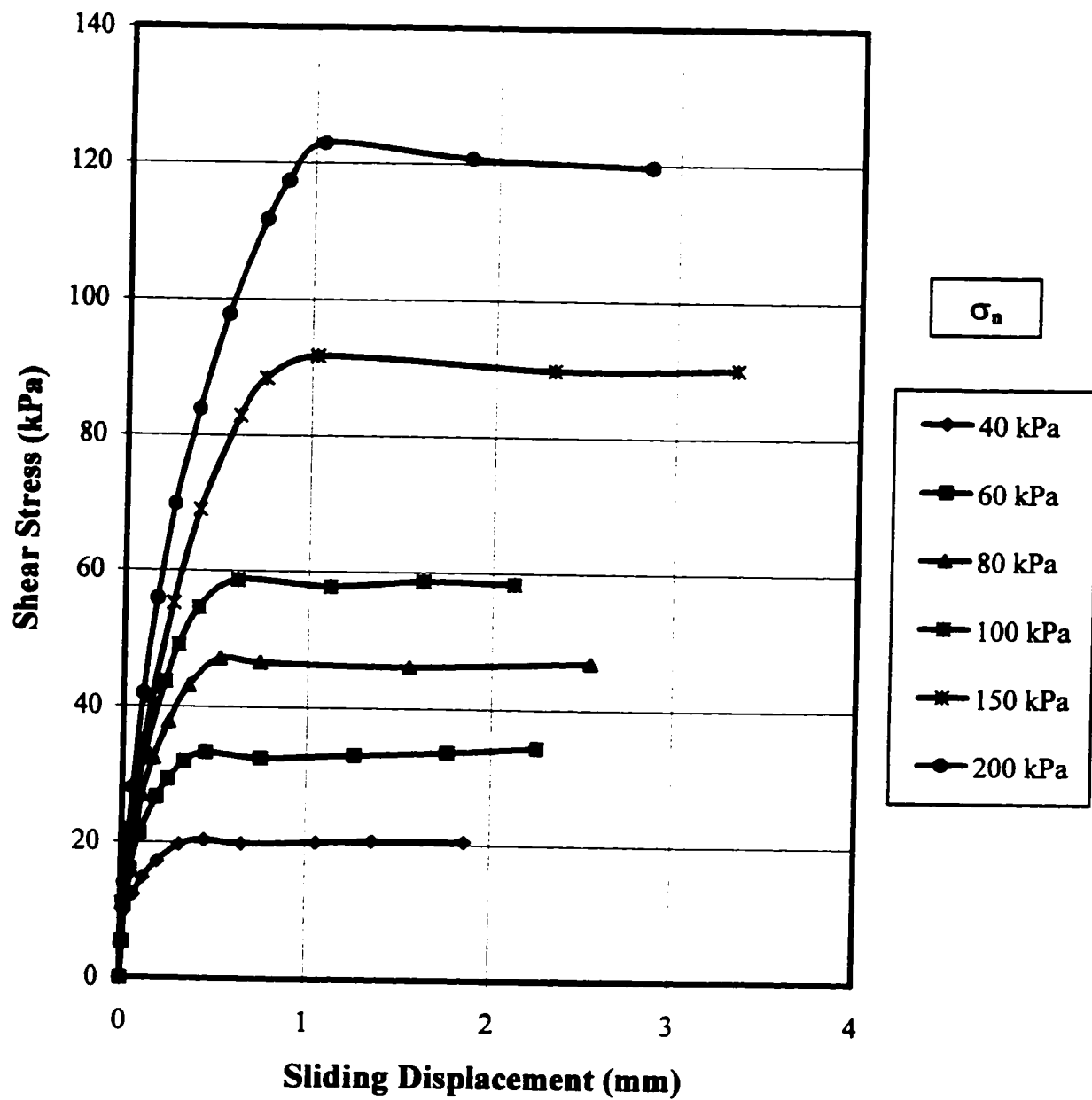


Figure 4.5. Shear stress and sliding displacement relationships between tire rubber and dry (site) sand.

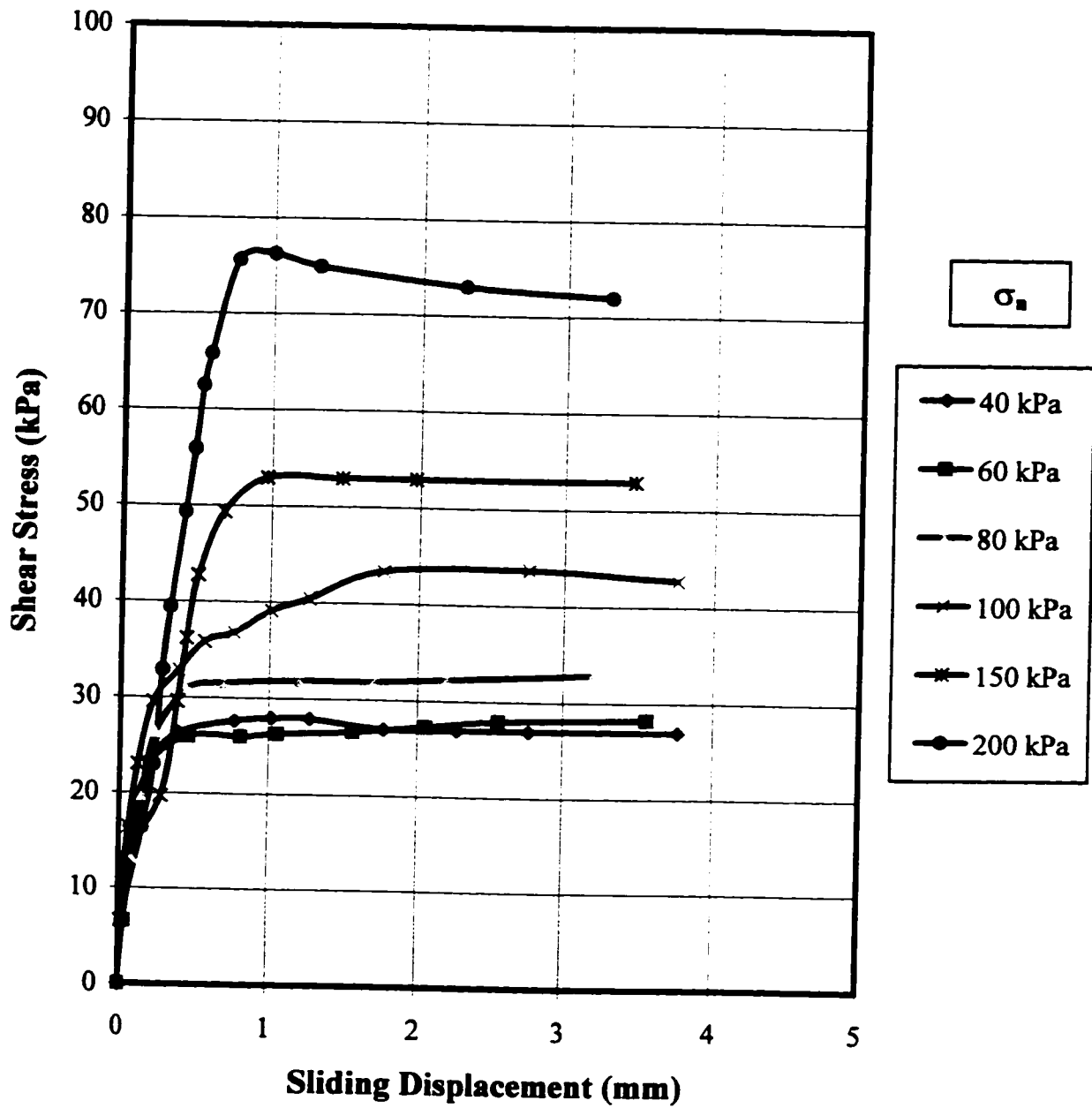


Figure 4.6. Shear stress and sliding displacement relationships between tire rubber and on-site cohesive backfill under undrained conditions.

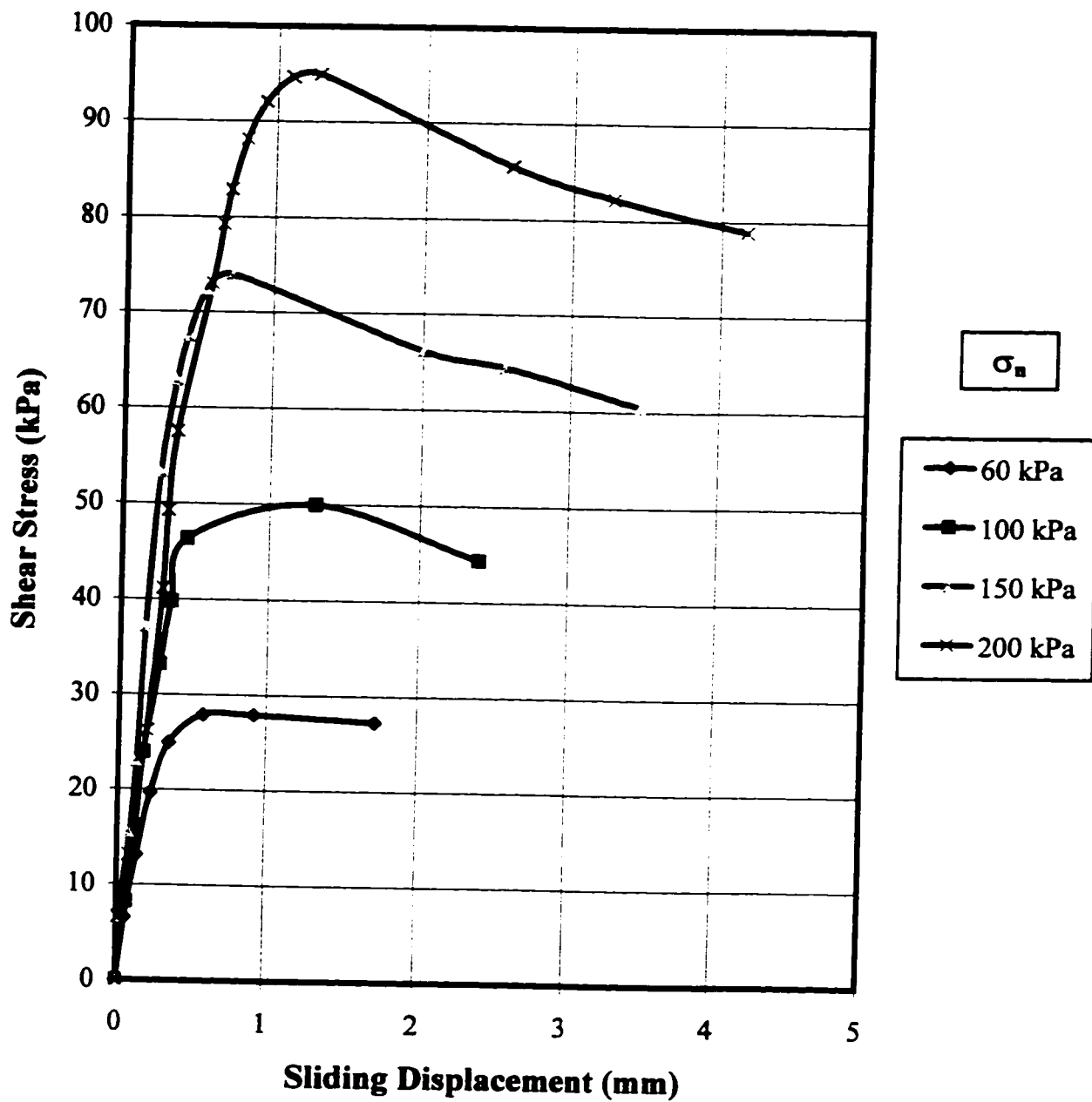


Figure 4.7. Shear stress and sliding displacement relationship between tire rubber and on-site cohesive backfill under drained conditions.

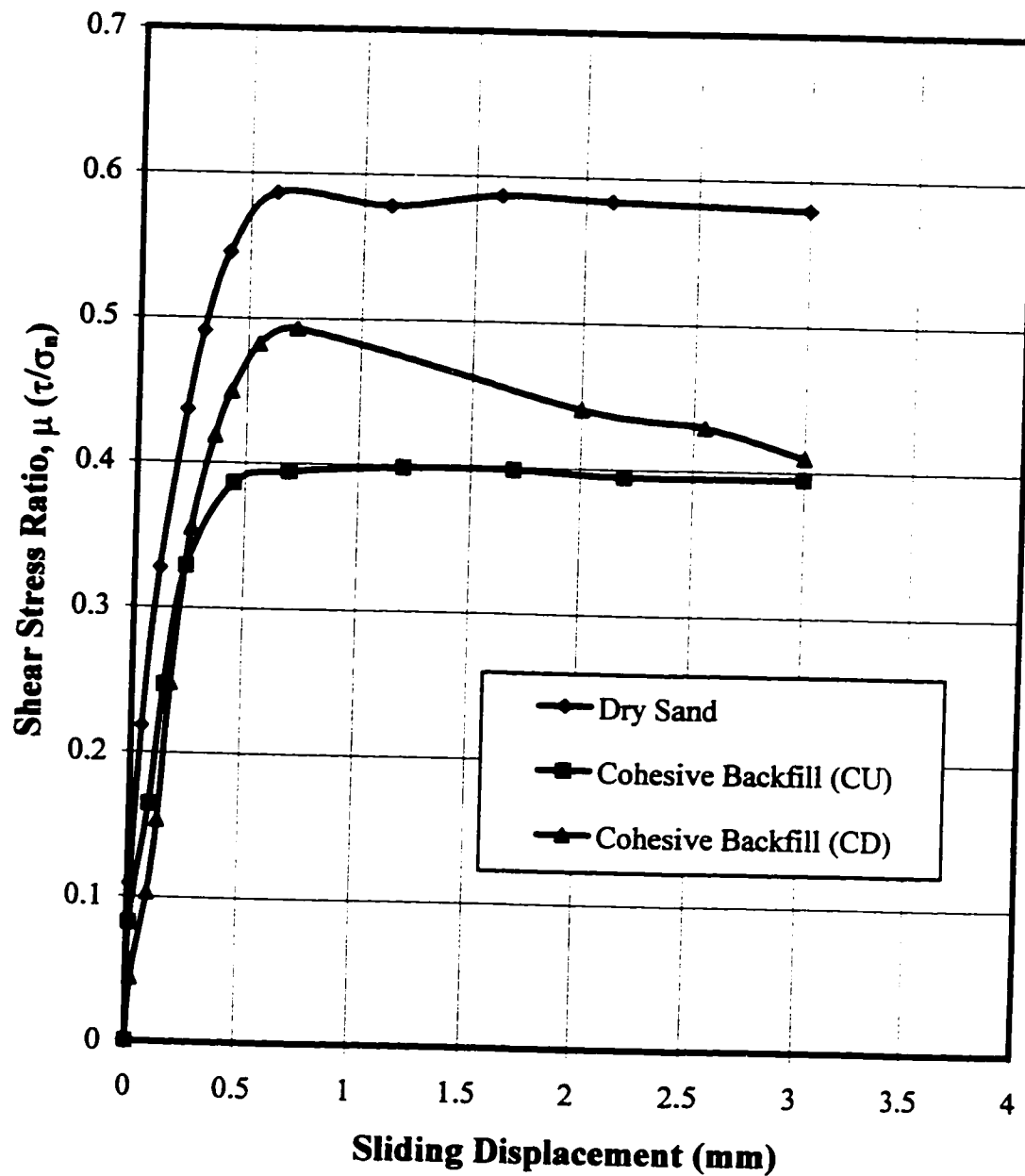


Figure 4.8. The behaviour of interface friction between tire rubber and soils.

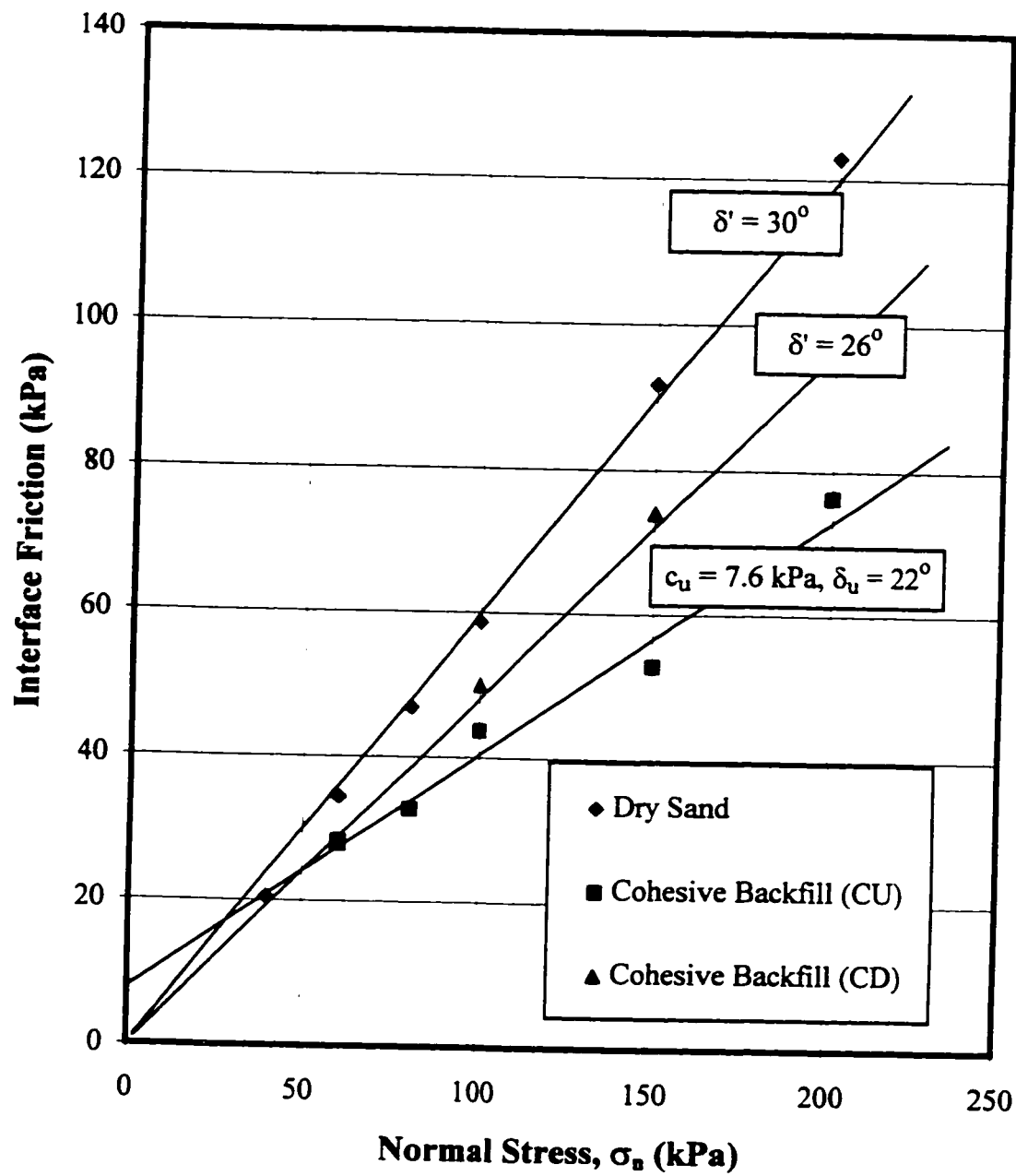
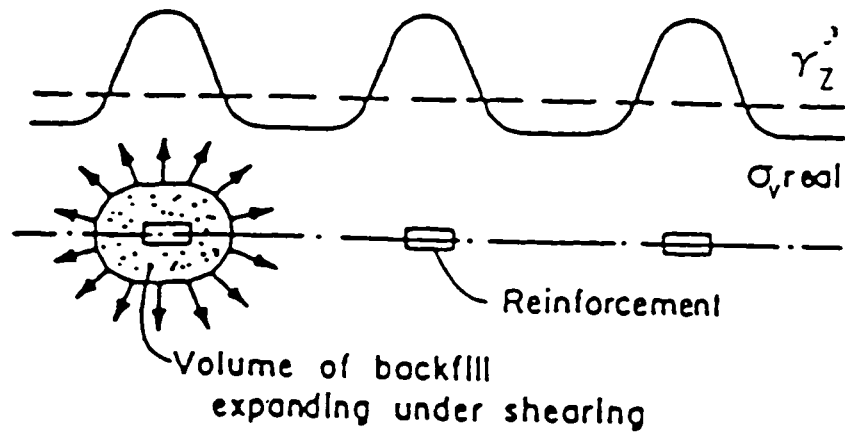


Figure 4.9. The interface friction and normal stress relationship of soils used in the test embankment.



$$\mu = \frac{\bar{\tau}}{\gamma_z'} > \mu^*_{real} = \frac{\tau_{real}}{\sigma_{v real}}$$

μ^* : Apparent friction coefficient

$\bar{\tau}$: Mobilized average shear stress

γ_z' : Overburden stress

Figure 4.10. Schematic illustration of dilatancy effect for reinforced soil (Schlosser et al. 1978).

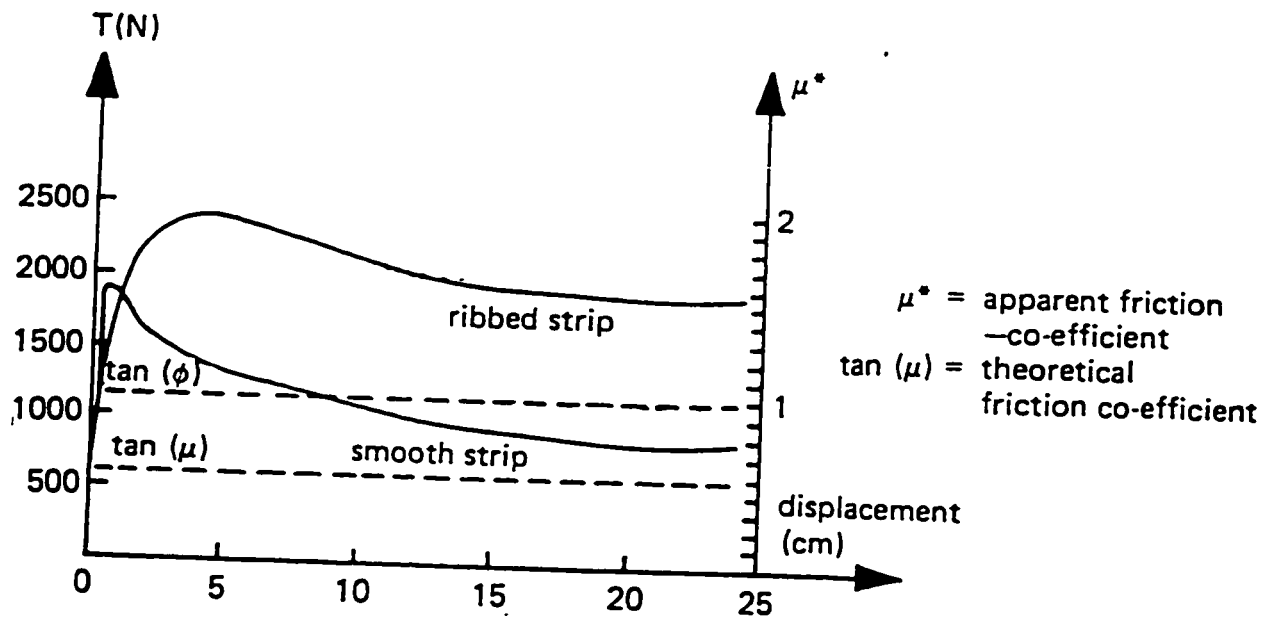


Figure 4.11. Influence of surface properties on the apparent friction coefficient (Jones 1988).

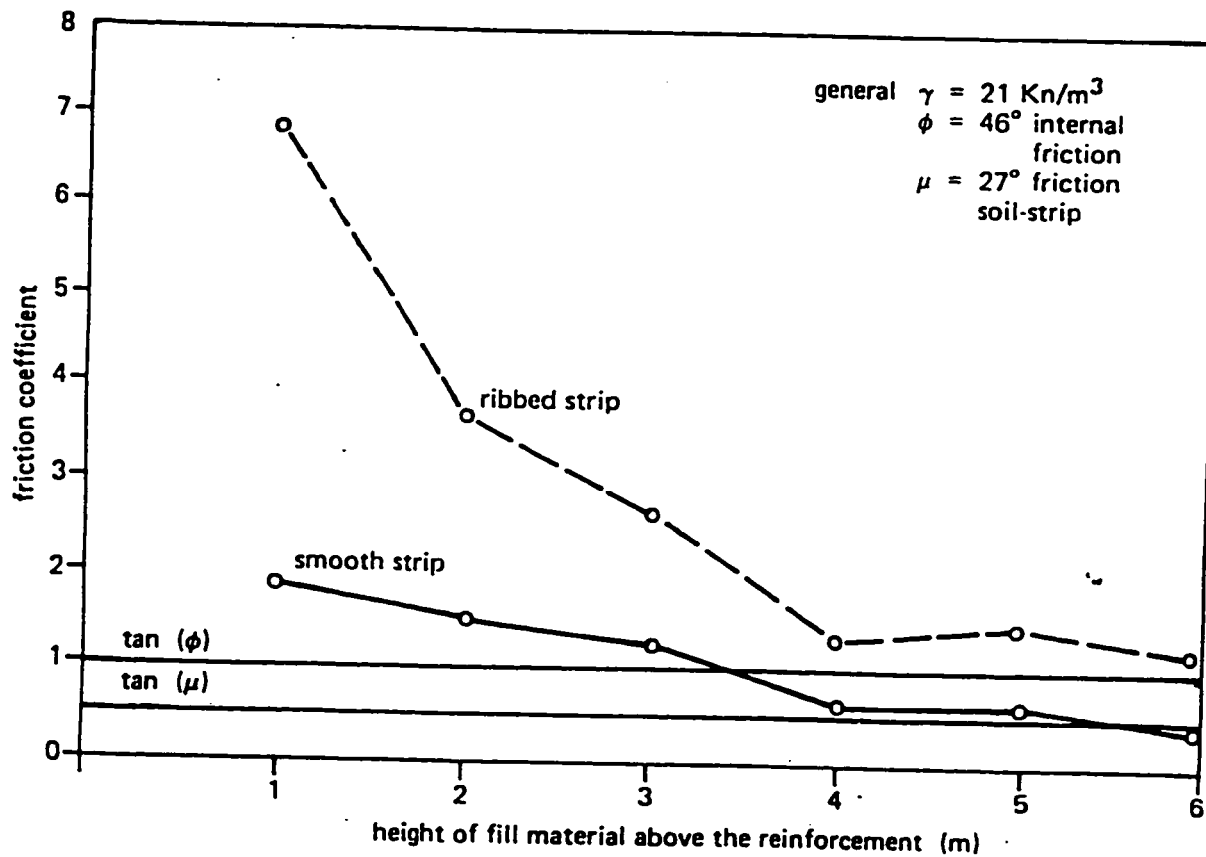


Figure 4.12. Influence of the overburden stress on the apparent friction coefficient from pull-out tests (Guilloux and Schlosser 1979).

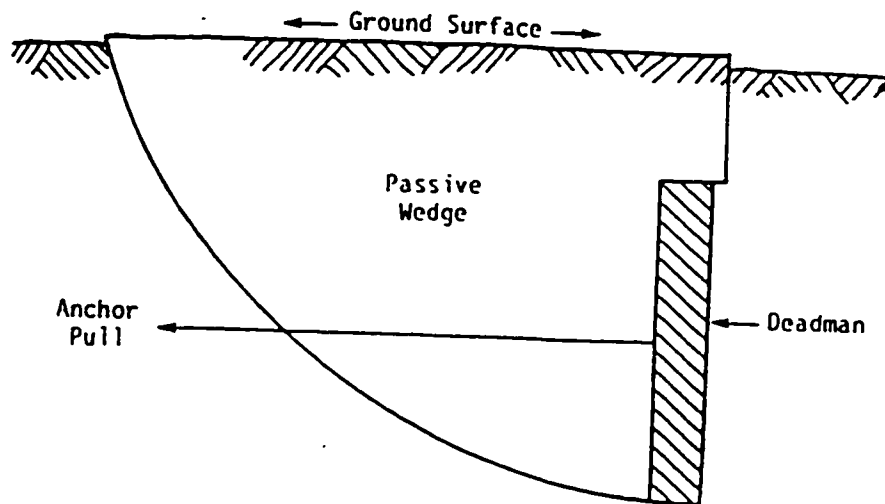


Figure 4.13. Load transfer by passive soil resistance.

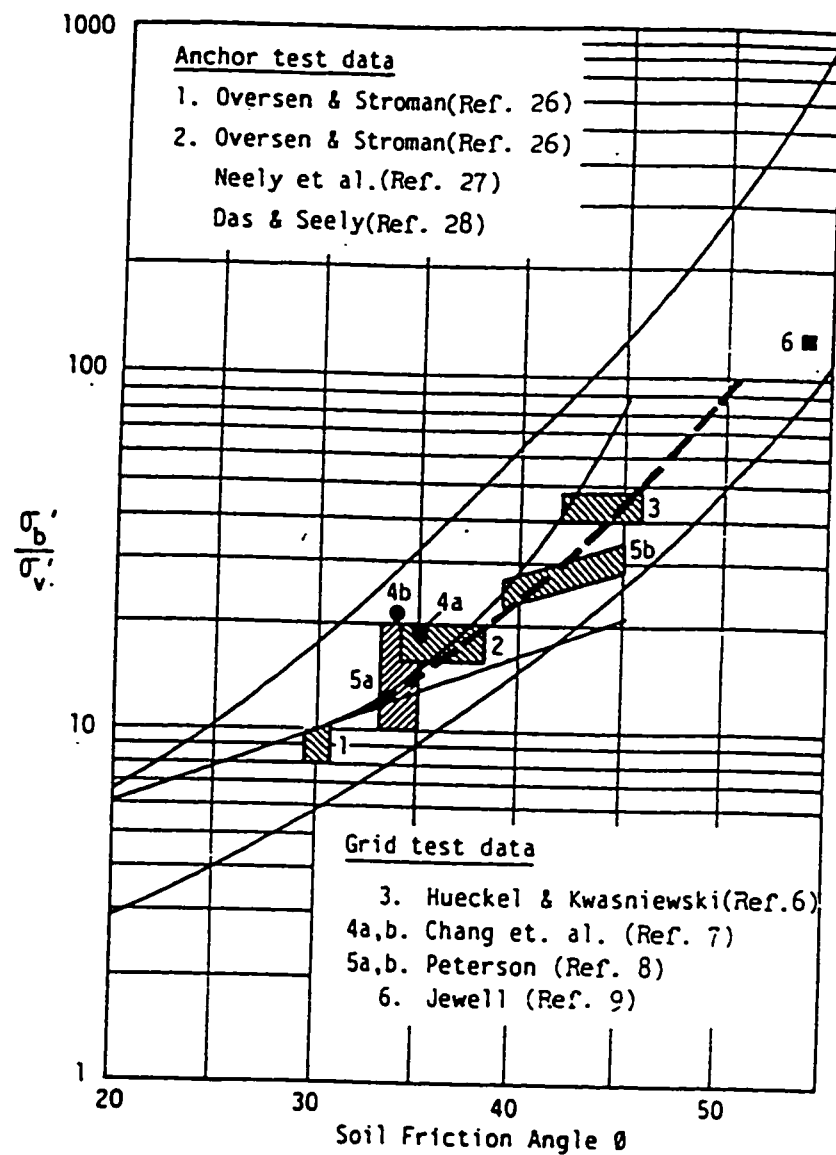


Figure 4.14. Theoretical relationship and experimental results for bearing stress (factor) as a function of the soil friction angle (after Jewell et al. 1984).

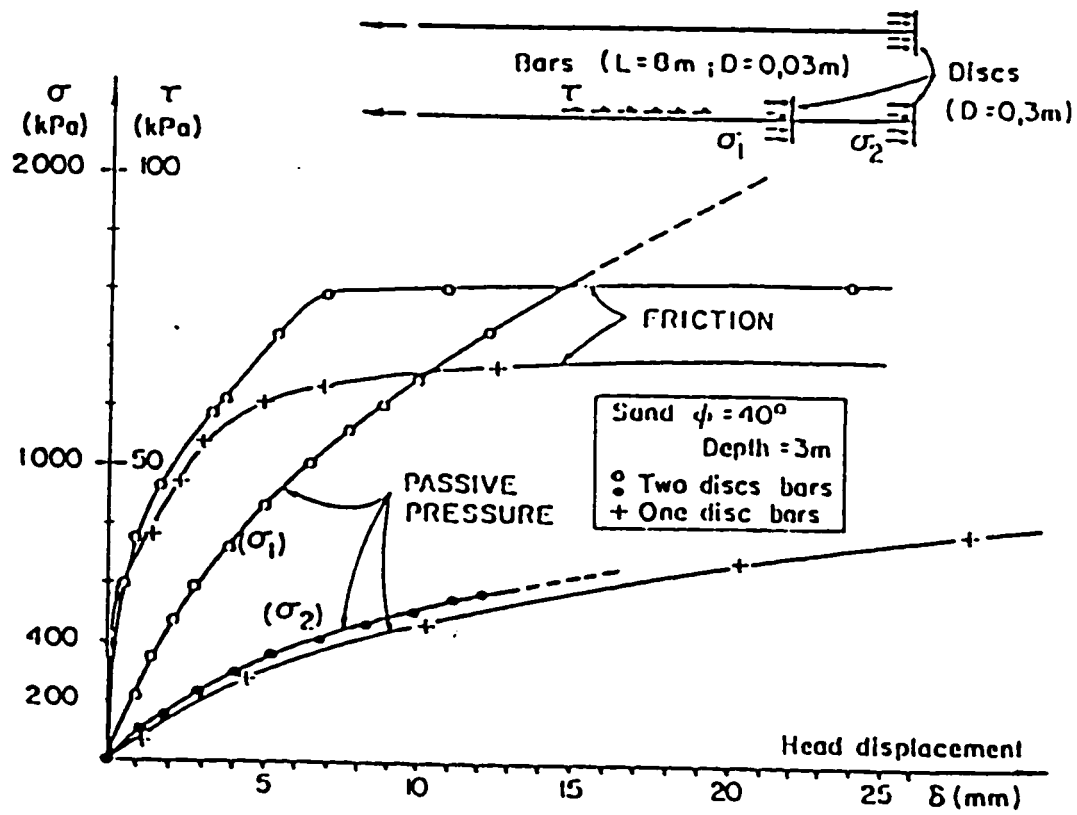


Figure 4.15. Mobilization of friction and passive stress on bars equipped with circular anchoring plates (Morbois and Long 1984).

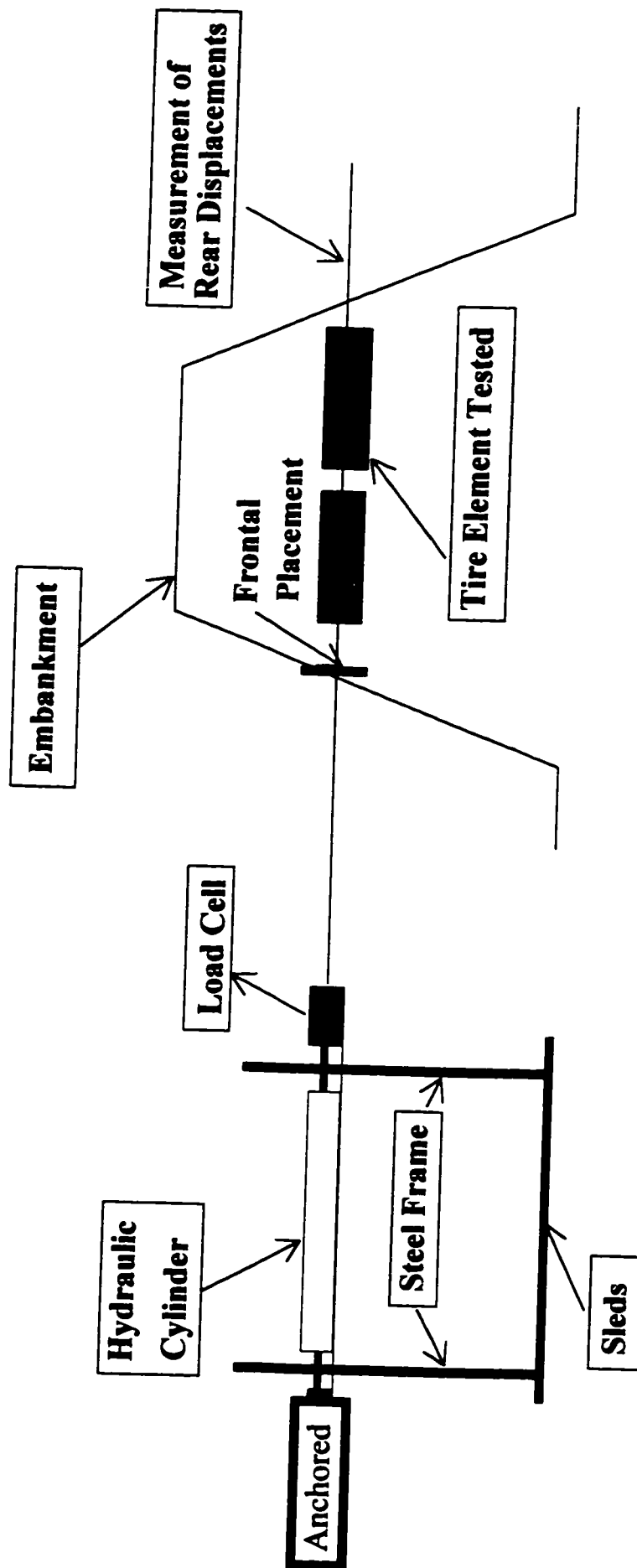


Figure 4.16. Schematic diagram of the principal components of the pull-out test.

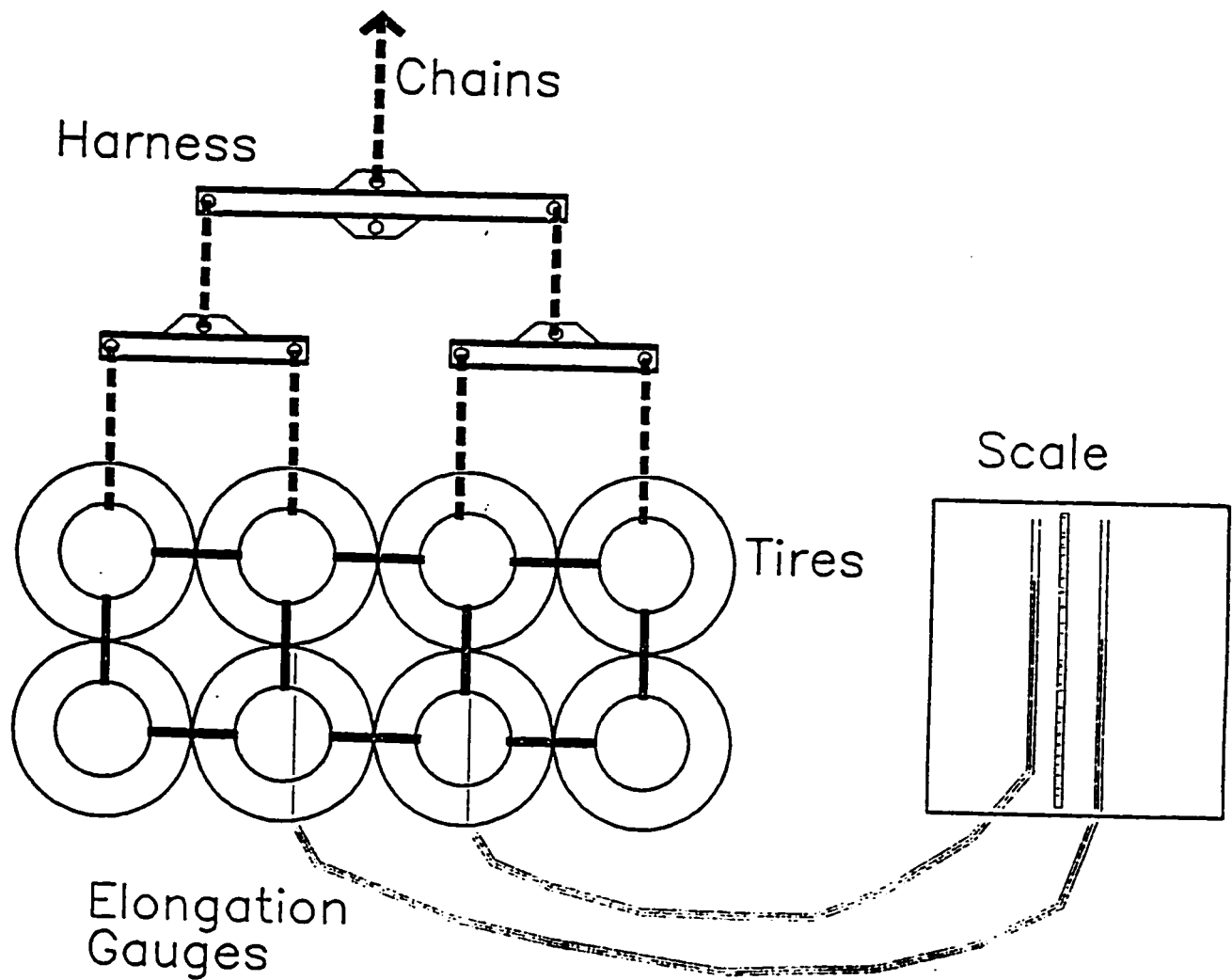


Figure 4.17. The tire pull-out assembly, showing harness system, a 4x2 tire reinforcing configuration, and rear displacement gauges (not to scale).

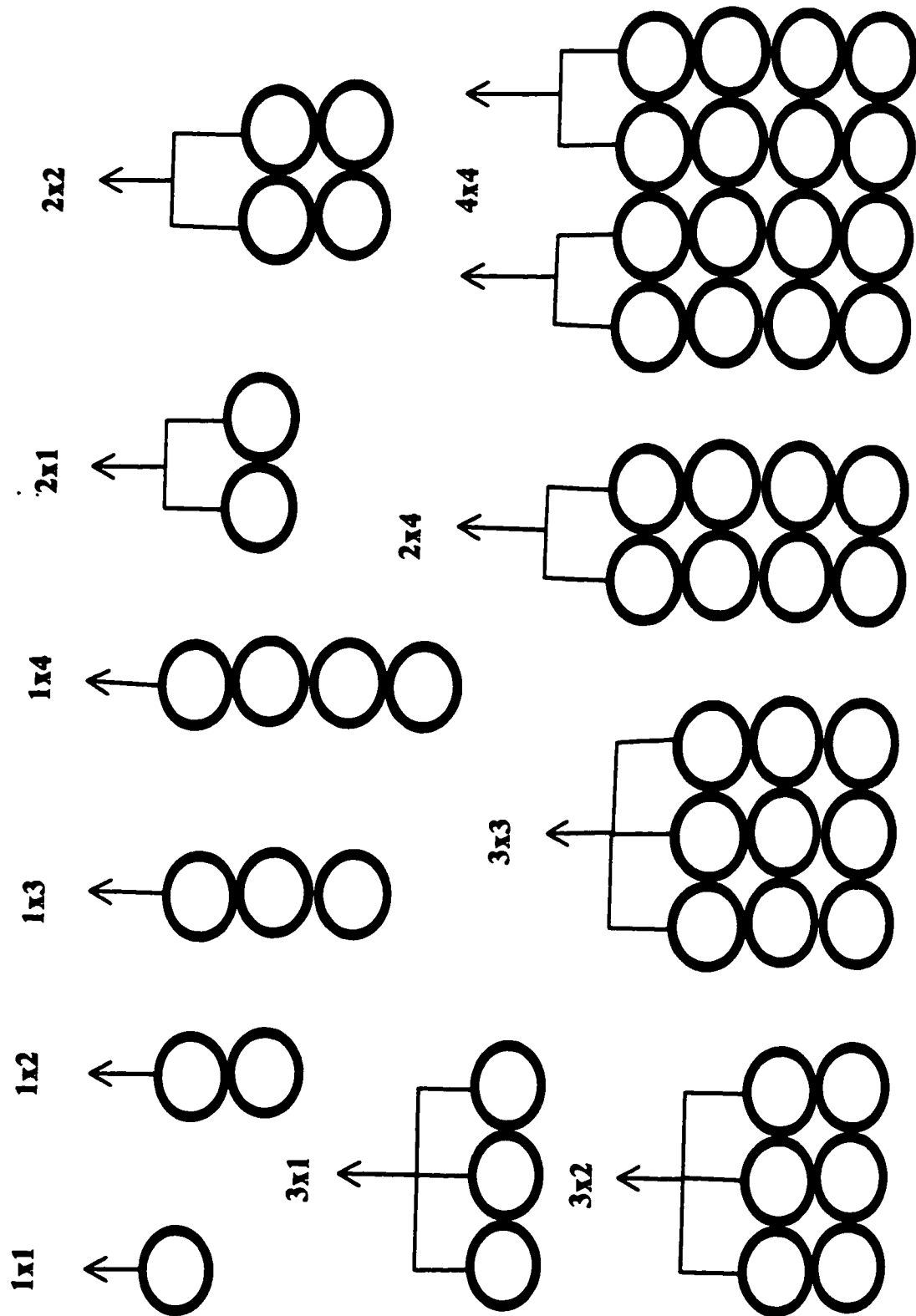


Figure 4.18. The different tire configurations used in the pull-out tests.

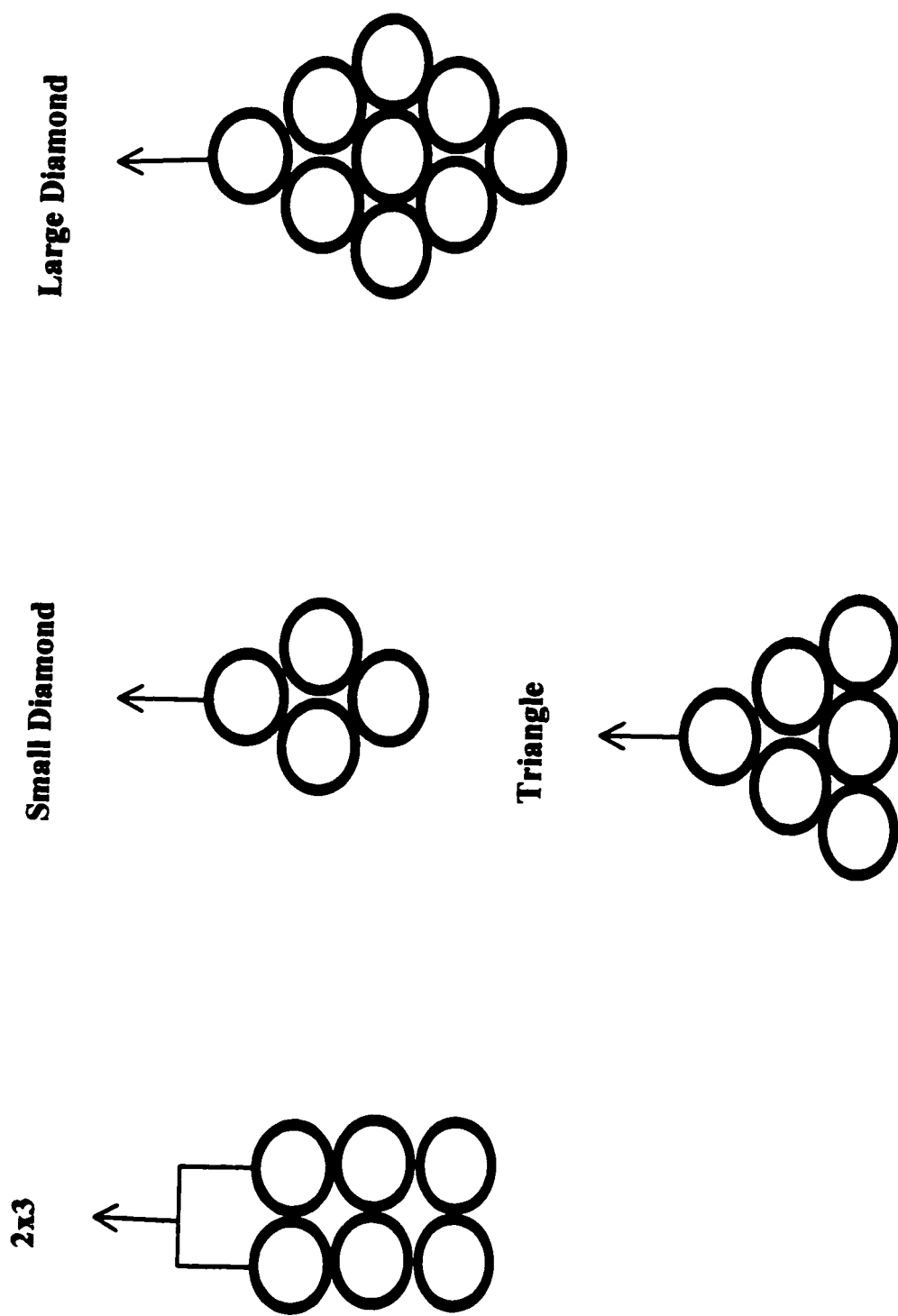


Figure 4.18. The different tire configurations used in the pull-out tests, continued.

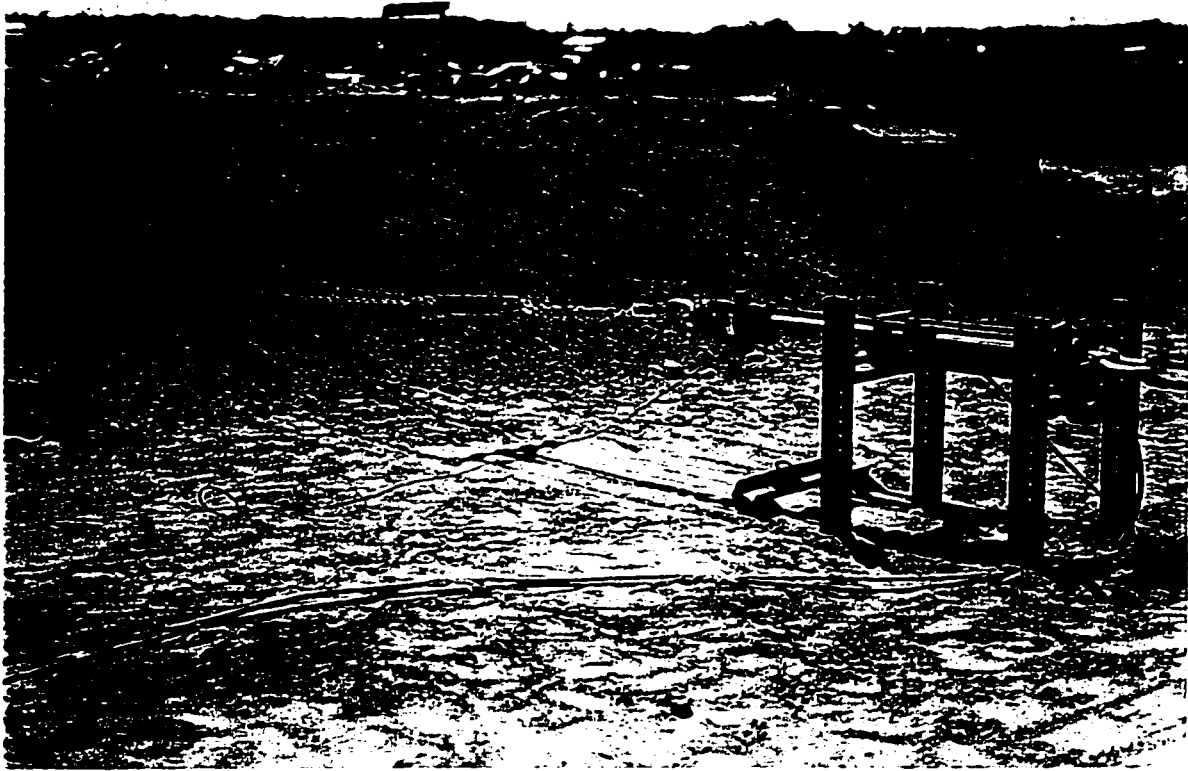


Figure 4.19. The principal components of a pull-out test (test in progress).

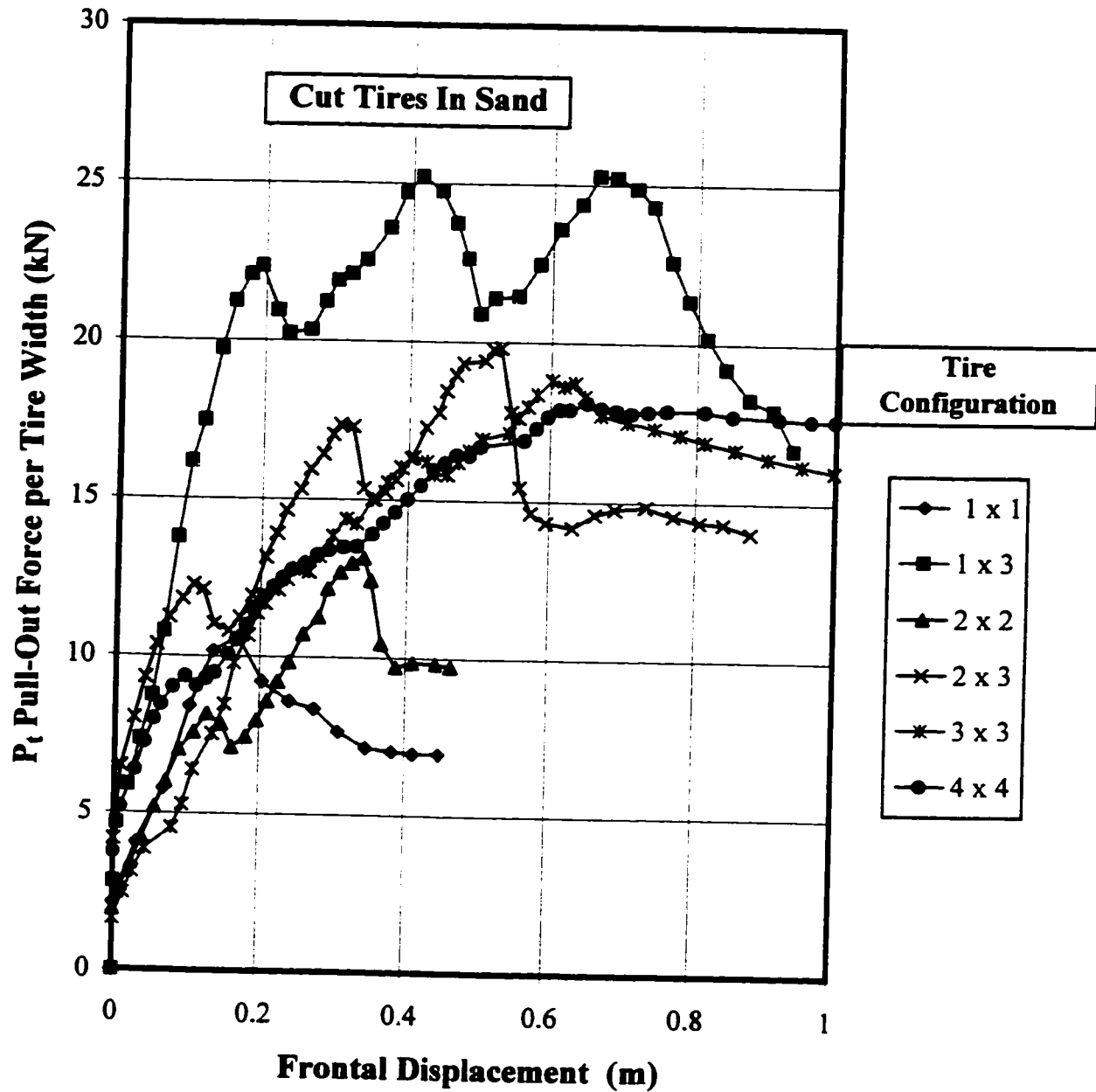


Figure 4.20. The pull-out force per tire and frontal displacement relationships for different cut tire configurations in sand, placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

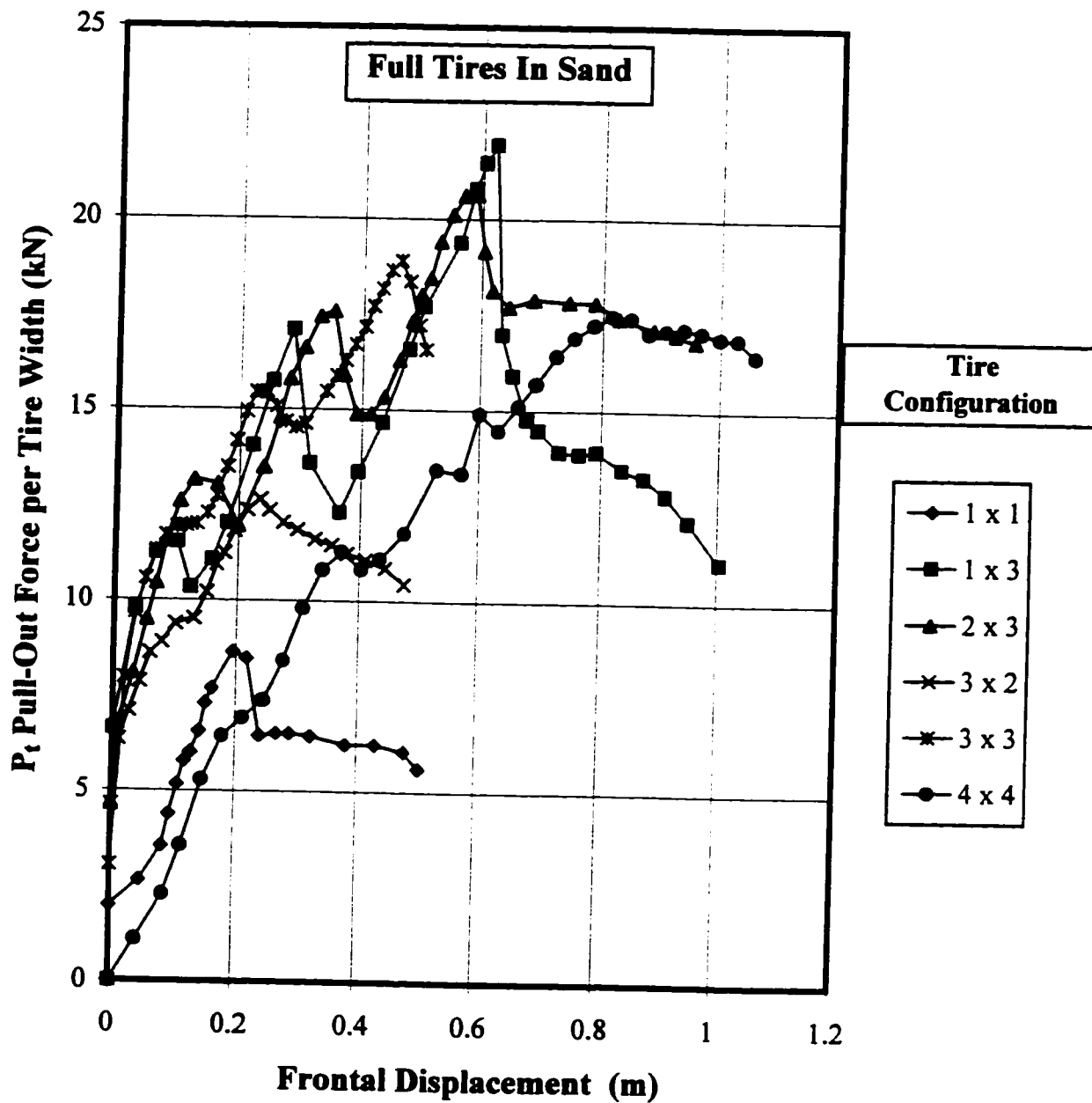


Figure 4.21. The pull-out force per tire and frontal displacement relationships for different full tire configurations in sand, placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

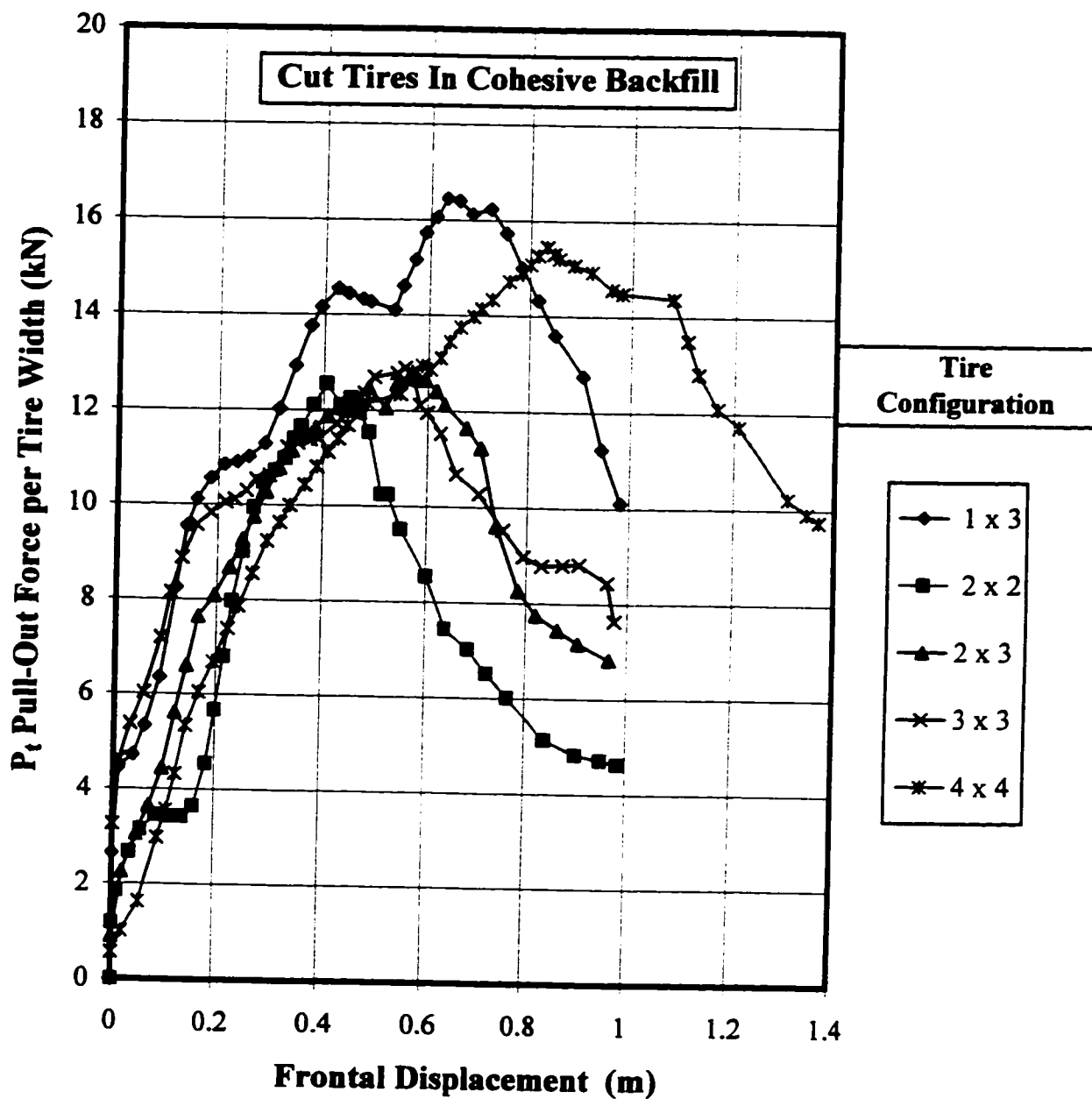


Figure 4.22. The pull-out force per tire and frontal displacement relationships for different cut tire configurations in cohesive backfill, placed 1 m behind the edge of the testing berm and under a confining stress of 10 kPa.

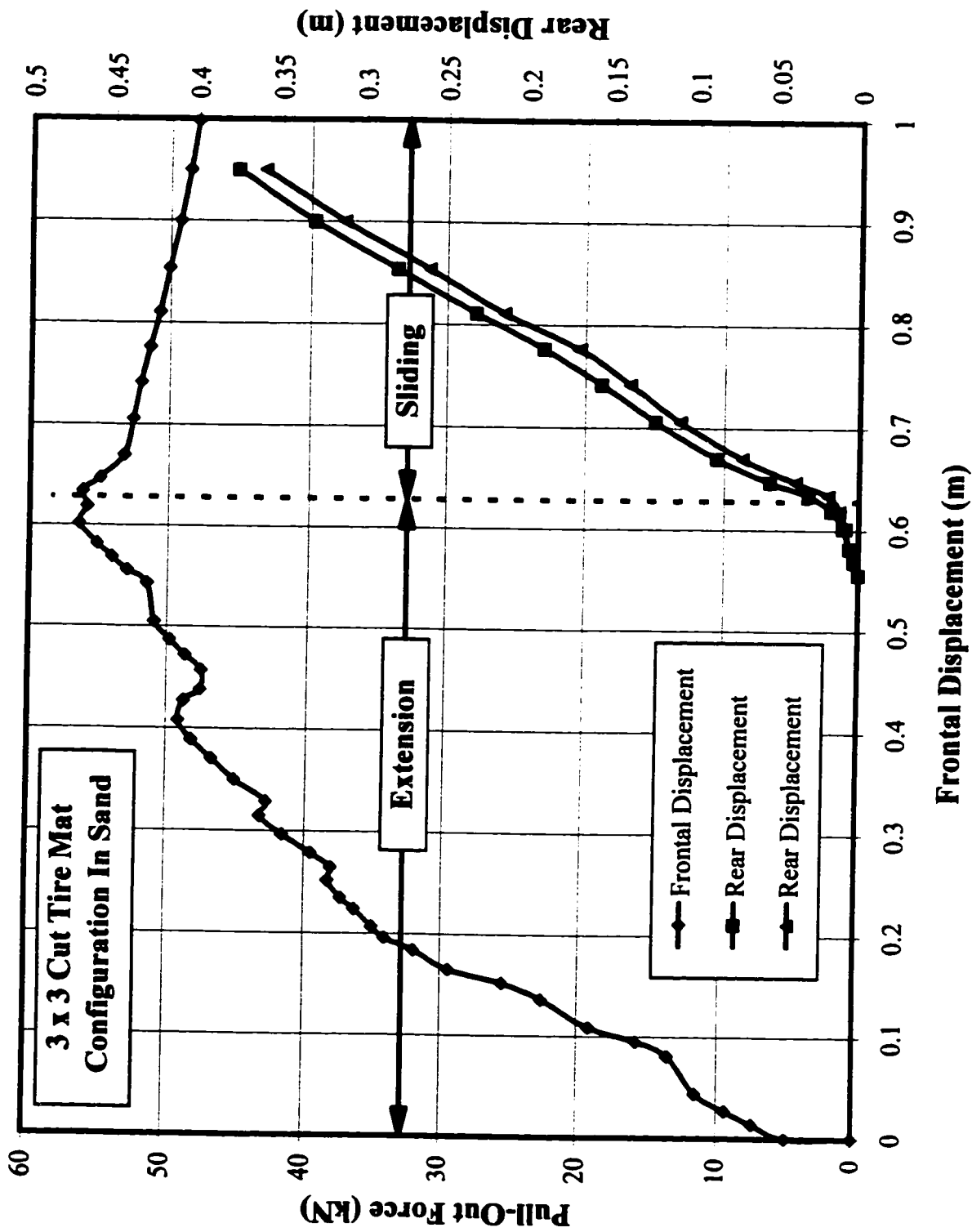


Figure 4.23 Pull-out force-frontal and rear displacement for a 3 x 3 cut tire mat configuration in sand, showing zones of reinforcement extension and sliding.

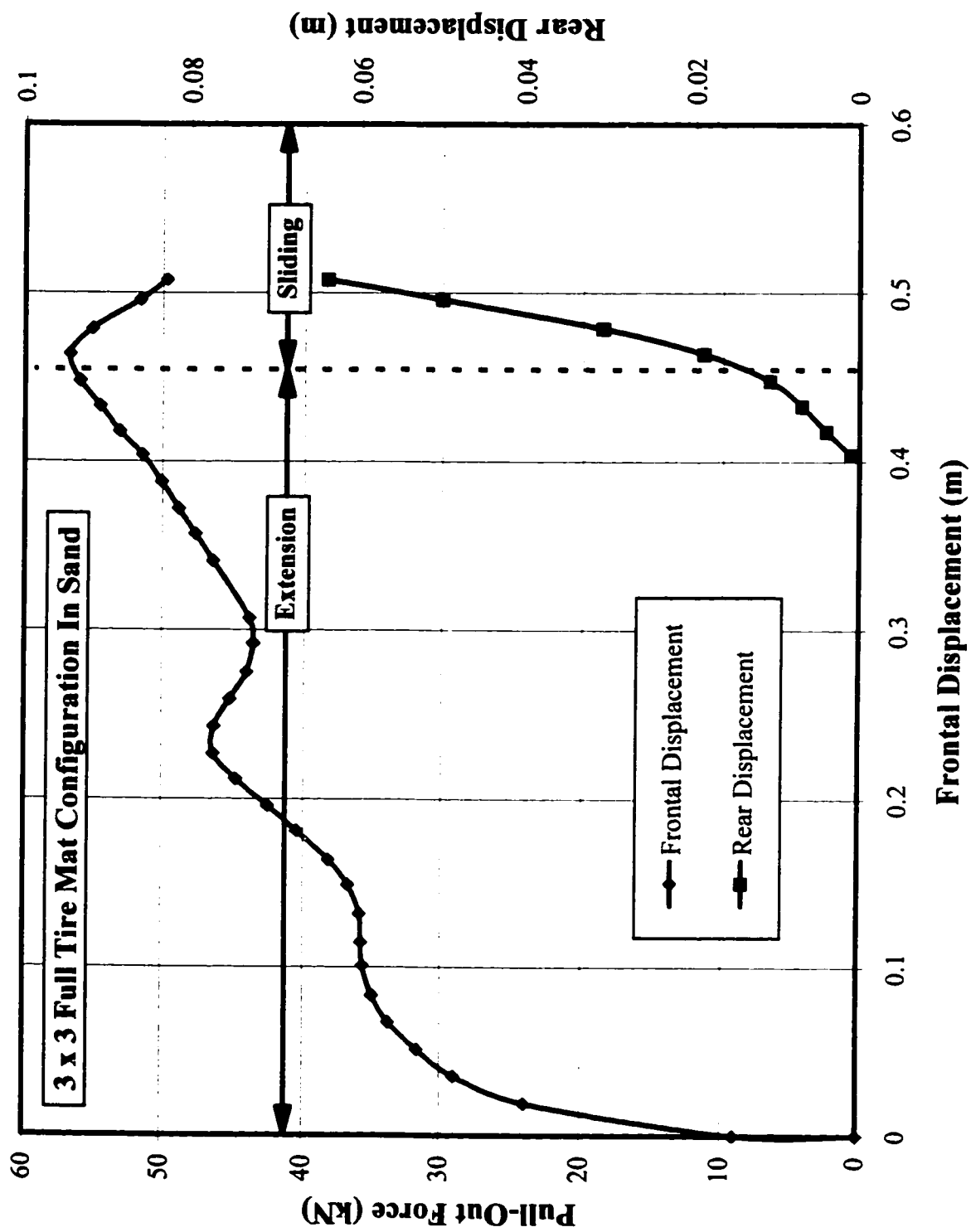


Figure 4.24. Pull-out force-frontal and rear displacement for a 3 x 3 full tire mat configuration in sand, showing zones of reinforcement extension and sliding.

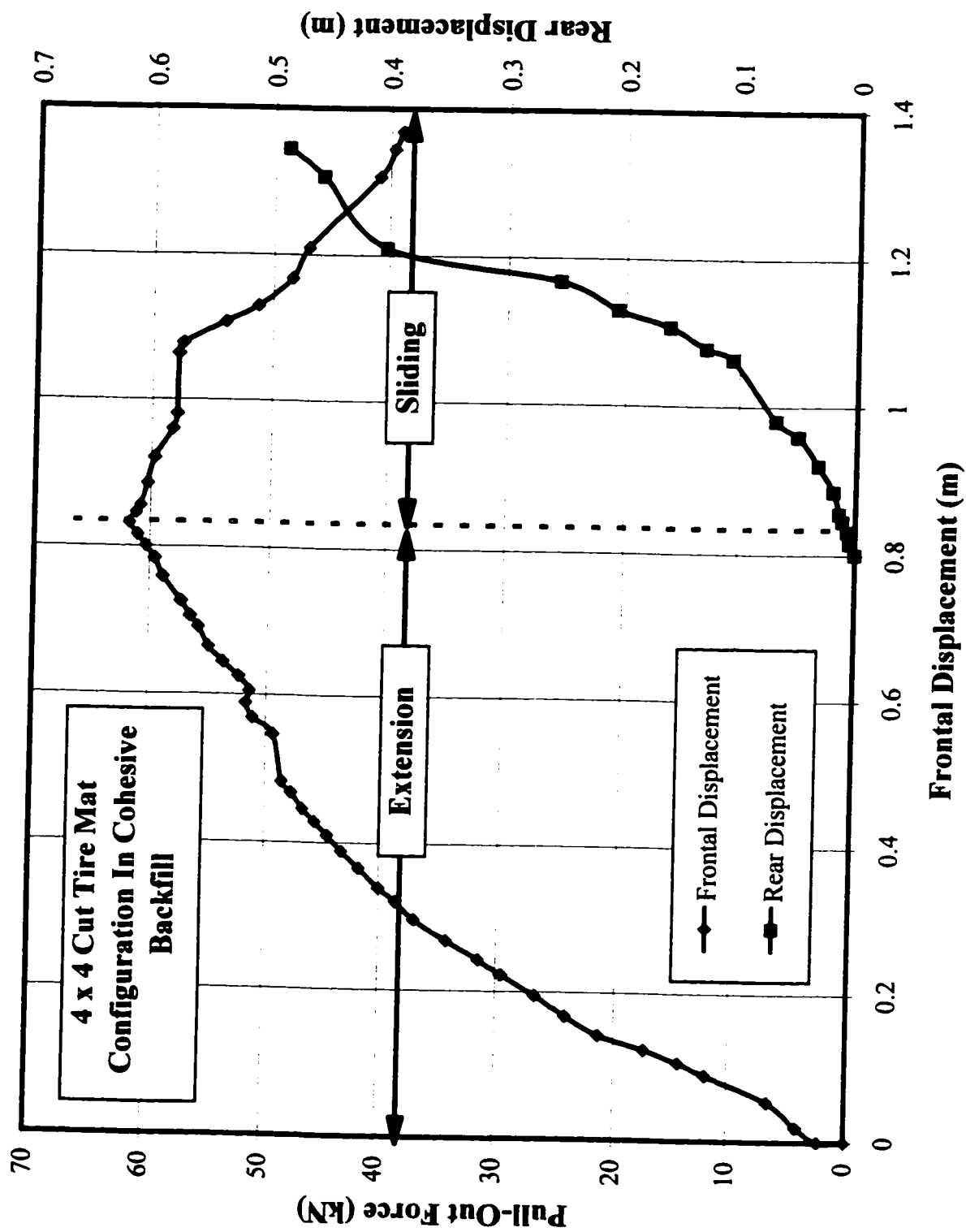


Figure 4.25. Pull-out force-frontal and rear displacement for a 4 x 4 tire mat configuration in cohesive backfill, showing zones of reinforcement extension and sliding.

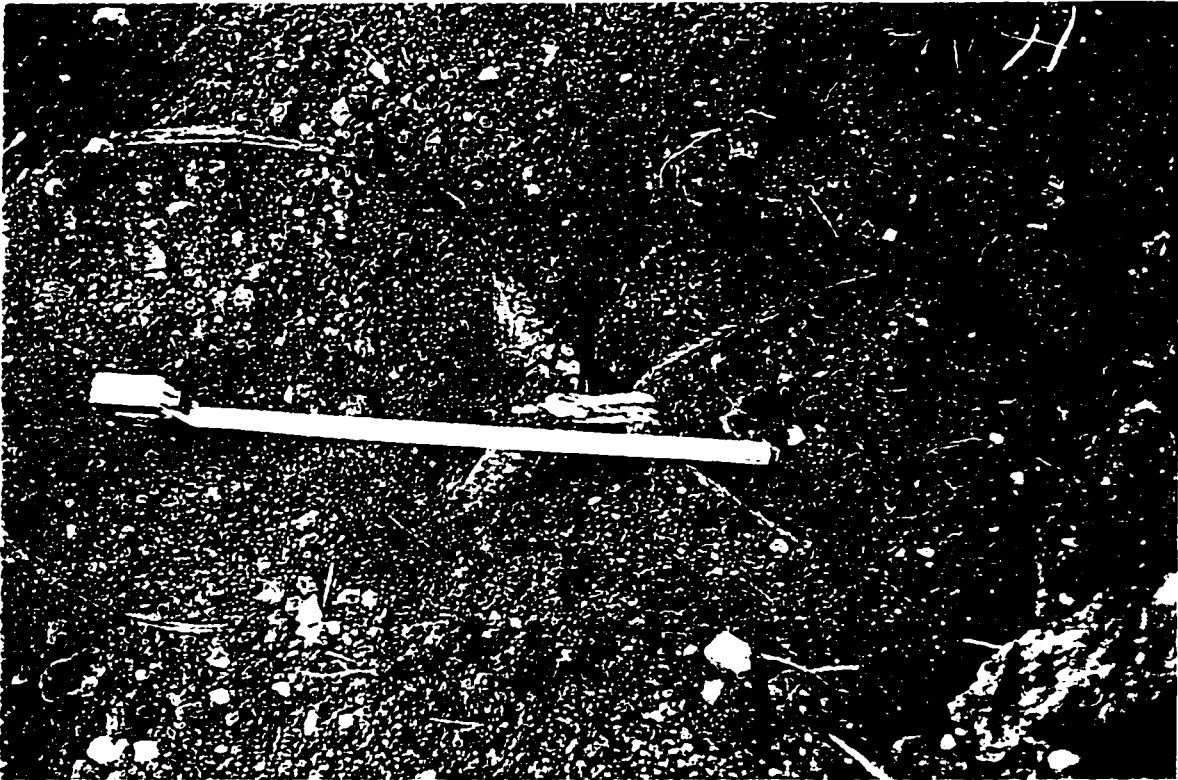


Figure 4.26. Elongation of the rope and engagement of the second row of tires.

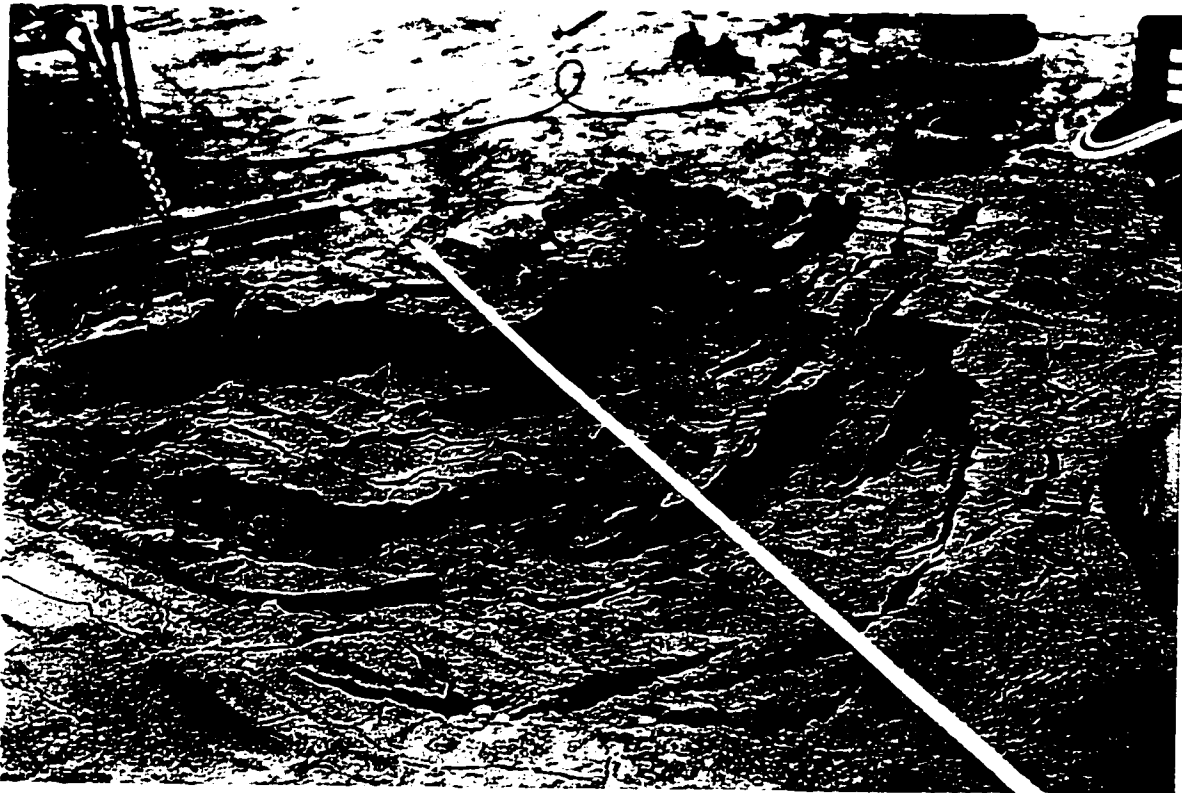


Figure 4.27. A series of photographs showing the progressive failure of a 3x3 cut tire mat reinforcement in sand.

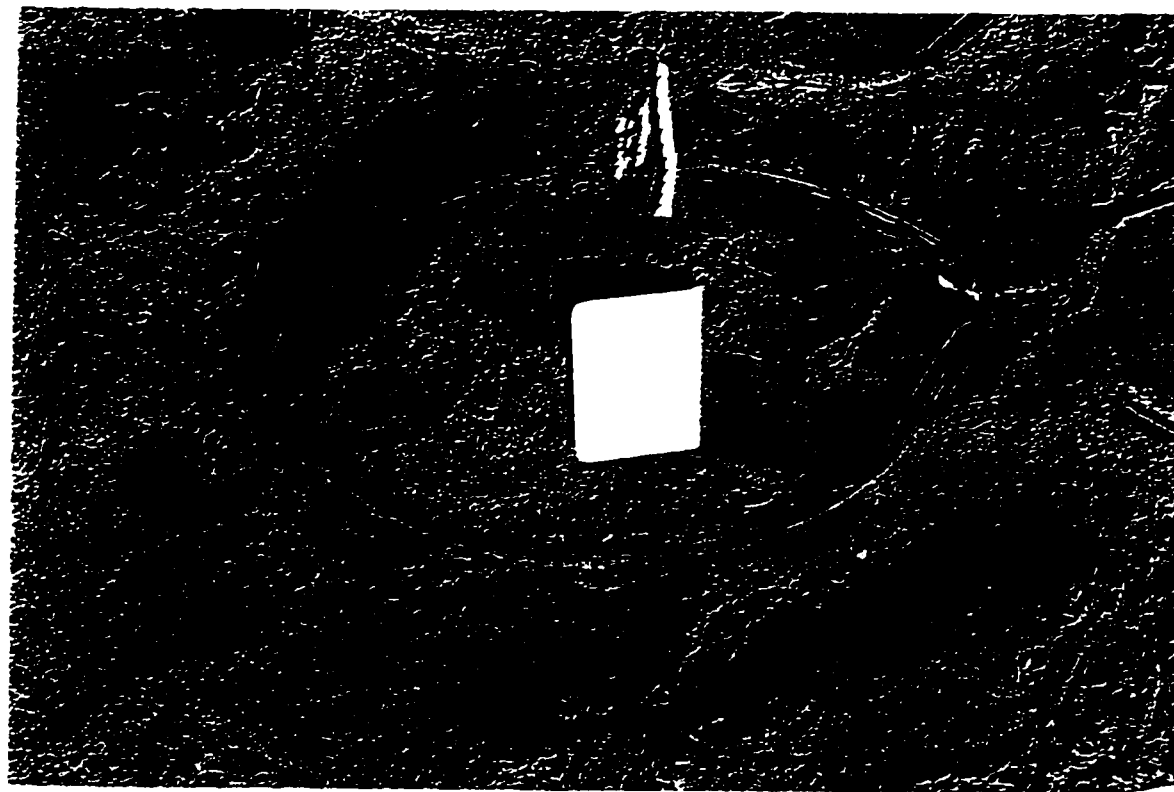


Figure 4.28. The deformed oval shape of the two tire elements (cut and full tire) after a pull-out test.

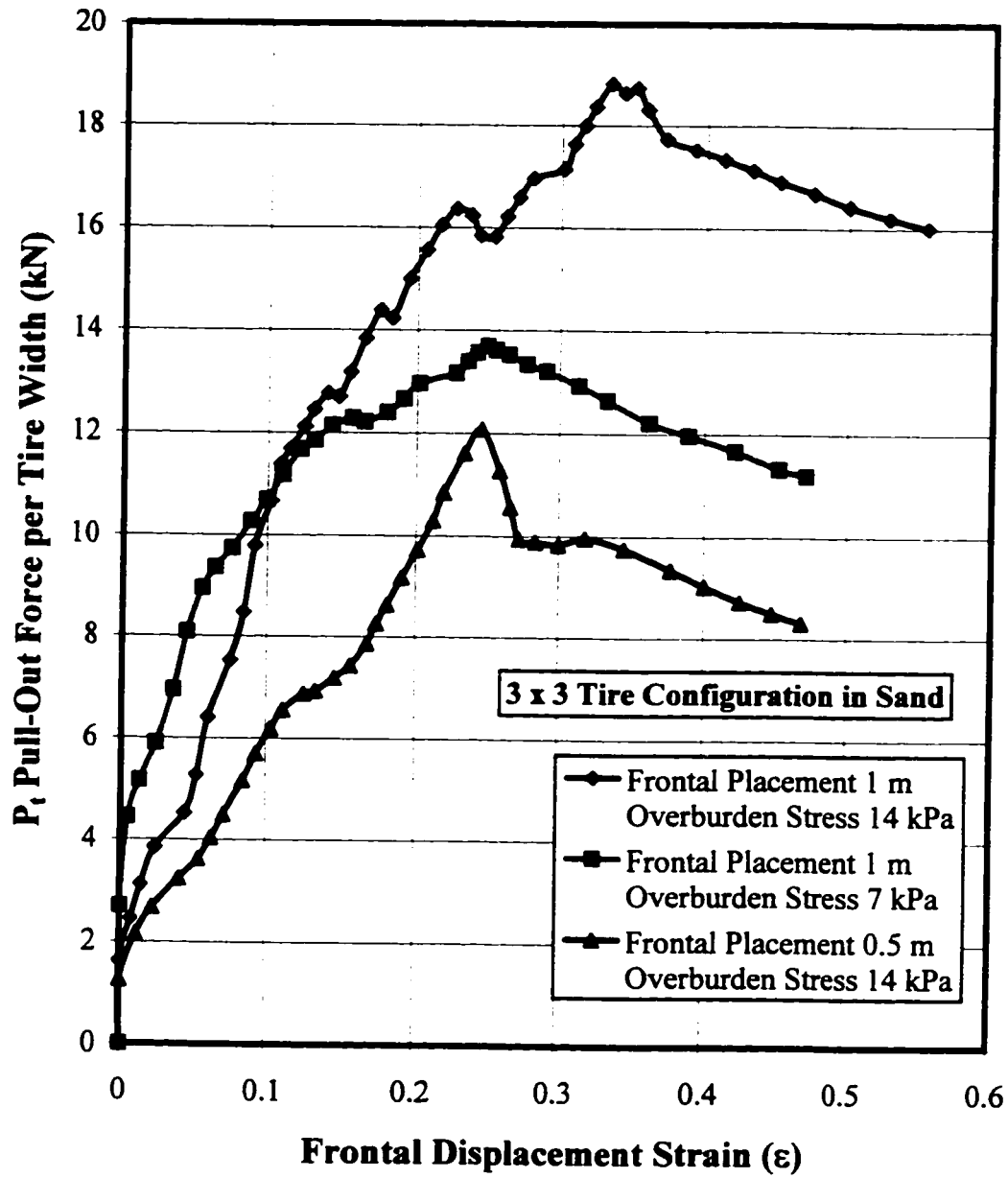


Figure 4.29. Comparison of the different pull-out resistance for the three major initial boundary conditions.

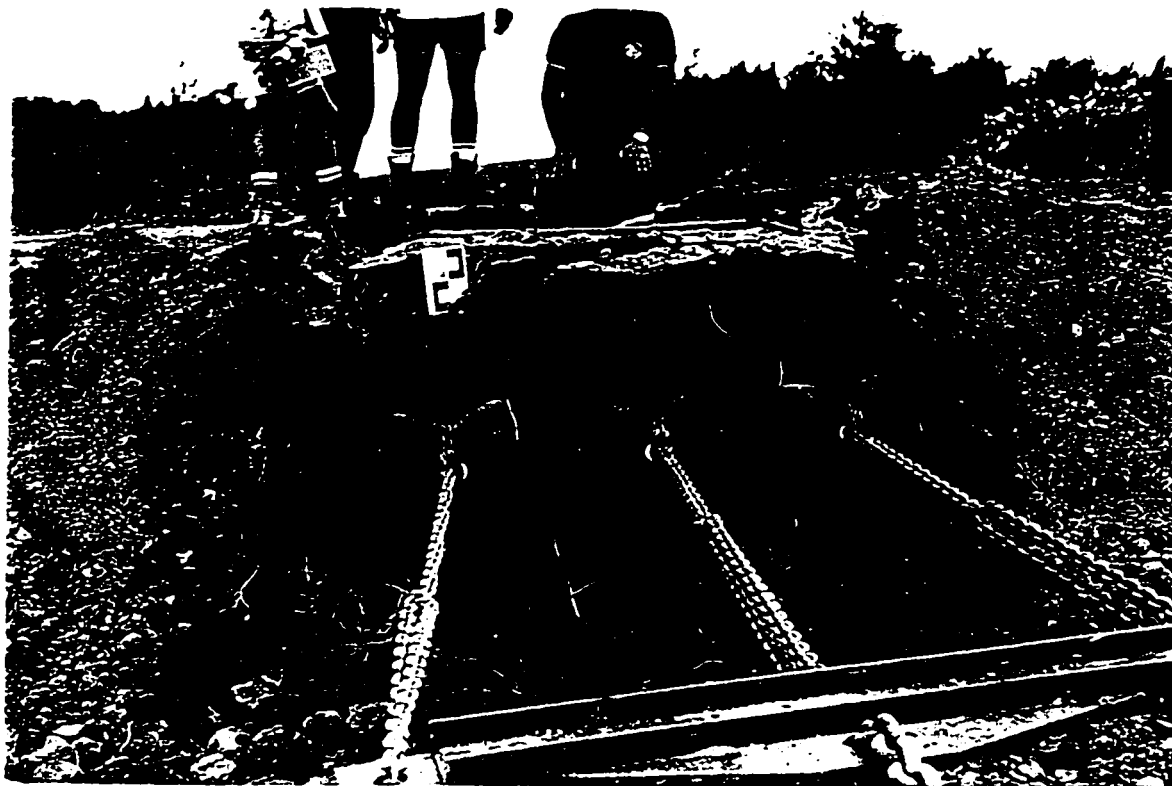


Figure 4.30. Extension of the frontal row of tires during pull-out, frontal placement of tires was 0.5 m.

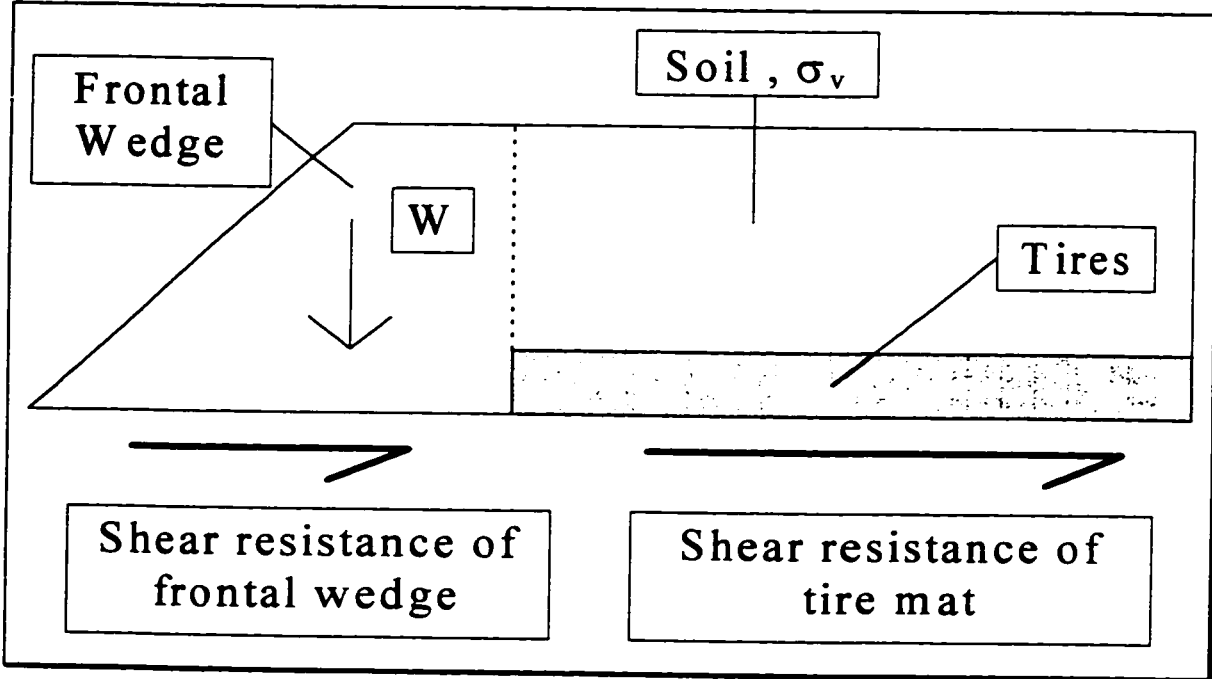


Figure 4.31. Shearing resistance mobilized during pull-out tests of tire reinforcement.

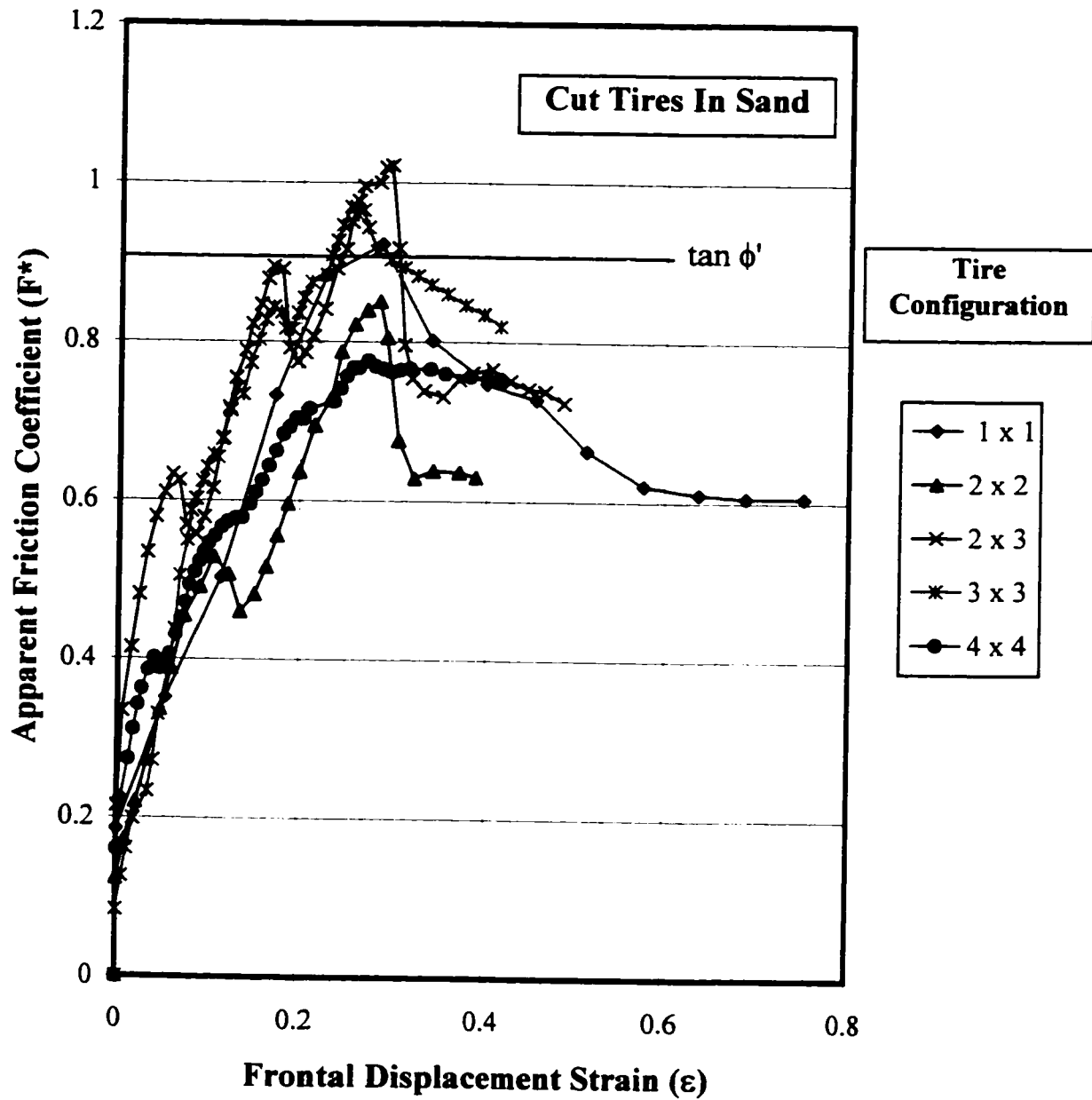


Figure 4.32. The variation of the apparent friction coefficient (F^*) for different cut tire configurations in sand, placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

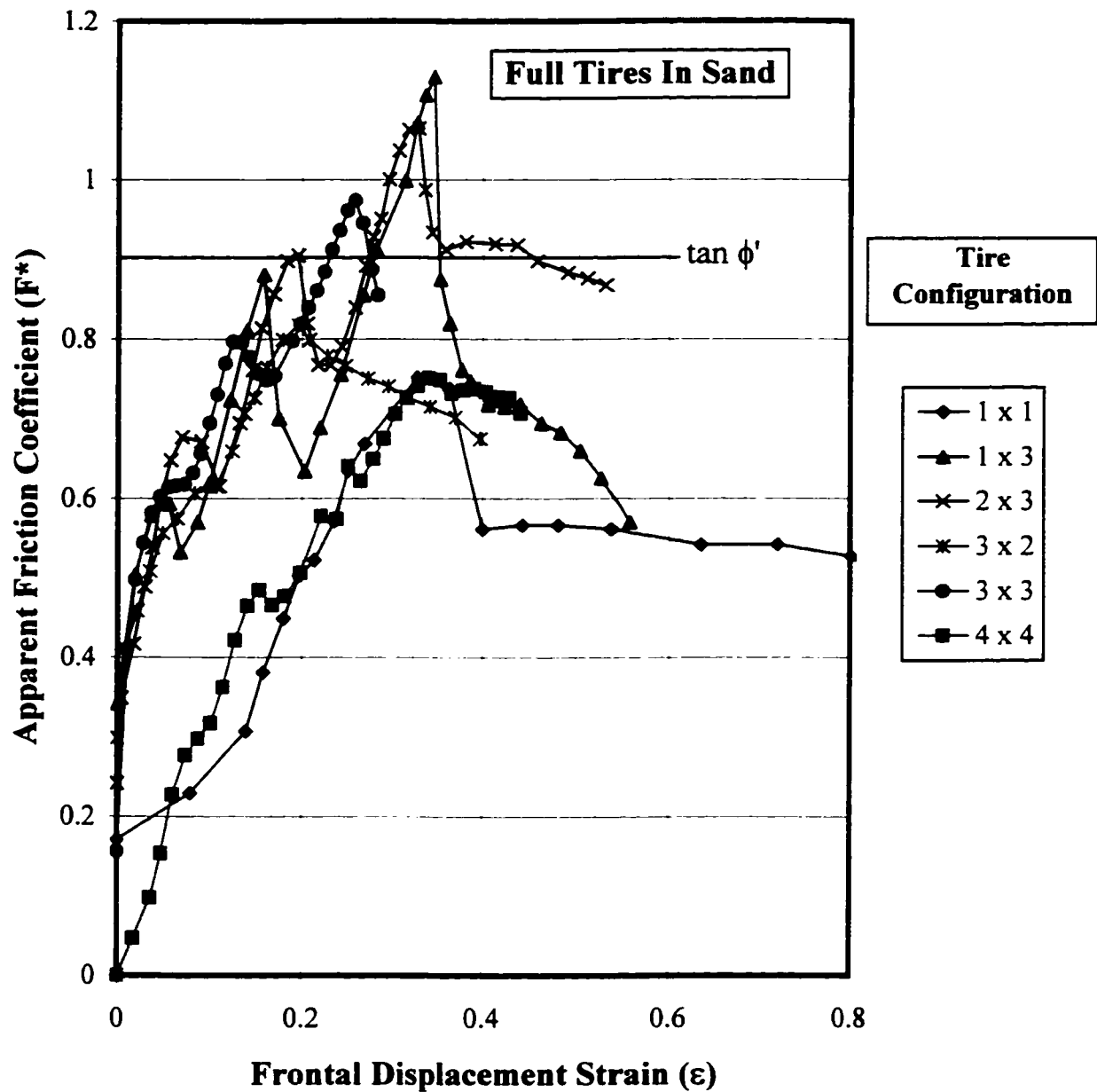


Figure 4.33. The variation of the apparent friction coefficient (F^*) for different full tire configurations in sand, placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

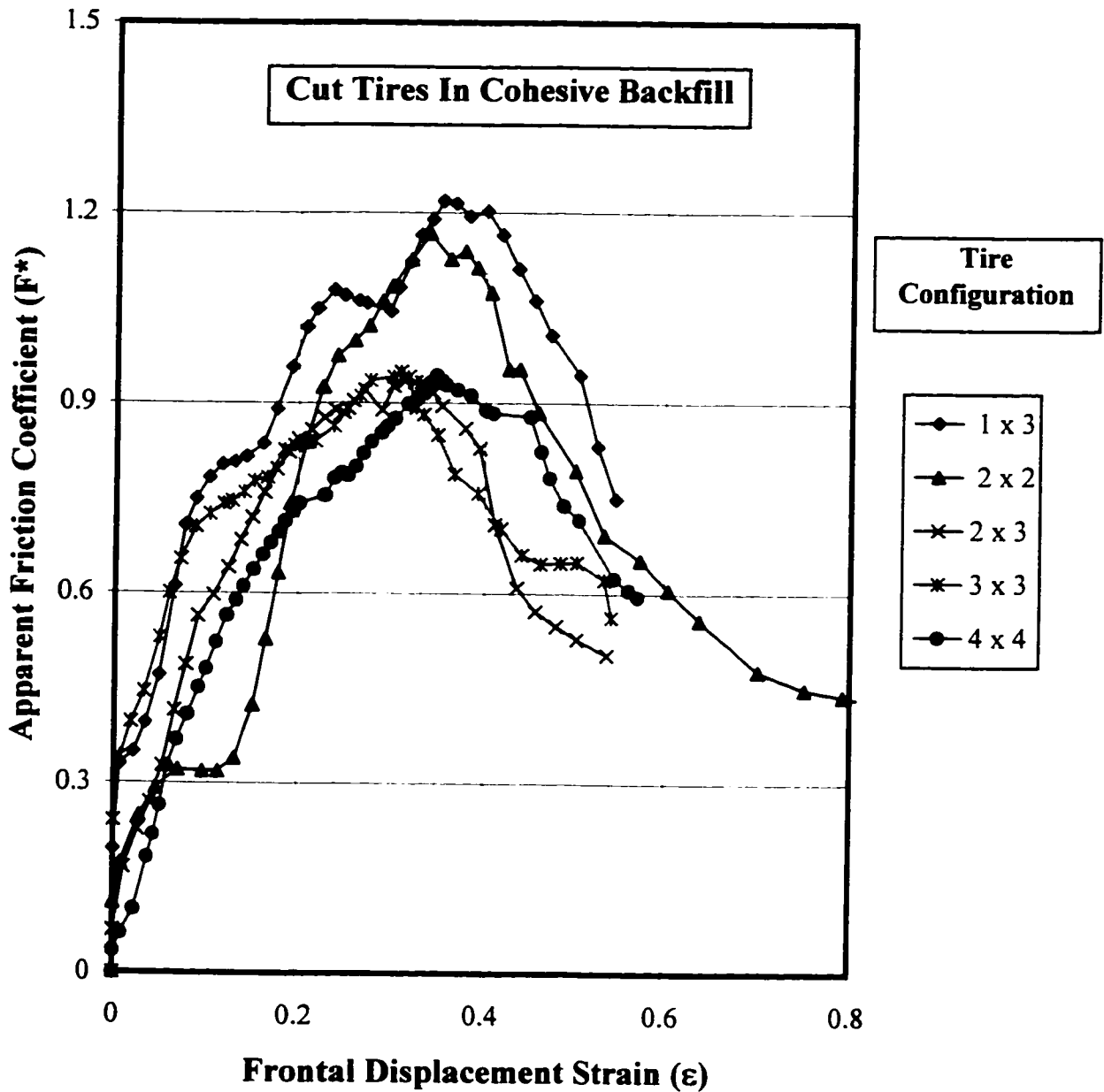


Figure 4.34. The variation of the apparent friction coefficient (F^*) for different cut tire configurations in cohesive backfill, placed 1 m behind the edge of the testing berm and under a confining stress of 10 kPa.

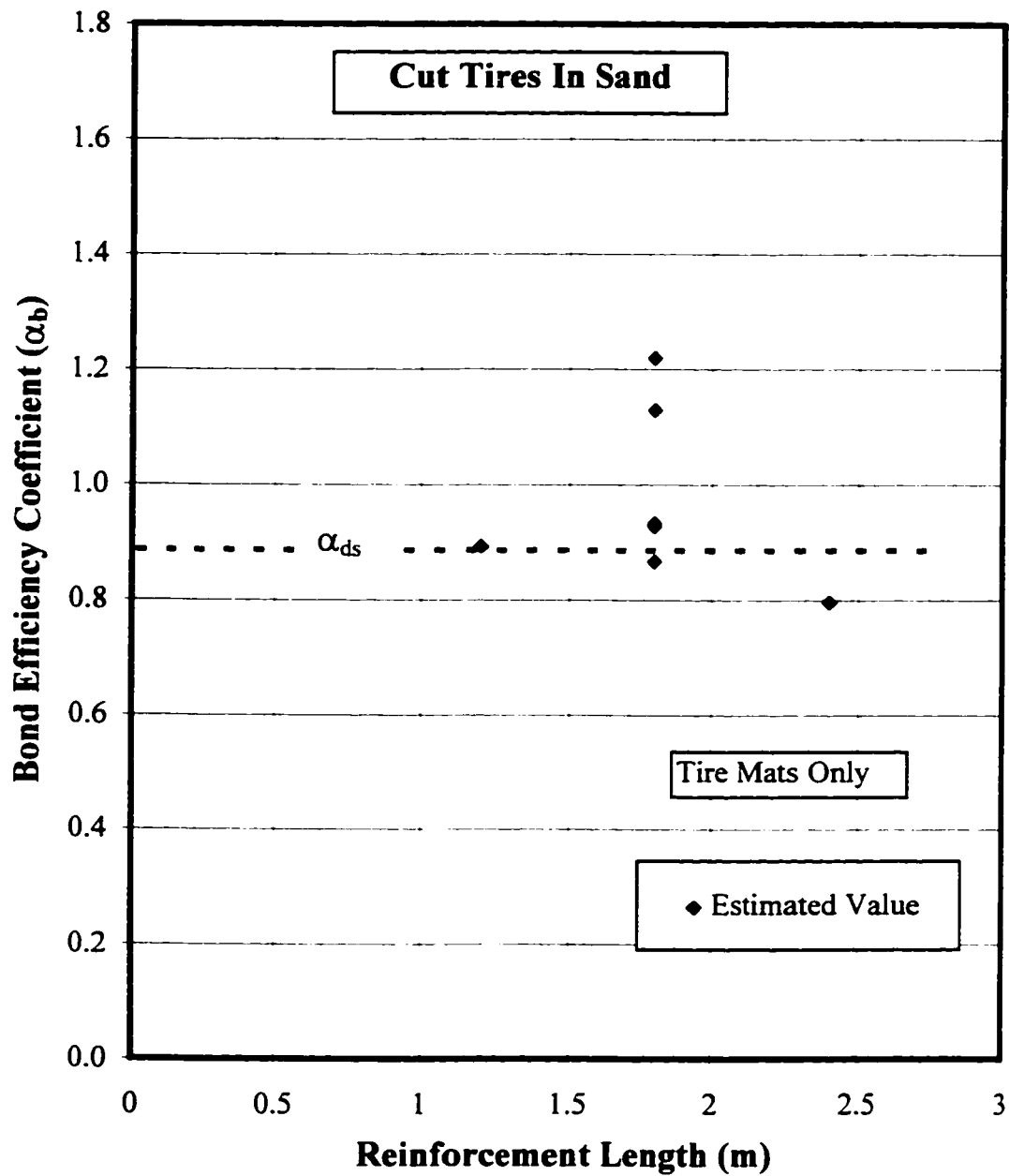


Figure 4.35. The variation of bond efficiency coefficient (α_b) with reinforcement length for cut tires placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

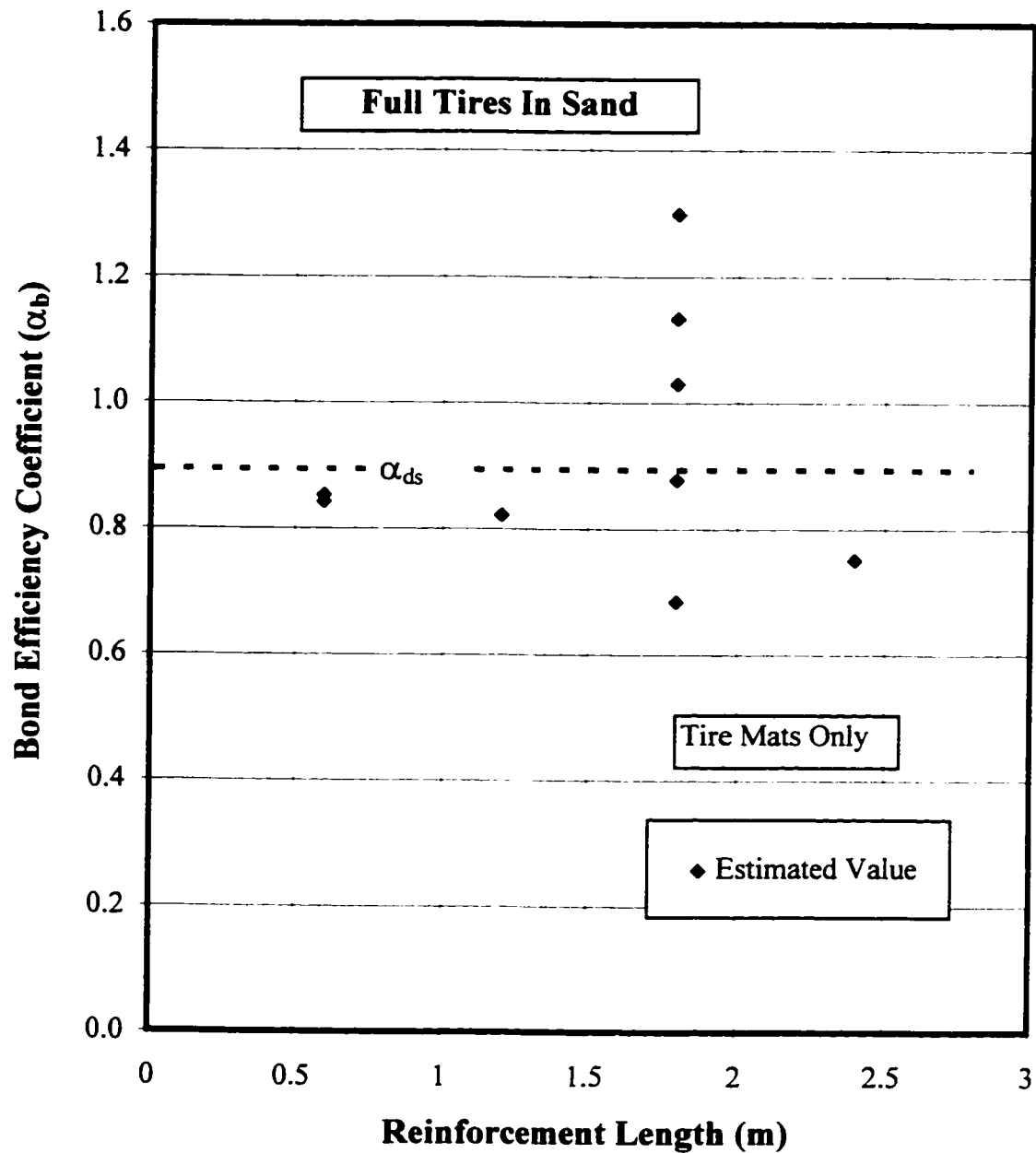


Figure 4.36. The variation of bond efficiency coefficient with reinforcement length for full tires placed 1 m behind the edge of the testing berm and under a confining stress of 14 kPa.

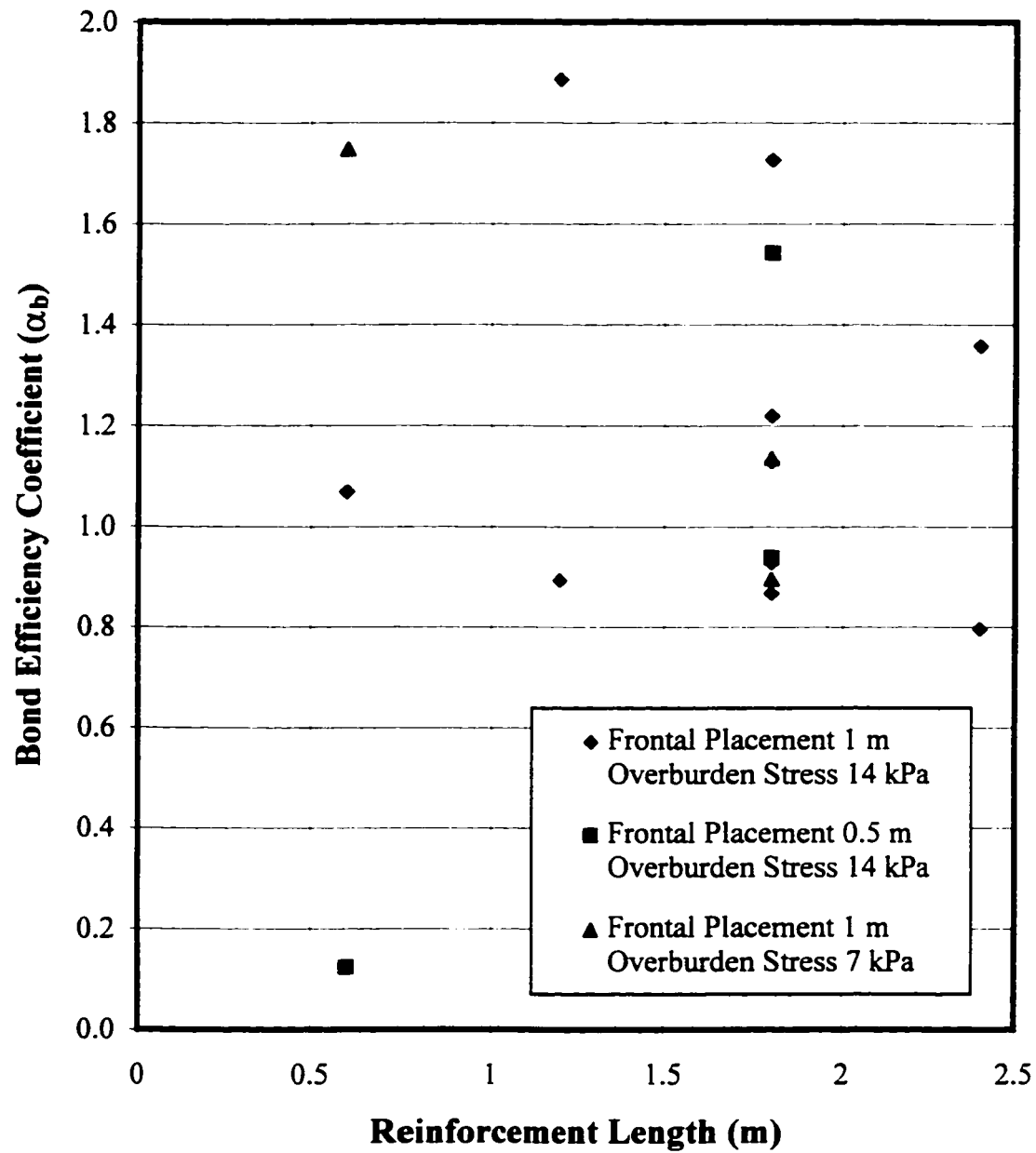


Figure 4.37. The variation of bond efficiency coefficient with reinforcement length for cut tires placed in sand under different boundary conditions.

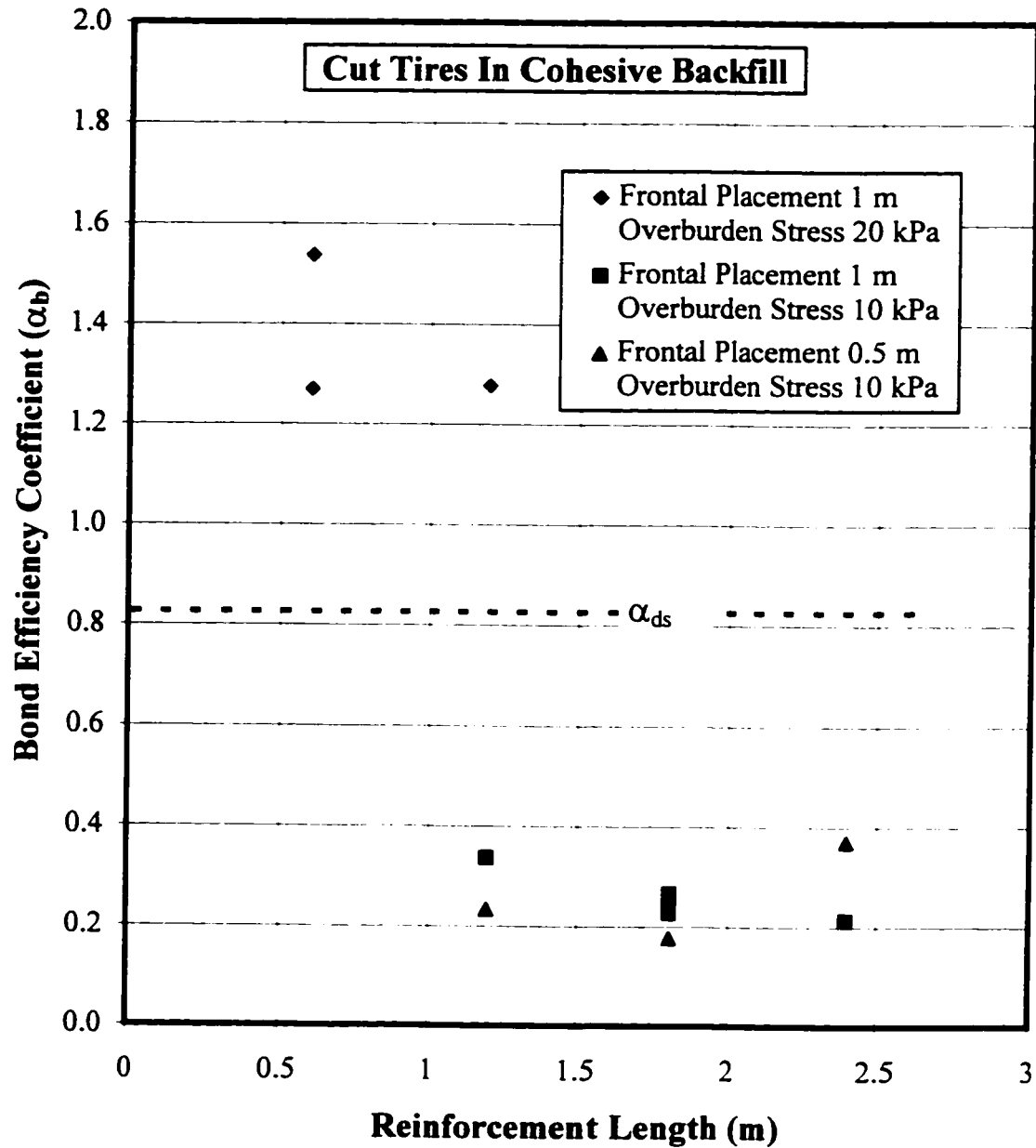


Figure 4.38. The variation of bond efficiency coefficient (α_b) with reinforcement length for cut tires placed in the cohesive backfill.

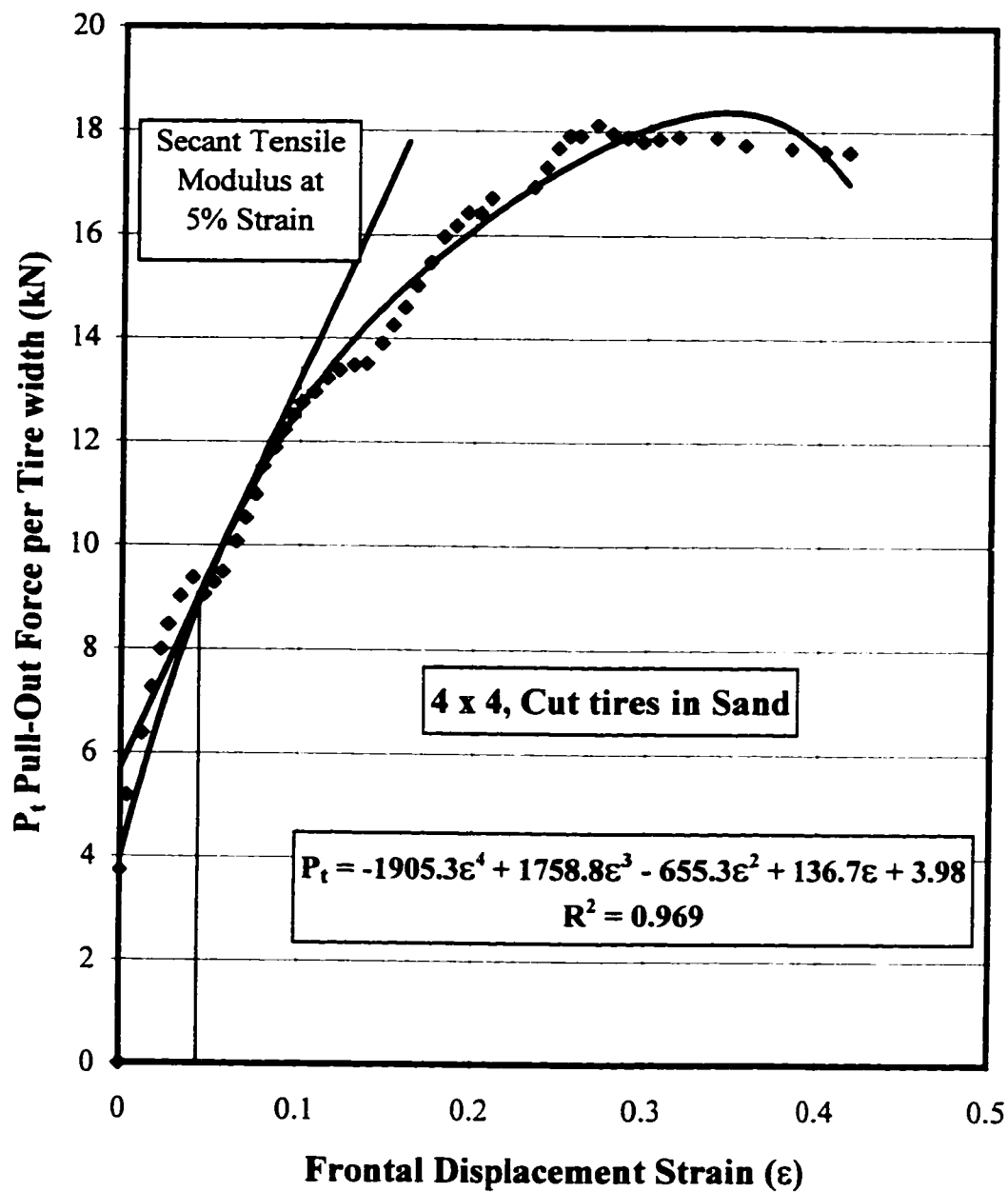


Figure 4.39. Fitting of the measured data with a 4th degree polynomial and indicating the secant tensile modulus at 5% strain.

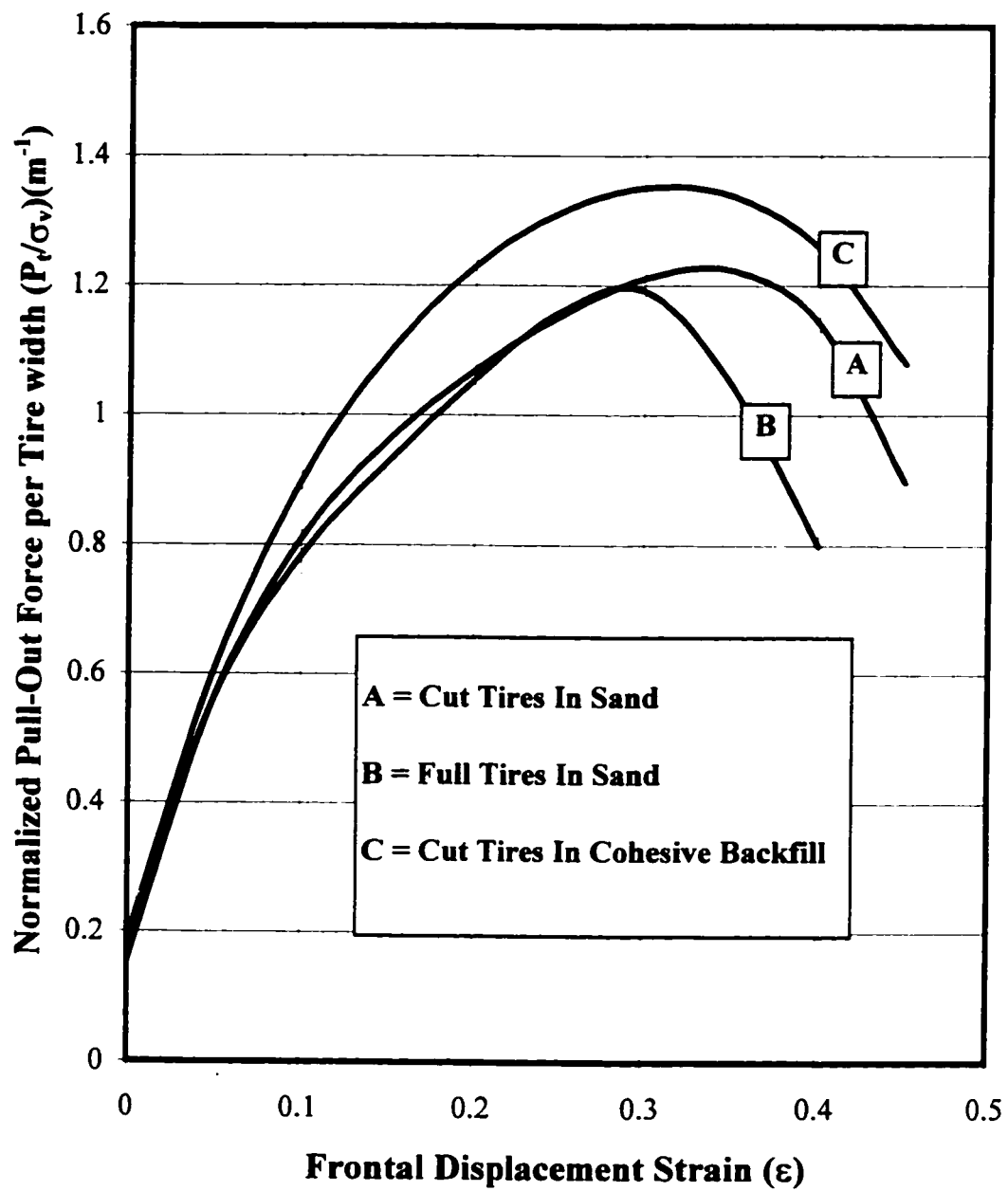


Figure 4.40. Normalized pull-out force per tire width versus frontal displacement strain.

CHAPTER 5

DESIGN, CONSTRUCTION, AND PERFORMANCE OF A TEST EMBANKMENT

5.1 Introduction

The performance standards for design and construction of conventional gravity retaining walls, and reinforced earthfill structures are well established. However, in Canada, experience with reinforced soil structures constructed with scrap tires is very limited. Engineers and contractors have little experience, or none, with this type of reinforcing system. Therefore, it is important to identify the various important design parameters, construction procedures, and performance criteria. Reinforced soil structures should be designed to be able to support the self weight and any externally applied forces. Likewise the retaining walls should remain stable under externally applied loading as well as be able to withstand lateral earth pressure. Reinforcement orientation, degree of compaction, induced compaction stress, and damages are important parameters related to construction. Where prior experience is unavailable, as in the case of tire reinforced structures, then the performance can best be evaluated by an analysis from instrumented structures. In the Canadian practice, the guidelines for the design of retaining walls and reinforced fills using conventional reinforcing materials such as geogrids, are well established (e.g. Canadian Foundation Engineering Manual (CFEM), 1992). It was therefore of interest to examine if the same, well accepted, approach could equally be applied to tire reinforced structures. Therefore, during the course of this investigation, a prototype tire reinforced embankment which incorporated various sections of reinforced fill and retaining walls was designed according to these guidelines, constructed and instrumented to monitor field performance.

This Chapter first presents the design considerations. The construction of the prototype embankment and the instrumentation is also described. The results of post-construction field instrumentation monitoring, as well as the results of large plate loading tests are also presented. The latter were performed to assess the compressibility and bearing capacity of tire reinforced earth fills.

5.2 Design Considerations: Failure Modes

(a) Retaining Walls

A retaining wall constructed with tire reinforcement in which the tire mats are stacked on top of each other (similar to a crib retaining wall), and the voids filled with soil, if properly designed, should perform as a “homogeneous” composite material. The various potential failure mechanisms are illustrated in Figure 5.1. The potential failure modes are:

- sliding of the reinforced volume at the base;
- overturning of the retaining wall, including overturning at some elevation above the toe;
- bearing capacity failure or excessive settlement of the foundation soil;
- deep seated stability failure, or slip along an internal plane of weakness;
- loss of serviceability due to excessive deformation.

(b) Reinforced Slope

Failure modes for reinforced slopes over stable foundations are classified as internal and compound (Figure 5.2). Internal failure occurs when the critical failure surface passes through the reinforced zone. The earth structure fails either by pull-out of the reinforcement or rupture of the reinforcing element (tensile capacity of the reinforcement is surpassed). External failure occurs when part of the critical slip surface passes outside the reinforced zone leading to subsequent sliding along the base of a reinforcement layer. This potential mechanism is

similar to that described for retaining walls; consequently, the sliding mass may be considered as a unit.

5.3 Preliminary Design Procedure

The recommended design procedures based on the results of the current investigation are presented in Chapter 8. The following general design procedures were used to establish preliminary factors of safety for the various stability requirements during construction of the test embankment.

5.3.1 Retaining Wall

The following design procedures are valid for the types of structures investigated in this research (a gravity retaining structure and reinforced slope). External stability governs the design for the tire gravity walls. This structure must be able to resist the horizontal earth pressure from the soil retained behind the tire wall and any load applied at the top of the wall. The different failure mechanisms have been outlined in Section 5.2.

Failure due to sliding along the base of the wall. Sliding along the base of the tire wall occurs when the driving force is greater than the frictional resistance. The resisting force at the base of the tire wall must be large enough to withstand horizontal sliding forces applied to the back of the wall (Figure 5.3). It is required that:

$$F.O.S_{\text{sliding}} = \sum \text{Horizontal resisting forces} / \sum \text{Horizontal sliding forces} \geq 1.5 \quad (5.1)$$

The resistance at the base of the tire wall is governed by the direct sliding efficiency coefficient (α_{ds}) and the unit weight (γ_{re}) of the tire wall. The direct sliding coefficient represents the minimum frictional properties at the base of the tire wall. The relevant expression is shown in Figure 5.3.

Failure due to overturning about the toe of the wall. The tire wall is considered to behave as a monolithic block of material. However, due to the flexibility of the structure, failure in this manner is questionable. Nonetheless, it is recommended that an adequate factor of safety should be used to limit excessive outward tilting and distortion of a suitably designed wall (Christopher et al. 1990). The sum of the resisting moments divided by the sum of the driving moments should be greater than or equal to the prescribed factor of safety (Figure 5.4).

$$F.O.S_{\text{overturning}} = \sum \text{resisting moments} / \sum \text{driving moments} \geq 2.0 \quad (5.2)$$

Bearing capacity failure. Stresses imposed by the self weight of the tire wall and surcharge should not exceed the bearing capacity of the foundation soil. The vertical stress at the base is estimated using the Meyerhof distribution (Figure 5.4). A minimum safety factor of 2 is generally accepted for reinforced soil walls. The flexible nature of the tire wall and the ability to accommodate large differential settlement reduces the required minimum factor of safety when compared to conventional reinforced concrete retaining walls ($FS \geq 3$).

Overall Stability. Both reinforced slopes and tire retaining walls must also satisfy an overall slope stability. The overall stability is determined using an appropriate classical slope stability analysis method. The global stability analysis to determine both internal and external stability should be performed. It is important to ensure that the tire retaining wall remains intact. The integrity of the structure can be verified by analyzing various failure planes and ensuring that there is sufficient resistance along the critical plane. Shear resistance along a potential failure surface is increased by providing a suitable overlapping pattern of tires and tying the frontal tire elements to those behind.

5.3.2 Reinforced Slope

The stability analysis of reinforced slopes and embankments are performed using modified versions of the classical limit equilibrium slope stability methods. The reinforcement increases the resisting force or moment by providing a tensile force, of specified amount, acting at their

installed location within the embankment or slope. Shear and bending strengths of the tire reinforcement are not considered and therefore the analysis is somewhat conservative. Generally, most methods provided essentially the same factor of safety, provided that the reinforcement forces are included in the equations of horizontal, vertical and moment equilibrium (Wright and Duncan 1991). The tensile capacity of the tire reinforcement layer is the minimum between the allowable (factored) pull-out resistance behind the potential failure surface and the allowable tensile strength of the attachment scheme (a detailed procedure is given in Chapter 8). Therefore, it is required that:

$$T_{dc} = \frac{P_r}{FS_p} \leq T_a \bullet R_a < T_r \quad (5.3)$$

Where:

T_{dc} = the design tensile capacity of the tire reinforcement which considers rupture and pull-out.

P_r = the unit pull-out capacity of the tire reinforcement (Equation 4.13 or 4.14).

FS_p = the factor of safety for pull-out. A value of 2.5 is recommended for the factored pull out capacity of the reinforcement. This factor takes in to account the allowable deformation of approx. 5% strain in such engineered structures. A higher factor may be used to limit the allowable strains further.

T_a = the allowable tensile strength of the attachment with respect to service life and durability (see section 2.9, Equation 2.2).

R_a = attachment coverage factor, relates the number of attachments per unit width of tire reinforcement.

T_r = the tire tensile strength per unit width of reinforcement. For tire mat reinforcement, a conservative value of 80 kN/m can be assumed (see section 2.14).

The internal stability analysis for a reinforced slope is outlined in Figure 5.5. The tensile force direction influences the calculated slope safety factor. The tire reinforcement layer is fairly stiff in the vertical direction (perpendicular to the failure surface), and therefore, the tensile force T_{dc} is assumed to act parallel to the reinforcement layer (a conservative approach).

An external stability analysis (compound or deep seated failures) of the reinforced slope assumes that the reinforced soil mass acts as a monolithic block of material which is able to withstand all external loads without failure. Depending on the use of the structure, and the type of backfill used (sandy or clayey soil), the external stability analysis may be required for both the short term (undrained) and the long-term (drained) conditions.

5.4 Test Embankment

A prototype test embankment was contracted to Deschenes Construction (Ontario) LTD on the private property of Conroy Auto-Parts Recycling, near Ottawa. Large numbers of tires were already stored on the site for several years. Also, this site had been operational as an auto recycling facility for considerable period of time. The site was licensed by the Ontario Ministry of the Environment as an approved site for scrap tire storage. The test fill incorporated three reinforced slope configurations, and three tire reinforced gravity wall sections. In the reinforced slope, the tires were used either as whole tires, or with one side wall removed. The latter is referred to as the cut tire. In the reinforced fill section, a mat of tires tied together was placed followed by a compacted backfill layer of soil, 0.3m thick, followed by the next layer of the tire mat. Hence in the reinforced slope, the tire layers were separated by a layer of compacted soil. In the construction of the retaining wall section, however, the tires were stacked on top of each other in a staggered manner. The voids were filled with soil and compacted before the next layer was placed. Hence in the retaining walls, a sandwiched layer of soil was not provided. More details are provided in the following sections.

The test embankment was instrumented to provide a better understanding of the engineering behaviour of retaining walls and reinforced slopes using scrap tires. To evaluate any toxic effects of buried used tires on the surrounding groundwater, a drainage system was installed below the embankment and the effluent collected in three wells. The results of the analysis of water quality data are presented in Chapter 7.

5.4.1 Embankment Design

The reinforced embankment was designed to use conventional construction techniques and to reuse old tires with the minimum of processing in order to maximise economic benefits. The test embankment was constructed over a sand drainage blanket, and has a height of 4 m (excluding the 2 m high final surcharge). The plan dimensions of the test embankment are 17.4 m width and 57 m length. The embankment geometry, tire mat lay-out, soil type are presented in Figure 5.6. The corresponding cross-sections A, B, and C are provided in Figures 5.7 to 5.9, respectively. The lay-out of the different instruments used to monitor the test embankment are also provided in these cross-sections. Figure 5.10 shows a longitudinal section of the entire structure.

The reinforced earth structure is composed of three 10 metre long sections, independent of each other, Section A, B, and C. Each section was constructed of two types of tire mat reinforcements for use as a retaining wall structure and as a reinforced steep slope. The three different configurations of retaining wall and reinforced slope are as follows: Section A comprised of cut tires filled with sandy soil; Section B with entire tires filled with sandy soil and Section C, constructed of cut tires filled with cohesive soil. The cohesive soil, in reality a plastic sandy silt was obtained from a stockpile at the recycling location. The sand was imported from a quarry. These materials are the same as those reported for the tire pull out tests.

The design procedures outlined in Section 5.3 were used to determine the various factors of safety. Estimated factors of safety for the different types of structures employed in the prototype test embankment are provided in Table 5.1. In Table 5.1, the different factors of safety for the three retaining walls were calculated using Rankine active earth pressure theory (K_a) and a trial-wedge method. A Rankine active earth pressures distribution behind the different retaining walls was assumed for the preliminary calculations before construction. However, field results from pressure cells placed behind the retaining walls reported larger lateral earth pressures (see section 5.6). Results of the study indicated that the reinforced tire wall may be more compressible than the backfill. Consequently, the interface wall friction acts up (not down, as shown in Figures 5.3 and 5.4) developing negative wall friction. The stability analysis was repeated using a trial-wedge

in the active state and a corresponding negative wall friction ($-\delta$) component. Interface friction angles determined by interface direct shear tests, outlined in Chapter 4, were used. All tire walls had factors of safety greater than one.

It was particularly difficult to estimate the in-situ density of the tire reinforcing layers. The degree of in-filling of the tires was uncertain, especially for whole tires. A 10% reduction in the dry unit weight based upon proctor tests was assumed for whole tires in sand, while 5% reduction was used for cut tires reinforcements. These reduction values compared very well with in-situ measurements, as discussed in section 5.4.4. However, the amount of voids within the tire mat reinforcement will tend to decrease with increasing confining pressure, as the tire wall is constructed. Also, tires will tend to distort as the height of the wall increases. The composite density of soil and tires will approach that of the backfill as the depth increases. The estimated pull-out resistance was based upon a factored reinforcement length of two tires (1.2 m) for serviceability considerations, even though a greater length (L_e) was provided, and a safety factor (FS_p) of 2.5 was used. The tensile strength of the rope attachment per tire (T_a) was determined to be 40 kN (each frontal tire in general was anchored to two or more rear tires with 2 wraps of rope). Short term strength of the rope was only considered, strength reduction for creep behaviour was ignored. The bearing capacity factor of safety was much greater than 2, since the prototype embankment was constructed over a very competent and stiff foundation, composed of a mixture of boulders, gravels and sands and at least 1 meter thick.

5.4.2 Instrumentation

A plan view of the instrumentation lay-out is given in Figure 5.11, and the different cross-sections also provide information on instrumentation locations. Four inclinometers casings were installed in each section; two for each retaining wall structure and two for each reinforced slope in order to monitor lateral movements. Deep magnetic settlement gauges were placed in the reinforced slope and the unreinforced section between the retaining wall and steep slope to observe settlements. Surface topographical monuments were also installed after the final height was reached.

Drilling experience at landfill sites indicates that it is often very difficult to drill through tires, since the steel belts and wires entangle with the rotating bit. Consequently, during construction of the fill, care was taken to ensure that no tires would be located along the vertical alignment of the drill holes which would subsequently be drilled for the installation of instrumentation. This was ensured by using a system of reference guide wires to locate the pre-planned drill hole positions.

Three pneumatic pressure cells were placed behind each of the three retaining walls at different heights to measure the lateral stresses, and to determine the lateral earth pressure coefficients. Their locations are at a depth of approximately 1.0 m, 2.0 m and 3.75 m (height of the wall is 4 metres). Each pneumatic pressure cell was calibrated by RST Instruments in an air pressure chamber. Surface monuments were also installed at various locations in all three sections to evaluate the overall settlement of the structure. These monuments were measured using an automatic level.

5.4.3 Construction of the Test Embankment

The field construction commenced with the levelling and compaction of a 300 mm thick clean sand drainage blanket (Figure 5.12) which included the installation of a 76 mm perforated pipe wrapped in a geotextile for effluent water collection under each section. A great number of tires stockpiled at the construction site required the removal of the inner steel rim. The inner rims were removed with a “derimming machine” (Figure 5.13). The tires were sliced on-site to remove one sidewall by using a specially designed and transportable machine, fabricated in Ontario (Figure 5.14). This machine performs the tire cutting by supporting the tire horizontally while a hydraulic jack (cylinder) fitted with a slicing carbon-steel blade cuts as the tire is rotated. A two-man crew was able to slice 300 cut tires per hour, if the tires were clean. The sliced side wall was placed inside the tire in order to avoid creation of unnecessary waste.

The gravity retaining wall and reinforced steep slope were constructed simultaneously. These two structures were built in an embankment with the retaining wall and reinforced steep slope on

opposite sides. Layers of tires were placed side by side, tied together with three turns of 9.5 mm diameter polypropylene rope (Figure 5.15), filled with the appropriate soil and compacted. The sand and clay backfills were spread using both front-end loader and a DC-3 bulldozer, shown in Figure 5.16. It is important to mention that the cohesive soil used was lumpy and wet and would not have met the accepted construction specifications for an earth fill. Poor quality clay fills are generally not acceptable for conventional embankment construction. Tire mats were vertically staggered in the construction of the retaining wall to promote interlocking between layers. Slope reinforcement was achieved by tying tires together in layers spaced at a vertical distance of 0.5 m. These procedures were repeated for the entire 4 m height of the structure. A series of photographs depicting the retaining tire walls at different stages of construction are presented in Figures 5.17 to 5.20, respectively. A frontal view of the different reinforced slope sections after completion is given in Figure 5.21. The construction procedures were interrupted only occasionally for installation of pressure cells located behind the retaining wall and a few inclinometer tubes. After the completion of the embankment, the settlement gauges and the remaining inclinometer tubes were installed by drilled holes. Most of the instrumentation was placed in the centre of each section to measure plane strain conditions and to eliminate any edge effects.

Relatively little construction equipment and work force was required. The construction of the prototype fill was completed within 2 months by a three-man crew with a front end loader and a self propelled vibrating drum roller. The construction time includes the installation of instrumentation and the placement of the final surcharge load. This surcharge load consisted of approximately 600 m^3 of low quality backfill, equivalent to a 2 m height of fill, which was placed with a small excavator (Figure 5.22) and lightly compacted. It should be noted that additional external loads generated by construction equipment were not considered in the design analysis. This omission would over-estimate the factors of safety. Slippage along the base of the retaining tire walls constructed using sand as a backfill (section A and B) was not observed which may indicate the possible development of higher frictional resistance between the different reinforcing layers. However, minor slippage at the base of the tire wall constructed with the cohesive backfill was observed (Figure 5.51) which may be the result of lower bonding between the tire

reinforcement and the cohesive soil when subjected to undrained conditions. A careful control during construction of retaining walls using cohesive soils is recommended, since the bond between cohesive soils and tire reinforcement can be poor and will be further reduced if positive pore pressures are developed. The fill was compacted by a lightweight smooth drum vibrating roller, a Super Pac 540c model that was able to deliver a centrifugal force of 67 kN. At the initial stage of construction, the retaining wall was compacted (Figure 5.23); however, as the wall was being built up, compaction induced stresses resulted in large lateral deflections of the wall. As a result, compaction at higher heights was restricted to the back of the wall. Figure 5.24 shows the degree of in-filling and compaction of cut tire reinforcements constructed with the cohesive backfill material (retaining wall section). A photograph showing the prototype test embankment at completion, viewed from the reinforced slope sections is given Figure 5.25.

Three plate loading tests were performed after completion of the embankment but before the placement of the 2 m high final surcharge load.

5.4.4 In-Situ Density Determination

The in-situ density of the compacted sand layers were verified by the Rubber Balloon Method (ASTM D-2167). The variation of the dry density and unit weight of the sand layers with depth, for both section A and B, is given in Figures 5.26 and 5.27, respectively. The average dry density of the sand sections was 1680 kg/m^3 . The degree of compaction, based upon the Standard Proctor Test (ASTM D-698) was approximately 90%. The unit weight of the sand layers, on average, was 18 kN/m^3 , representing a 10% reduction in weight compared to the value estimated from the Standard Proctor Test.

The in-situ density of the compacted cohesive backfill was determined by pushing a calibrated hollow cylinder of known weight and dimension into the soil. The difference in mass between the empty cylinder and the cylinder with soil divided by the volume of the cylinder established the density of the backfill. The variation of unit weight with depth in section C is given in Figure 5.28. The density of the cohesive backfill exceeded the expected value evaluated from the

Standard Proctor test. The average in-situ unit weight of the cohesive backfill was approximately 20 kN/m^3 with a corresponding dry density of 1700 kg/m^3 .

5.4.5 Tire-Soil Density

The in-situ density of tire-soil material was also determined by means of a box arrangement, during the construction of the test embankment. The tire was placed in a square rigid box, open at one end, covered with the next layer of soil (0.5 m) and compacted. The box was then excavated from the surrounding soil, removed and the excess soil was trimmed off. Care was taken not to disturb the soil within the box. The weight of soil and tire within the box was determined by suspending it by a cable from a tripod that was equipped with a calibrated load cell. To account for the variability in tire sizes, three boxes were used. The square boxes were constructed out of 19 mm thick plywood, reinforced with steel angles. The dimensions of the three boxes were:

- 660 mm x 660 mm x 254 mm, a volume of 0.110 m^3 ;
- 711 mm x 711 mm x 254 mm, a volume of 0.128 m^3 ;
- 761 mm x 761 mm x 305 mm, a volume of 0.177 m^3 .

The reinforcement layer composed of cut tires filled with sand indicated an average unit weight of approximately 17 kN/m^3 . The reduction in the total density was about 5% when compared to the measured in-situ density of the sand layer. The small decrease in density is attributed to the lower density of the tire elements ($\sim 1100 \text{ kg/m}^3$). Tests performed on whole tire reinforcing mats filled with sand gave an average unit weight of 16 kN/m^3 , representing a decrease in total density of about 12% when compared to the in-situ density of the sand. The greater decrease in density when compared to cut tires filled with sand is associated with the formation of a void space at the top of the tire element. Compaction of whole tires, which are initially partially filled with soil, results in the collapse of the sidewall, as shown in Figure 5.29. The soil trapped within the tires is pushed further into the tire element resulting in the formation of a small empty gap.

The collapse of the sidewall is beneficial, because it decreases the amount of unfilled space within the tire element and produces a tire reinforcement of greater density.

A complete infilling of the cut tire reinforcement with the lumpy cohesive backfill was difficult. The measured average unit weight of 16.4 kN/m^3 reflects this observation. This represents a 20% decrease in density when compared to the compacted cohesive backfill layer. Compression of the void spaces may lead to an undesirable amount of settlement. The lower unit weight of the reinforcing layer should be considered in design when using cohesive soils or waste materials as backfill. With the benefit of the experience, it is recommended to perform compaction of cohesive soils in smaller lifts and to use a compactor with a “sheep’s foot” vibrating drum (a smooth vibrating drum was used on this project).

5.5 Plate Load Tests

The bearing capacity and modulus of compressibility of each section were determined by performing a large plate load test. The square loading plate $1.2 \text{ m} \times 1.2 \text{ m}$, comprised of two 12.7 mm thick steel plates. Smaller steel plates were stacked over the large square plate to reduce bending effects. The plate load test closely followed the ASTM D1194 -72 procedure. The setup for conducting the plate load test is presented in Figure 5.30. The impressive supporting platform was loaded up to 40 tonnes, and was able to apply a stress greater than 225 kPa to the soil structure (Figure 5.31). A reference beam in the shape of an “H” attached with dial gauge and liquid transducers manufactured in the university workshop were used to measure the plate and surrounding soil movements. A photograph showing the location of the plate and loading system with respect to the reference beam is given in Figure 5.32. The location of the transducers is given in Figure 5.33. The precision of the dial gauges was $\pm 0.01 \text{ mm}$, and of the liquid transducers, $\pm 0.05 \text{ mm}$. The plate was loaded in increments ranged from 10 to 20 kPa; this load was maintained until settlements had essentially ceased. The test was terminated when the peak loading capacity of the system had been reached or until the ratio of load increment to settlement reached a minimum, steady magnitude.

5.5.1 Results

The load per unit area (q) versus settlement (s) relationship for footing tests performed in sand reinforced with cut and full tires are given in Figures 5.34 and 5.35, respectively. At the maximum loading capacity of the system, 225 kPa, the ultimate bearing capacity (q_{ult}) for both tests were not reached. The ultimate bearing capacity, defined by a plate loading test, is the lesser of the load which produces a settlement of 25 mm or when failure occurs. A conservative estimate of the ultimate bearing capacity can be made by extrapolating the q - s relationship to a settlement of 25 mm. The q - s relationship is assumed to be linear and is, therefore, defined as:

$$s = C_m \bullet q \quad (5.4)$$

where q is the load per unit area (kN/m^2); s is the settlement of the plate (m); and C_m is the modulus of compressibility (m^3/kN). The estimated ultimate bearing capacities, based on Equation 5.4 and Figures 5.34 and 5.35, for sand reinforced with cut and full tires are 830 kN/m^2 and 460 kN/m^2 . The ultimate bearing capacity was reached for the cohesive backfill reinforced with cut tires and is approximately equal to 240 kN/m^2 (Figure 5.36).

In conventional reinforced soils, the improvement of the ultimate bearing capacity of the foundation soil is usually expressed in a non-dimensional form called the bearing capacity ratio (BCR). The bearing capacity ratio (BCR) is defined as:

$$\text{BCR} = q_{ult(R)} / q_{ult} \quad (5.5)$$

where $q_{ult(R)}$ is the ultimate bearing capacity with soil reinforcement, and q_{ult} is the ultimate bearing capacity without reinforcement. Plate loading tests were not performed on unreinforced sections of the prototype embankment. However, the bearing capacity of the unreinforced sand and cohesive backfill can be predicted by Terzaghi's theoretical expression. The ultimate

bearing capacity of a square footing placed on the surface of a soil can be estimated by the following relationship:

$$q_{ult} = 1.3cN_c + 0.4\gamma BN_\gamma \quad (5.6)$$

where γ is the unit weight of the soil; c is the undrained cohesion of the soil; B is the width of the plate; and N_c and N_γ are bearing capacity factors. The theoretical value of q_{ult} for the sand, based on Equation 5.6 and Terzaghi's bearing capacity factor for a friction angle of 40° ($N_\gamma = 100$) is approximately equal to 875 kPa. The friction angle for sand determined from laboratory shear box tests was 42 degrees. However, due to the variations in-situ density, a friction of 40 degrees was assumed to be more representative. Also, Terzaghi's bearing capacity coefficients are very sensitive to small variations in friction angle at high friction value. Due to the short time period in which the plate load test was performed, an undrained condition is assumed in the calculation of q_u for the cohesive backfill ($\phi_u = 0$, $N_c = 5.7$). The estimated q_{ult} is approx. 370 kPa. The modulus of compressibility based upon Equation 5.4 and the bearing capacity ratio predicted by Equation 5.5 for each section are given below.

Section and Reinforcement	Modulus of Compressibility (C_m) (m^3/kN)	Bearing Capacity Ratio (BCR)
Section A Cut tires in sand	3.0×10^{-5}	0.95
Section B Full tires in sand	5.4×10^{-5}	0.52
Section C Cut tires in cohesive backfill	1.0×10^{-4}	0.65

The calculated BCR were based on a mixture of theoretical and experimental results. Therefore, the BCR values should be considered qualitative in nature. The bearing capacity ratios for all sections were less than one. Therefore, the tire reinforcement used in this investigation decreased the bearing capacity of the soil. The tire reinforced soils did not fail in shear but rather in excessive displacement of the plate, even though the test loading plate was located 2 m from the slope. The stress imposed on the foundation was able to compress the soil surface beyond a distance of 1.2 m from the edge of the plate. The compression of void spaces within the tire reinforcement layer were responsible for reducing the bearing capacity by allowing excessive settlements to occur. As a result, full tire reinforcement gave the lowest BCR value while cut tires in sand gave the highest, almost unity. Also, tire reinforcements are highly extensible, and therefore, a considerable amount of settlement may be necessary before benefits are realized. Clearly, in tire reinforced fills, the settlement criteria is the predominant one. It is of interest to note that even in the case of whole tires in sand, the load test indicates that a settlement of only 12 mm had occurred at the maximum loading of 225 kPa (a loading equivalent of a ten-story high building). This settlement is half of the usual criteria of 25 mm for acceptable settlements in conventional structures. The results also indicate that should a more stringent settlement criteria become necessary, then recourse should be made to the use of cut tires. It should also be remembered that the plate loading tests were carried out on top of the berm, with the test plate located only 1.8 m from the edge of the slope. Placement of the loading plate so close to the slope must also have caused some lateral deformations which were not monitored. Hence the observed settlement data are likely to be conservative. The fact that no bulging of the slope due to plate loading test in the cohesive soil section occurred testifies to the effectiveness of this reinforcement in minimising lateral deformations.

The results of the plate load tests also indicated that tire reinforced wall can be more compressible than the compacted backfill, particularly, if the infilling material used in the tire reinforced zone is identical to that of the backfill. The resulting downward movement of the tire wall relative to the backfill will generate negative wall friction. Hence, the active thrust will act upward against the back of the tire retaining wall. The direction of the active thrust is critical for

stability analysis. The modulus of compressibility can be useful in estimating the settlement of the tire retaining wall relative to the backfill.

5.6 Lateral Stress Distribution From Earth Pressure Cells

The lateral stress distribution behind each retaining wall was directly measured by the pressure cells. The pneumatic pressure cells used in this study were calibrated in a pressurized air chamber. However, when a pressure cell is buried in a soil, it has different deformation properties from the surrounding soil, as a result, the stress pattern in the vicinity of the cell is modified. Small deflections of the flexible diaphragm will modify the stress distribution across the face of the cell, and hence, the stress measured. Consequently, a pressure cell calibrated in a pressurized air chamber will not register the same stress as when embedded in a soil. An accurate assessment of soil stresses can only be made by first calibrating the pressure cell in an air chamber and then in the soil of interest subjected to known stresses. This calibration procedure gives rise to an “action factor” for the cell. Action factors will vary according to soil type, porosity, moisture content, particle size distribution, particle shape, compressibility, stress ratio and stress history apart from the characteristics of the cell itself (Dunn and Billam 1966).

Action factors for the pressure cells installed at the back of each tire retaining wall were not determined. Therefore, the measured stresses used to calculate the lateral earth coefficients should be considered an approximation of the true state of stress in the backfill.

An estimate of the lateral earth pressure coefficients behind tire walls constructed with the sand backfill (section A and B) are shown in Figures 5.37 and 5.38, respectively. Lateral earth pressure coefficients were also evaluated for the tire wall constructed with the cohesive backfill (Figure 5.39). Field results are compared with both Rankine and Coulomb theoretical active earth pressure coefficients. Rankine determined the state of stress for a cohesionless soil mass adjacent to a wall with a frictionless back-face. However, it has become evident that wall friction can significantly affect the earth pressures acting on the wall (Quigley and Duncan 1978). Coulomb’s theory and its subsequent development by others is able to consider such factors as wall friction,

a sloping backfill, and inclination of the wall. Results from the plate load tests and settlement measurements (Section 5.7) indicate that the tire retaining walls are more compressible relative to the backfill, resulting in the development of a negative wall friction. Consequently, the lateral active pressure coefficients, using Coulomb's theory, were determined for a level backfill, a wall inclination of 80° and a negative wall friction equaling the backfill friction angle ($\delta = -\phi'$). Lateral earth pressure coefficients based on Rankine's theory which does not consider negative wall friction are unconservative, especially at large negative values of wall friction (Figures 5.37 to 5.39).

During construction, high horizontal soil stresses were developed behind each tire retaining wall. The high pressures were attributed to the compaction process. Broms (1961) summarized that the effect of compaction behind unyielding walls was to increase the horizontal pressures, above K_0 values within backfills behind low walls or near the surface of backfills behind high walls. K_0 values for sand and cohesive backfills, based on the theoretical relationship $1 - \sin \phi'$, are 0.33 and 0.47, respectively. The state of stress behind each wall was well above the K_0 condition. However, compaction of a soil can greatly influence the measured response of a pressure cell, especially in sands (Dunn and Billam 1966). Regardless, compaction behind tire retaining walls should be carefully carried out in order to limit the development of high lateral stresses. (Wall failure by compaction)

Retaining walls constructed with tire mat reinforcements which are filled with soil and placed in successive layers produces a flexible structure. Tire retaining walls, unlike conventional concrete gravity retaining walls, are able to yield substantially as to fully induce an active state of stress within the backfill adjacent to the wall. The reduction in lateral earth pressure registered in the lower cells, during construction, is attributed to the progressive yielding of the wall. After the final 2 m surcharge was placed, outward movement of the walls resulted in a substantial reduction in the horizontal soil stresses which indicates that sufficient wall yielding had occurred to fully diminished compaction induced stresses. Lateral earth pressure coefficients for all three walls were reduced to values between Rankine and Coulomb theoretical coefficients. The

increase in horizontal stress with time, after the placement of surcharge, resulted from the development of negative wall friction. This gradual increase in the earth pressure coincided with the continuing vertical deformation of each tire wall (Figures 5.43 to 5.45). Regrettably, the two deeper earth pressure cells, located in Sections A and B failed after one year of operation. The progressive development of negative wall friction has significant impact on stability of tire retaining wall, and must be addressed in design. Tschebotarioff (1973) reported the bulging of the upper portion of a 10.2 m high, double-cell crib wall. The wall was constructed with precast-concrete headers and stretchers filled with loose granular material. The sloping backfill (12°) behind the wall was compacted. He surmised that the observed distress probably resulted from an increase in active pressure on the back of the wall caused by downward settlement of the wall with respect to the backfill and a corresponding change in wall friction from a positive to a negative value. This peculiar relative movement between wall and backfill was attributed to an overload of the foundation below the heel of the wall.

5.7 Settlement

The overall settlement of each section, measured from the settlement gauges, are presented in Figures 5.40 to 5.42. The total settlement of sections A and B, tire reinforcements in sand, was approximately 25 mm. There was no substantial difference between the unreinforced and reinforced sand in each of these sections of the prototype embankment which suggests that the unreinforced sand was well confined.

The total settlement measured in section C, cut tires in a cohesive backfill, was greater than 200 mm. This large settlement arose from the difficulty in assuring a good compaction of the tire-soil layer (as discussed earlier, not all void spaces were eliminated) and the high plasticity of the backfill, especially since it was wet. The initial settlement of 150 mm, after just 76 days, originated from the compression of the void space as a result of on-going creep deformation of the structure, while consolidation of the fill is responsible for the remaining amount of settlement. The amount of settlement substantially decreased after approximately one year after the completion of the prototype embankment. Consequently, proper compaction during

construction will minimize long term settlements. If the amount of settlement is in question, and the use of cohesive backfill is unavoidable, then other construction techniques could be used to reduce the potential settlement problems such as preloading the structure, and construction in stages. This is similar to measures adopted with conventional cohesive backfills. However the results strongly indicate that only cut tires should be used with cohesive backfill at this stage.

The settlement of the ground surface of the prototype embankment, i.e. on top of the permanent surcharge, for each section, measured by surface monuments, are given in Figures 5.43 to 5.45. The least amount of surface settlement was measured in section A, while the greatest was in section C. For sections A and B, the difference between the internal measured settlements (based on settlement gauges) of the reinforced section and surface settlement from measurements of the surface monument included the settlement of underlying foundation and surcharge layer. The foundation layer comprising of boulders, gravels and sand was thickest underneath section C and may explain the similarity between internal settlement gauges and surface monument measurements. The voids within the tire reinforcement generated during infilling (section B: full tires in sand and section C: cut tire in cohesive backfill) were compressed with time, as the overburden pressure increased with embankment height and surcharge. The compression of voids within the tire retaining wall would generate vertical deformations. Also, the development of a potential failure plane in the poorly compacted surcharge backfill layer produced slumping of the ground surface as the retaining wall deflected outward. Hence, the surface monument in each section located near the retaining wall reported the greatest vertical movement. The reinforced slope constructed with full tires in sand settled, in general, approximately 30% more than the same slope constructed with cut tire reinforcement.

5.8 Lateral Displacement

The magnitude of lateral displacement of an earth structures constructed with tire reinforcements depends on several factors such as: compaction effort, extensibility of the tire-soil reinforcement including creep characteristics, horizontal and vertical spacing, reinforcement length, strength

and deformation characteristics of both the backfill and the foundation soil, surcharge intensity, type and location of any external loading, and construction procedures.

5.8.1 Reinforced Slopes

Lateral displacements for the reinforced slope for each section are given in Figures 5.46 to 5.48, respectively. The original slope geometry is outlined as a dashed line in which the horizontal scale is reduced by a factor of 10 (horizontal scale 1:10). The lateral displacements shown in these figures represent the actual measured horizontal movement (horizontal scale 1:1). The measured horizontal displacements between the two inclinometers, in each of the three sections, were fairly consistent. The lateral displacement in the reinforced slope arose from deformation within the reinforced zone and movement within the unreinforced drainage blanket foundation. The minimum factor of safety for the internal stability of the reinforced slope was 1.4 for the cohesive section (Table 5.1) which included a serviceability reduction. To limit excessive horizontal deformation, the pull-out capacity was limited to 5% strain. The maximum lateral movement in each section after 745 days after surcharge was: section A: 89 mm, section B: 106 mm, and section C: 70 mm. Most of the horizontal displacement occurred within approximately one year after the completion of the prototype embankment. These deformations would fall within an acceptable range for most tire reinforcement applications. However, limiting strains to 5% may require that the reinforced slopes be constructed with large number of layers of tire mat reinforcements.

5.8.2 Retaining Walls

The measured lateral displacement of the three tire walls are shown in Figures 5.49 to 5.51. In Figures 5.49 and 5.50, the initial design of the retaining wall is outlined with a dashed line in which the horizontal scale is reduced by a factor of 5 (horizontal scale 1:5), and for Figure 5.51, the horizontal scale is reduced by a factor of 3 (horizontal scale 1:3). The lateral displacements shown in these figures represent the actual measured horizontal movements (horizontal scale 1:1). The horizontal displacement relative to the base were minor below the self-stable slope

which is represent by the interface shear strength behaviour between successive layers of tire mat reinforcements. The slope of this line is equal to the internal friction angle of the backfill, ϕ' , times the direct sliding efficiency coefficient α_{ds} , ($\alpha_{ds} * \phi'$). Results indicate that most of the horizontal displacements occurred above the self-stable slope. The lateral displacement in the three tire retaining walls arose from deformation within the tire reinforcement, deformation of the unreinforced zone behind the wall, and movement due to construction. The settlement of the foundation would also affect the horizontal displacement of the tire retaining wall.

The various displacement ratios (defined as the maximum displacement, Δ , divided by the wall height, H , or the height at which the inclinometer tube exits the wall face) for each retaining wall is given are Table 5.2. The wall constructed with cut tires in sand demonstrated the highest stiffness ratio (greatest resistance to horizontal movement), while the wall using the cohesive backfill showed the lowest stiffness ratio (least amount of resistance to horizontal movement). The post construction monitoring has shown a continuous horizontal movement with time but with decreasing rate after a period of approximately one year.

The tire retaining wall in section C, constructed with cut tires in a cohesive backfill, reported the largest lateral displacement (395 mm after 745 days) indicating its high level of flexibility. An inward movement of this tire wall into the backfill below the slope line representing the interface angle is observed (Figure 5.51). This observation may indicate that the reinforced wall section is experiencing a Poisson's ratio effect and is squeezing laterally. Consequently, the bottom of the wall at the face is moving out as well as the heel of the reinforced zone into the backfill.

The least lateral movement in the retaining wall was monitored in Section A with cut tires filled with sand. The recorded movement after placing the surcharge was approximately 80 mm. The advantage of using cut tires is clearly evident. The deformations in the sand filled sections, especially using cut tires, is similar in magnitude to that experienced with geosynthetic reinforced walls (Christopher et al. 1990). The post-construction deformations in the cohesive soil section are large (11% post construction). These latter measurements indicates that a more careful control of placement, water content, and the degree of compaction are necessary with clayey

soils. Also, as discussed below and in the following Chapter, a larger extent of reinforcement may be required to reduce lateral deformations.

Horizontal Deformation due to Construction

Horizontal deformations occurring during construction were primarily due to compaction of the overlying layers. The tire retaining wall constructed using cut tires and sand reported a normalized horizontal deformation ($\Delta/H, \%$) of about 1.2%, while for full tires, it was approximately 1.9%. The degree of infilling of full tires with sand was slightly less than for cut tire which resulted in a retaining wall of lesser density (a reduced unit weight). As a result, the retaining wall constructed with cut tire was able to provide a greater resistance to lateral forces induced by the compaction effort. Also, as the confining pressure increased with increasing height of the wall, the cut tire reinforcement was able to generate greater interlocking forces between successive tire reinforcing layers. This was primarily the result of a thin and well compacted interface layer which limited deformations and transferred stresses to the stiffer tire reinforcement.

The tire wall constructed using cut tires and the cohesive soil showed the largest lateral movement of about 4.7%. This represents, on average, a 3 fold increase in the measured horizontal deformation due to construction when compared to the other two walls. It was difficult to ensure proper infilling of the cut tires with the cohesive backfill even after compaction, since voids were still visible, as shown in Figure 5.24. The presence of voids would reduce the overall stiffness of the tire reinforcement (greater deformability). Also, it was found to be impractical to level off the cohesive soil surface with the top of tire reinforcement. This produced an uneven interface layer of various thickness between the different tire reinforcing layers. As a result, the interlocking strength or bonding resistance between the tire and soil was reduced, especially under undrained conditions. These conditions were prevalent during the construction stage. Therefore, some slippage between successive tire mat reinforcements has likely occurred. Since the tire wall was constructed in successive lifts, the outward movement developed in the lower reinforcing layers, over which the subsequent tire mat reinforcement was aligned, resulted in a moving datum. Cumulative adjustments had to be made at each incremental

level which increased the outward movement as the wall height increased. This problem was most apparent in the cohesive backfill. Also, the reduction in interface strength coupled with a highly plastic backfill produced a tire retaining wall of greater deformability.

It is difficult to predict the magnitude of this type of construction movement. If the amount of lateral deformations are an important criteria (a limited amount of horizontal movement is permitted), it would be appropriate to provide a pre-batter to the retaining wall or to brace the tire wall face using simple props. However, the amount of horizontal movement attributed to construction accounted for approximately 30% of the total measured displacement, after 745 days of monitoring.

Horizontal Deformation in the Reinforced Tire Wall

The inclinometer located behind the retaining wall was installed after construction but before the placement of the surcharge load. Hence, the measured displacement does not include deformation related to compaction. Therefore, the difference in the measured horizontal displacement between the inclinometer located within the tire wall (1.2 m behind the tire wall face), minus construction movement, and the one located behind the tire wall (unreinforced zone), essentially indicates post construction lateral deformation that the reinforced tire wall has undergone (Table 5.2). For the retaining wall in section A, cut tires in sand, showed movement within the wall after 745 days (normalized lateral deformation, $\Delta/H = 2.3\%$) which were less than those measured within the unreinforced zone by an inclinometer located behind the wall ($\Delta/H = 2.7\%$). As stated earlier, cut tires were able to provided strong interlocking resistance when using a good quality backfill. This structure behaved as an integral mass, and therefore, little or no deformations occurred within the reinforced tire wall.

The measured normalized horizontal deformation in the reinforced zone B using full tires and sand after 745 days (excluding displacements associated with construction) was approximately 1.1% (Table 5.2). Since the amount of wall face movement was not measured, the inclinometer located within the retaining wall was assumed to represent the maximum displacement of the structure. Deformation within the retaining wall represented 20% of the total measured

displacement. The weaker interlocking resistance generated at the interface (since it is difficult for full tires to interlock with each other), and the presence of voids within the tire reinforcing mat reduced the relative stiffness of the reinforcement and produced higher strains. Also, the voids within the reinforcement during the construction stage were compressed with time, as the overburden pressure increased with wall height and surcharge. This settlement would cause additional movements to occur within the tire retaining wall.

The normalized horizontal deformation within the reinforced zone of the tire wall constructed with cut tires and a low quality backfill (section C) was 5.6% (excluding construction movement). This displacement represents 35% of the total amount of movement after 745 days of monitoring. The large displacements are the results of several factors: lower shear strength of the fill (i.e. smaller friction angle ϕ) which resulted in greater forces being generated within the reinforcement to maintain equilibrium and hence larger deformations; an uneven and thicker soil interface of greater plasticity and of lower strength; difficulties in assuring complete infilling of tire during construction which resulted in a more flexible reinforcement; and an increased outward deflection due to greater vertical compression of the retaining wall. Field observations indicate that proper infilling of tires and a more careful control on water content, layer thickness and the degree of compaction are necessary when using a low quality backfill.

Horizontal Deformation behind the Tire Wall (Unreinforced Zone)

In all three walls, there existed a portion of unreinforced retained fill near the top of the wall located above the interface slope angle, indicated by an inclination $\alpha_{ds}\phi'$ to the horizontal, and extending beyond the reinforced tire wall (see Figures 5.49 to 5.51). The measured horizontal displacements from inclinometers installed behind both retaining walls constructed using the sand backfill reported similar results ($\Delta/H = 2.7\%$, for section A; $\Delta/H = 2.5\%$, for section B). Movement of the unreinforced backfill (sand) behind the cut tire wall (section A) accounted for 77% of the total amount of the observed movement, while for the full tire retaining wall, it accounted for only 45%. For the tire retaining wall in section C, deformation of the unreinforced backfill accounts for 35% of the observed lateral movement, representing a normalized horizontal deformation of 5.5%. This represents a two-fold increase in the measured horizontal

deformation in the unreinforced zone (movement behind the tire wall) when compared to the other two walls. The lower shear strength and greater deformability of the cohesive backfill resulted in an increase in horizontal deformation of the reinforced tire wall. The greater movements are related to a combined effect of an increase of the internal deformation within the retaining wall mass and the enlargement of the unreinforced zone which results in the development of larger strains behind the retaining wall.

Many factors govern the horizontal deformation of tire retaining walls. They are the shear strength and deformability characteristics of the backfill, reinforcement stiffness and density (which are strongly influenced by voids), interface characteristics of the tire reinforcement and the backfill, the extent of the reinforcement, external loading conditions and possible creep behaviour. A parametric study was therefore undertaken to determine the effects of reinforcement length, reinforcement layout, shear strength and deformability of backfill used, and external loadings on the deformation of tire reinforced walls. The results of the study are presented in the following Chapter.

Table 5.1 Estimated factors of safety for the different types of structures used in the test embankment.

Type Of Structure	Factors Of Safety		
	Sliding [1.5]	Rotation [2.0]	Slope Stability [1.3]
<u>Retaining Wall</u>			
Cut Tires In Sand using: Rankine (K_a)	3.0	3.6	1.8
Trial-Wedge Method (K_a)	1.7	2	
Full Tires In Sand using: Rankine (K_a)	2.9	3.5	1.7
Trial-Wedge Method (K_a)	1.6	1.9	
Cut Tires In Cohesive Backfill During Construction (Undrained Conditions)	1.9 ^A	1.4 ^A	3.5
Cut Tires In Cohesive Backfill Long Term (Drained Conditions) using: Rankine (K_a)	1.9	2.0	1.4
Trial-Wedge Method (K_a)	1.1	1.2	
<u>Reinforced Slope *</u>			
Cut Tires In Sand	>> 1.5	----	2.2
Full Tires In Sand	>> 1.5	----	2.0
Cut Tires In Cohesive Backfill During Construction (Undrained Conditions)	>> 1.5	----	2.5
Cut Tires In Cohesive Backfill Long Term (Drained Conditions)	>> 1.5	----	1.7

[] Indicates the minimum recommended value

* Slope stability analysis reflects the internal stability of the structure. The reported F.S. value include a serviceability reduction.

A = Lateral earth pressure distribution was based upon actual measured values obtained from pressure cells located behind the wall, just after the placement of the surcharge load, see section 5.6.

Table 5.2. The different stiffness ratios or normalised lateral deformation for each section.

Section and Reinforcement	Inclinometer located at 1.2 m behind the tire wall face			Inclinometer located behind the tire wall*
	After Construction ($\Delta / H, \%$)	Surcharge + 745 Days - Construction Movement ($\Delta / H, \%$)	Surcharge + 745 Days ($\Delta / H, \%$)	
Section A Cut tires in Sand	(a) 1.2	(b) 2.3	(c) 3.5	(d) 2.7
Section B Full tires in Sand	1.9	3.6	5.5	2.5
Section C Cut tires in Cohesive Backfill	4.7	11.1	15.8	5.5

* The inclinometer located behind the wall was installed after construction but before the placement of the surcharge load. Hence, the measured displacement do not included movements related to construction activities.

Note: displacements occurring within the tire walls are defined by (b) - (d): Section A = -0.4%

Section B = 1.1%

Section C = 5.6%

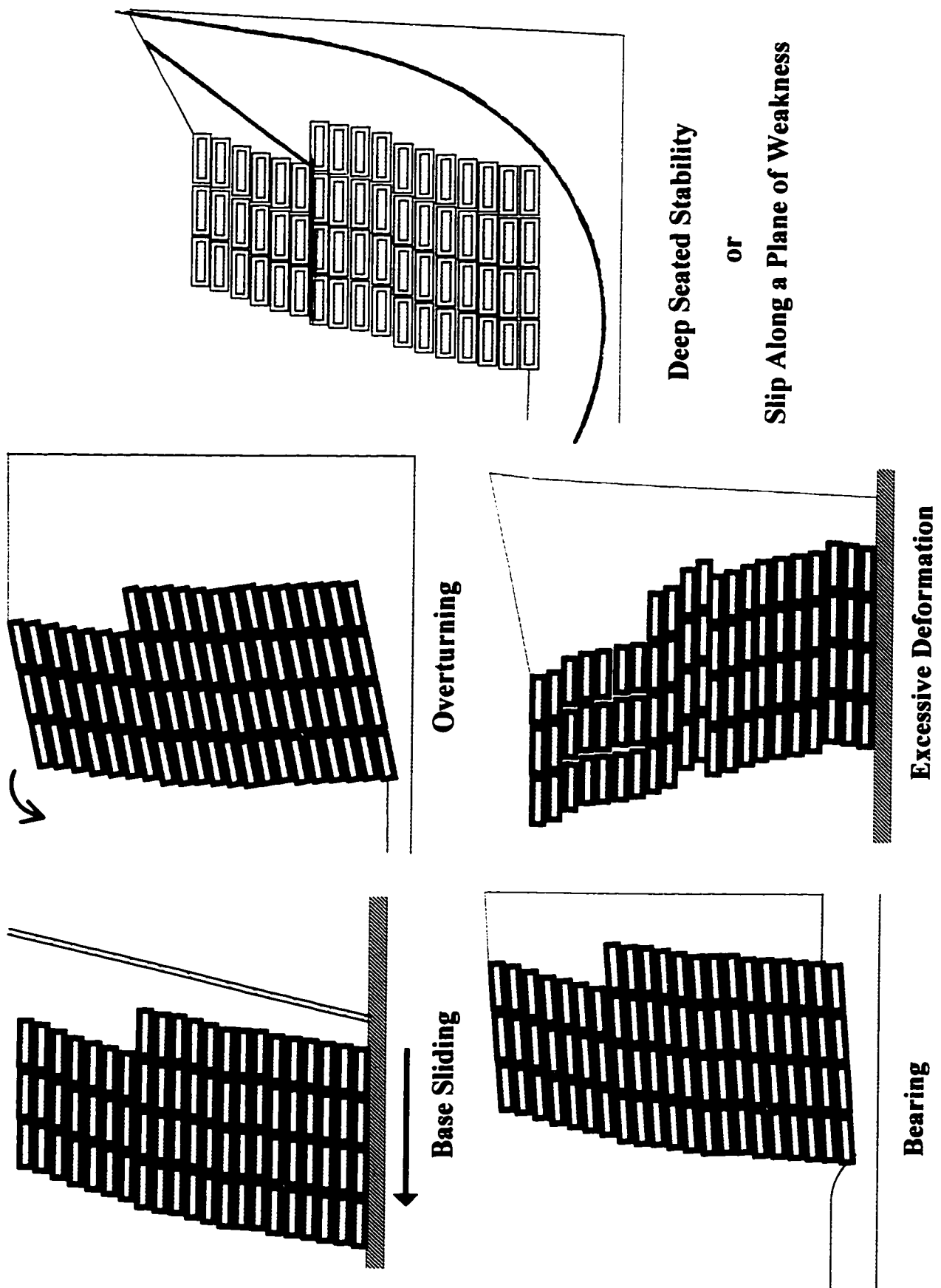


Figure 5.1 Potential failure mechanisms for a retaining wall constructed with tire reinforcement.

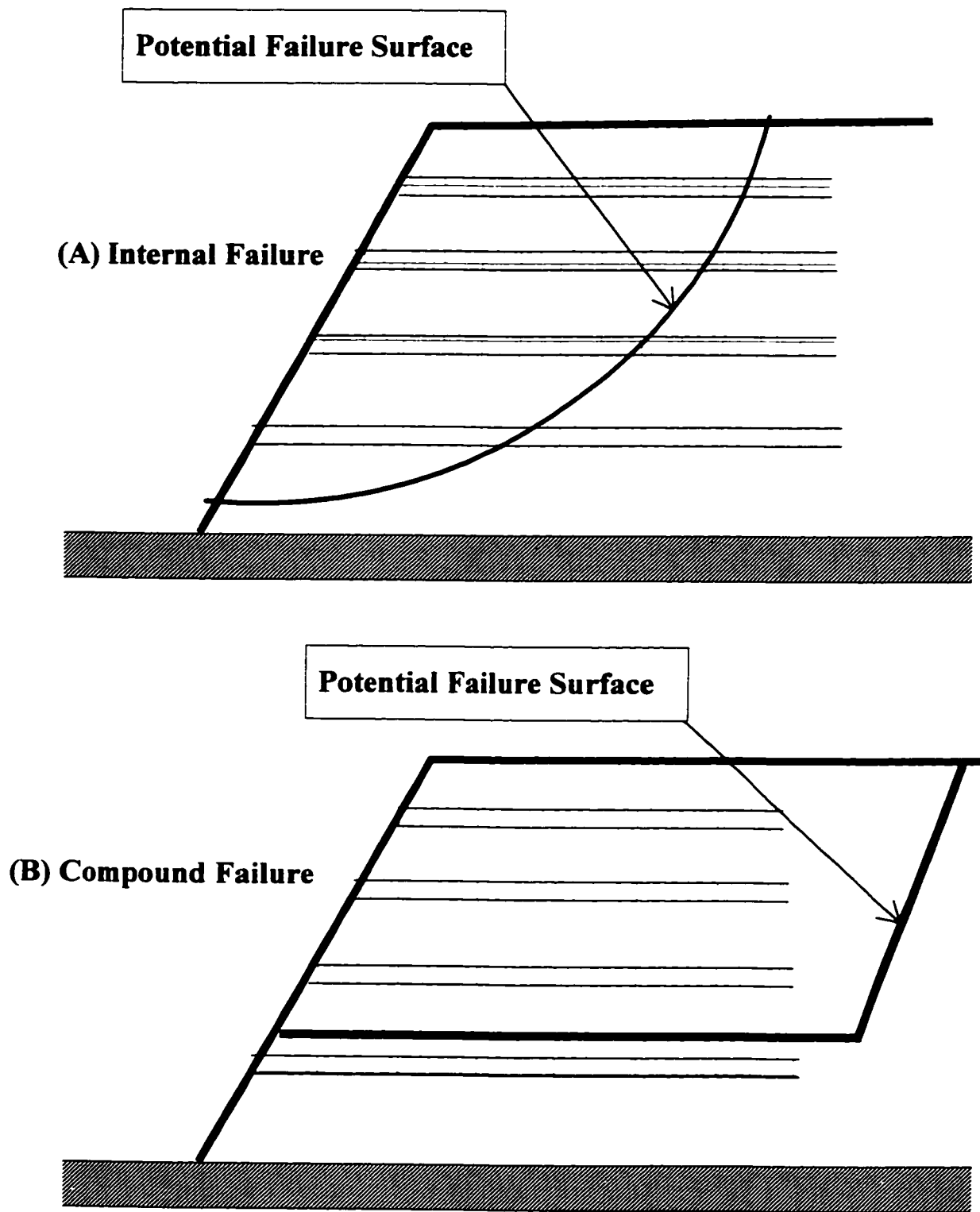
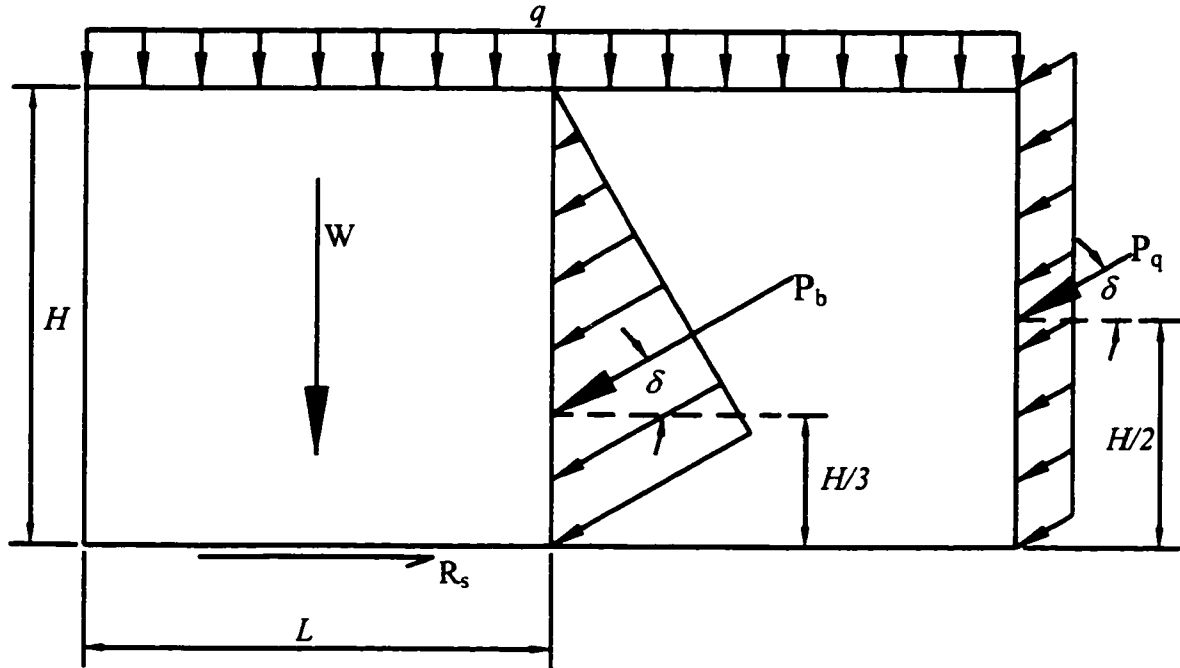


Figure 5.2. Potential failure mechanisms for reinforced slopes (CFEM 3rd Edition).



$$W = \gamma_r HL$$

$$P_b = \frac{1}{2} K_a \gamma_b H^2$$

$$P_q = K_a q H$$

$$R_s = \text{Resisting Force Minimum of } \tan \phi'_b, \tan \phi'_f, \tan \phi'_{re}$$

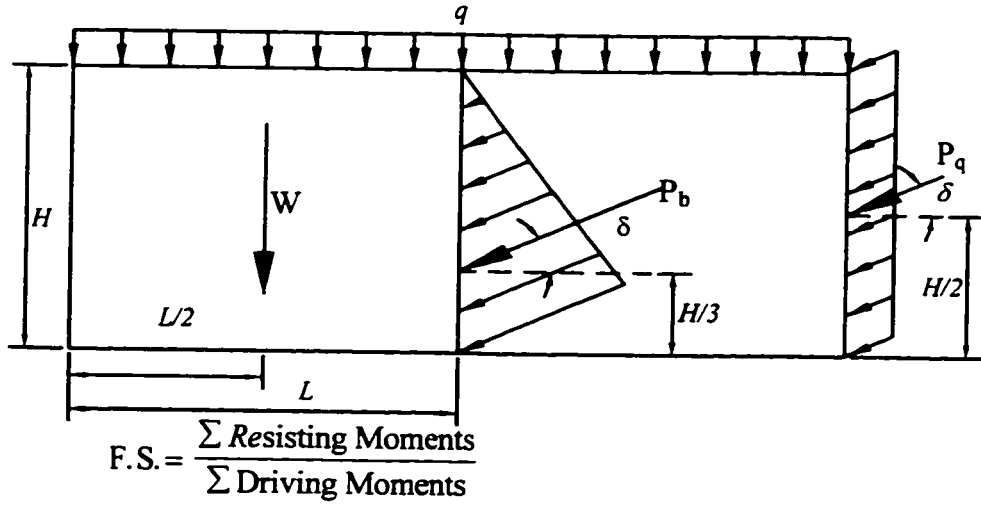
$$F.S. = \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}}$$

$$F.S. = \frac{(W + qL + P_b \sin \delta + P_q \sin \delta) \alpha_{ds} \tan \phi_{(b,f,re)}}{(P_b \cos \delta + P_q \cos \delta)}$$

$$F.S. \geq 1.5 \text{ (Elias et al. 1996)}$$

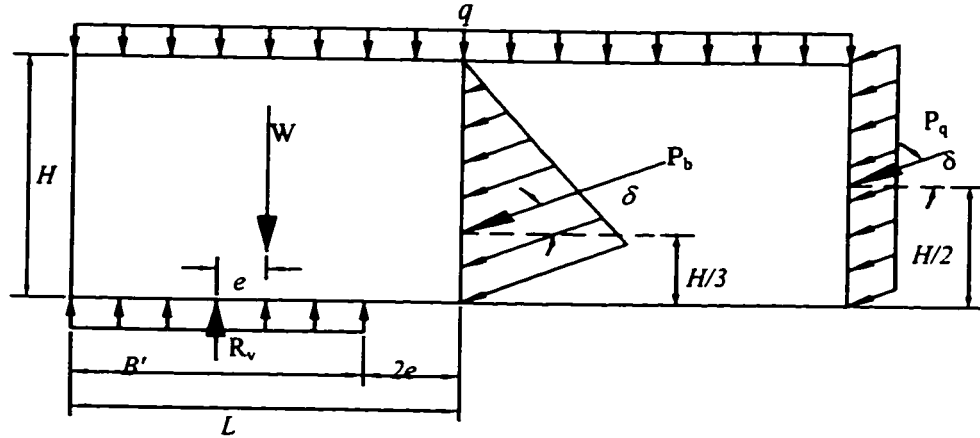
Figure 5.3 - Calculation of the factor of safety against base sliding.

Note: The above general case is valid where the retaining wall is less compressible than the backfill. In case where the retaining wall is more compressible, P_b will act upwards.



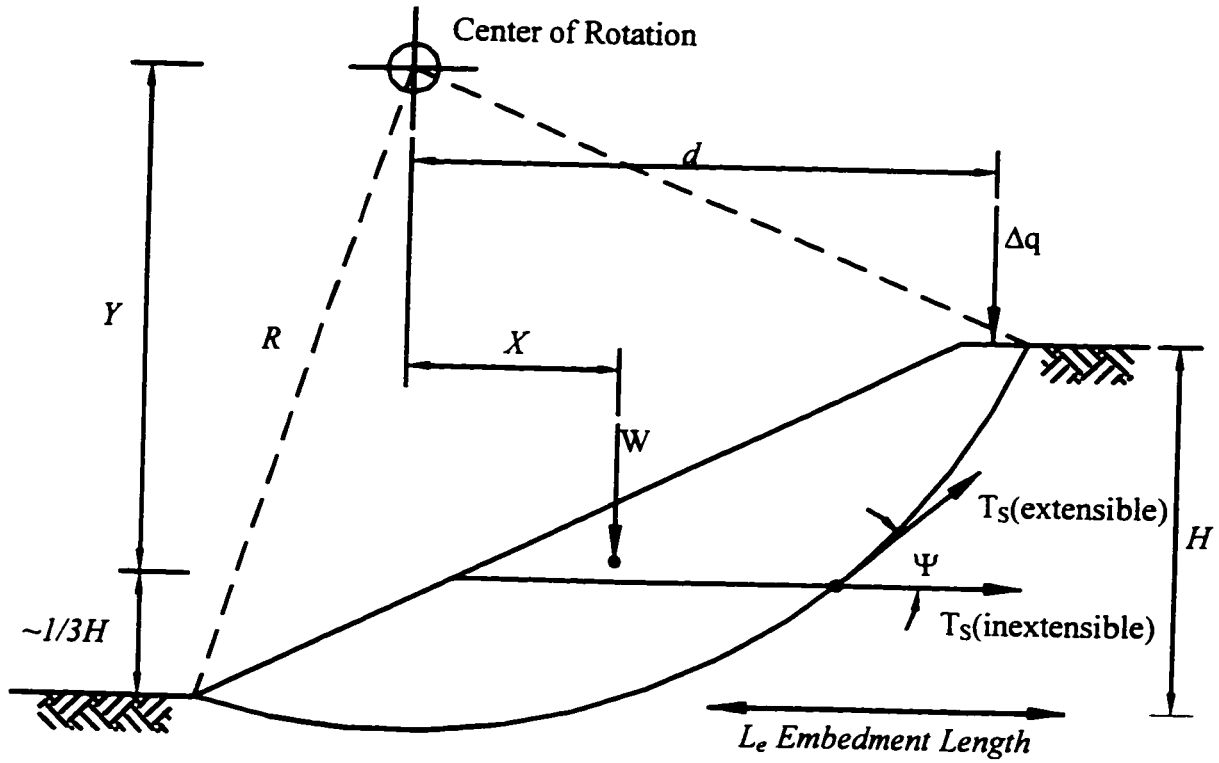
$$F.S. = \frac{(W + qL)(L/2) + (P_b \sin \delta + P_q \sin \delta)L}{(P_b \cos \delta)(H/3) + (P_q \cos \delta)(H/2)} \geq 2.0 \quad (\text{Elias et al. 1996})$$

Figure 5.4a - Calculation of the factor of safety against overturning.



$$F.S. = \frac{cN_c(\phi') + 1/2 \gamma B' N_\gamma(\phi')}{\sigma_v} \geq 2.0 \quad (\text{Elias et al. 1996})$$

Figure 5.4b - Calculation of the factor of safety against bearing capacity.



Factor of Safety of Unreinforced Slope:

$$F.S._u = \frac{\text{Resisting Moments } (M_R)}{\text{Driving Moments } (M_D)} = \frac{\int_0^{L_{SP}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)}$$

where: W = weight of sliding earth mass

L_{SP} = length of slip plane

Δq = surcharge

τ_f = shear strength of soil

Factor of Safety of Reinforced Slope:

$$F.S._r = F.S._u + \frac{T_s \cdot D}{M_D}$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers

D = moment arm of T_s about center of rotation

= R for extensible reinforcement

= Y for inextensible reinforcement

Figure 5.5. The rotational shear approach to determine the required strength of reinforcement (Elias et al. 1996).

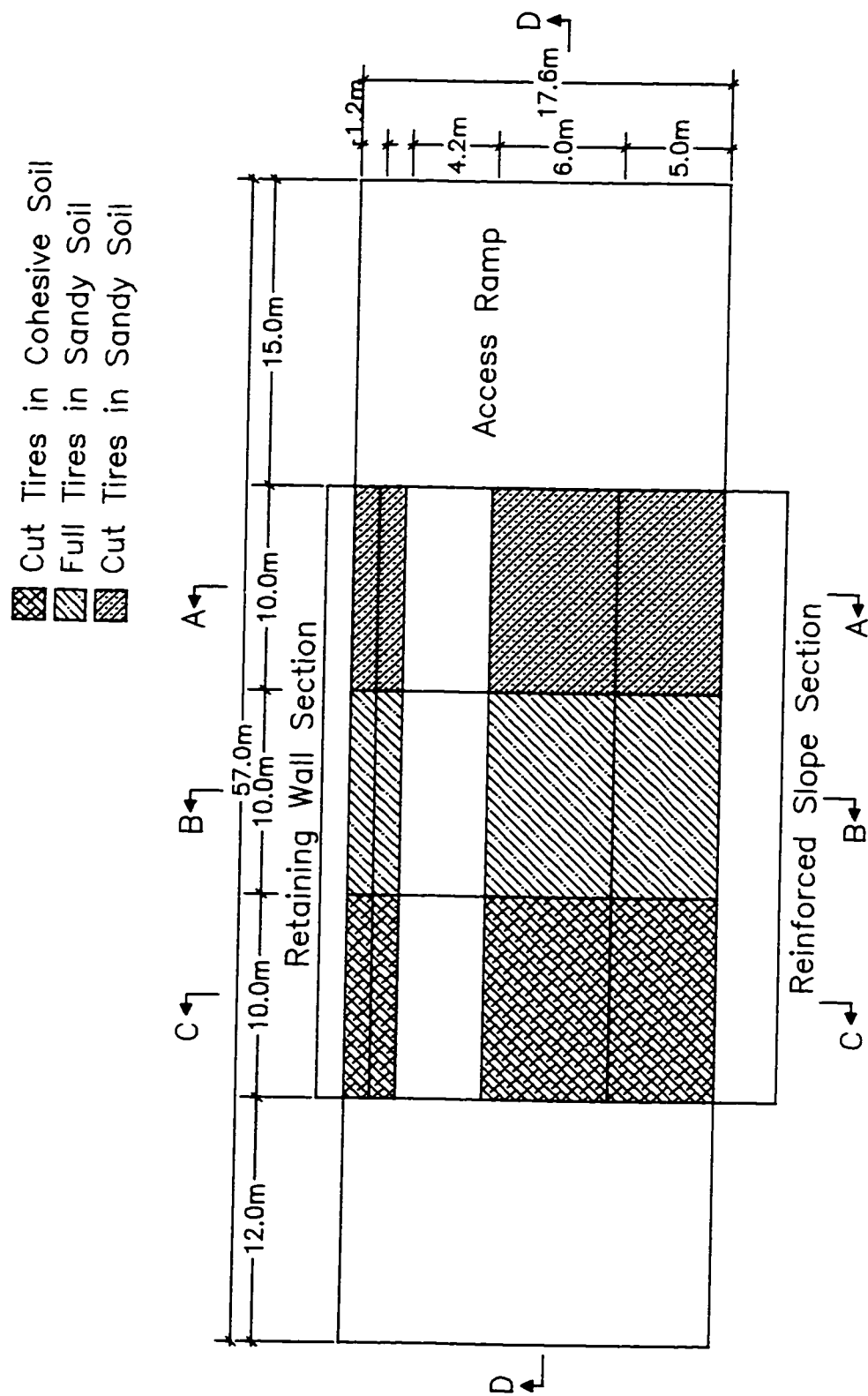


Figure 5.6. Plan view of the test embankment, showing the geometry, tire mat layout and soil type.

Cross-Section A-A

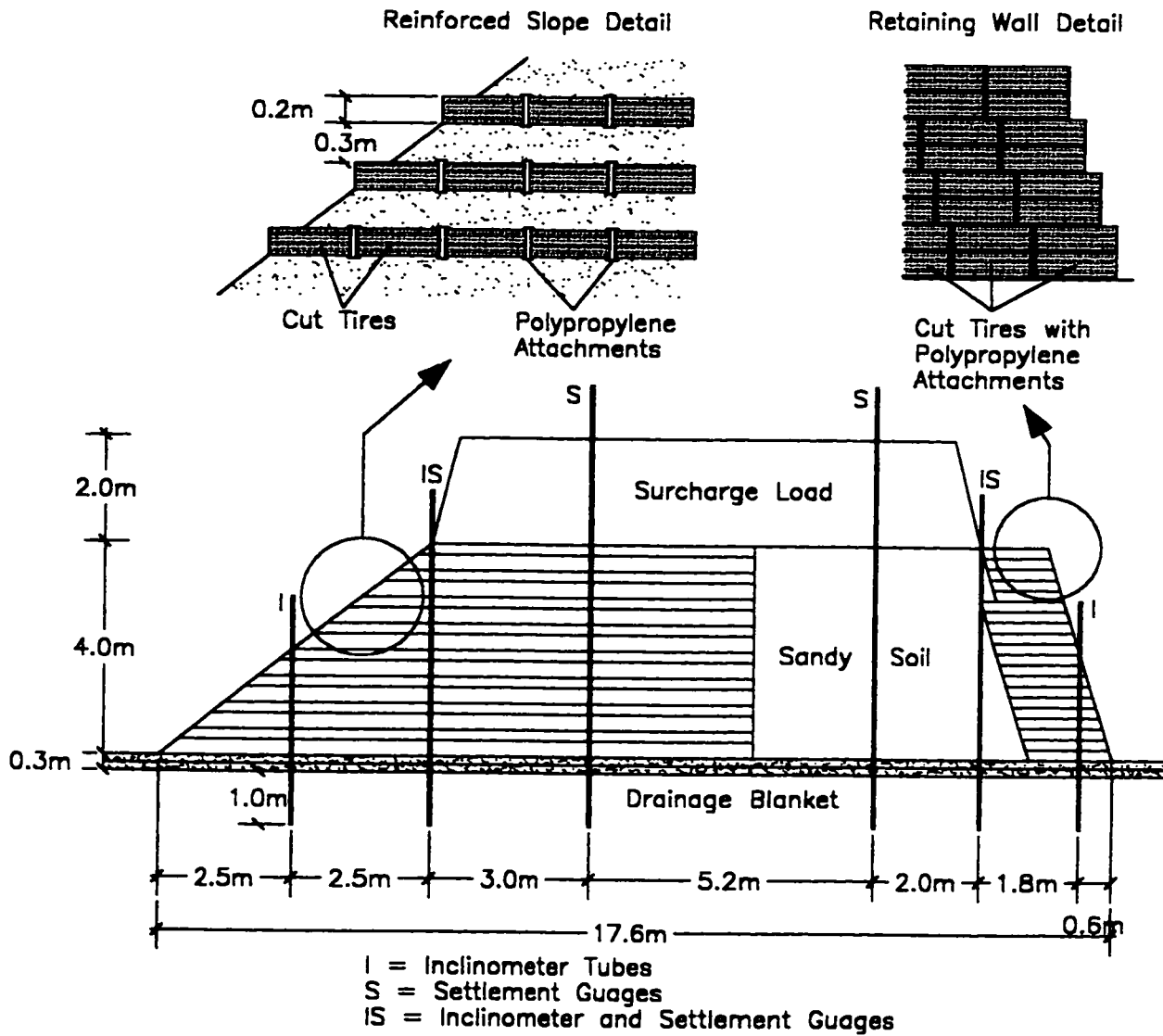


Figure 5.7. View of cross-section A.

Cross-Section B-B

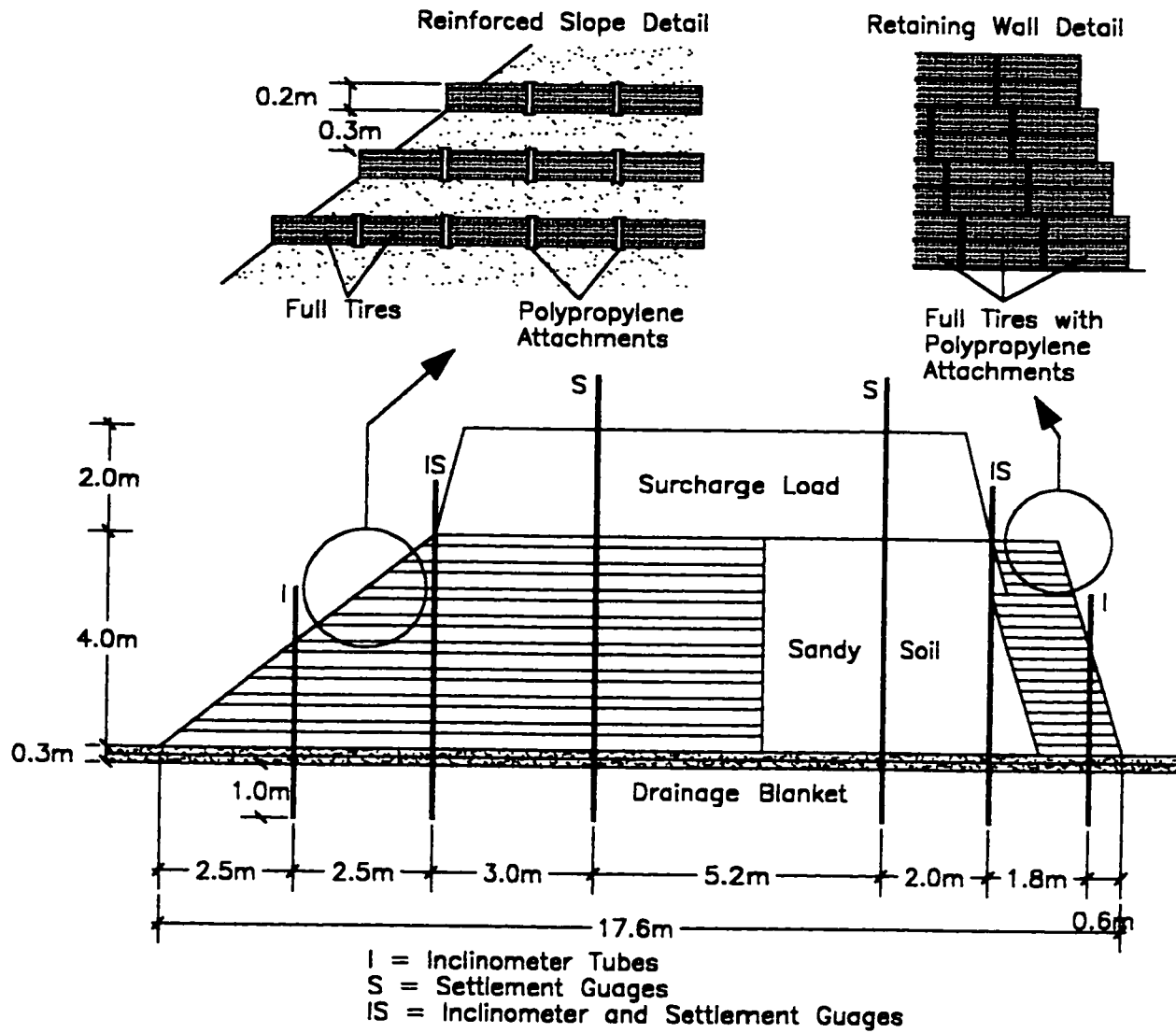


Figure 5.8. View of cross-section B.

Cross-Section C-C

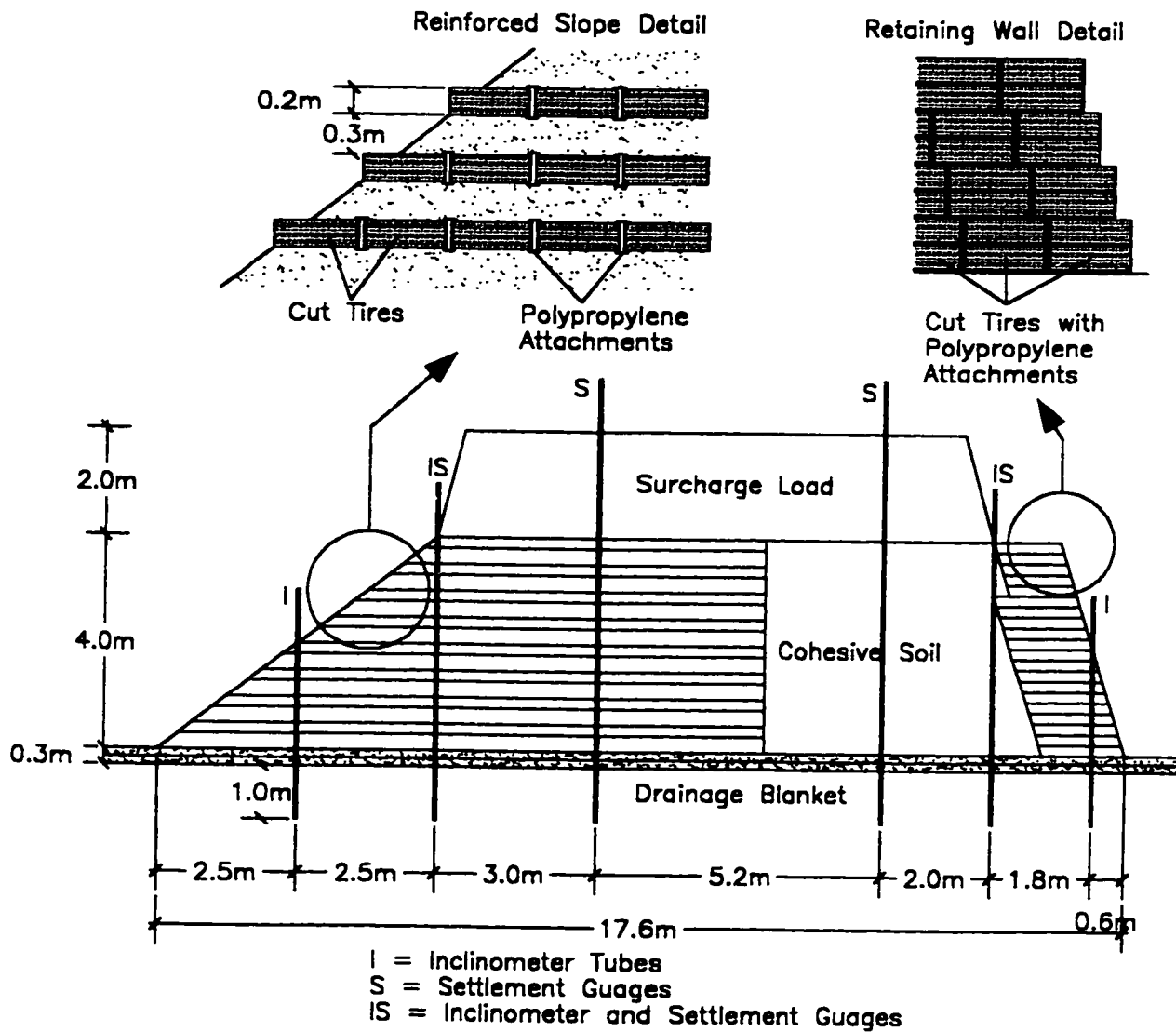


Figure 5.9. View of cross-section C.

Longitudinal Section D - D

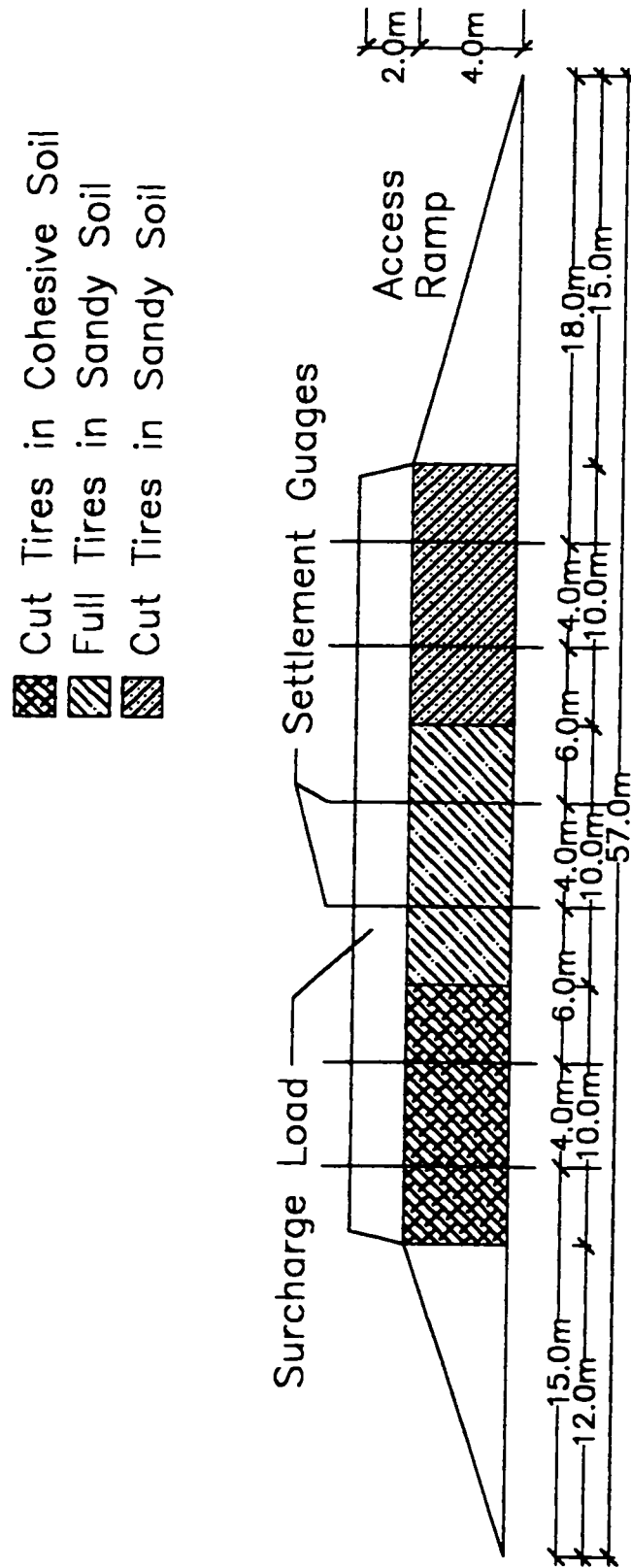


Figure 5.10 Longitudinal section DD of entire embankment.

- Inclinator Tubes
- ▲ Inclinator and Settlement Gauges
- ◆ Settlement Gauges

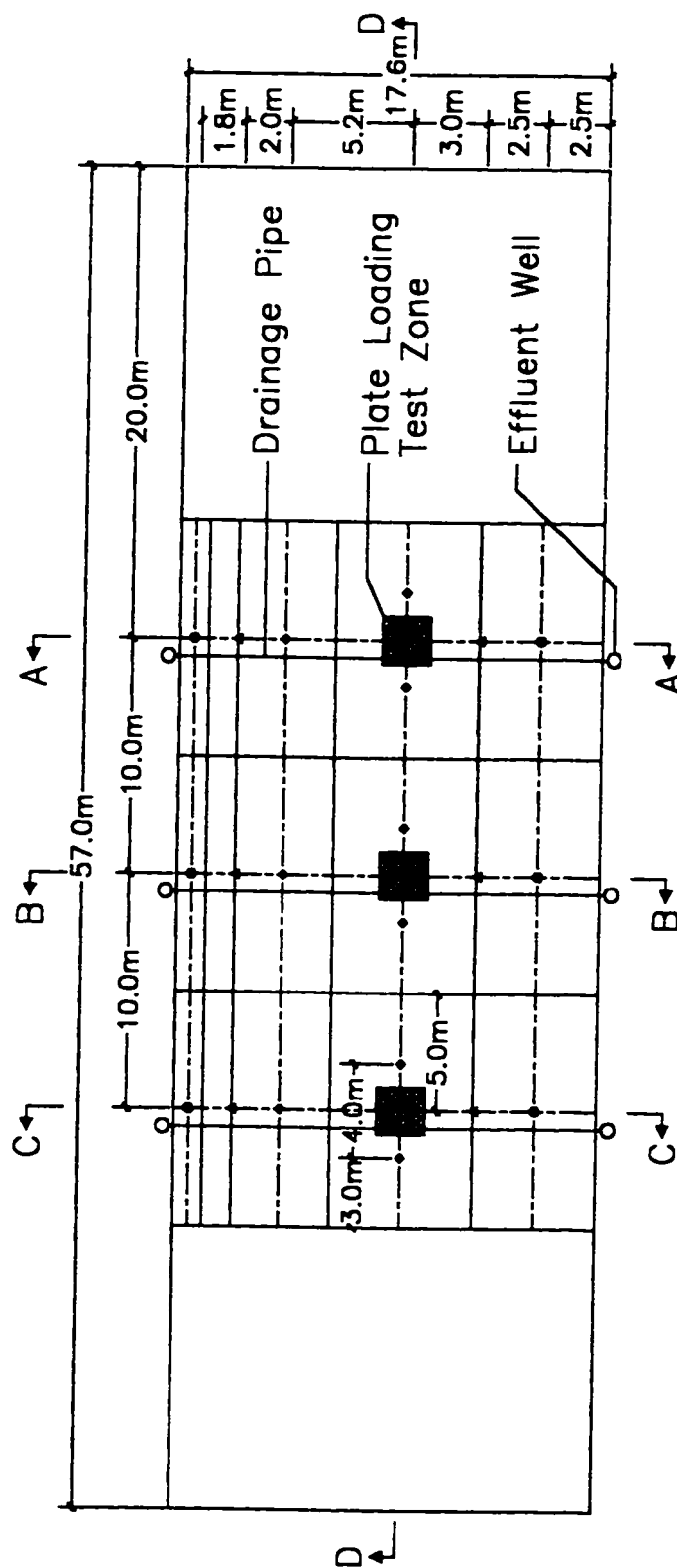


Figure 5.1.1 Plan view of instrumentation used in the test embankment and location of the plate loading test.

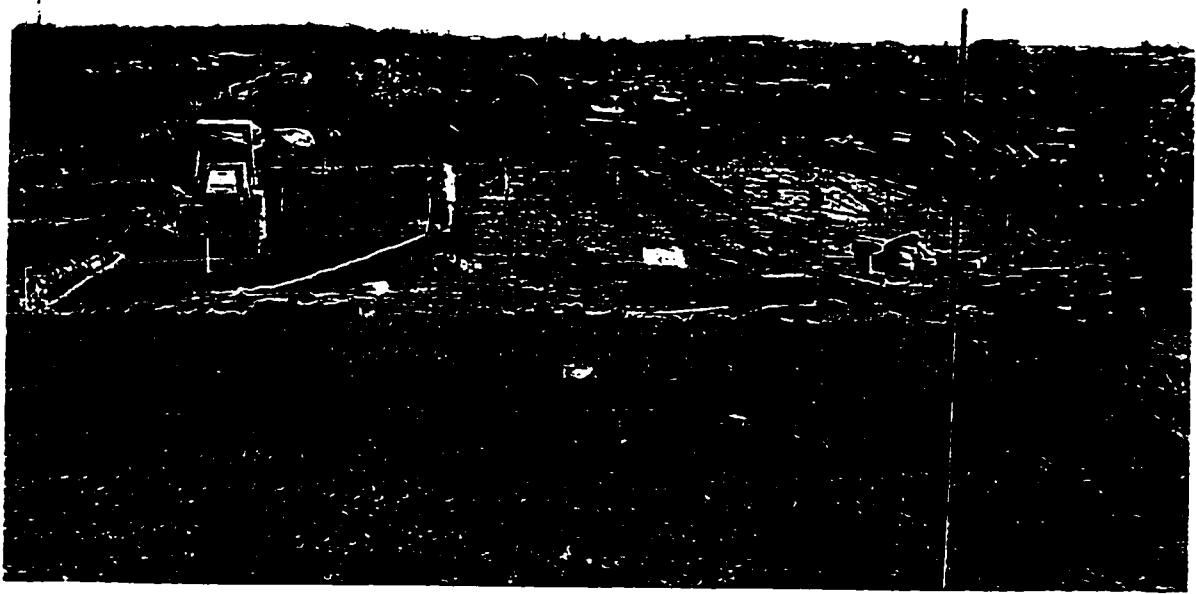


Figure 5.12. The installation of the drainage blanket.



Figure 5.13. The "derimming" machine.

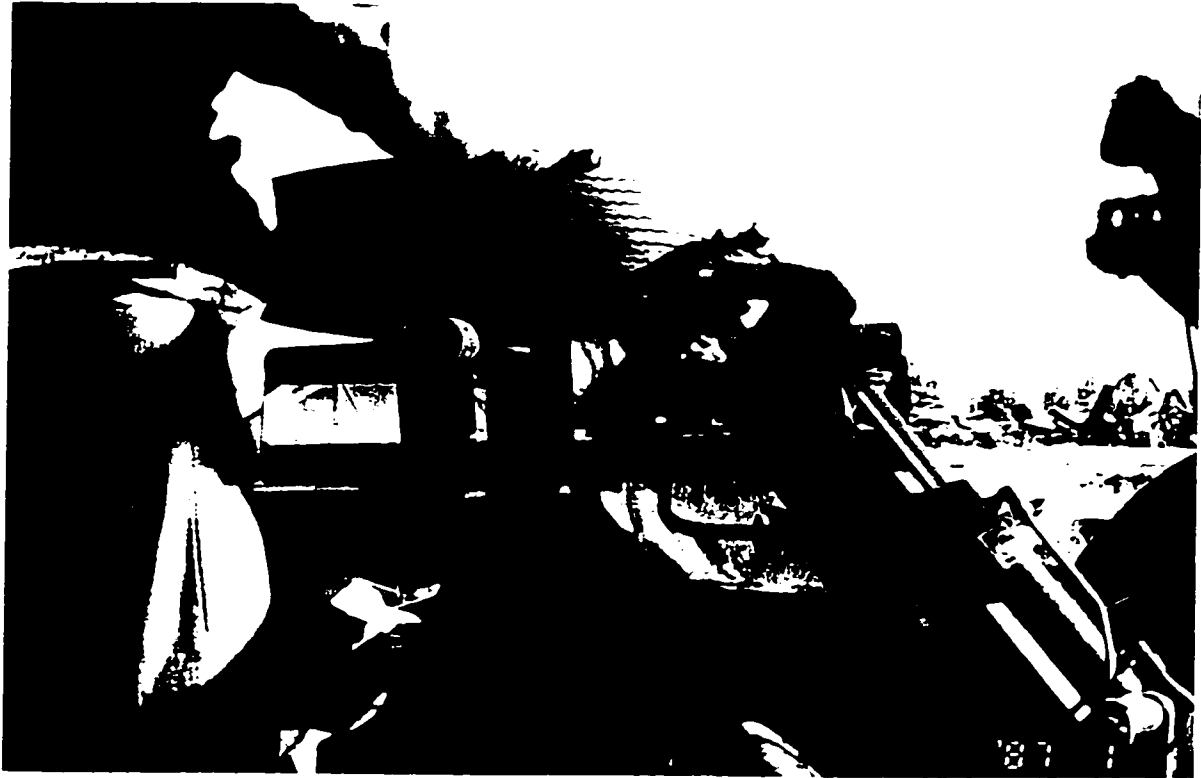


Figure 5.14. The tire slicing machine.



Figure 5.15. A series of photographs showing the fabrication of the tire mats.



Figure 5.16. Infilling of the tire mat reinforcements.



Figure 5.17. Initial stage of construction of the tire retaining wall.

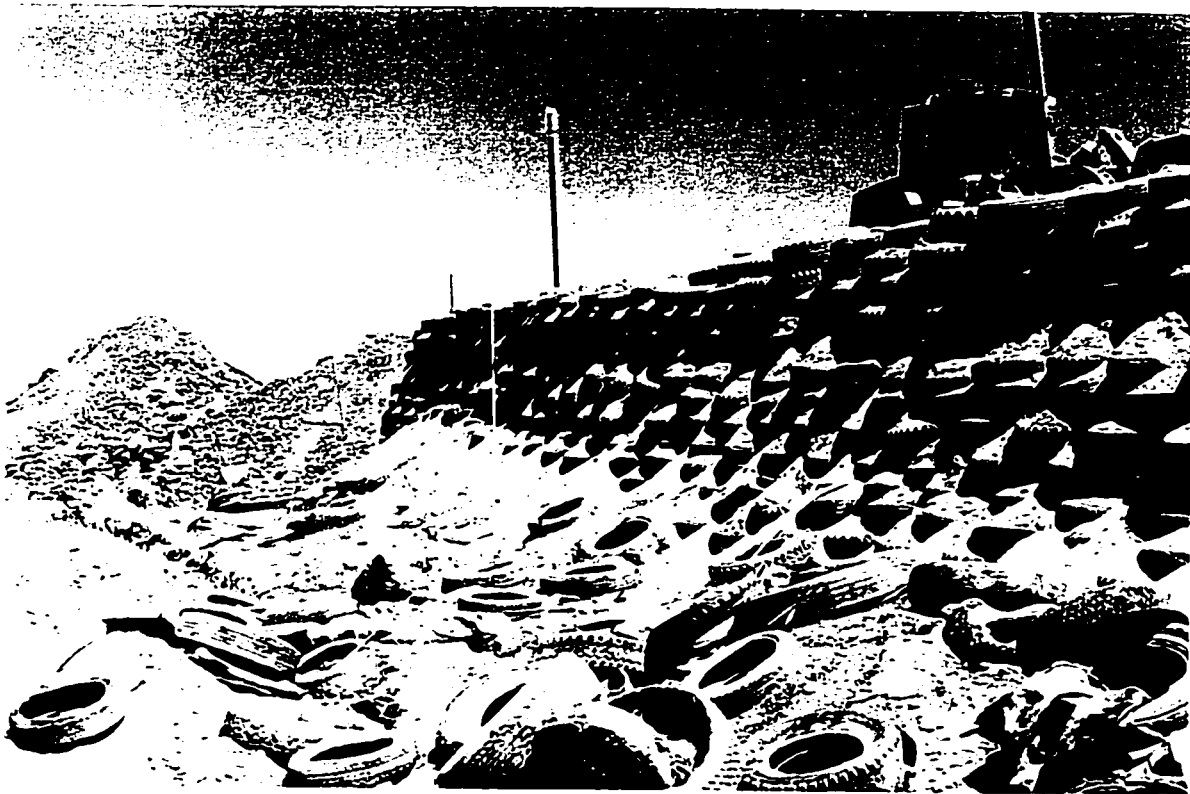


Figure 5.18. Mid-stage of construction of the tire retaining wall (approx. height 2m).



Figure 5.19. Completion of the tire retaining wall (height 4m).



Figure 5.20. The tire retaining wall section after placement of the 2 m surcharge load.

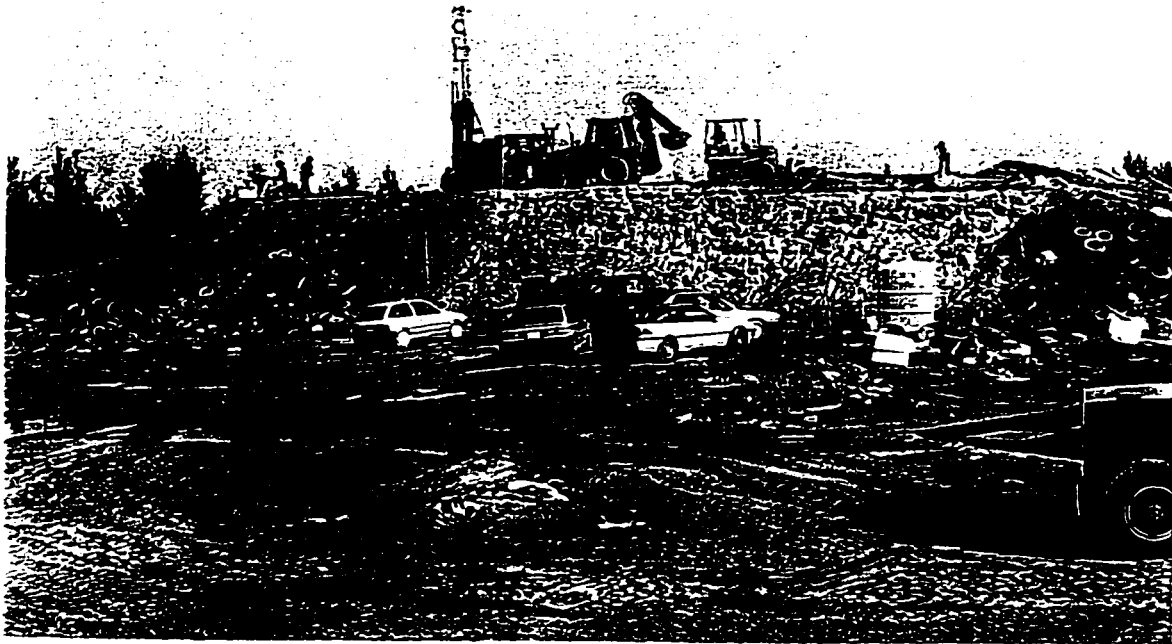


Figure 5.21. A frontal view of the completed reinforced slope section.



Figure 5.22. The placement of the surcharge load.

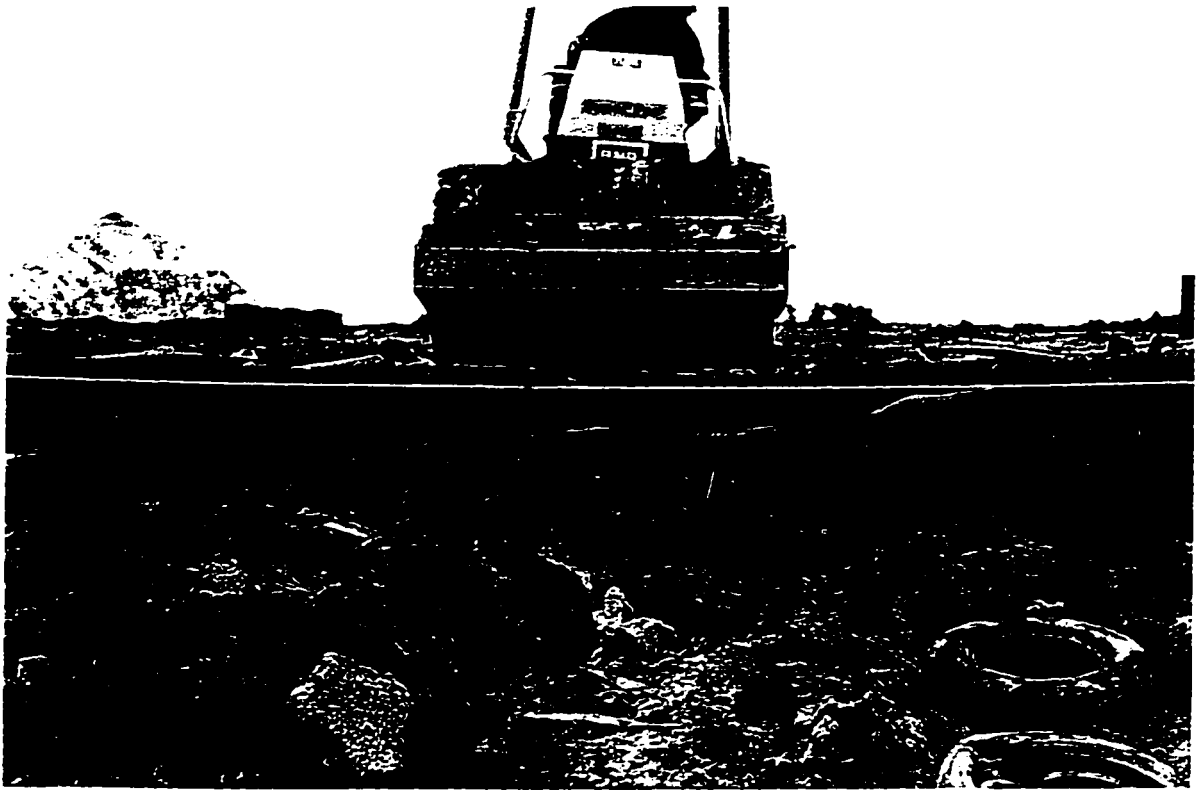


Figure 5.23. Compaction of the retaining wall at the initial stage (Section C).



Figure 5.24. A photograph showing the degree of infilling and compaction of cut tire reinforcement placed in the cohesive backfill material.



Figure 5.25. The completed prototype test embankment, viewed from the reinforced slope sections.

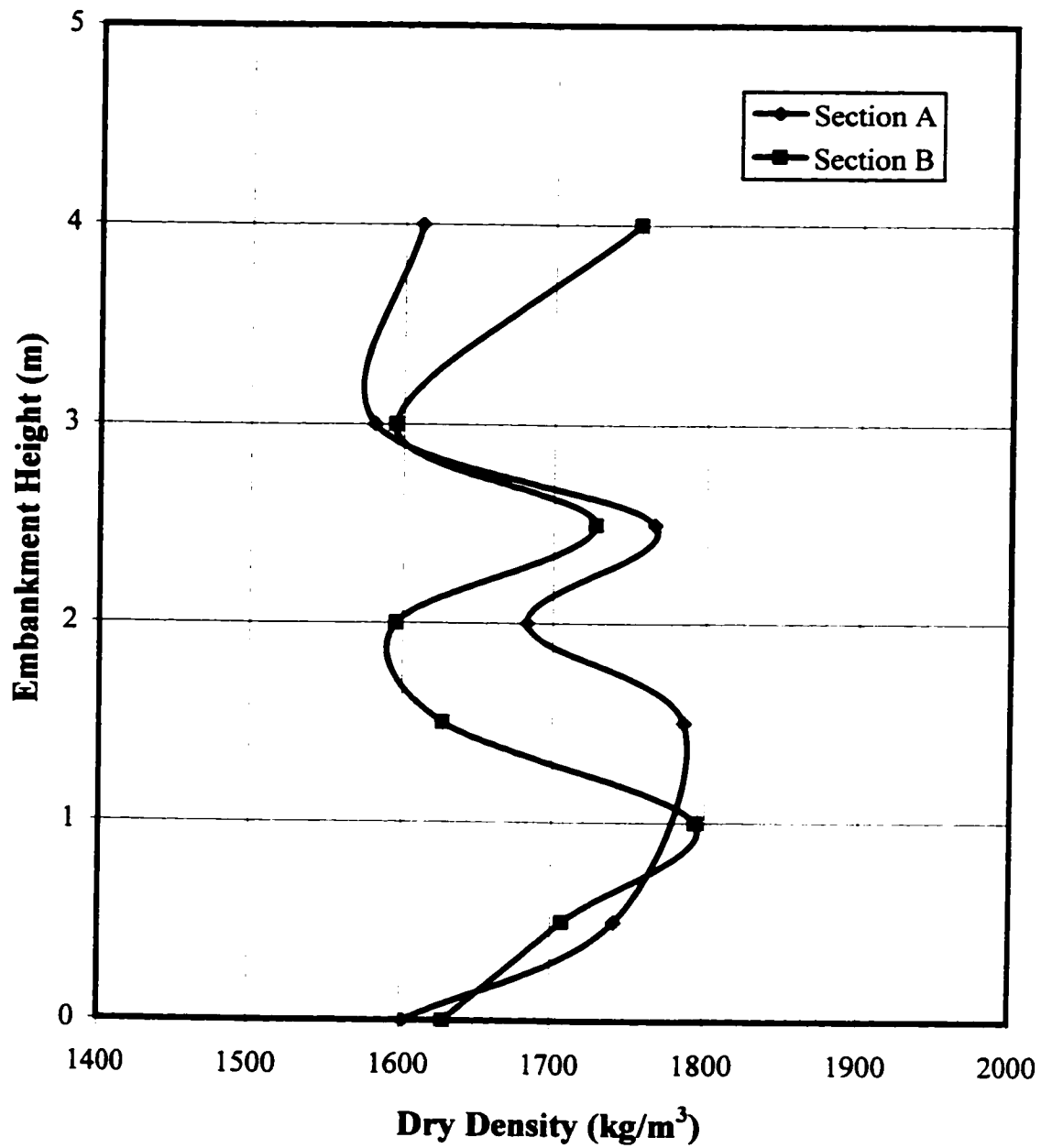


Figure 5.26. The variation of the measured dry density of sand with embankment height (Sections A and B).

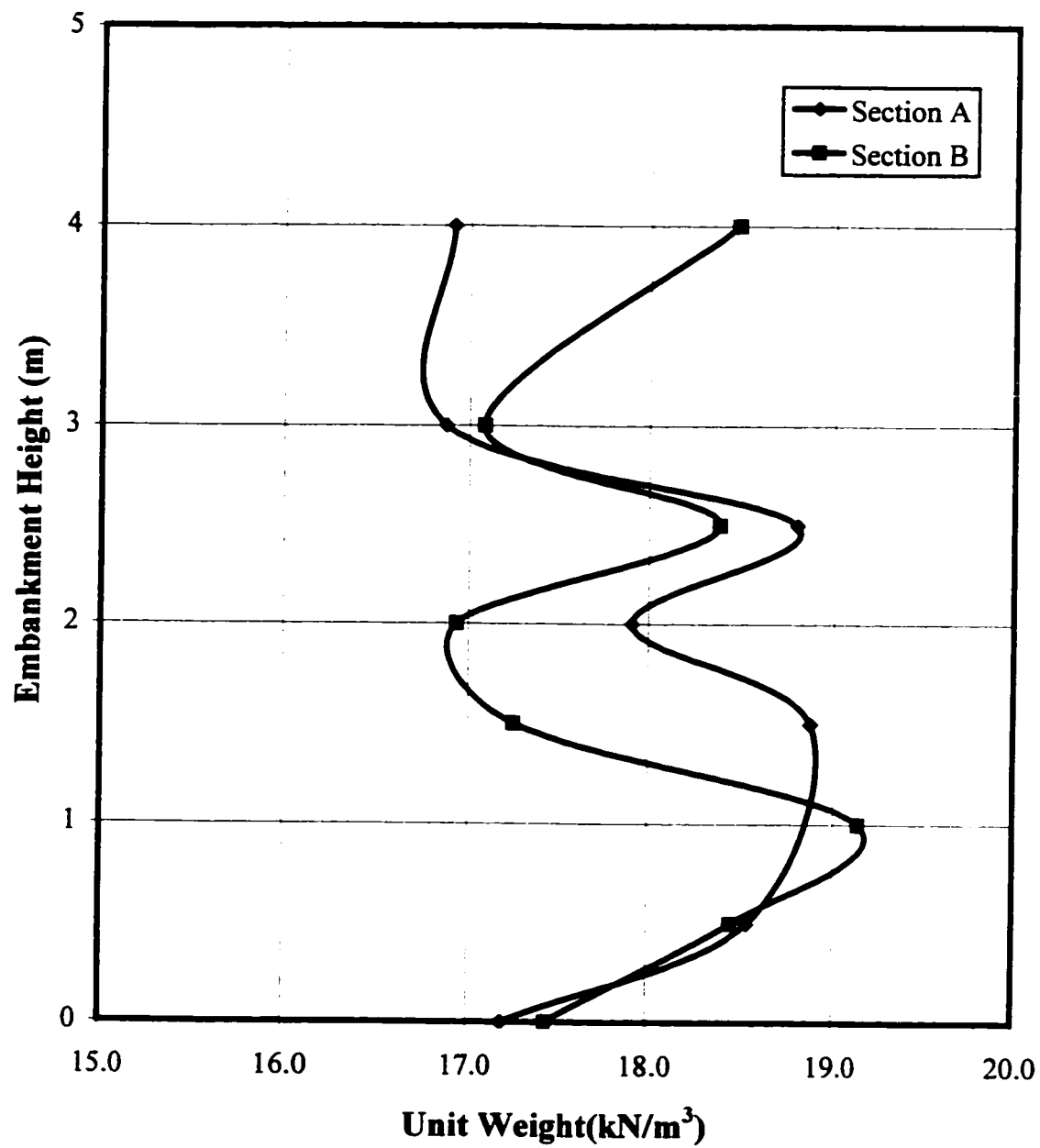


Figure 5.27. The variation of the measured unit weight of sand with embankment height (Sections A and B).

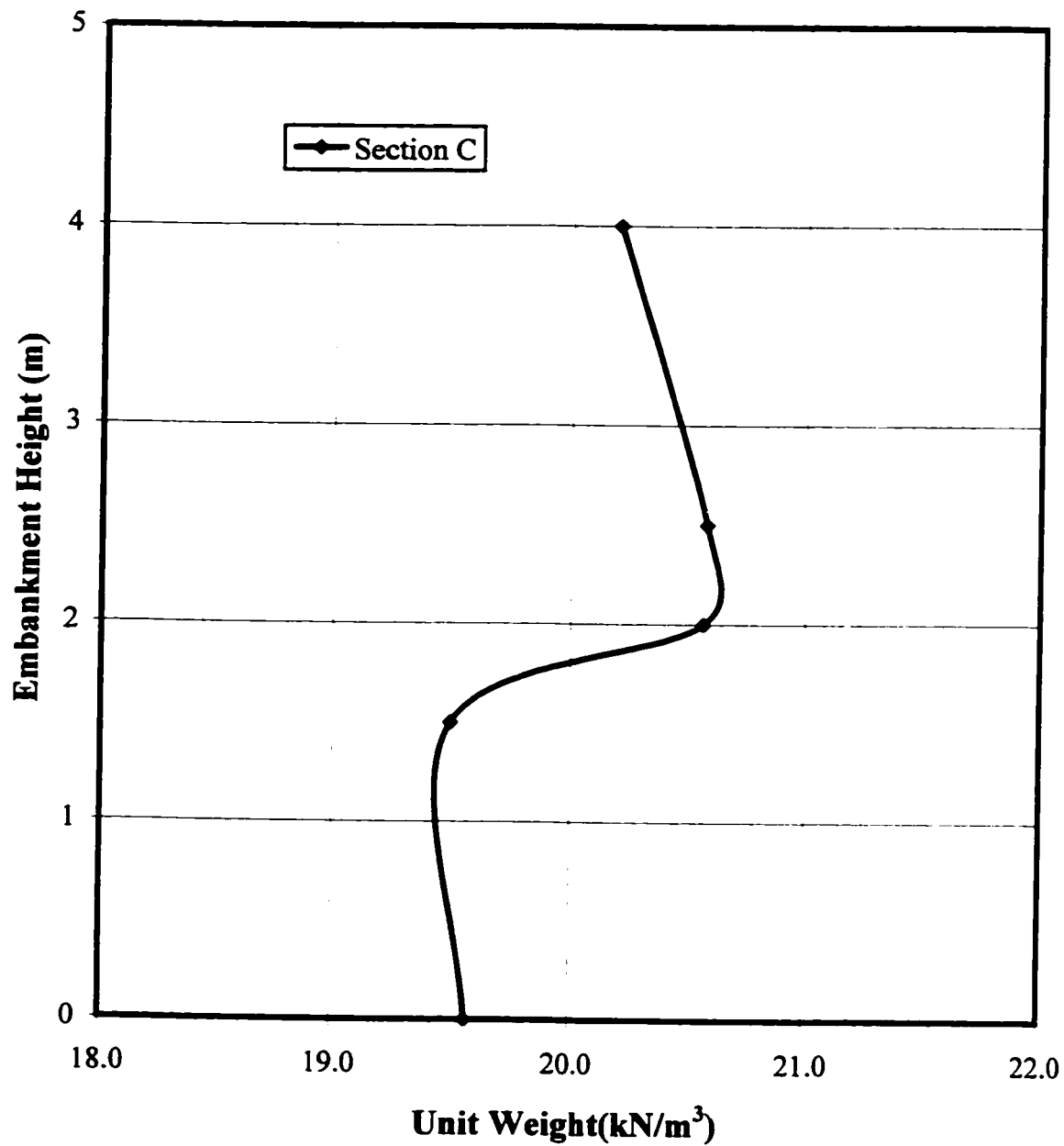


Figure 5.28. The variation of the measured unit weight of the cohesive backfill with embankment height (Section C).

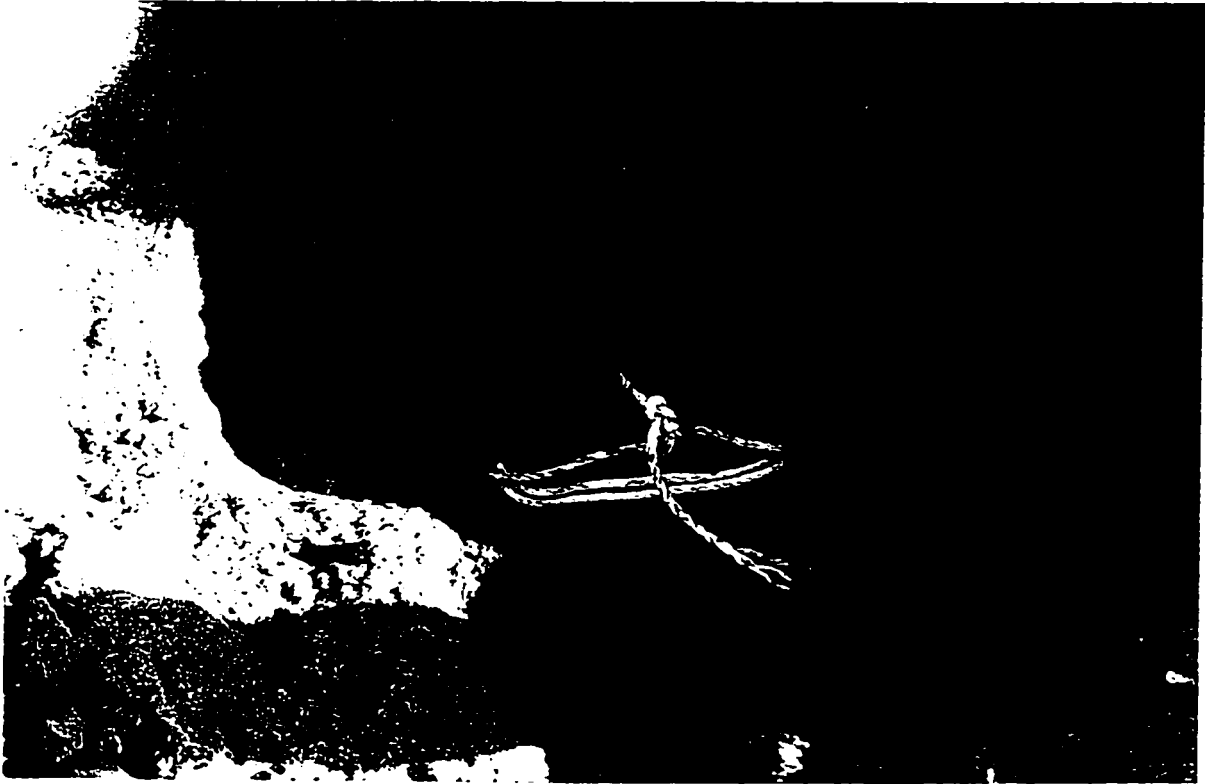


Figure 5.29. Photograph showing the collapse of the sidewall during compaction of a whole tire, and the formation of a small void at the top of the tire element.

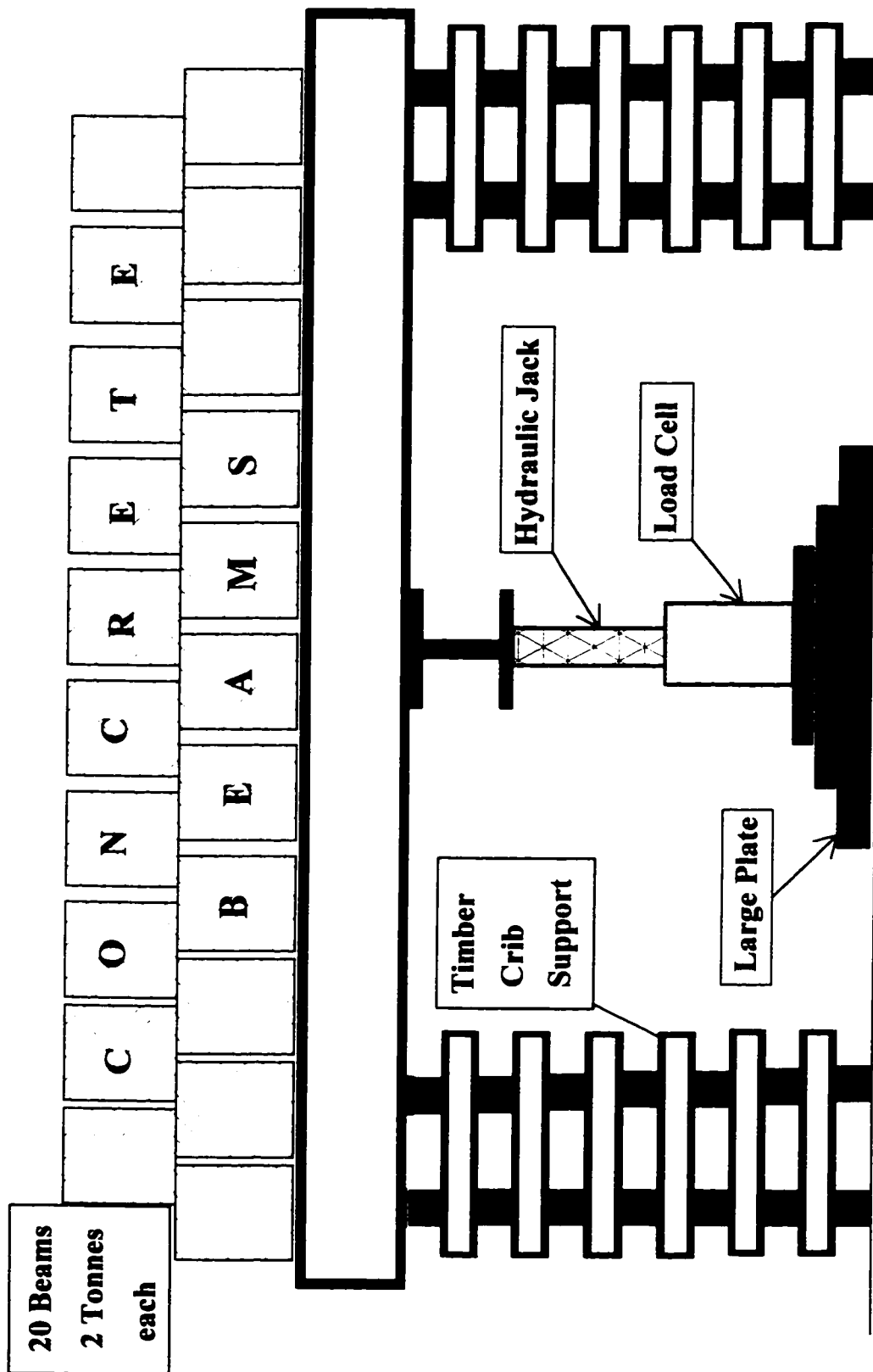


Figure 5.30. General concept of the plate loading test.

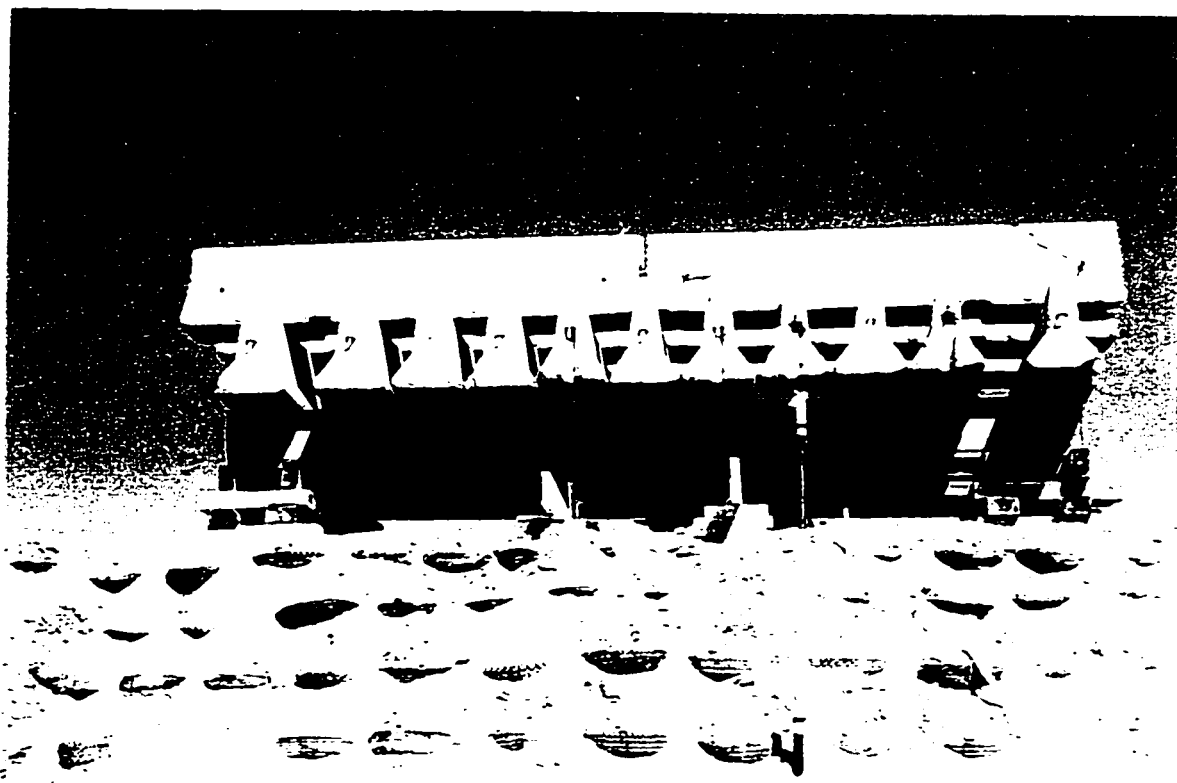


Figure 5.31. Photograph showing the loading platform for the plate load test.

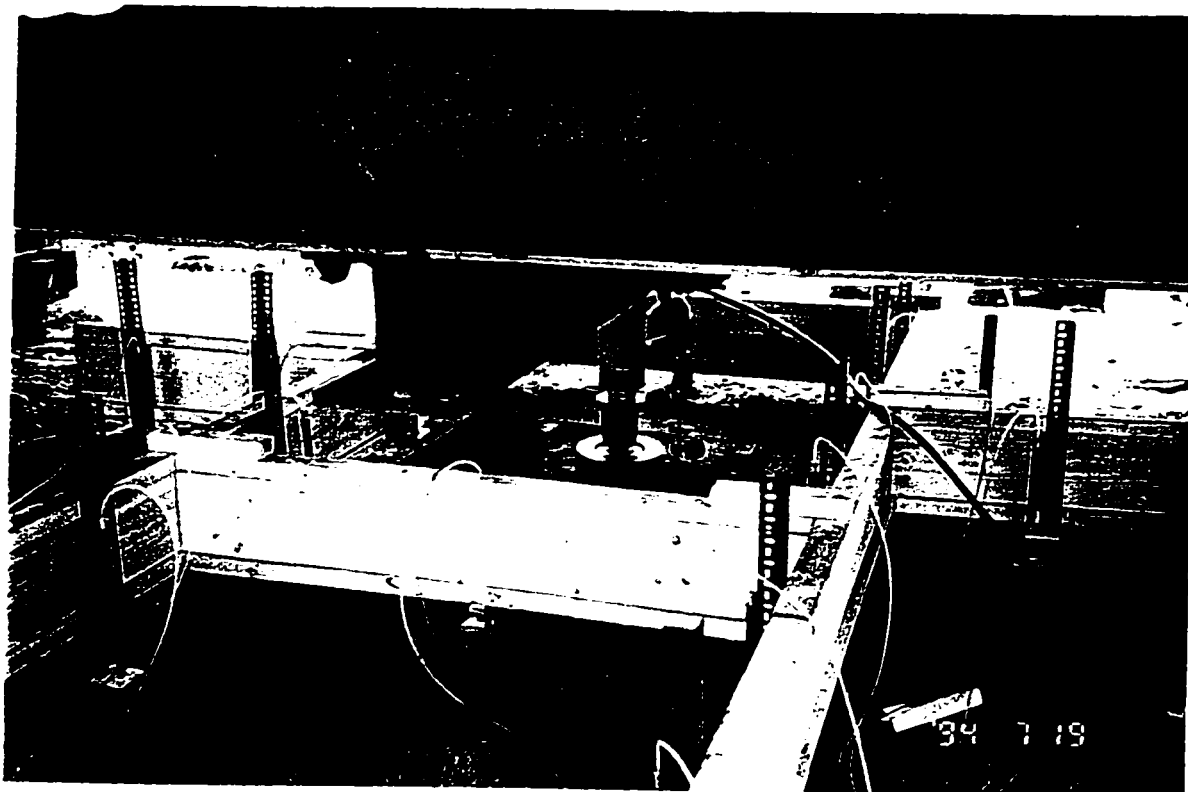


Figure 5.32. Location of the plate and loading system with respect to the reference beam.

Dimensions: Centimeters

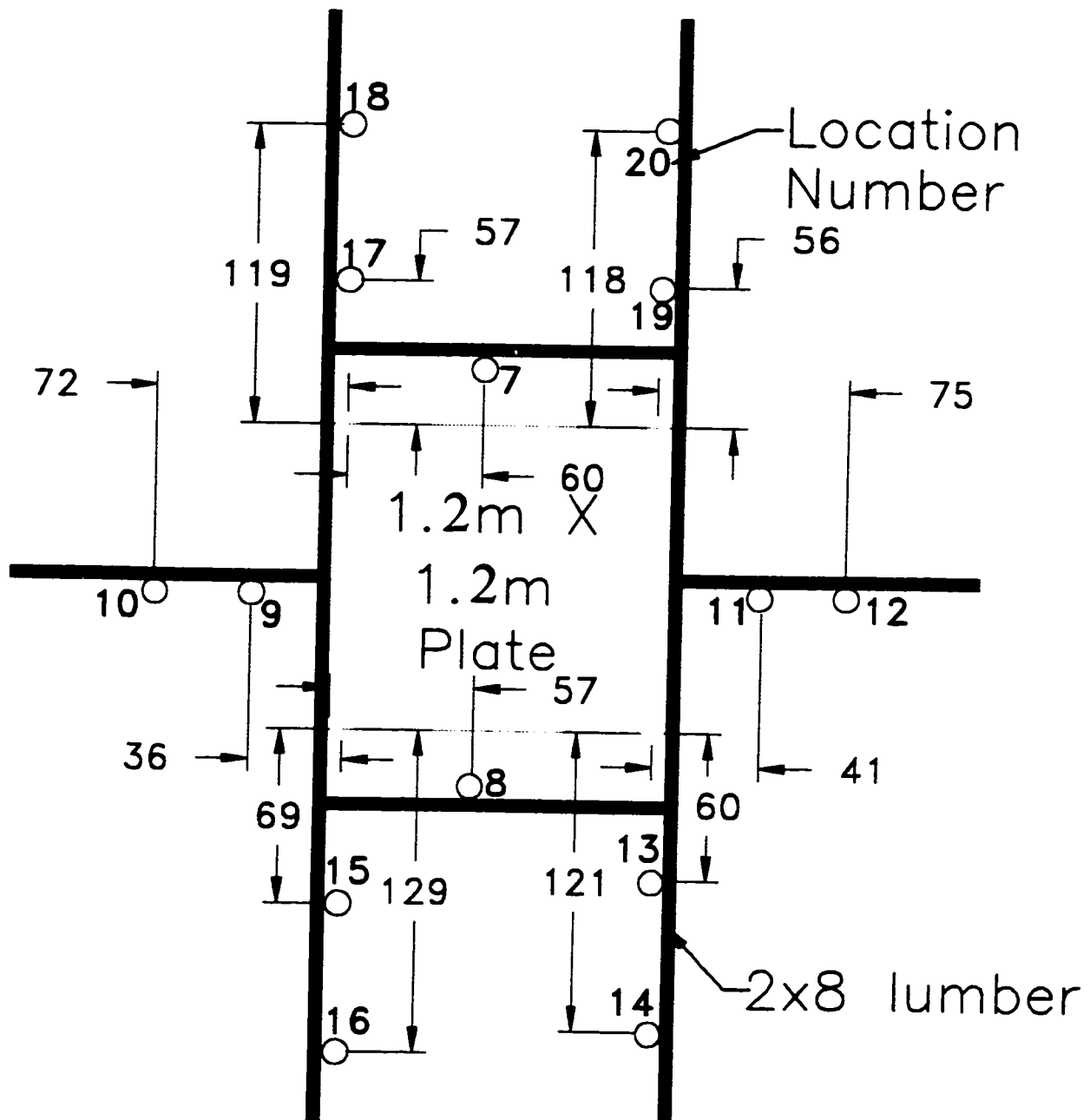


Figure 5.33. Location of the liquid transducers.

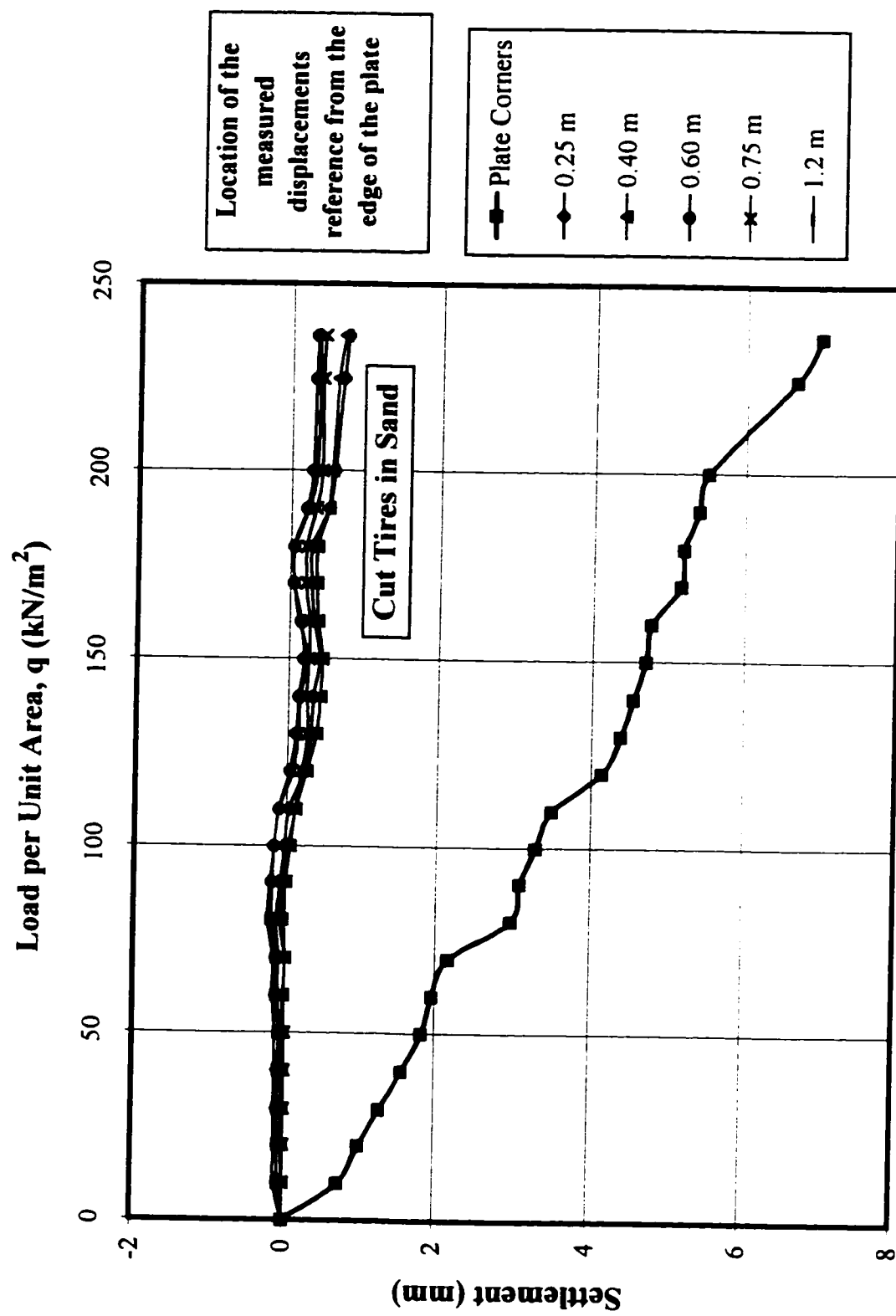


Figure 5.34. Results of the plate loading test performed on Section A, cut tires in sand.

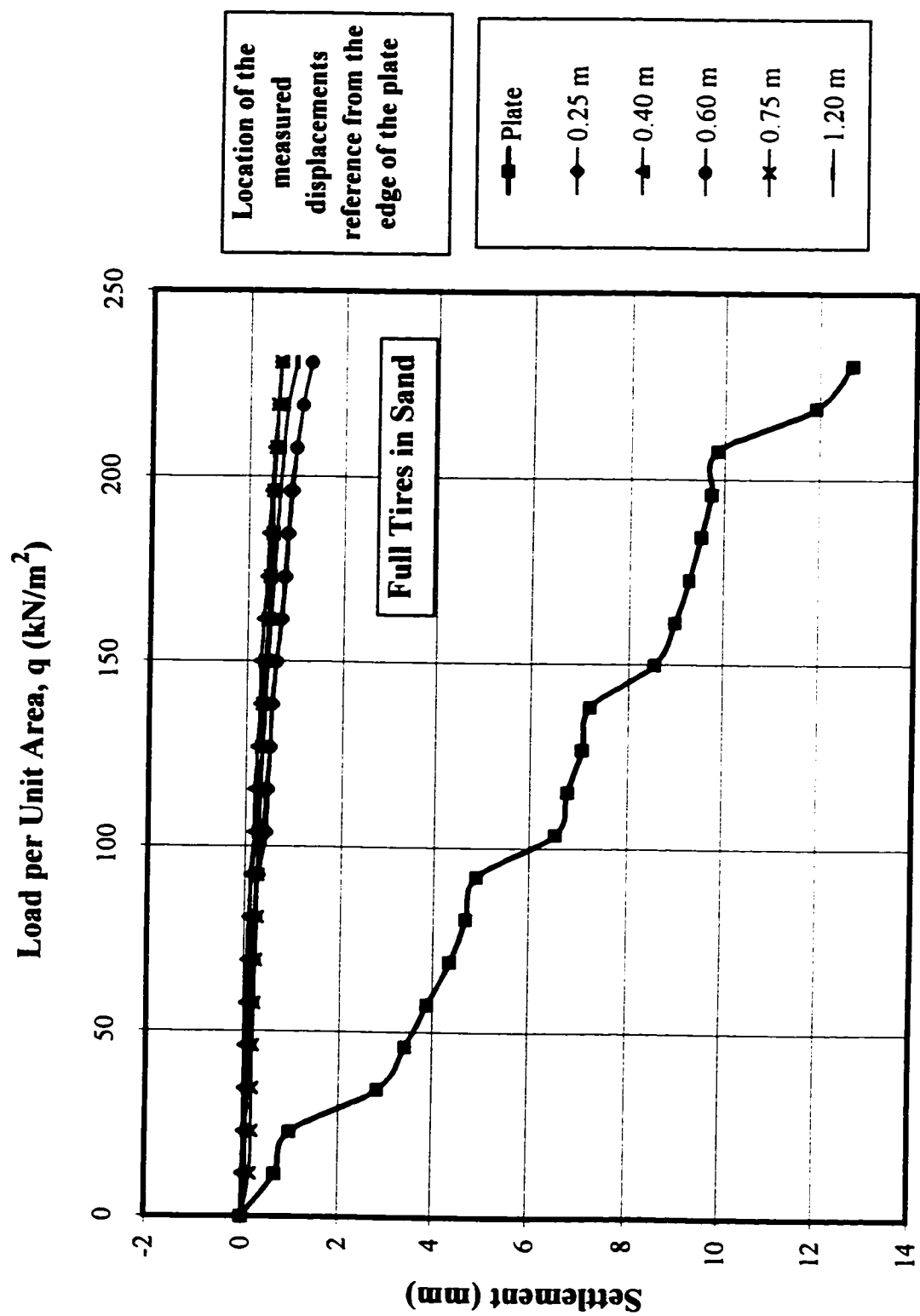


Figure 5.35. Results of the plate loading test performed on Section B, full tires in sand.

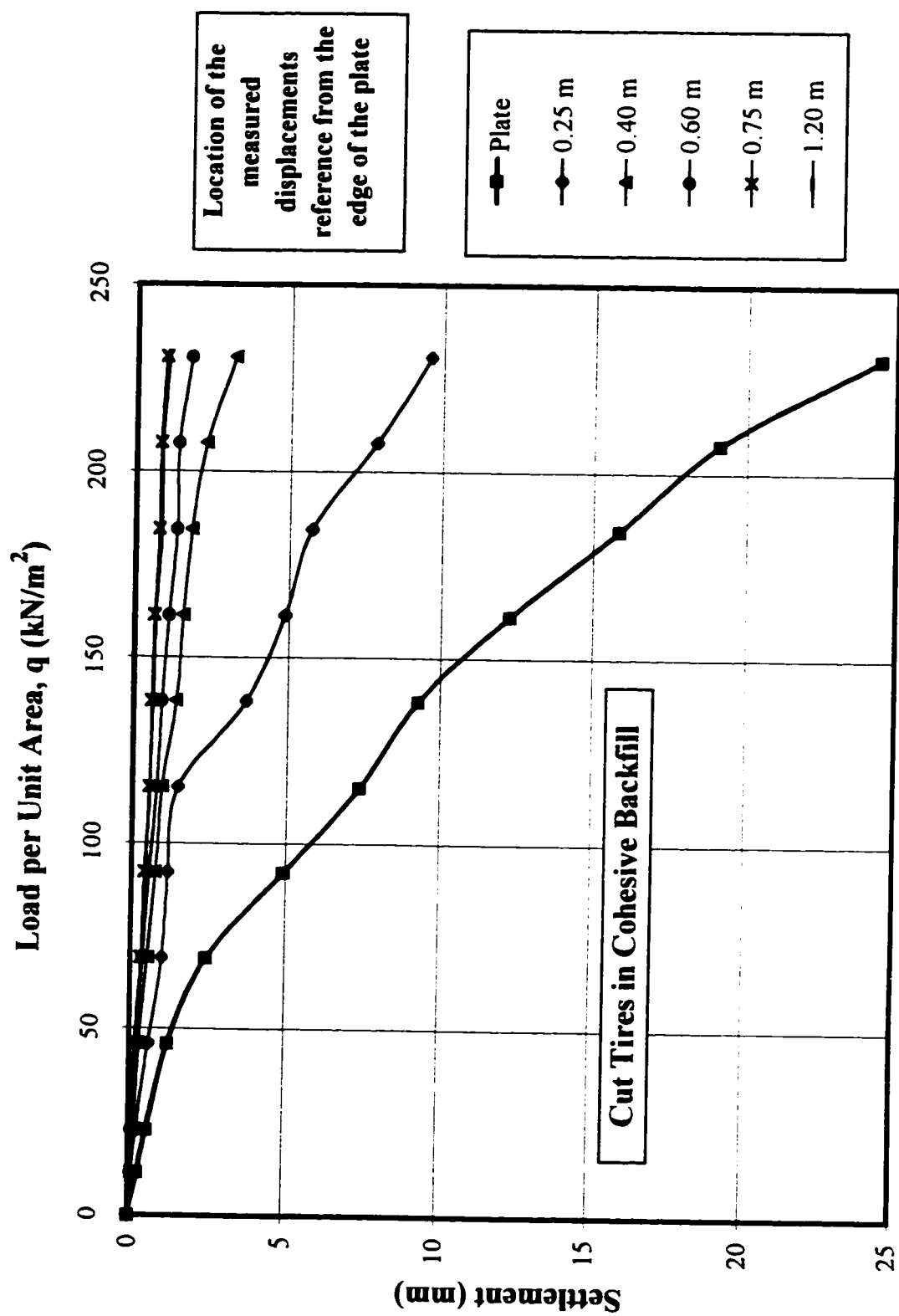


Figure 5.36. Results of the plate loading test performed on Section C, cut tires in cohesive backfill.

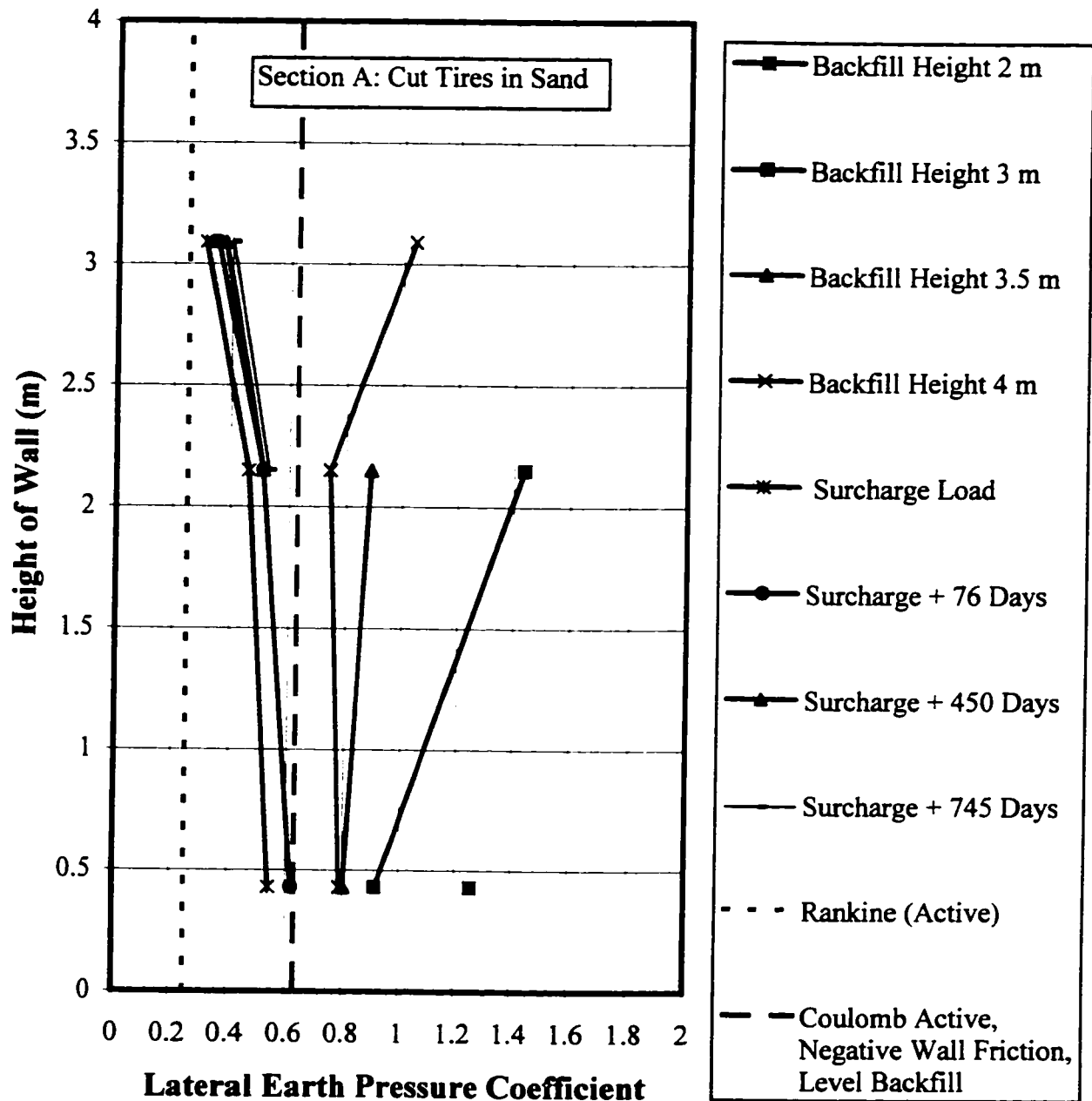


Figure 5.37. Lateral earth pressure coefficients behind the wall constructed with cut tires in sand.

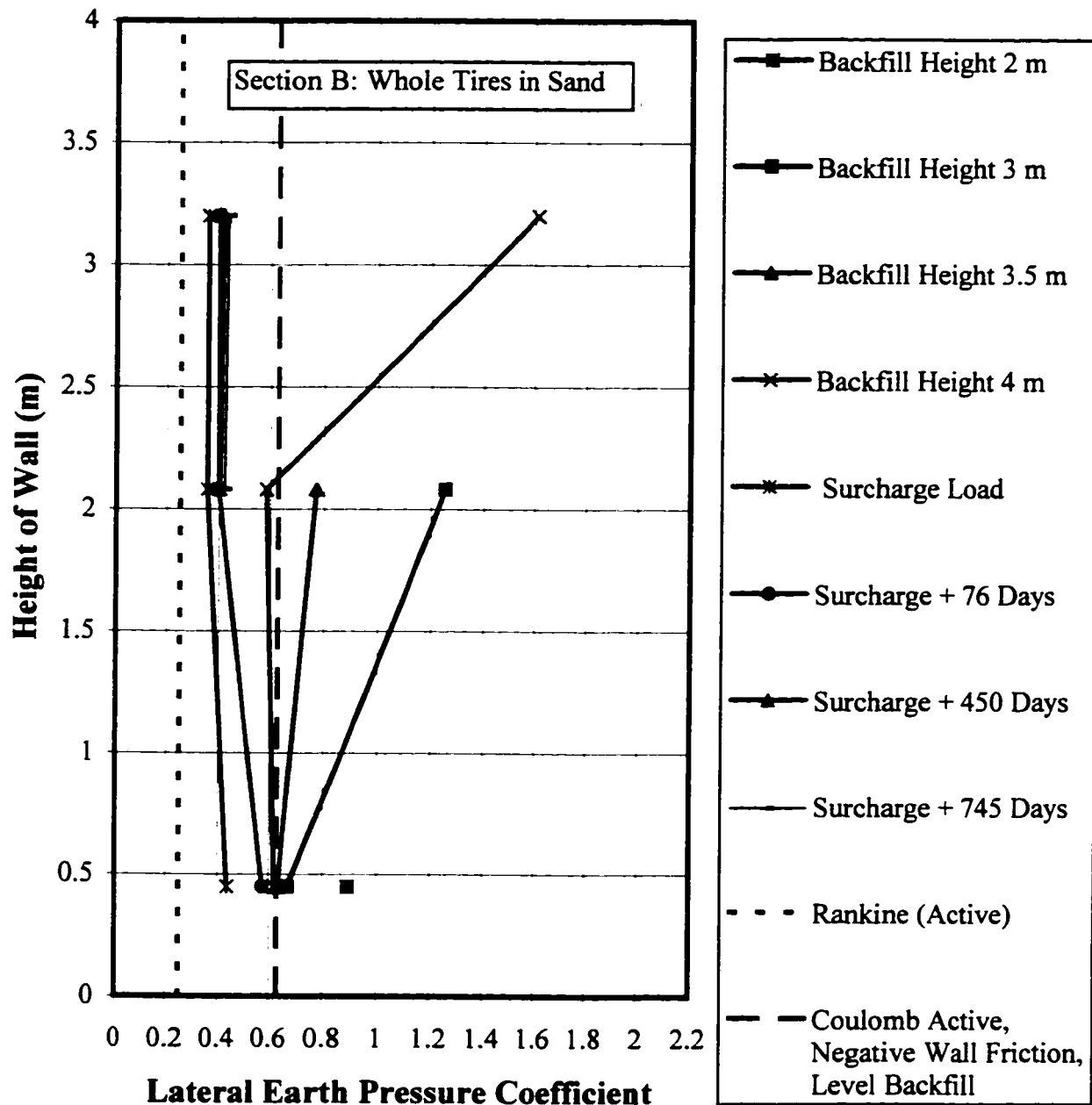


Figure 5.38. Lateral earth pressure coefficients behind the wall constructed with full tires in sand.

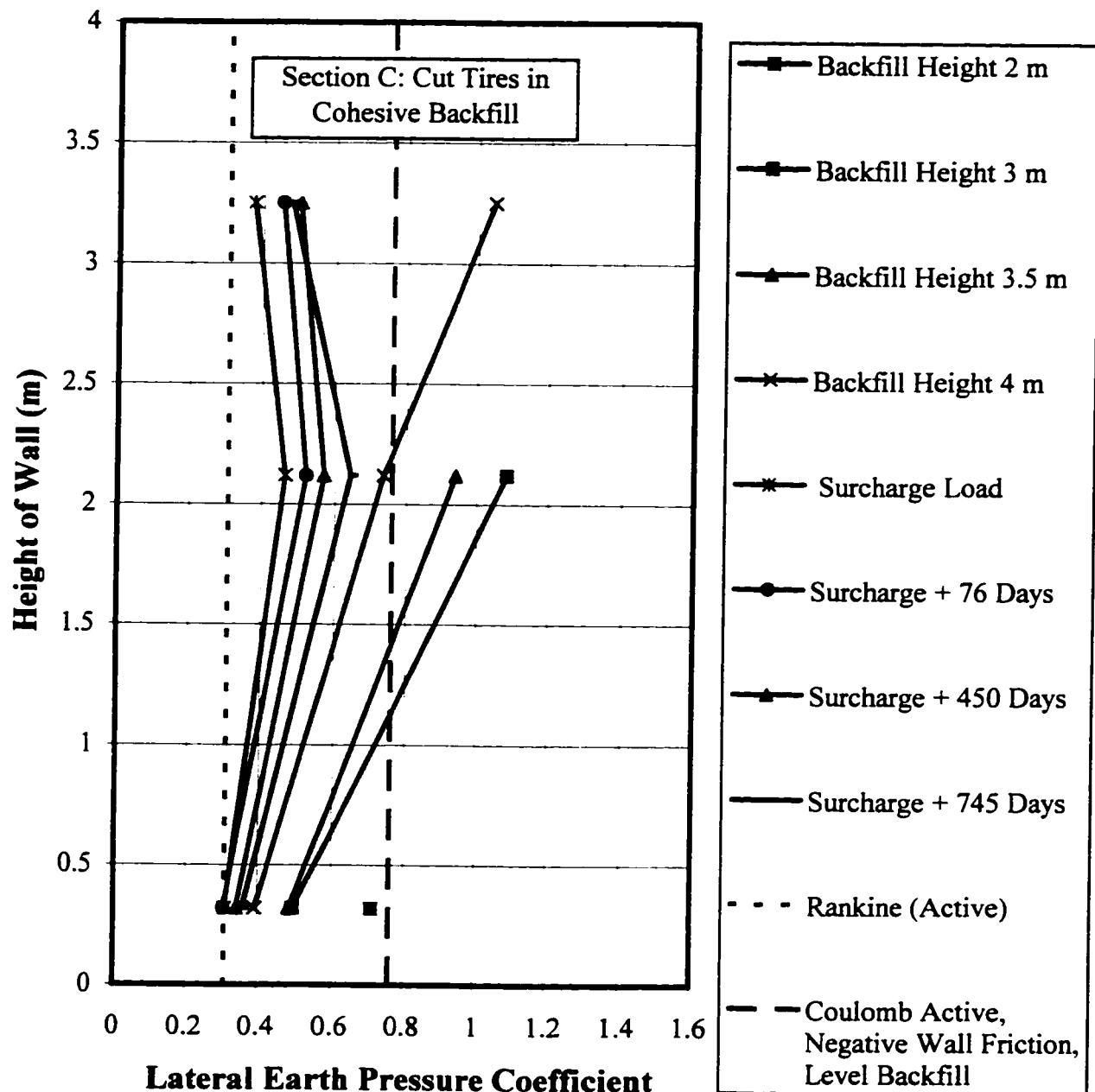


Figure 5.39. Lateral earth pressure coefficients behind the wall constructed with cut tires in cohesive backfill.

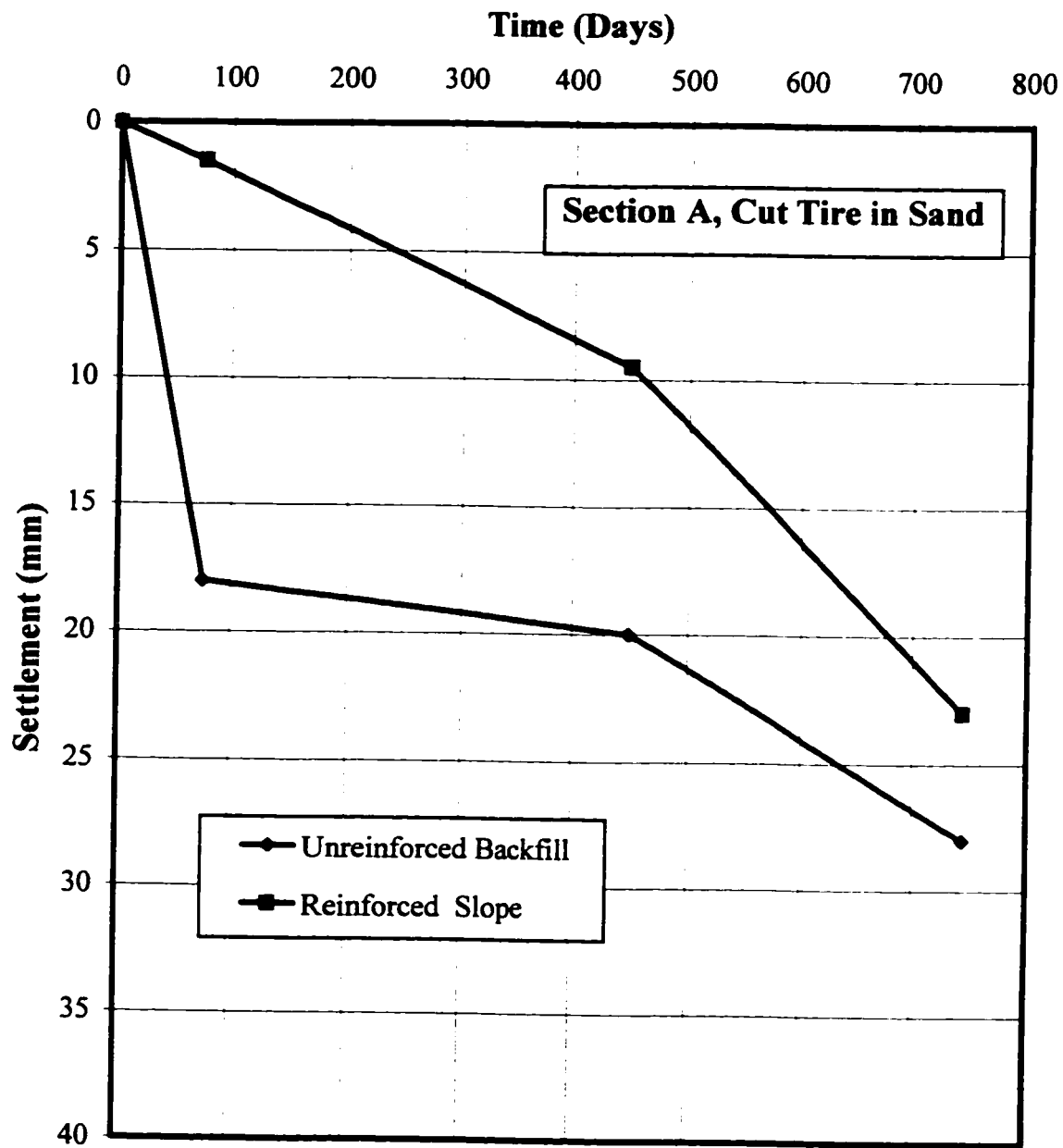


Figure 5.40. The overall settlement of Section A with the number of days after surcharge, for both reinforced slope and unreinforced backfill.

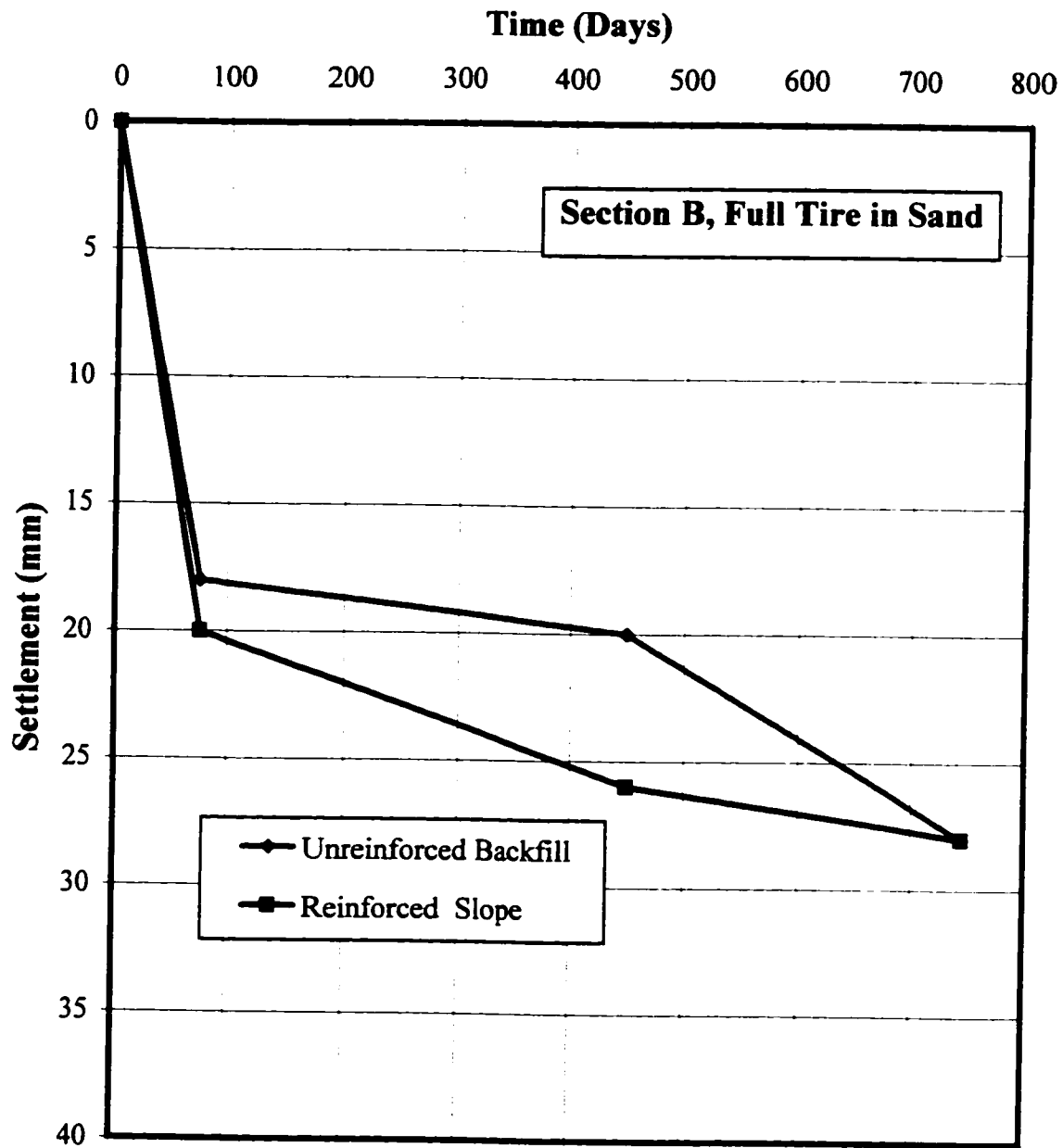


Figure 5.41. The overall settlement of Section B with the number of days after surcharge, for both reinforced slope and unreinforced backfill.

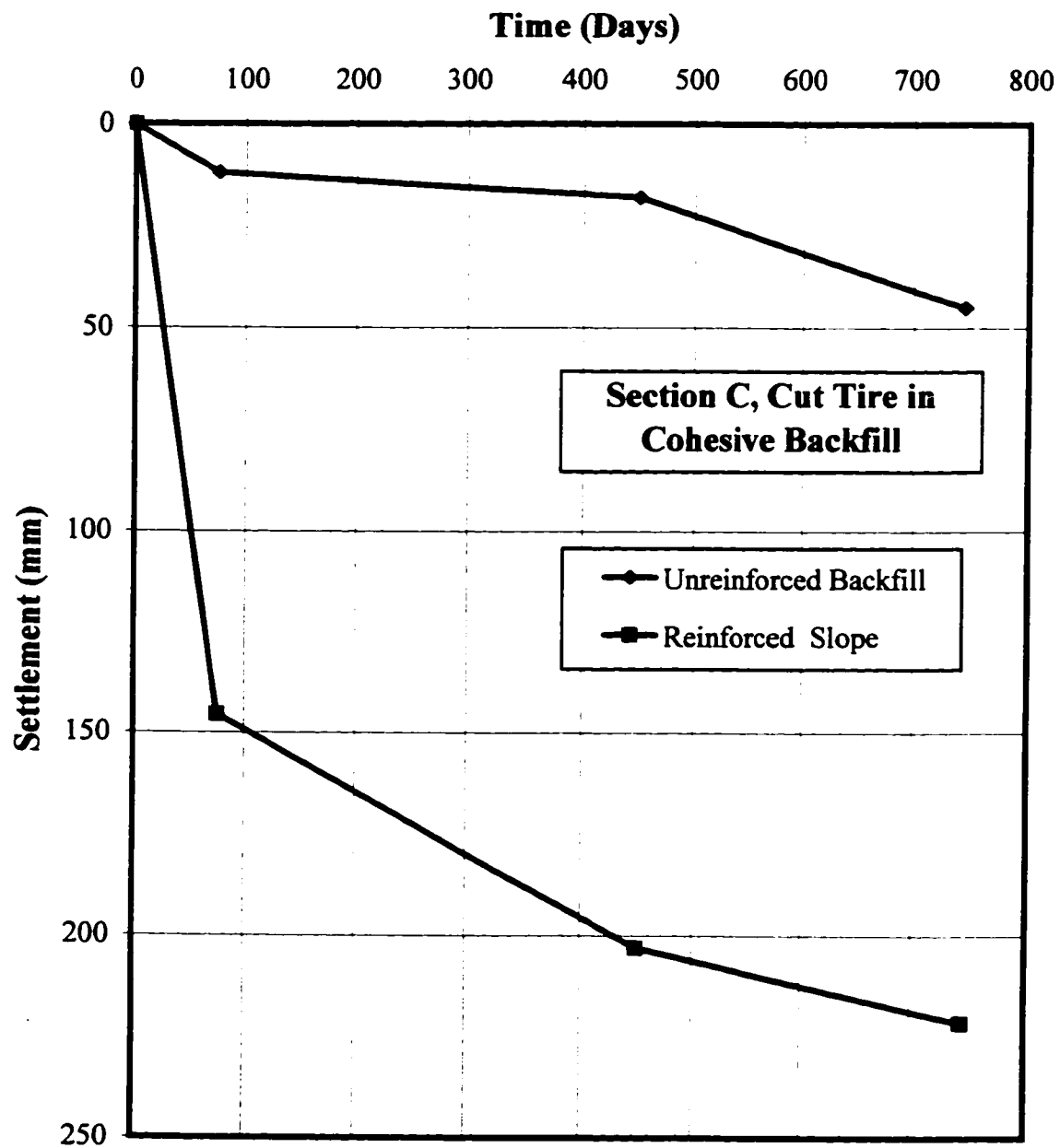


Figure 5.42. The overall settlement of Section C with the number of days after surcharge, for both reinforced slope and unreinforced backfill.

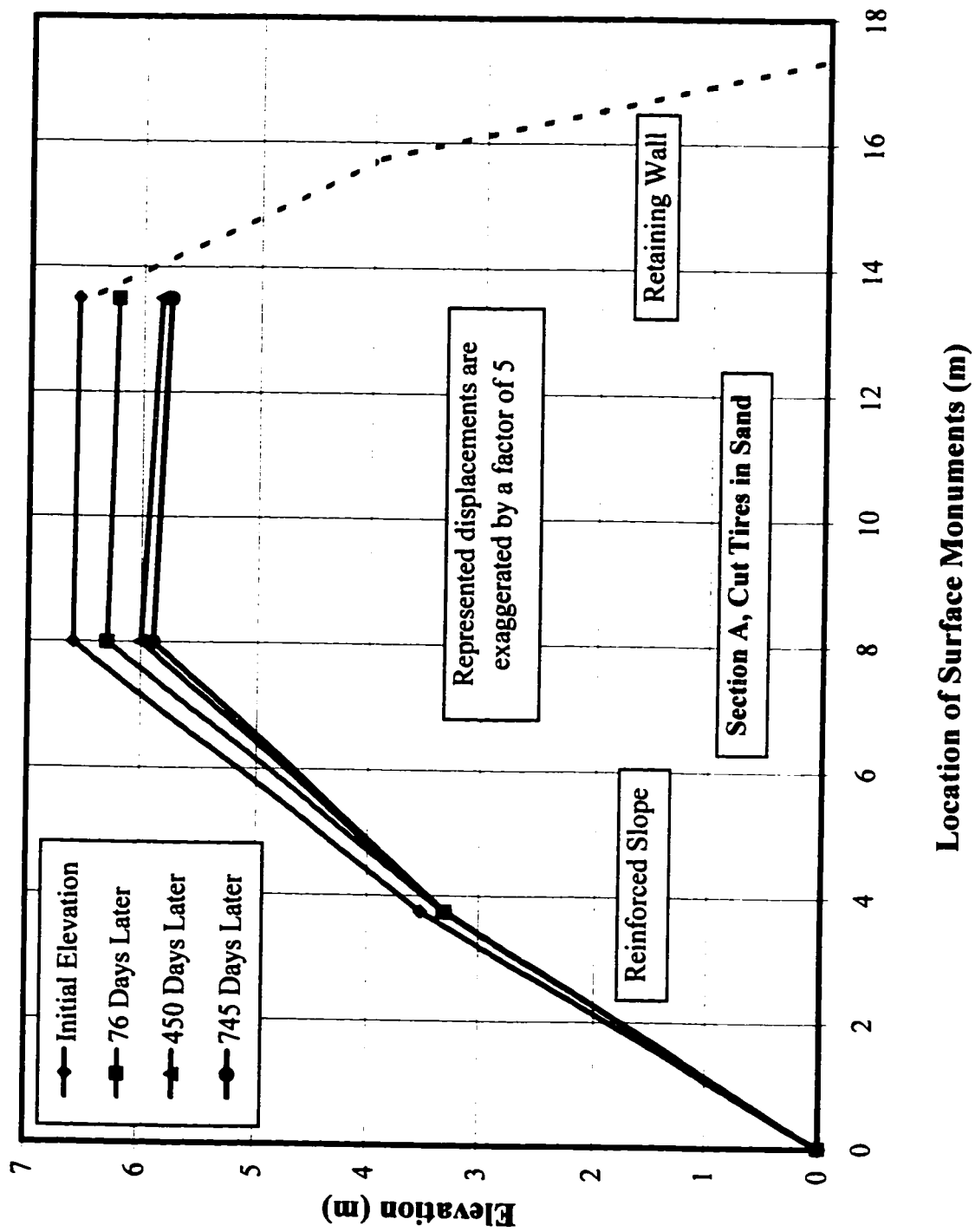


Figure 5.43. Settlement of surface monuments with time for Section A, cut tires in sand.

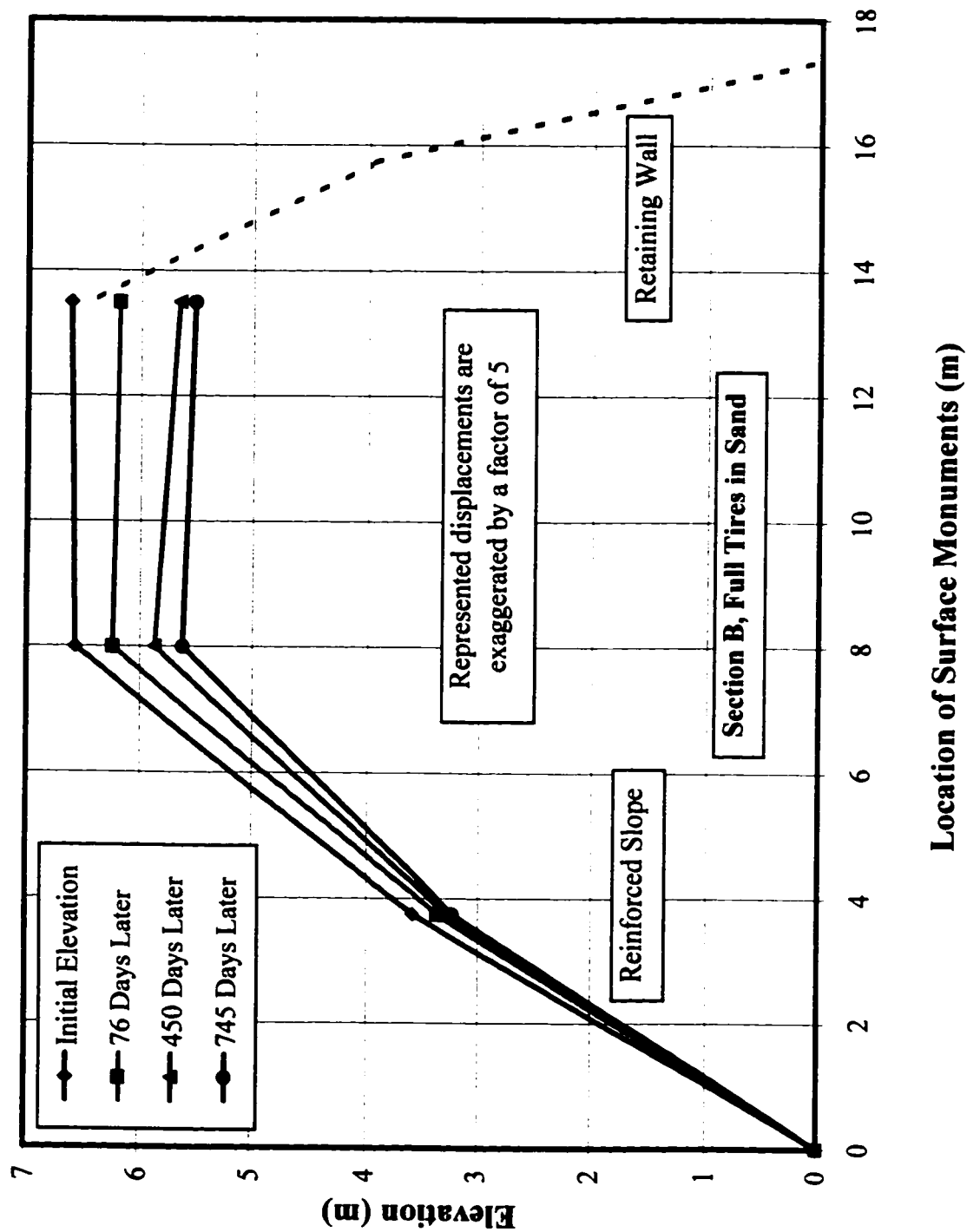


Figure 5.44. Settlement of surface monuments with time for Section B, full tires in sand.

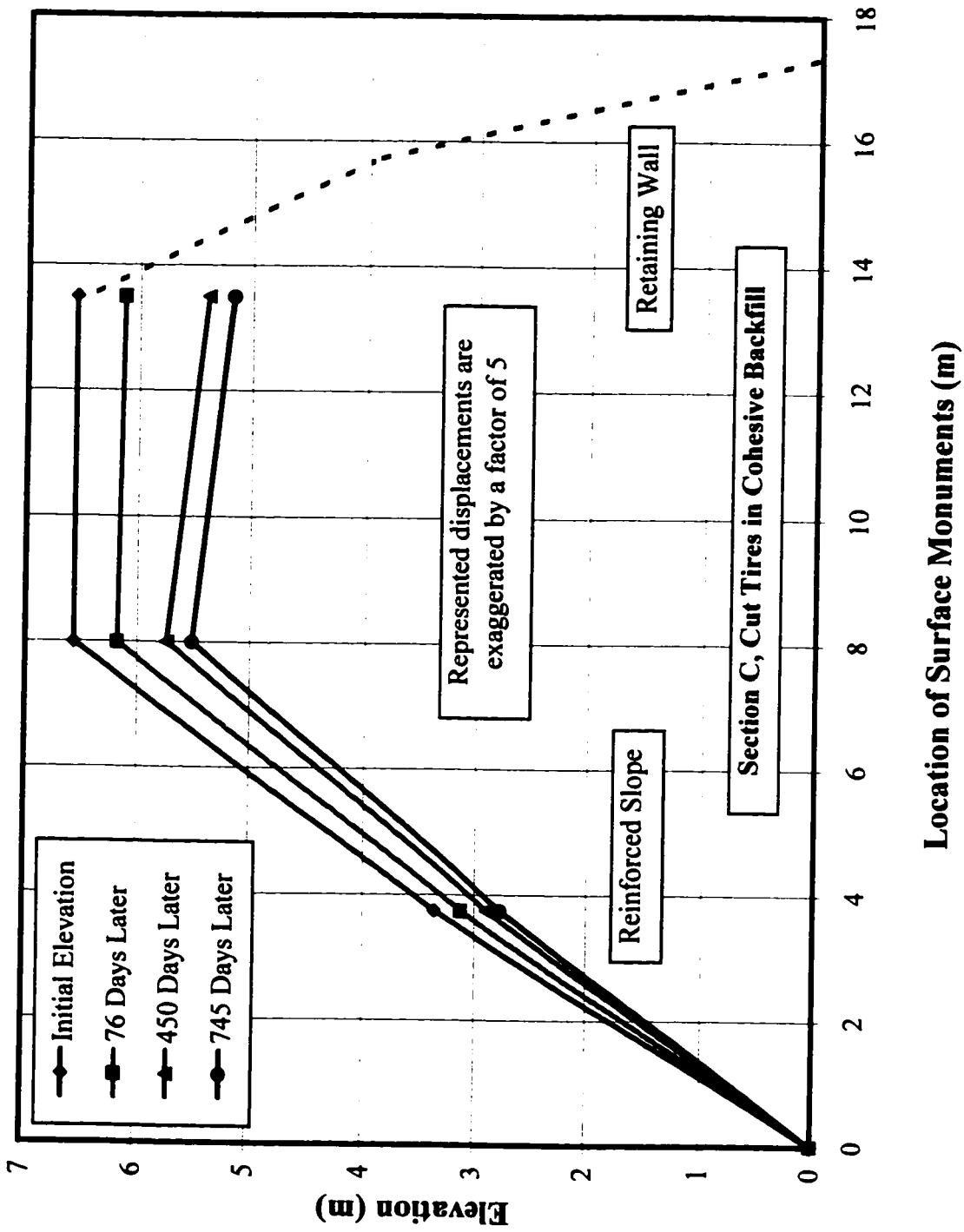


Figure 5.45. Settlement of surface monuments with time for Section C, cut tires in cohesive backfill.

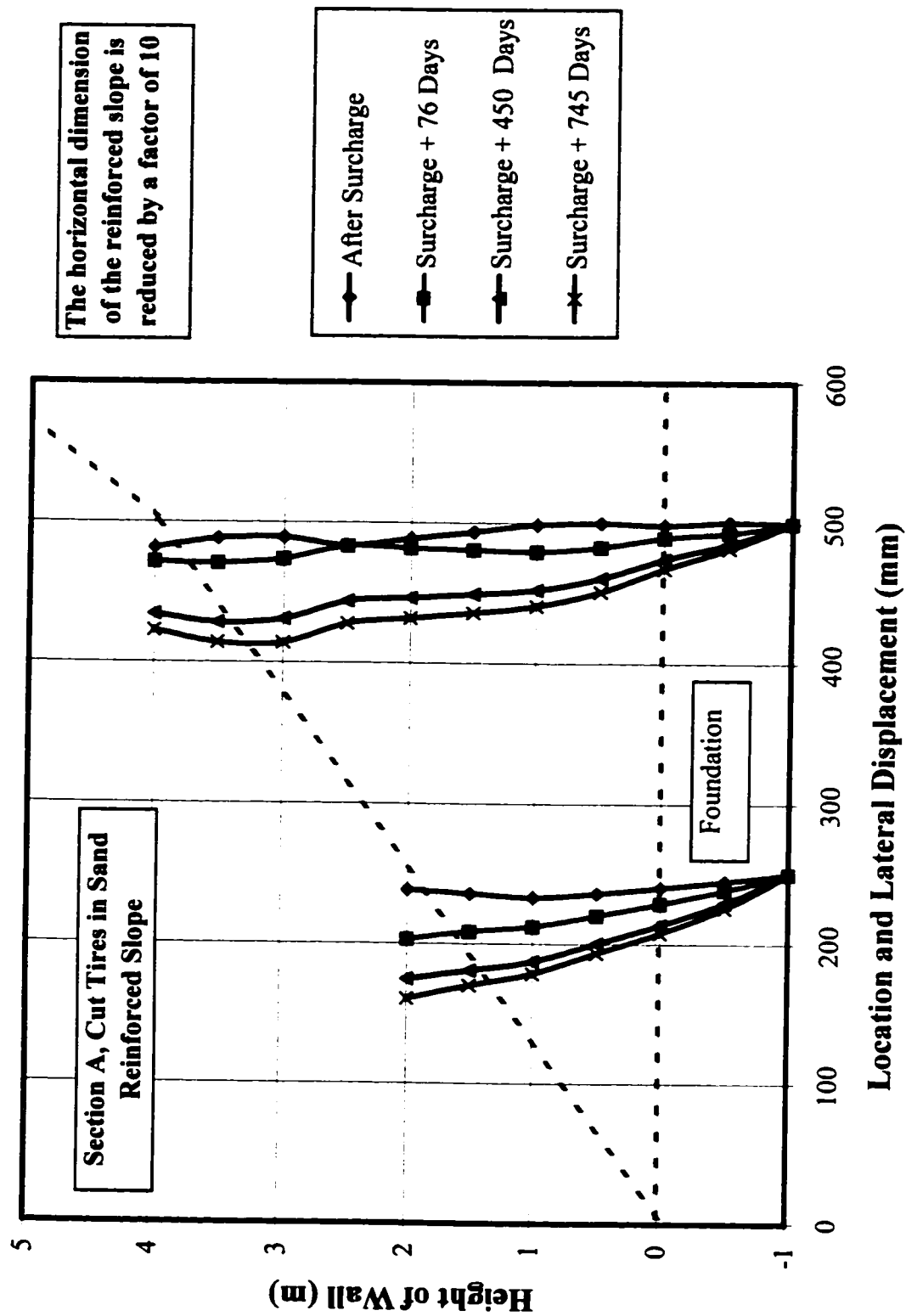


Figure 5.46. Lateral displacement of the reinforced slope in Section A, cut tires in sand, with reference to inclinometer location and time.

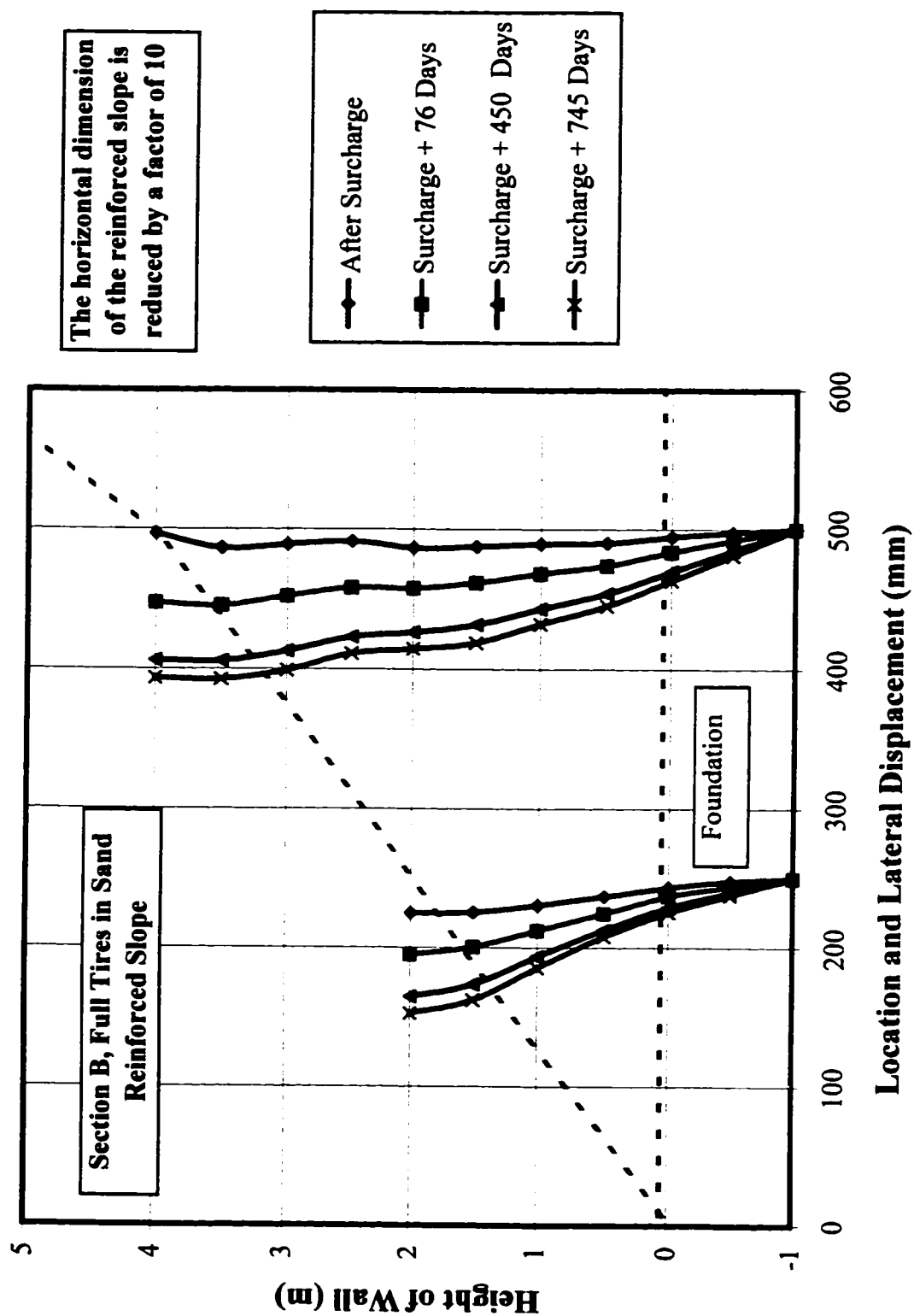


Figure 5.47. Lateral displacement of the reinforced slope in Section B, full tires in sand, with reference to inclinometer location and time.

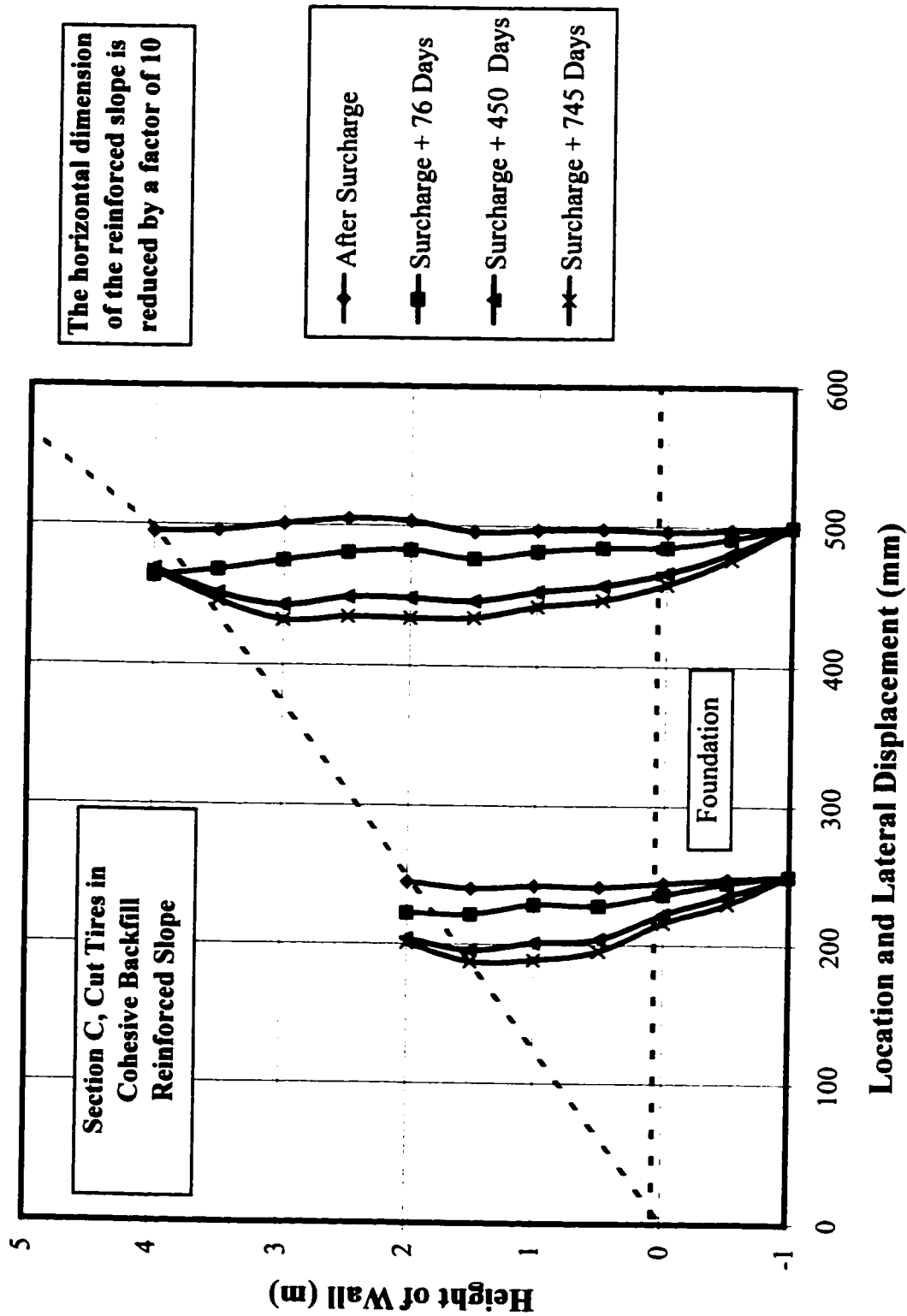


Figure 5.48. Lateral displacement of the reinforced slope in Section C, cut tires in cohesive backfill, with reference to inclinometer location and time.

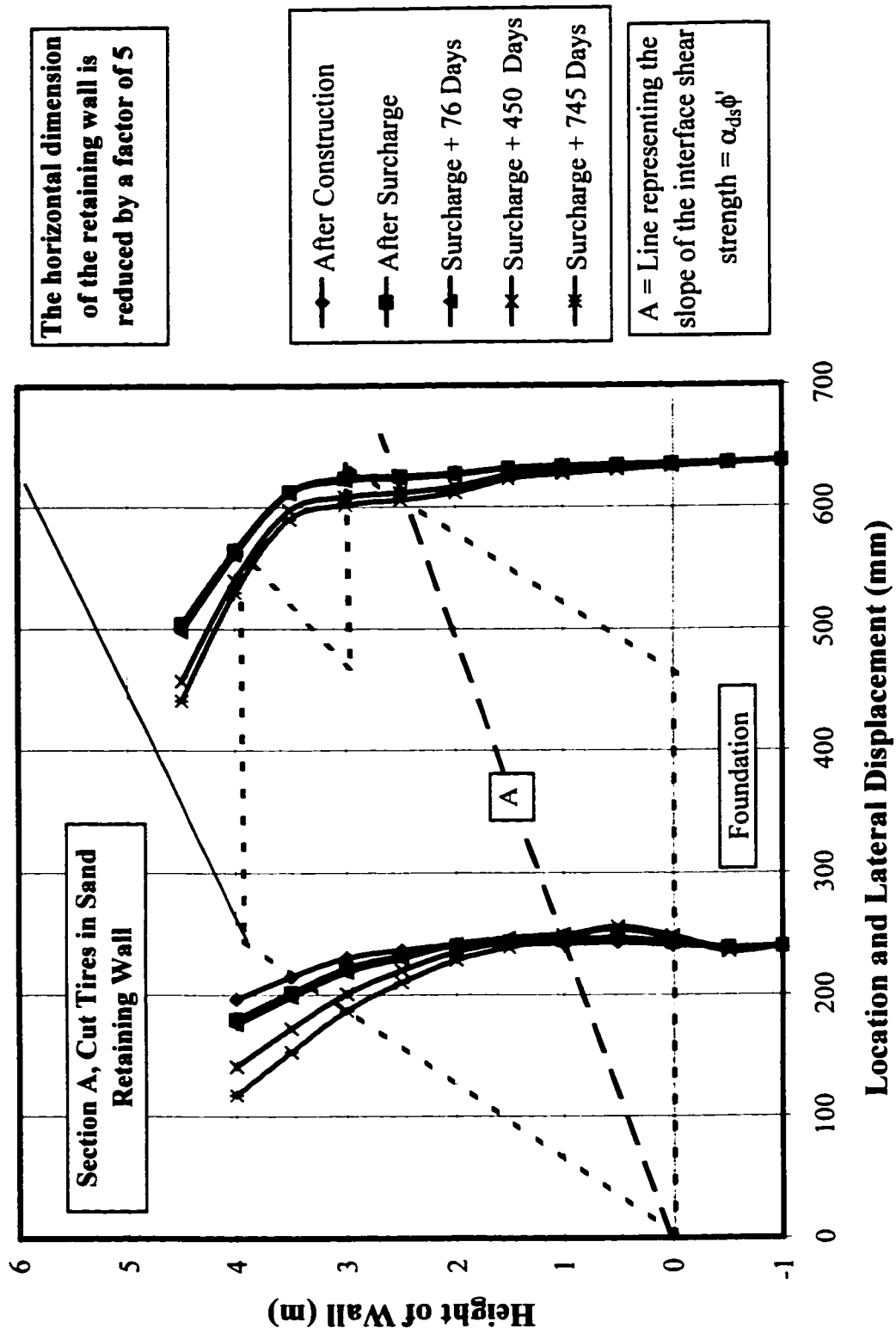


Figure 5.49. Lateral displacement of the tires wall in Section A, cut tire in sand, with reference to inclinometer location and time.

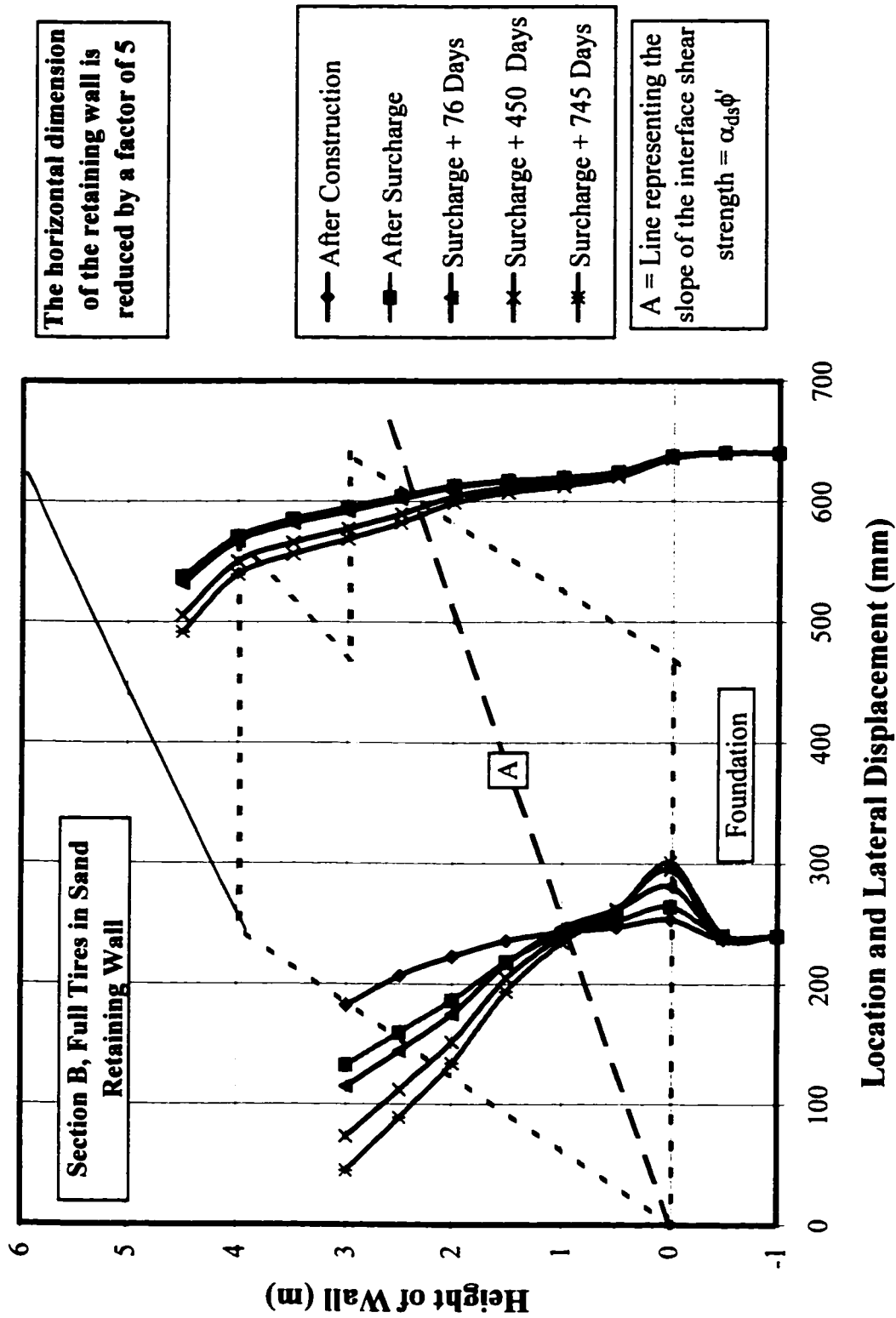


Figure 5.50. Lateral displacement of the tire wall in Section B, full tires in sand, with reference to inclinometer location and time.

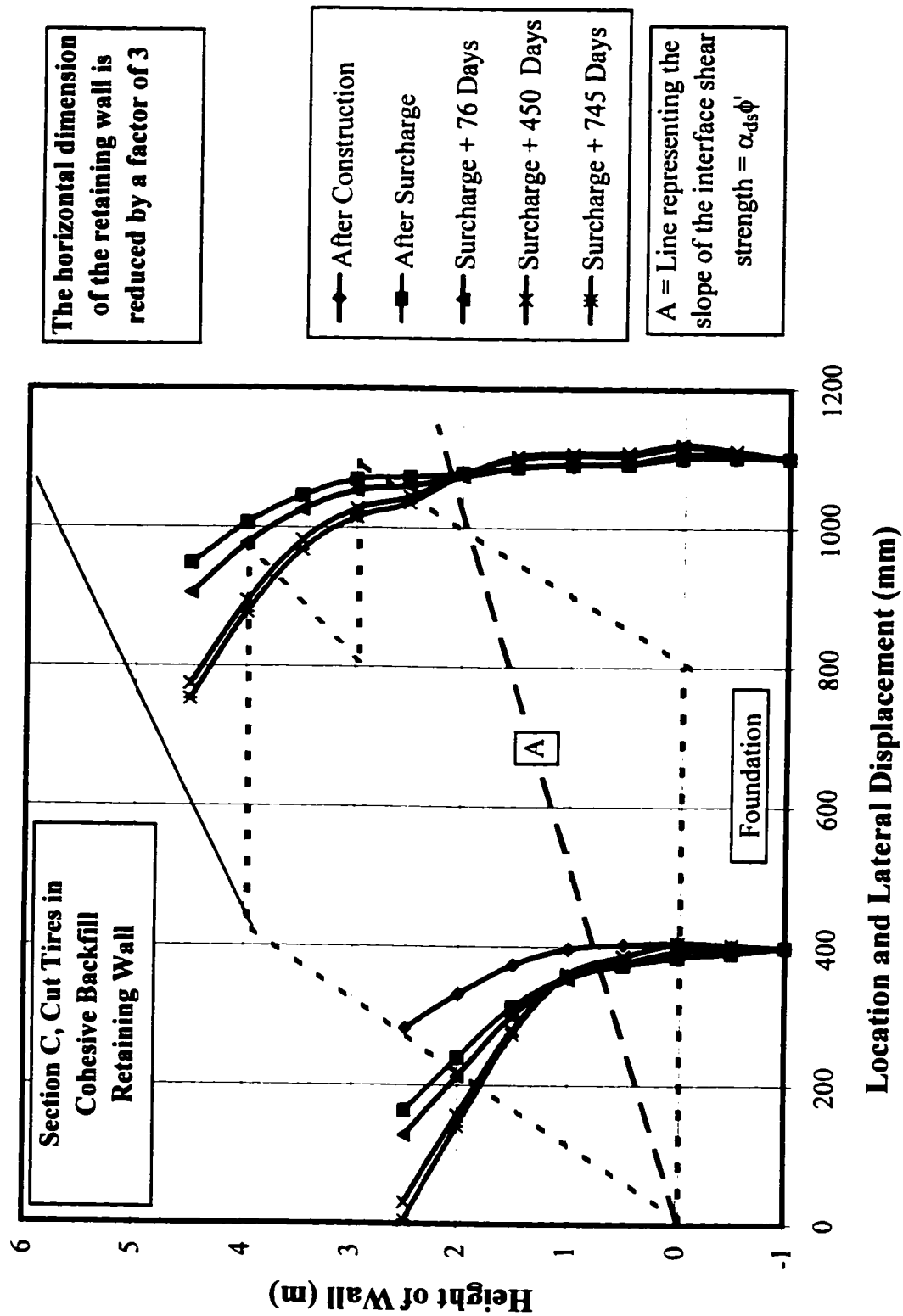


Figure 5.51. Lateral displacement of the tire wall in Section C, cut tires in cohesive backfill, with reference to inclinometer location and time.

CHAPTER 6

PARAMETRIC STUDY OF REINFORCED TIRE WALL DEFORMATION

6.1 Introduction

Results of the field analysis (Chapter 5) indicate that excessive wall deformation occurred when using a low quality backfill. The prototype embankment used conventional empirical design procedures which considered the overall stability of the structure. These design procedures do not explicitly take into account the wall deformation under working stress. Under certain conditions, limiting the amount of lateral deformation may become an important project requirement. There may be additional economical benefits of using cohesive soils (clays of medium to high plasticity) or waste materials (mine waste, pulverized fuel ash). Wall deformation might also become critical for walls with large external loadings or sloping backfills in which. Hence, an estimate of the wall deformation is desirable for a satisfactory design for some types of walls.

Unfortunately, there is no simple procedure to estimate the amount of lateral deformations which would occur in a tire reinforced retaining wall. Therefore, a parametric study based on simple finite element analyses was undertaken to determine the relative impact of several key parameters on tire wall deformation. The study examined the effects of reinforcement length, reinforcement layout, shear strength and deformability of backfill used, foundation compressibility, and the external loadings on the deformation of tire reinforced walls. The results of the study provide useful insight into tire retaining wall deformation behaviour. Based on the analysis, a simplified design procedure for estimating wall deformation can be established.

6.2 Method of Analysis

The study used a numerical approach. The analysis was performed using the commercially available computer program SIGMA/W, developed by GEO-SLOPE International. This two-dimensional finite element program is able to analyze stress and deformation problems in earth structures. The program is able to analyze a variety of conditions ranging from a simple linear-elastic case to more complex effective-stress nonlinear problems with stage loadings. However, the finite element code cannot directly model compaction induced stresses, and therefore, the analysis was limited to post-construction movement. Monitoring of the prototype structure showed that post-construction deformation, including surcharge load, accounted for on average 70% of the total measured movement.

6.2.1 Modelling of Wall Components

The stress-strain behaviour of the backfill soil was modelled as having an elastic-plastic relationship with a Mohr-Coulomb failure criterion. The tire reinforcement was modelled as an anisotropic elastic structural element. Based upon tire pull-out test results, it is desirable to consider tire reinforcement as having different stiffness values in two orthogonal directions. The interaction between soil and tire reinforcement was modelled with thin interface elements having a non-linear elastic properties. The interface stiffness increased with increasing confining stress and reinforcement length (discussed below). The foundation was modelled as a linear elastic medium; a requirement of the program when using infinite elements. A list of the different constitutive soil models used in the analyses is given in Table 6.1.

6.2.2 Selection of Model Parameters

The unit weight of the sand backfill was determined from field measurements. The internal friction angle was evaluated from direct shear box tests. However, no conventional triaxial tests were performed on the backfill soil; as a result, the stress-strain modulus had to be assumed. Bowles (1977) give stress-strain moduli for dense sand between 50 and 80 MPa, and a value of

75 MPa was selected to represent the compacted sand. A typical value of 0.25 was chosen for the Poisson's ratio, ν . Abdurahman et al. (1995) performed a numerical analysis on the behaviour of retaining walls reinforced with tires, using a homogenization procedure. The authors presented a list of figures relating the different parameters (E_x , E_y , G) as function of the material properties of both the tire and backfill soil. Values representing a single tire reinforcing element was used. The interface parameters were evaluated from interface direct-shear tests between tire rubber and soil presented in Chapter 4, section 4.3. The relationship between the shear stress and the relative displacement at the interface between tire rubber and sand was expressed using the hyperbolic formulation presented by Clough and Duncan (1971). The following values were obtained, a shear stiffness number (K) of 65, a shear stiffness exponent (n) of 0.65, and a failure ratio (R_f) of 0.97. Estimation of the numerical interface parameters from pull-out tests was not possible due to the high extensibility of the tire reinforcement. It is difficult to separate the interface deformation characteristics from the actual physical elongation of the reinforcement (Wilson-Fahmy and Koerner 1994). Hence, pull-out tests are performed on short reinforcement length to determine the appropriate interface parameters. The test embankment was built over a stiff foundation, and therefore, stress-strain modulus of a 100 MPa was chosen.

To validate the model parameters, a finite element analysis for the retaining wall was performed and compared to the measured displacements observed in the tire wall in section A (cut tires and sand), minus the lateral deformations associated with construction activities. To produce a comparable profile, in terms of displacement and deformation characteristics, the interface stiffness modulus number of 65 had to be increased by 30% for a reinforcement length of 2.4 m. It should be recalled that the upper portion of the retaining wall was constructed with a reinforcement length of 1.8 m (3 tire width) while the bottom portion used a reinforcement length of 2.4 m (4 tire width). This result demonstrated that the interface stiffness may be influenced by the length of the reinforcement. A similar behaviour was observed during pull-out of tire mat reinforcement of various lengths. The secant tensile modulus (force per tire width) measured at a 5% strain showed a 30% increase in stiffness per unit length of tire reinforcement, for tests performed in the sand backfill (Figure 6.1). The interface stiffness number K was, therefore, increased by 30% for each additional unit tire length (0.6 m) from the base value of 65. A K

value of 65 was assumed to represent a reinforcement length of 3 tires (1.8m). A list of the key parameter values for the backfill soil, tire reinforcement, soil-reinforcement interface and foundation is given in Table 6.1.

6.2.3 Validation of the Model

The measured and simulated lateral deformation at 1.2 m from the toe of the retaining wall in section A, cut tires placed in sand, is shown in Figure 6.2. The predicted wall facing deformations were quantitatively in close agreement with the amount of lateral movement (quantitative) but did not quite match the shape of the deformation profile when compared to the field values.

The model was able to match the observed quantitative lateral deformations of the tire wall in section C, cut tires in cohesive soil. The stress-strain modulus of the backfill soil was evaluated at about 18 MPa (prior to construction) for clay, based on consolidation tests (Figure 3.14). The tire reinforcement anisotropic parameters were adjusted accordingly and the interface stiffness coefficient (K) was reduced by a factor of 4 (the stress-strain modulus ratio of the two soils). The remaining parameters were determined as outlined in Chapters 3 and 4. Therefore, the model is able to predict wall deformations for various soil types having different strength and deformation characteristics.

However, since the interface stiffness parameter (K) was calibrated with field observations corresponding to one structural geometry and the lack of any further validation from field structures, the model predictions can not be considered as absolute values. This type of analysis is classified as a "Class C1" prediction by Lambe (1973). Thus, the program was used to study relative, rather than absolute, wall deformations. Hence, the results of the analysis could be used to provide a reasonable estimate of the expected lateral deformation to occur within a tire retaining wall structure after construction.

6.3 Cases Studied

The influence of the various parameters on lateral deformation were compared against a baseline case. The baseline case (wall 1) is 6 m high with a uniform tire reinforcement length of 4.2 m, providing a length to height ratio (L/H) of 0.7. The wall configuration and soil properties are given in Figure 6.3. The major elements of the mesh used for the analysis of the baseline case are shown in Figure 6.4. The complete FEM mesh for the baseline case consisted of 900 nodes, defining 345 soil elements, 168 tire reinforcing elements, and 168 interface elements.

The parametric study was performed by selectively varying input parameters and comparing the predicted deformation with those obtained for the baseline case. A list of the various cases analyzed is given in Table 6.2. In addition to varying strength and deformation characteristic of the backfill soil, different wall geometries were also investigated, such as reinforcement length, reinforcement layout and sloping wall. The length of the tire reinforcement was varied uniformly from 3 to 9 m, as shown in Figure 6.5. The three different reinforcement layouts are shown schematically in Figure 6.6. A shorter reinforcement length of 1.8 m was prescribed for the upper 3 m of the wall while the bottom half remained at 4.2 m (Wall 5). Wall 6 is the reverse of Wall 5, a shorter reinforcement length of 1.8 m was used in the bottom half of the wall while the upper 3 m remained at 4.2 m. For the trapezoidal Wall 7, the reinforcement length was extended beyond the stable slope ($\phi' = 40^\circ$). The slope of the tire reinforced wall was varied between 80° and 70° from the horizontal while maintaining the reinforcement length at 4.2 m (Figure 6.7). The effects of a uniform surcharge load and sloping backfill on lateral wall deformation were examined. The influence of foundation settlement on wall face movement was also explored. Results from the analysis indicated that none of the cases considered above gave evidence of being close to an overall failure. Factors of safety were above unity for the various failure criterion (base sliding, overturning). The minimum safety value of 1.37 against base sliding was evaluated for the baseline wall when the backfill soil had an effective friction angle of 25° .

6.4 Results

The predicted wall face deformations (Δ) have been normalized in terms of wall height (Δ/H , %). A Deformation Index (DI) was used to relate the predicted maximum lateral deformations of each case (Δ) to maximum lateral deformation of the baseline case. Therefore, DI is defined as $\Delta/\Delta_{\text{baseline}}$. A DI of 2 means a 100% increase in maximum wall face movement compared to the baseline case. The value reported here represent post-construction deformations and do not include compaction induced movements. A summary of the analysis is given in Table 6.3.

The Baseline Case

The lateral wall face deformation profile for the baseline case is shown in Figure 6.8. A view of the deformed mesh is given in Figure 6.9 where the deformations are exaggerated by a factor of 10 for clarity. The original outline of the structure is also shown. The magnitude of the predicted maximum deformation is 42 mm ($H/140$) which is similar to expected values for conventional reinforced walls which varies between $H/250$ for inextensible reinforcement and $H/75$ for extensible reinforcement (Christopher et al. 1990).

The baseline wall was also modelled in 3 steps of 2 m in height to simulate stage construction. Stage construction increased the outward movement by 12% , representing 5 mm, compared to the 1 step baseline case. Based on field observations of the test embankment, compaction induced deformations during construction can be estimated at 60 mm. This represents a twelve fold increase compared the additional movements produced by the simulated stage construction. Hence, analysis in steps could not properly model lateral deformation resulting from compaction induced stresses. The parametric study was, therefore, limited to post-construction deformations.

Reinforcement Layout

The tire reinforcement layout influences the magnitude and shape of the deformation pattern (Figure 6.8). For shorter reinforcement at the top half of the wall (Wall 5), there is a distinct increase in the outward deflection of the wall face, above a wall height of 3 m. This structural geometry resulted in a deformation index of about 1.5. Similar results were reported by Chew

and Schmertmann (1990) from a numerical analysis of conventional wall using inextensible reinforcement. In contrast, the shortening of the reinforcement in the bottom half reduced the outward wall deflection by about 100%. The longer reinforcement at the top of the wall forced the active soil wedge to be developed along the back of the retaining wall equal to the top reinforcement length. As a result, the soil elements located within the lower portion of the wall acted as part of the retaining structure. And since the soil stiffness parameters were higher than those of the interface, the overall stiffness of tire wall was increased. This type reinforcement layout was not used in the test embankment, and therefore, a comparison is not possible.

The lateral deformation were further decreased for the trapezoidal reinforcement layout in which the reinforcing length was extended beyond the stable slope (Wall 7). A deformation index of 0.21 was reported, representing a 360% reduction in lateral movement. This reinforcement layout generated the least amount of post-constructional movement. Rowe and Ho (1993) stated that minimal lateral movement in a conventional reinforced wall will occur when the unreinforced zone above the stable slope (ϕ'). Bathurst et al. (1988) reported negligible movement for a conventional reinforced wall in which the reinforcement length to wall ratio (L/H) was much greater than the inverse of the slope of the fill friction angle ($L/H = 1 > \cot \phi' = 0.75$).

Reinforcement Length

The lateral deformations are strongly influenced by the overall dimensions of the tire wall, similar to those observed for conventional reinforced walls (Christopher et al. 1990; Rowe and Ho 1993). As the reinforcing length to wall height ratio (L/H) was varied from 0.5 to 1.5 for the baseline case, the deformation index ranged from about 1.5 to 0.3, as shown in Figure 6.10. The corresponding decrease in lateral wall deformation is again attributed to the decreased unreinforced wedge area above the stable slope and the increase in wall stiffness resulting from longer tire reinforcements. It is interesting to note that the reduction in wall movement is no longer proportional to the reinforcing tire mat length for a L/H ratio greater than one. This may indicate that a further reduction in wall movement, greater than 250% when compared the baseline case, can be difficult to achieved by solely increasing the reinforcement length.

Wall Batter

The lateral deformations were reduced by 60% when the wall was inclined at 80 degrees and a further decrease of 220% , compared to the baseline case, was achieved when the wall was inclined at 70 degrees (Figure 6.11). The corresponding deformation indices were 0.62 and 0.31 respectively. The length of the reinforcement was maintained constant at 4.2 m. The decrease in wall face movement is associated with the reduction in the active thrust behind the wall due to the wall batter. This type of reinforcement layout is beneficial since it provides protection against tire overhang, i.e. maintains a positive slope during construction and can accommodate large deformation before the top of the wall has moved beyond the toe. It also has the additional benefit of reducing wall face movement while employing a reasonable length of tire reinforcement. For example, to produce a DI of 0.3 in a vertical wall, a reinforcing length of 9 m must be employed, while for a wall inclined at 70°, under identical conditions, a reinforcing length of 4.2 m is required. A reduction in the required length of tire reinforcement will significantly decrease the overall cost of the tire wall structure. This can be particularly important when dealing with soils of inferior quality.

Shear Strength of the Backfill

The internal friction angle (ϕ') was varied from 25° to 45° and the inclination of the baseline wall ranged between 70 and 90 degrees. Note that the remaining model parameters were kept constant. For the baseline wall, where the L/H ratio was 0.7, the corresponding deformation index ranged from 8.3 to 0.74 (Figure 6.12). A marked increase in the predicted wall deformation occurred for friction angles of less than 35 degrees. The reduction in shear strength of the backfill resulted in a larger force being generated within the reinforced zone to maintain equilibrium (increase in the active thrust). Thereupon, a greater portion of the shear strength is mobilized which induces larger deformations. The decrease in the friction angle also produced a larger zone of unreinforced soil mass above the stable slope and behind the retaining wall, confirmed by a $\cot \phi'$ value between 1.43 and 2.15 ($\phi' = 35^\circ$ to 25°), which is substantially greater than the L/H ratio of 0.7. It should be noted that in order to minimize the influence of the unreinforced zone, L/H ratio should equal $\cot \phi'$. An inclination of 80° reduced the predicted

wall deformation by a factor of approximately 2, when compared to the baseline case. A wall having a batter of 70° , further reduces the lateral movements by approximately 200%, for internal friction angles of less than 35 degrees.

The important implications of Figure 6.12 are that a substantial wall deformation can occur for soils having inferior strength properties. The selection of reinforcement length therefore becomes more critical in soils with lower friction angles. Also, providing a batter can significantly reduce wall deformation.

Deformability of the Backfill

The elastic modulus of the backfill was varied from 5 to 100 MPa, while the other model parameters were kept constant, as in the baseline case. The elastic-plastic relationship used to model the behaviour of the backfill, the elastic modulus parameter represents the initial linear-elastic stiffness of the soil. The deformation index varied from 12.1 to 0.74 for the baseline case (Figure 6.13). The deformation index curve demonstrated an exponential growth trend with a sharp increase at an elastic modulus of 30 MPa. The analysis demonstrates the importance of proper compaction of low quality backfill and a good control of the water content, especially for clay soil of high plasticity. Again the introduction of a batter to the baseline wall significantly reduced the outward deflection of the wall. Another important aspect of the analysis is that both curves indicated that no significant benefits would be derived with the use of very stiff soils. However, good quality backfills have superior strength characteristics, and as demonstrated in Figure 6.12, the internal friction angle has a great influence of the performance of the tire retaining wall. It is important to note, that soils of inferior quality have greater deformability which is usually accompanied by a lower shear strength. For example, the cohesive backfill used in the prototype embankment had an elastic modulus of 18 MPa and an internal friction angle of 32 degrees. In this case, the deformation index of the soil would be a combination of the two parameters. Hence, an estimate of the deformation index for the same wall geometry as the baseline case would be 12 (DI of 3 for ϕ' x DI of 4 for E) which would be equivalent to a normalized lateral deformation (Δ/H) of 8.3%.

Foundation Compressibility

The effects of a compressible foundation are demonstrated in Figure 6.14. The local overstressing of the foundation beneath the toe or overall settlement of the foundation cause additional movements to occur at the wall face. The wall deformation is dependent on the strength and stiffness of the foundation. A reduction in the foundation stiffness of the baseline case by a factor of 3 produced an increase the wall movement by 66%. A compressible foundation having a stress-strain modulus of 10 MPa resulted in a deformation index of 2.35. If the competence of the foundation is in question, a finite analysis should be performed to evaluate the effect of foundation deformations.

Uniform Surcharge

The provision of a uniform surcharge increased the outward movement of the wall while maintaining the same deformed shape attributes as at the zero-surcharge condition. The increase in wall deformation was proportional to the surcharge magnitude. For the baseline wall configuration, each 20 kN/m^2 increment of surcharge increased the maximum wall deformation by about 40% of the baseline value for the range of surcharge up to 80 kN/m^2 (Figure 6.15). Indicating that the placement of a uniform surcharge can produce a substantial amount of post-construction movement.

Sloping Backfill

The existence of a sloping backfill is very common in retaining wall applications. The backfill slope angle, α , was varied from 0 to 25 degrees; the corresponding deformation index ranged from 1 to 2 (Figure 6.14). There was a slight decrease of the wall face movement for a backfill slope angle greater than 10 degrees. This decrease in wall face movement was associated with the extent of the finite element mesh generated behind the retaining wall which was kept constant for all slope angles. A greater extent of the mesh may have been required to minimize influences from the imposed boundary conditions. Therefore, the increase in wall face deformation could be proportional to the backfill angle. For the baseline layout, an increase in the slope of the backfill for every 5 degrees will increase the maximum deformation by about 25% of the baseline value for the range of slope angles up to 25 degrees.

6.5 Estimation of Wall Deformation

A preliminary estimate of the tire retaining wall deformation can be computed by simply multiplying the standard deformation by the appropriate deformation indices, provided in Figure 6.8 and Figures 6.10 to 6.16. The combined effects of wall geometry, backfill strength and deformation characteristics, and external loading conditions, such as reinforcement length, batter, and uniform surcharge can be estimated by multiplying the deformation indices in series. With such an estimate of the possible deformation, the designer can modify the wall design to meet the deformation requirements of the project, such as adjusting the construction wall batter. These design aides may be more useful, especially when dealing with soils of inferior quality, to determine the overall dimensional requirements of the retaining wall. Other verifications for base sliding and overturning failures would also be required.

Table 6.1. Constitutive models and key parameters used in the baseline wall.

Constitutive Models and Parameters	Material			
	Backfill Soil	Tire Reinforcement	Soil- Reinforcement Interface	Foundation
Constitutive Model	Elastic-Plastic	Anisotropic Linear Elastic	Non-linear Elastic (Hyperbolic)	Linear Elastic
Parameters				
Unit Weight (kN/m^3)	18	17	0	0
Friction Angle, ϕ (Degrees)	40	n.a.	36	n.a.
Young's Modulus, E (MPa)	75*	—	—	100
E_x (MPa)	—	150	—	—
E_y (MPa)	—	225	—	—
Poisson Ratio (ν)	0.25		0.25	0.25
ν_x	—	0.30	—	—
ν_y	—	0.15	—	—
Shear Modulus, G (MPa)	—	50	—	—
Modulus Number, K	—	—	#	—
Modulus Exponent, n	—	—	0.65	—
Failure Ratio R_f	—	—	0.97	—

* Represents the initial linear-elastic stiffness of the soil

Stiffness of the interface was a function of the reinforcement length (L).

Number of Tire Elements	Reinforcement Length (m)	Modulus Number, K (Dimensionless)
3	1.8	65
4	2.4	85
5	3.0	110
6	3.6	145
7	4.2	190

The initial modulus number of 65 was based upon direct-shear tests interface results. Calibration of the finite element model with field results (embankment observations and pull-out test) indicated that the initial stiffness modulus number increased by 30% for each additional tire length (0.6 m), from the base value of 65 which represented a reinforcement length of 3 tires (1.8m).

Table 6.2. A list of cases studied.

Wall #	Cases Studied
1	Baseline
2	Reinforcement Length = 3.0 m
3	Reinforcement Length = 6.0 m
4	Reinforcement Length = 9.0 m
5	Short Reinforcement at the Top
6	Short Reinforcement at the Bottom
7	Trapezoidal Wall with a Reinforcement Length = $\cot \phi'$
8	Staggered Wall, inclined at 80° Reinforcement Length = 4.2 m
9	Staggered Wall, inclined at 70° Reinforcement Length = 4.2 m
1	Compressible Foundation
1	Uniform Surcharge, Varied from 20 to 80 kPa
1	Sloping Surcharge, Varied from 5 to 25 degrees
1	Shear Strength of the Backfill, Varied from 25 to 45 degrees
8	Shear Strength of the Backfill, Varied from 25 to 45 degrees
9	Shear Strength of the Backfill, Varied from 25 to 45 degrees
1	Deformability of the Backfill, The Elastic Modulus Was Varied from 10 to 100 MPa
8	Deformability of the Backfill, The Elastic Modulus Was Varied from 10 to 100 MPa
9	Deformability of the Backfill, The Elastic Modulus Was Varied from 10 to 100 MPa

Table 6.3. Summary of analyses.

Wall #	Description	Maximum Lateral Wall Face Deformation (mm)	Normalised Wall Face Deformation (Δ/H , %)	Deformation Index ($\Delta/\Delta_{\text{baseline}}$)
1	Baseline	42	0.70	1.00
2	Reinforcement length = 3.0 m	62	1.03	1.48
3	Reinforcement length = 6.0 m	24	0.40	0.57
4	Reinforcement length = 9.0 m	12	0.20	0.29
5	Short Reinforcement at the top	64	1.07	1.52
6	Short Reinforcement at the bottom	20	0.33	0.48
7	Trapezoidal Wall Reinf. Length = $\cot \phi$	9	0.15	0.21
8	Staggered Wall, inclined at 80° Reinforcement Length = 4.2 m	26	0.44	0.62
9	Staggered Wall, inclined at 70° Reinforcement Length = 4.2 m	13	0.21	0.31
1	Stress-Strain Modulus of Foundation = 25 MPa	70	1.17	1.66
1	Stress-Strain Modulus of Foundation = 10 MPa	99	1.64	2.35
1	Uniform Surcharge = 20 kPa	65	1.08	1.55
1	Uniform Surcharge = 40 kPa	85	1.42	2.02
1	Uniform Surcharge = 60 kPa	95	1.58	2.26
1	Uniform Surcharge = 80 kPa	110	1.83	2.62
1	Sloping Surcharge, 5 Degrees	54	0.90	1.29
1	Sloping Surcharge, 10 Degrees	66	1.10	1.57
1	Sloping Surcharge, 15 Degrees	74	1.23	1.76
1	Sloping Surcharge, 20 Degrees	80	1.33	1.90
1	Sloping Surcharge, 25 Degrees	85	1.42	2.02
1	Friction Angle of Backfill, $\phi = 25^\circ$	349	5.81	8.30
1	Friction Angle of Backfill, $\phi = 30^\circ$	185	3.08	4.40
1	Friction Angle of Backfill, $\phi = 35^\circ$	68	1.13	1.62
1	Friction Angle of Backfill, $\phi = 45^\circ$	31	0.52	0.74

Table 6.3. Summary of analyses (Continued).

Wall #	Description	Maximum Lateral Wall Face Deformation (mm)	Normalised Wall Face Deformation (Δ/H , %)	Deformation Index ($\Delta/\Delta_{\text{baseline}}$)
8	Friction Angle of Backfill, $\phi = 25^\circ$	170	2.83	4.05
8	Friction Angle of Backfill, $\phi = 30^\circ$	90	1.50	2.14
8	Friction Angle of Backfill, $\phi = 35^\circ$	37	0.62	0.88
8	Friction Angle of Backfill, $\phi = 45^\circ$	19	0.32	0.45
9	Friction Angle of Backfill, $\phi = 25^\circ$	100	1.67	2.38
9	Friction Angle of Backfill, $\phi = 30^\circ$	41	0.68	0.98
9	Friction Angle of Backfill, $\phi = 35^\circ$	18	0.30	0.43
9	Friction Angle of Backfill, $\phi = 45^\circ$	10	0.17	0.24
1	E Modulus of Backfill = 5 MPa	508	8.47	12.10
1	E Modulus of Backfill = 10 MPa	298	4.97	7.10
1	E Modulus of Backfill = 20 MPa	160	2.67	3.81
1	E Modulus of Backfill = 40 MPa	79	1.32	1.88
1	E Modulus of Backfill = 60 MPa	53	0.88	1.26
1	E Modulus of Backfill = 100 MPa	31	0.52	0.74
9	E Modulus of Backfill = 5 MPa	140	2.33	3.33
9	E Modulus of Backfill = 10 MPa	80	1.33	1.90
9	E Modulus of Backfill = 20 MPa	44	0.73	1.05
9	E Modulus of Backfill = 40 MPa	21	0.35	0.50
9	E Modulus of Backfill = 60 MPa	16	0.27	0.38
9	E Modulus of Backfill = 100 MPa	10	0.17	0.24

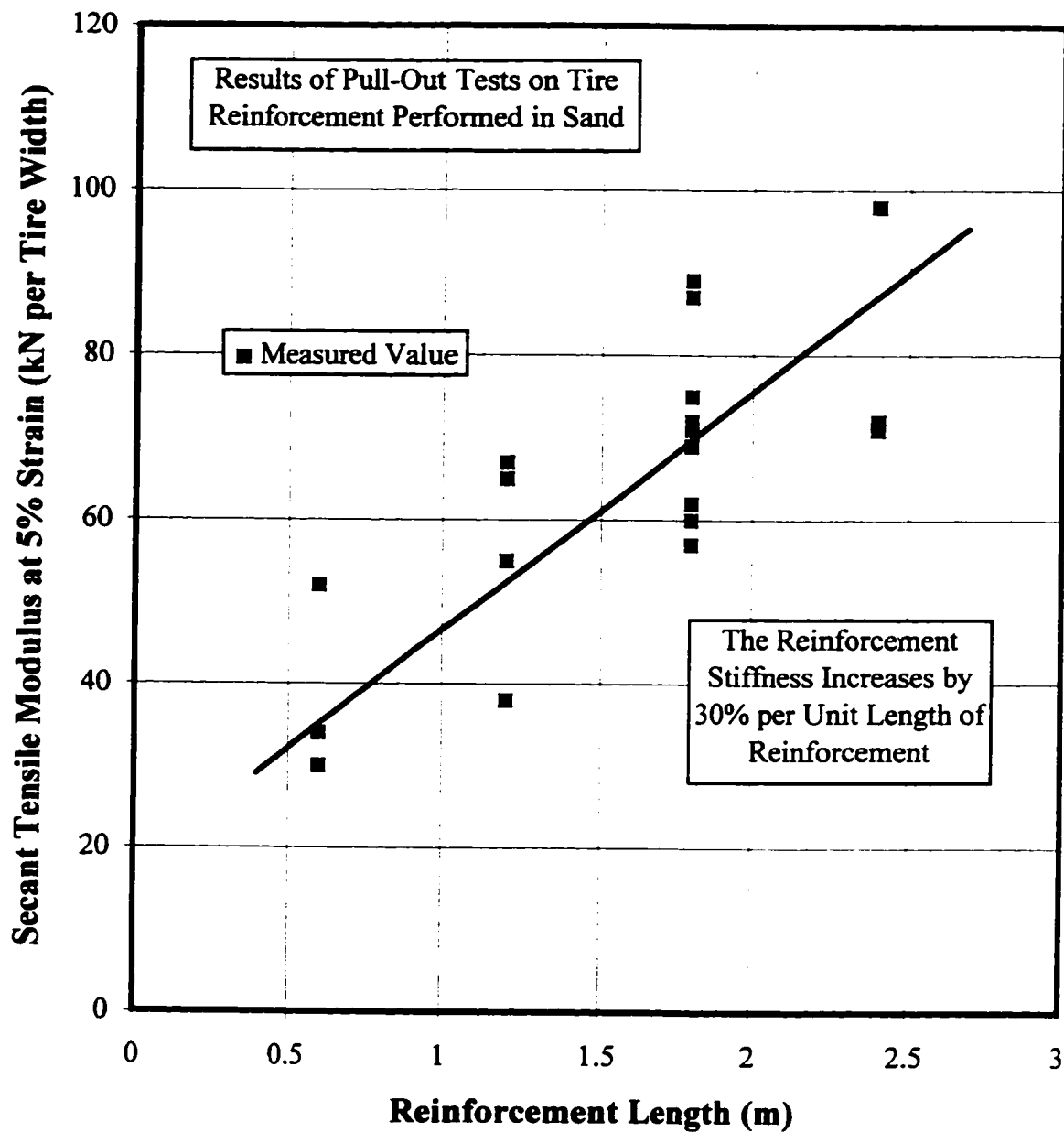


Figure 6.1. The relationship between tire reinforcement stiffness and reinforcement length based on pull-out tests results performed in sand.

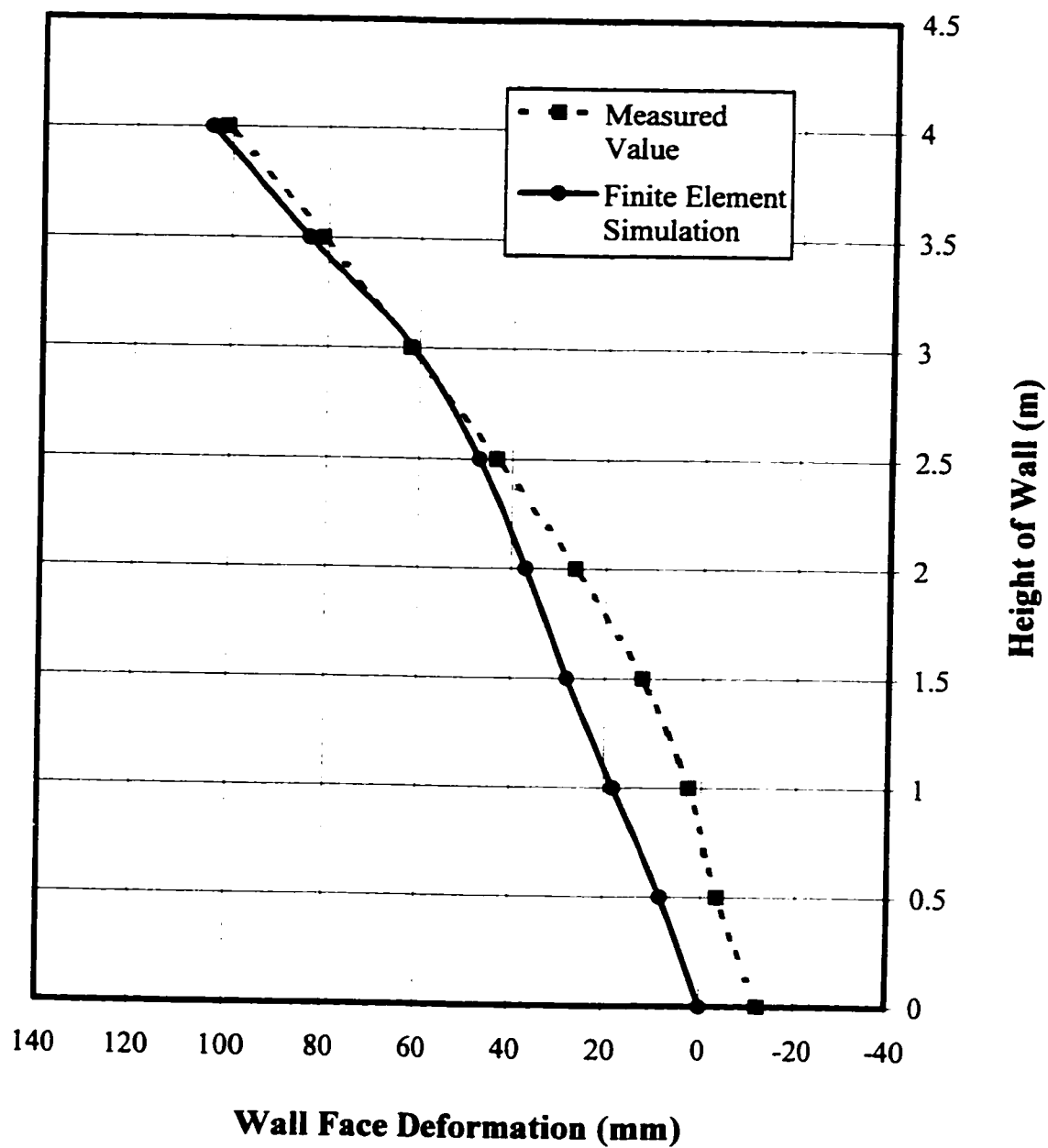


Figure 6.2. Measured and simulated lateral deformation at 1.2 m from the toe of the retaining wall in section A, cut tires placed in sand.

BASELINE CASE: WALL 1 $L/H = 0.7$

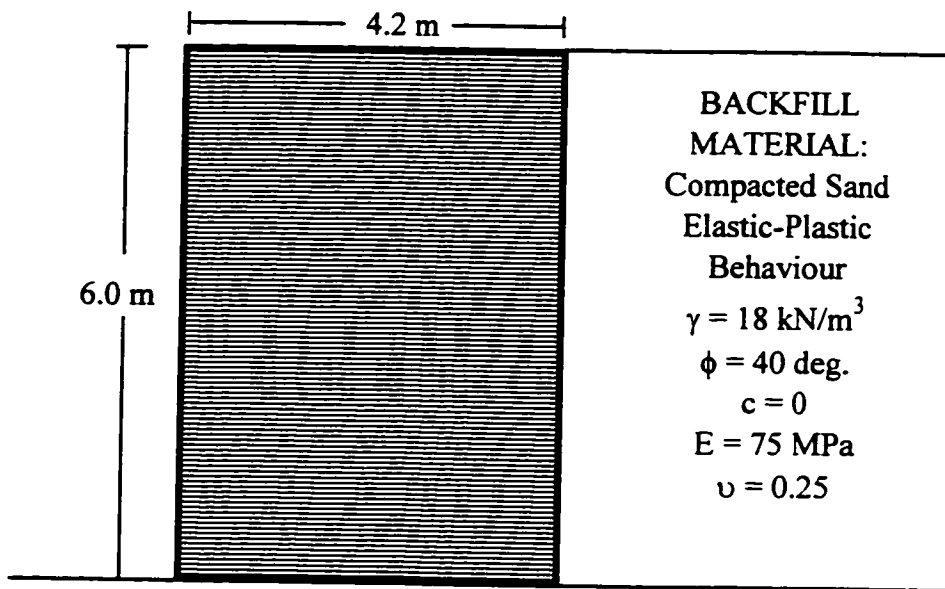


Figure 6.3. Baseline case for finite element analysis: Wall 1.

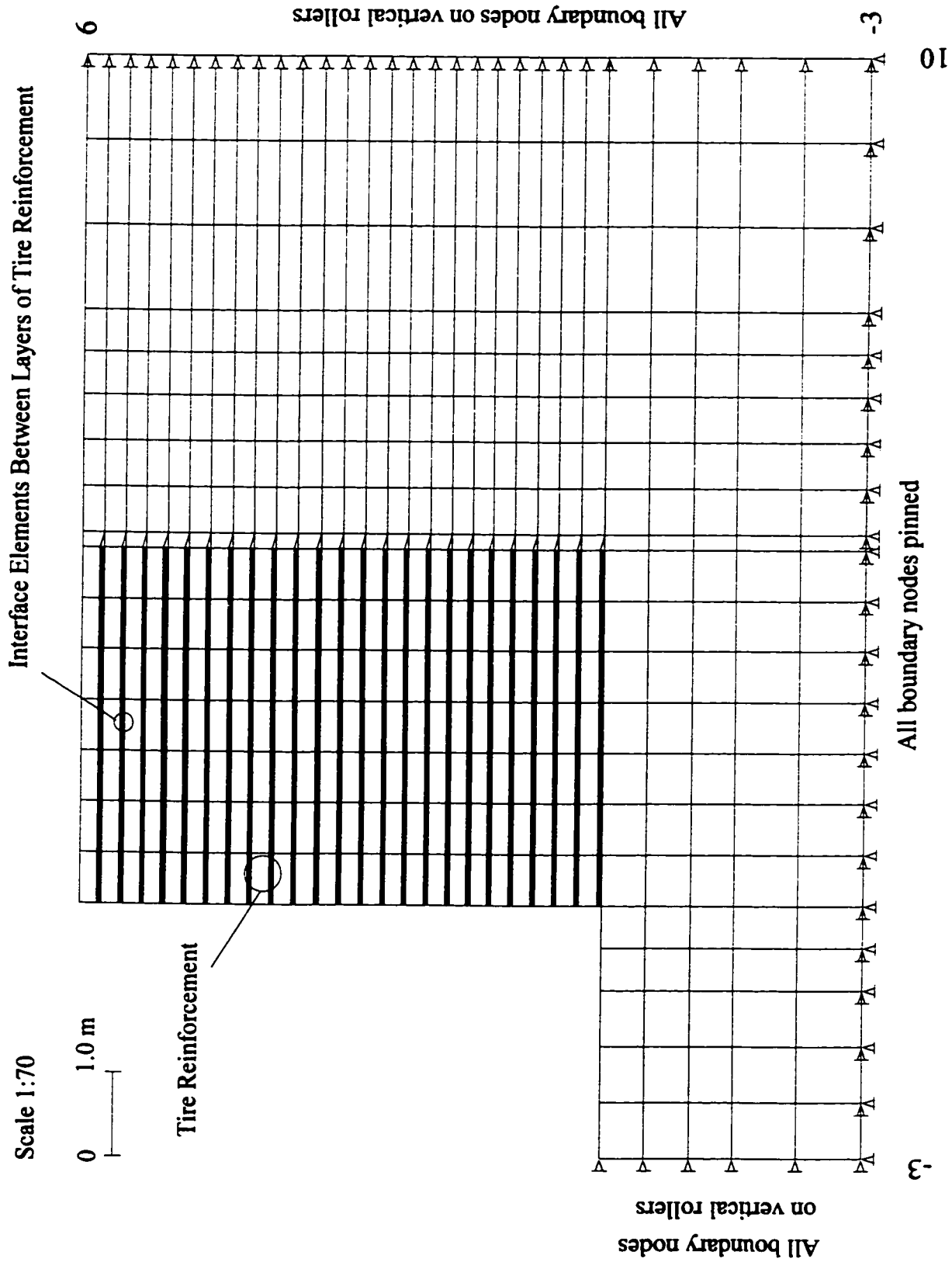
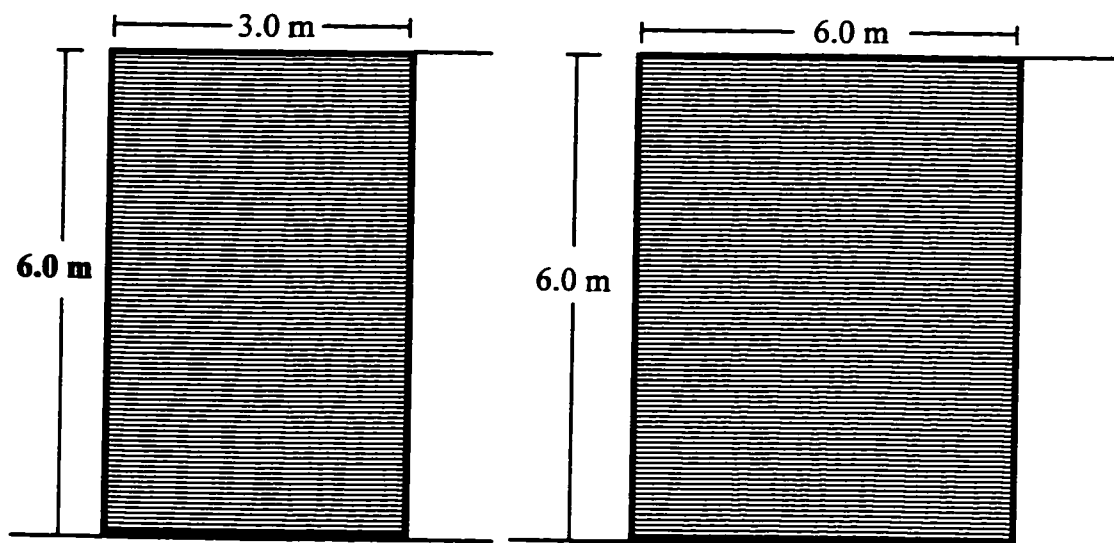
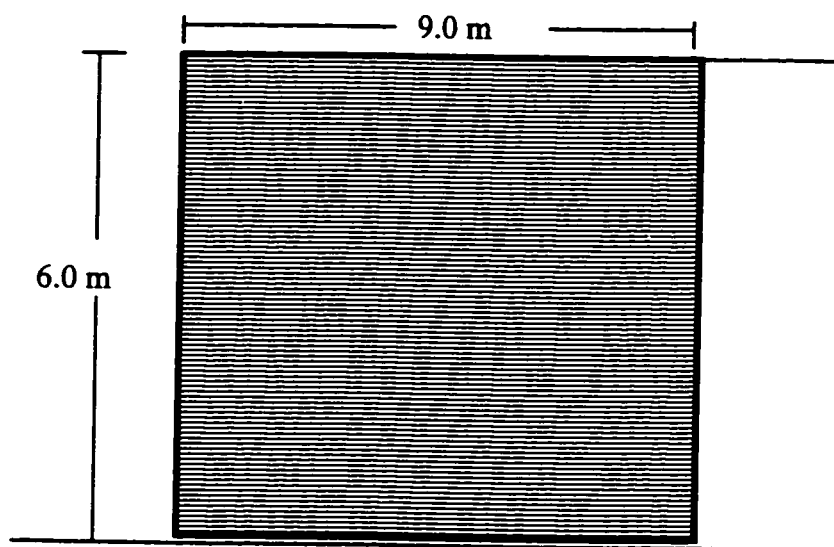


Figure 6.4. The finite element mesh for the baseline case.



WALL 2 $L/H = 0.5$

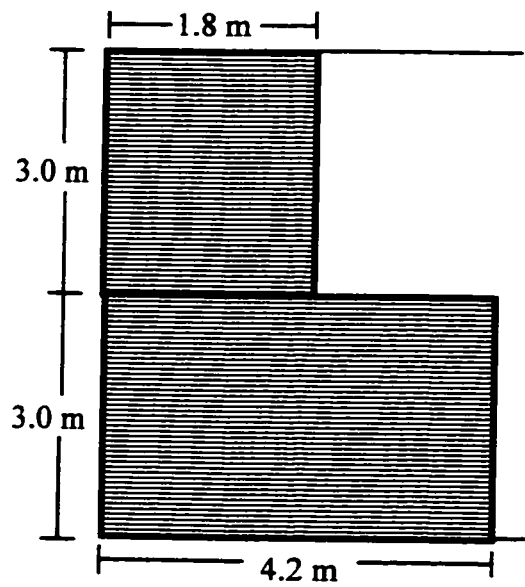
WALL 3 $L/H = 1.0$



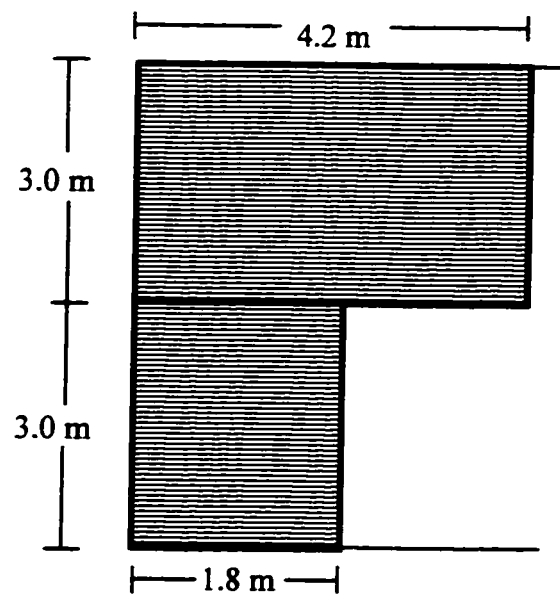
WALL 4 $L/H = 1.5$

Soil Properties: Same as Baseline Wall

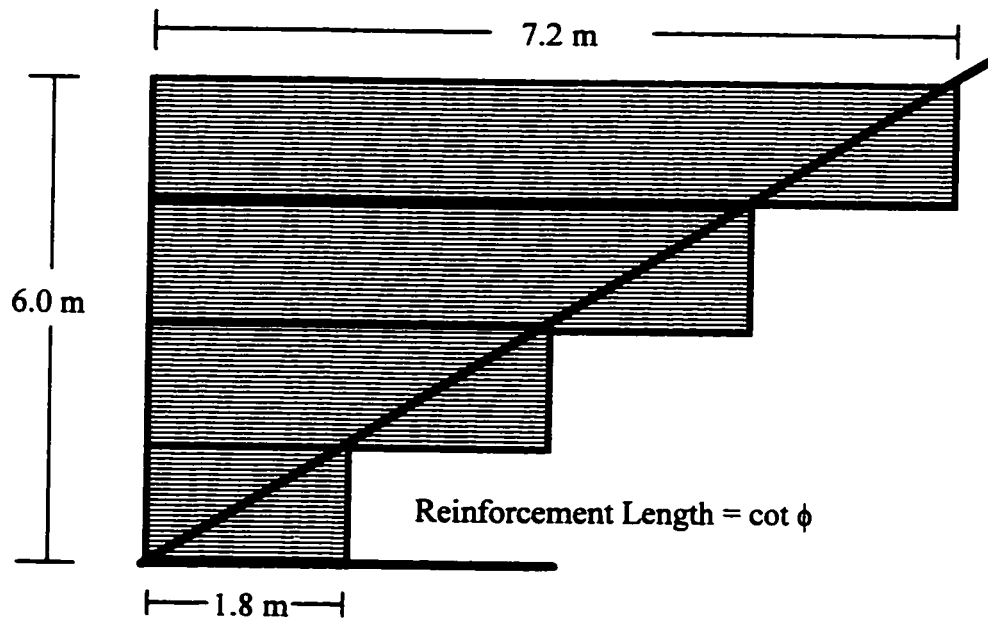
Figure 6.5. Variation in reinforcement length.



WALL 5



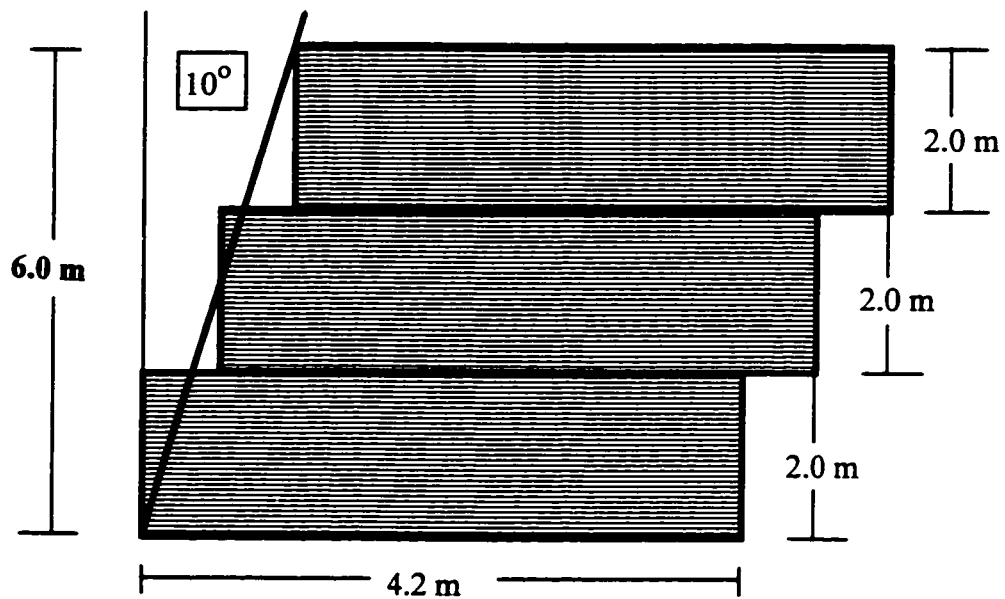
WALL 6



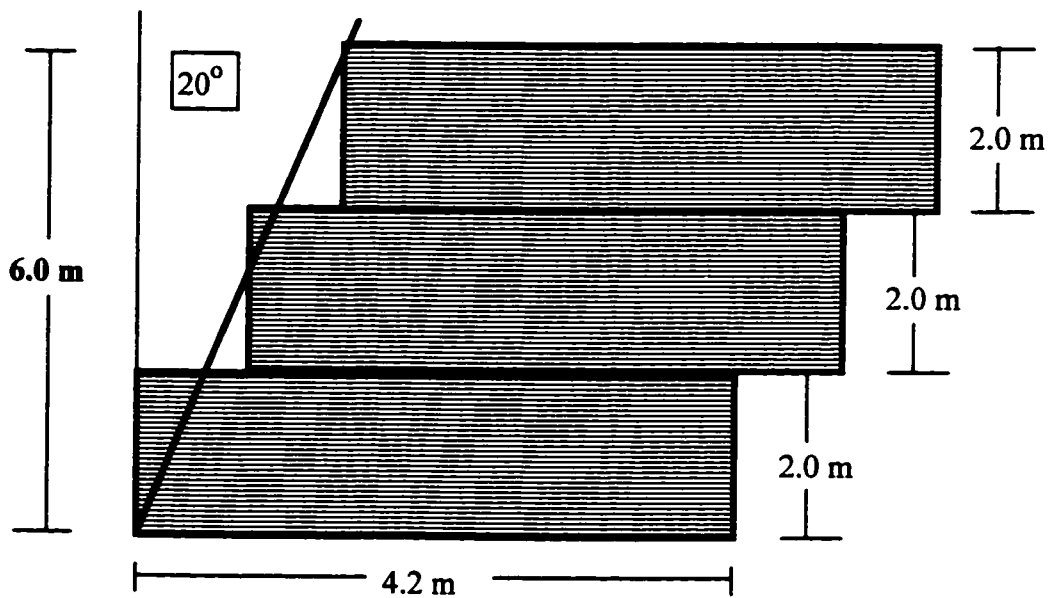
WALL 7

Soil Properties: Same as Baseline Wall

Figure 6.6. Variation in reinforcement layout.



WALL 8 Staggered Wall, inclined at 80 degrees



WALL 9 Staggered Wall, inclined at 70 degrees

Figure 6.7. Structural geometry used in sloping walls, 10° and 20° batter.

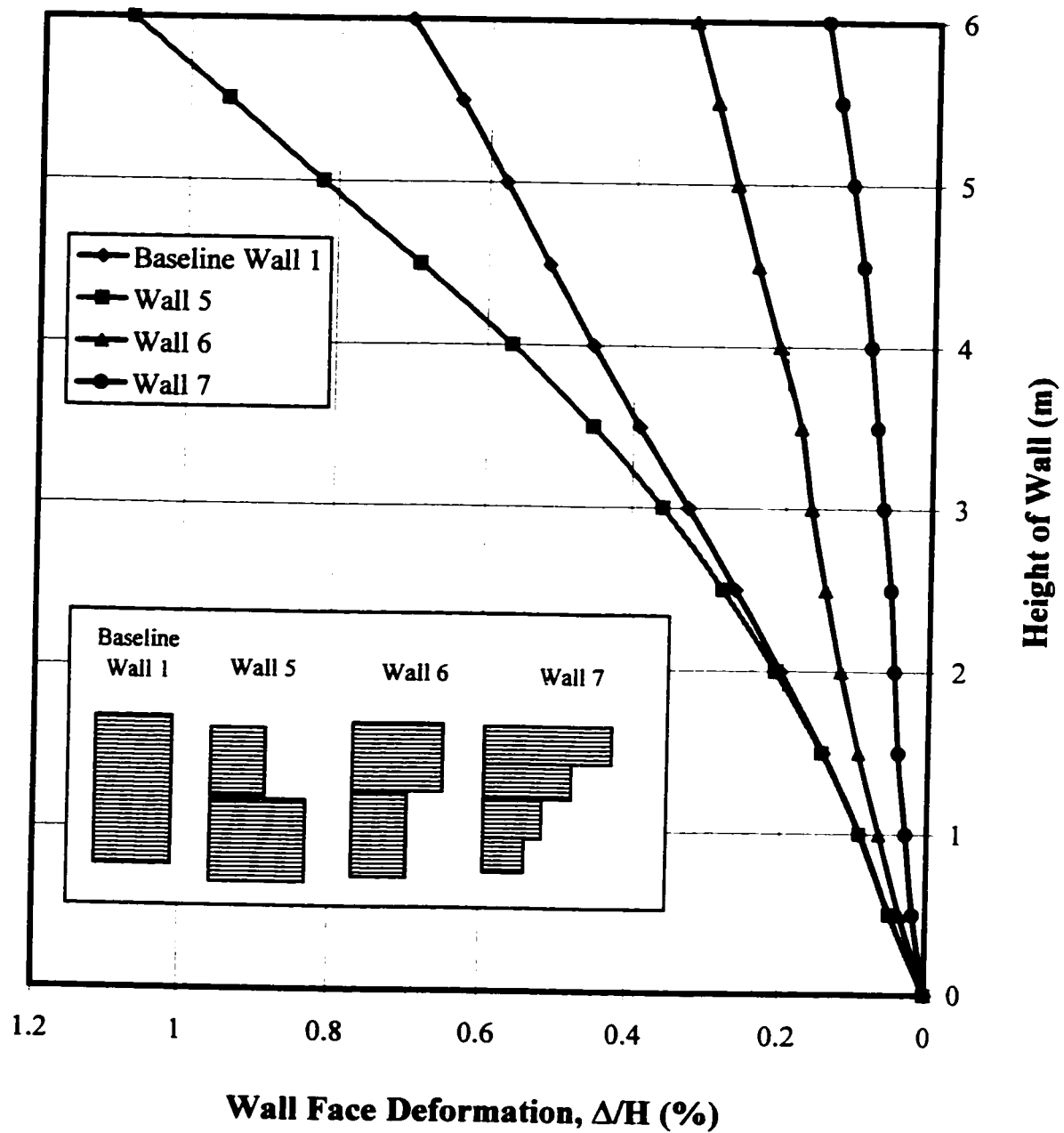


Figure 6.8. Lateral wall face deformation for different reinforcement layouts.

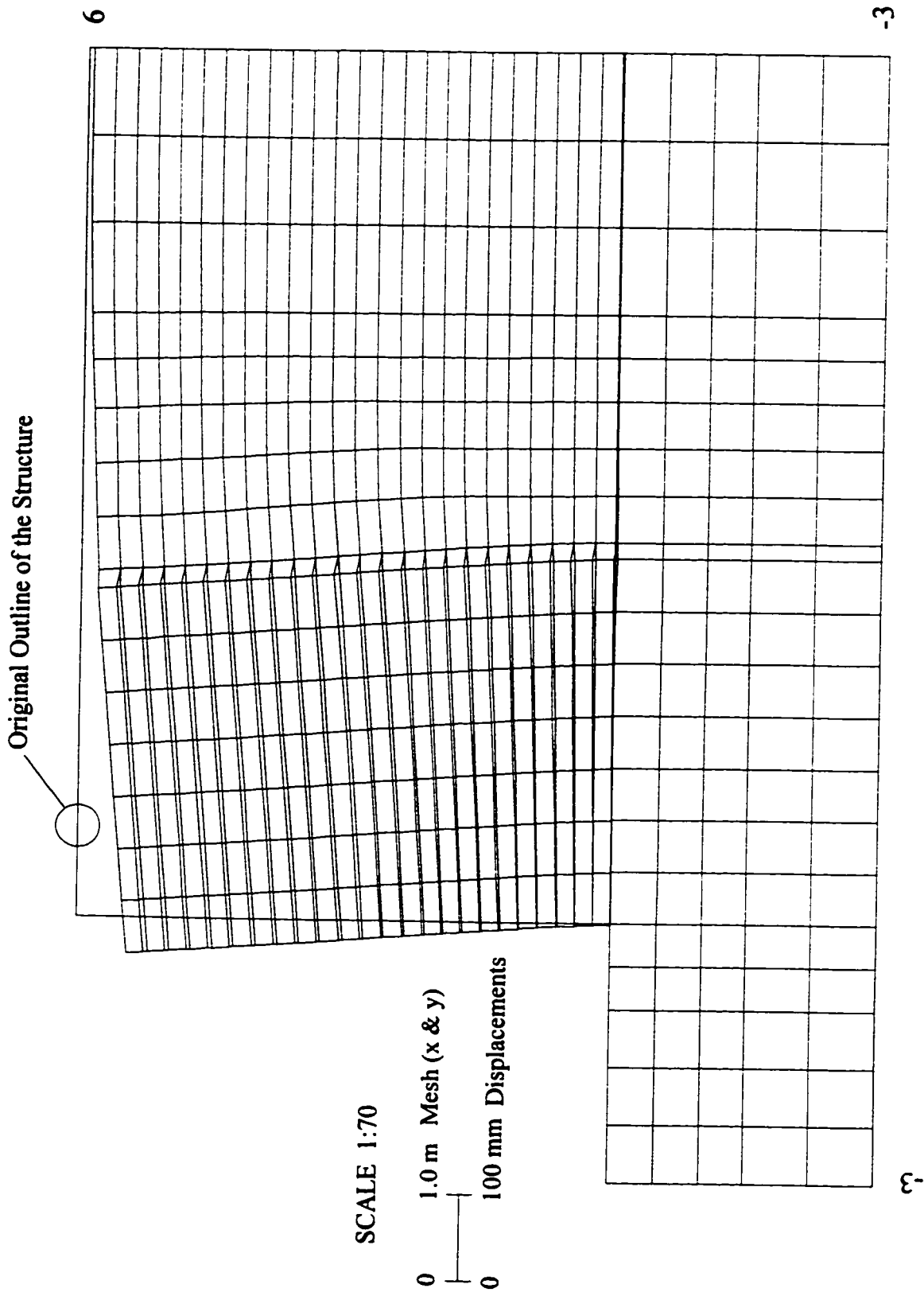


Figure 6.9. Deformation mesh for the baseline wall (no external loading).

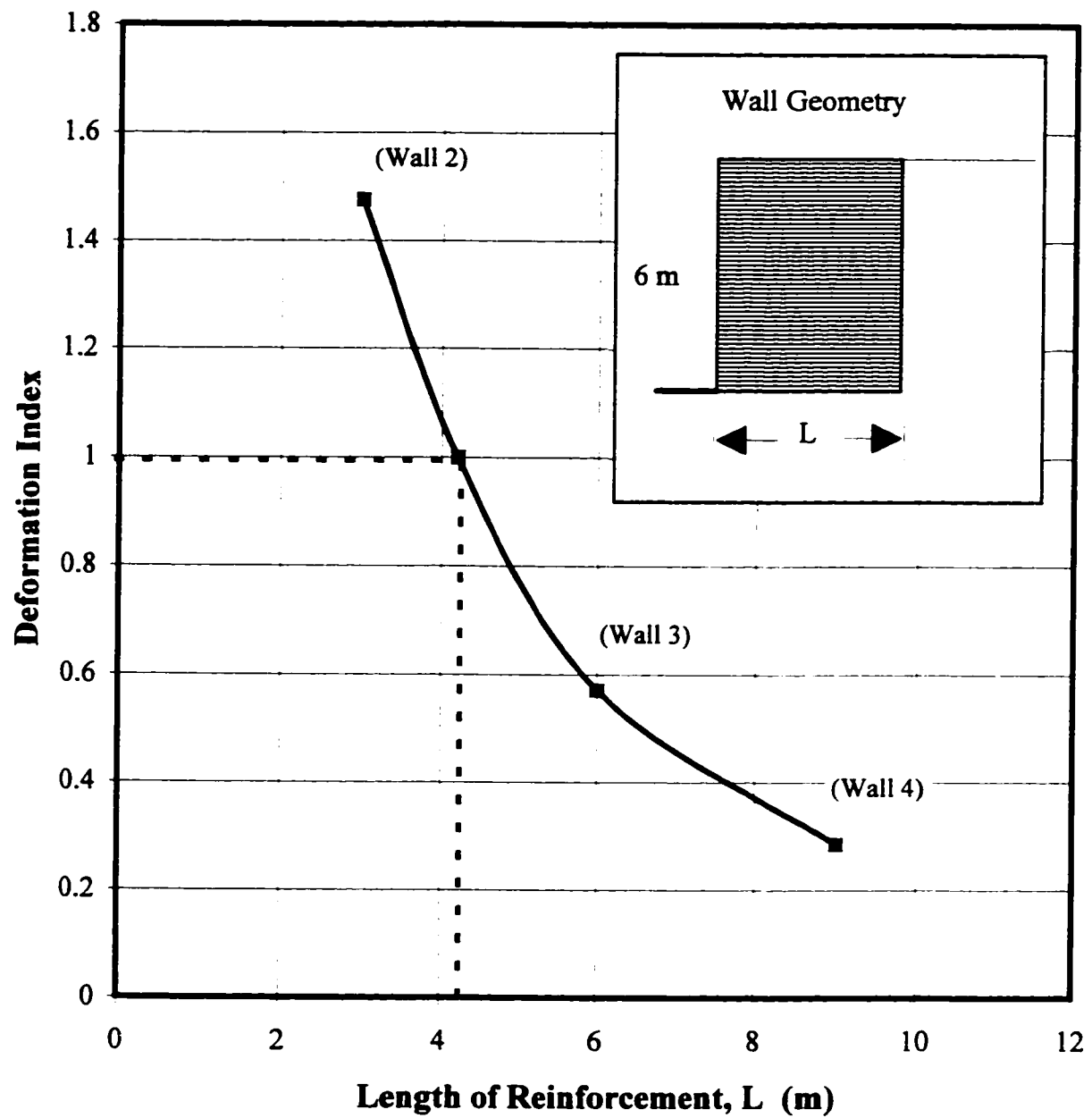


Figure 6.10. The effect of reinforcement length on the maximum wall face deformation.

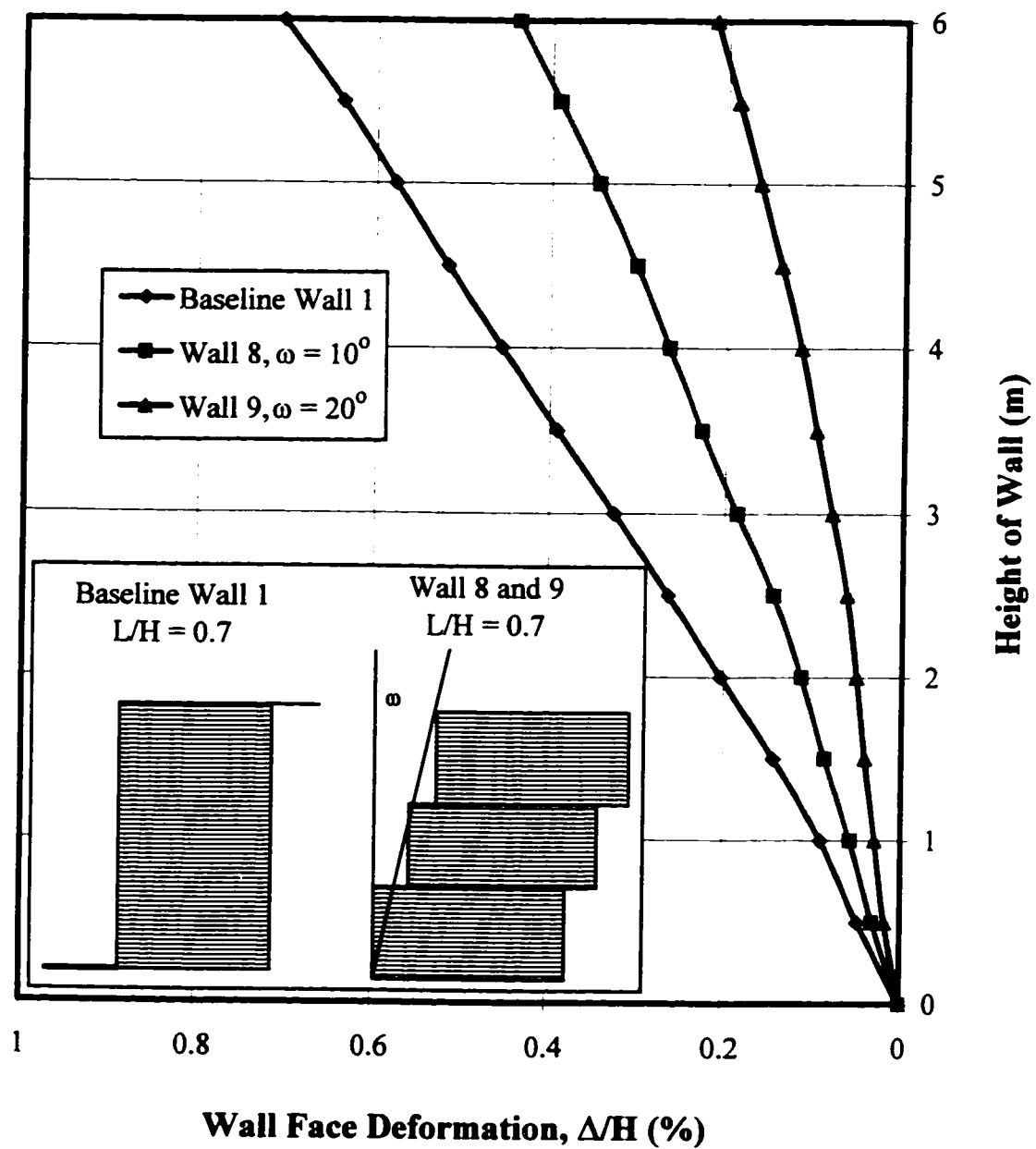


Figure 6.11. The influence of wall batter on wall face deformation.

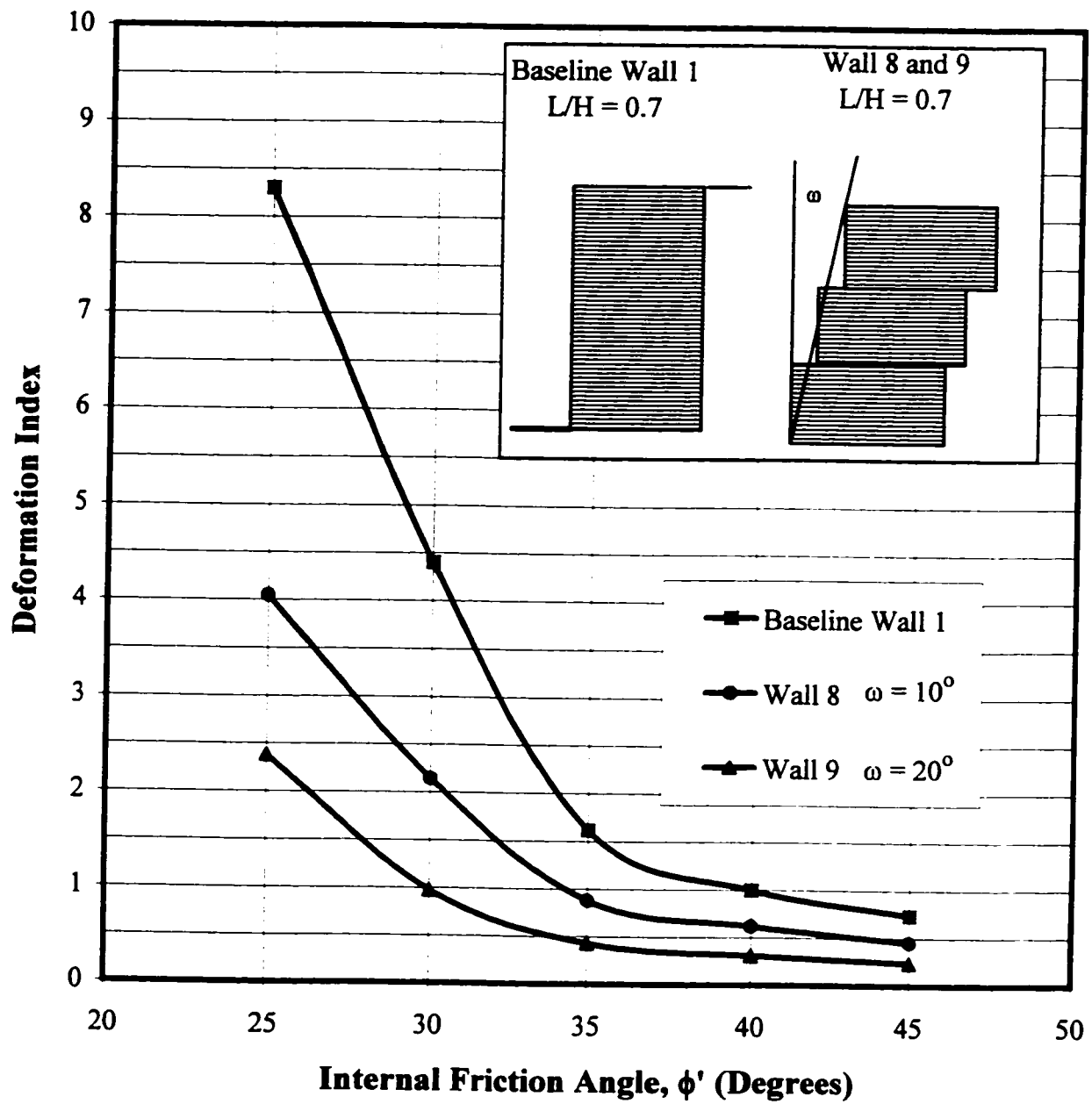


Figure 6.12. The variation in lateral wall deformation as a function of the internal friction angle.

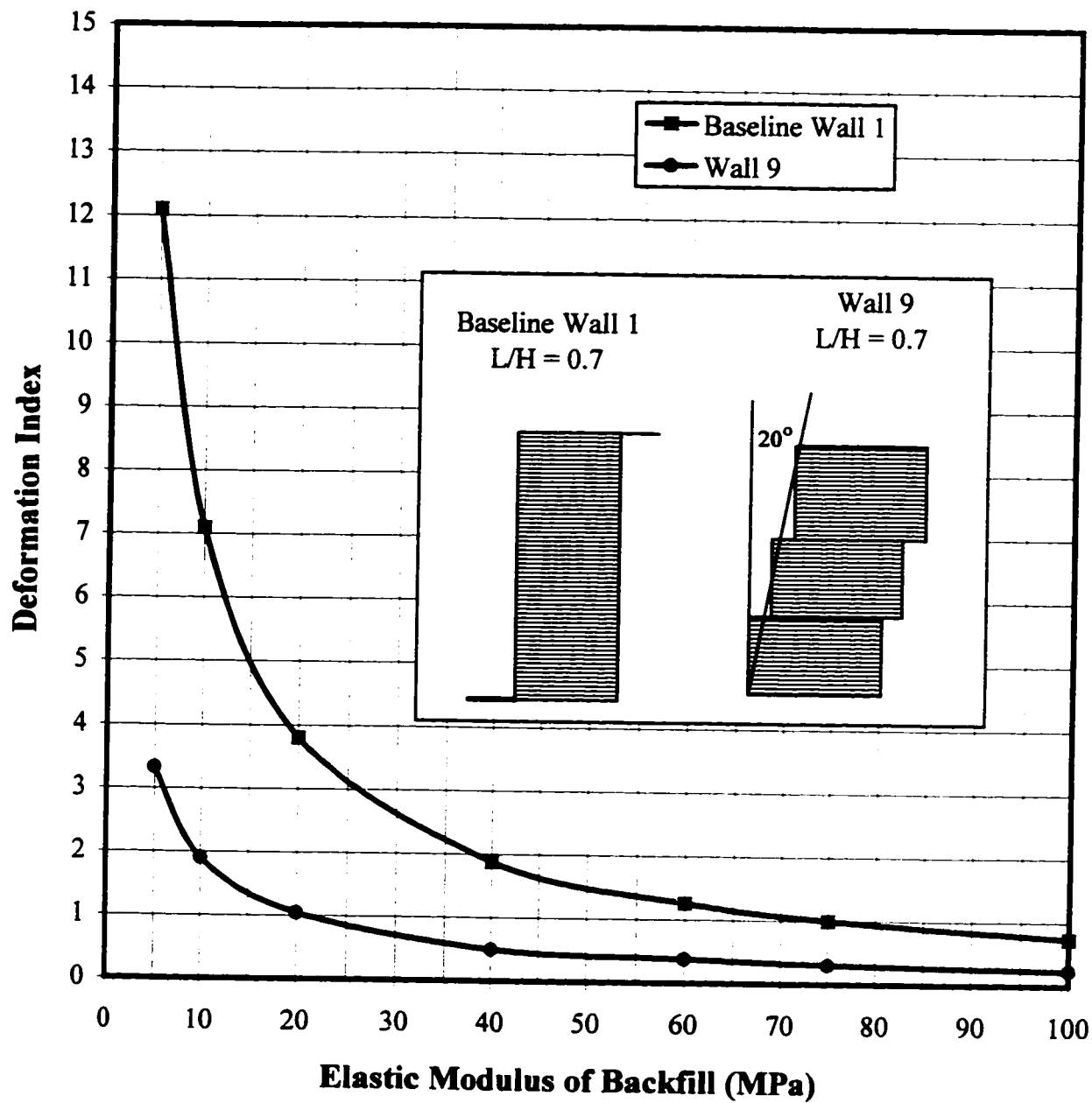


Figure 6.13. The influence of the deformability of the backfill on the lateral wall deformation.

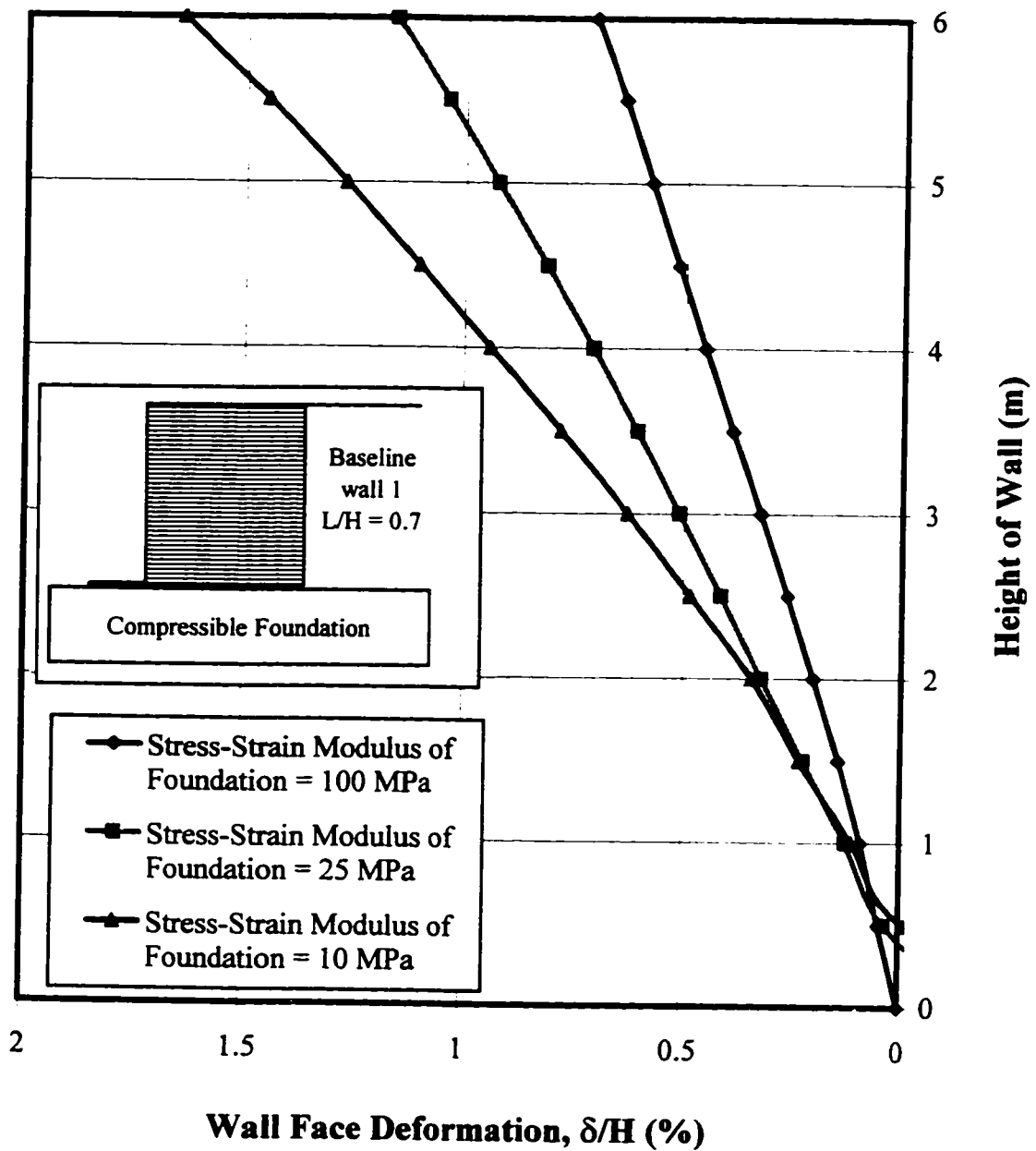


Figure 6.14. The effects of foundation compressibility on the outward movement of the wall face.

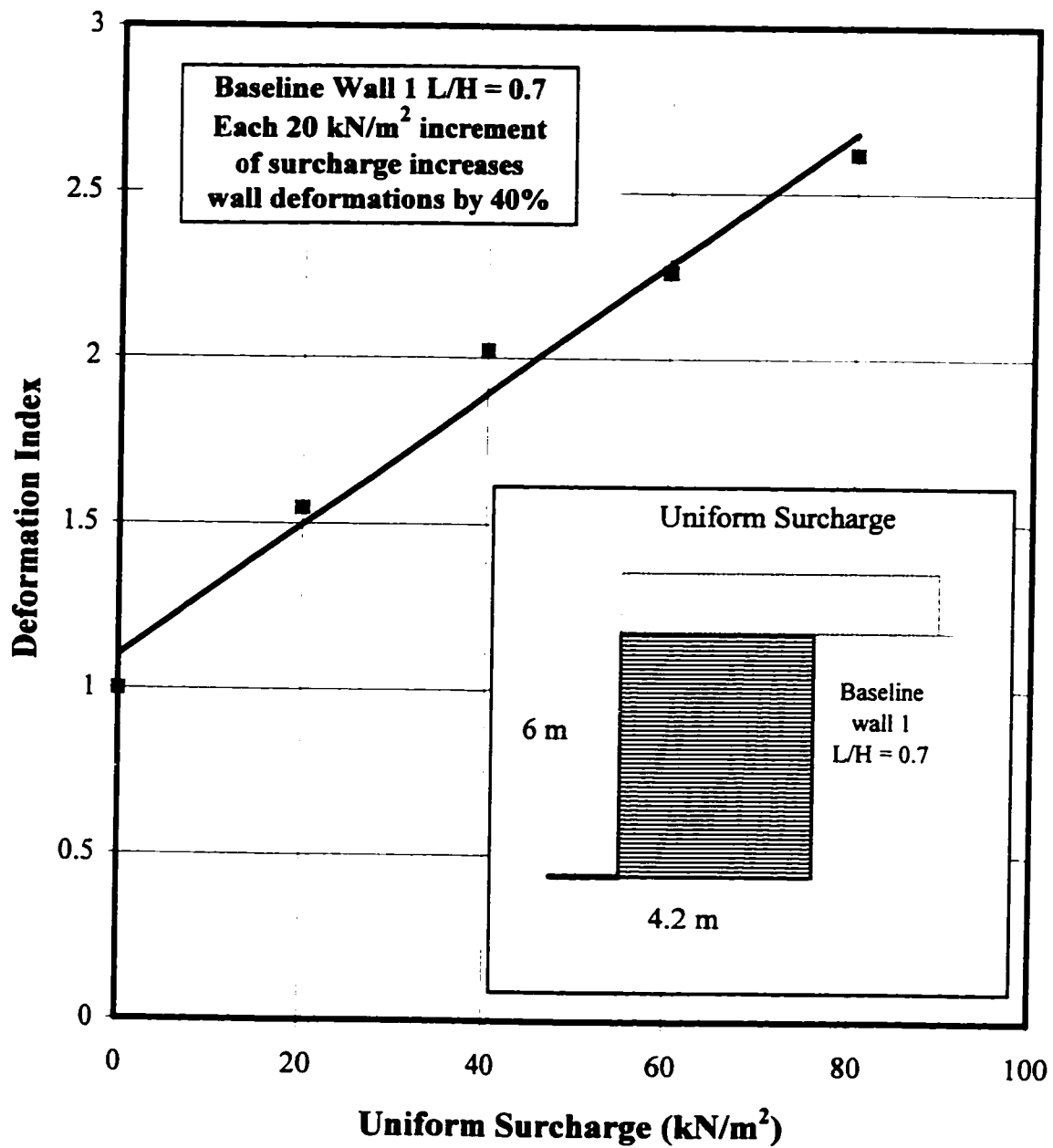


Figure 6.15. The effect of a uniform surcharge on the wall face deformation.

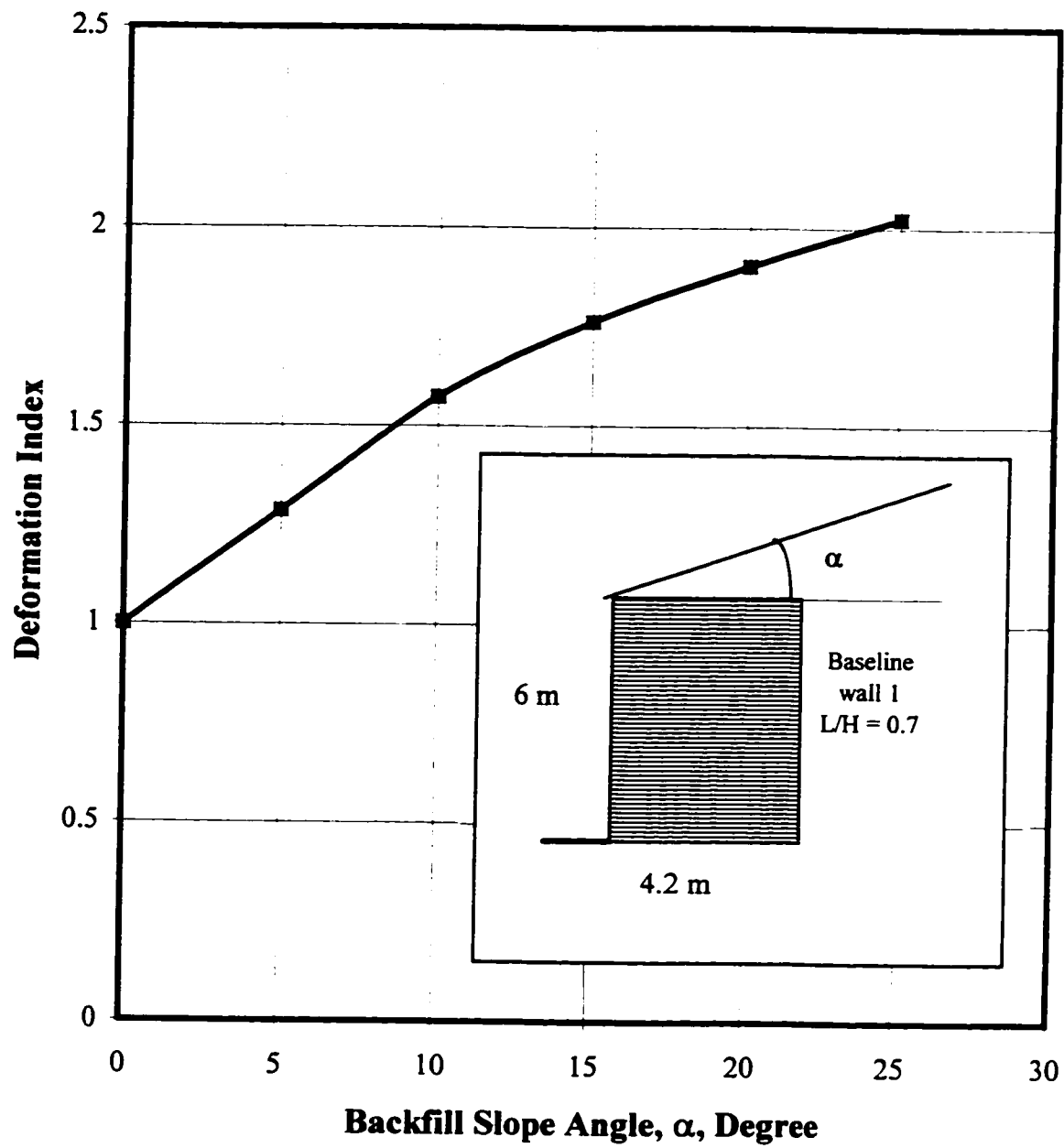


Figure 6.16. The effect of backfill slope angle on the wall face deformation.

CHAPTER 7

ENVIRONMENTAL EVALUATION

7.1 Background

One of the major concern in using shredded or whole tires in civil engineering applications is the potential for harmful substances to leach from the tires. To date, only a few reports specifically address this concern. While data from embankments constructed with tire chips has been reported, there is no published data on water quality from whole tires or tires with one sidewall removed embedded in soil.

The Minnesota Pollution Control Agency (MPCA) (1990) conducted laboratory leaching tests on tires chips subjected to different environmental conditions. These tire chips were not embedded in soil. The MPCA study reported that barium, cadmium, chromium, lead, selenium, and zinc are constituents of concern in acidic environments (pH of 3.5 to 5). Certain types of hydrocarbons (Polynuclear Aromatic Hydrocarbons, PAHs) may be released under basic conditions (pH of 8.0). A field sampling program was also implemented at two sites where waste tires had been used in road construction over wetlands. The field testing program collected soil and groundwater samples under the roadbed. Field studies did not identify significant differences between waste tire areas and control areas for soil samples. At one site, water samples had concentrations of cadmium, chromium, and lead which exceeded the Regulatory Allowable Limits. In addition, the reported concentrations levels of barium would have exceeded the Ontario Drinking Water Objectives. The Ontario Drinking Water Objectives are more stringent; element such as barium, chromium, lead, selenium, copper, and aluminum all have lower acceptable concentrations limits when compared to the USEPA standards. The hydrocarbon compounds did not exceed their respective recommended concentration limits. To minimize the potential environmental impacts, placement of tire materials above the water table,

in the unsaturated zone, was recommended by the MPCA. This would limit infiltration of water through the tire subgrade.

Eldin and Senouci (1992) examined the potential environmental impact of a test embankment constructed in the state of Wisconsin, using tire chips. The evaluation was performed by periodic analysis of water specimens collected from two lysimeters placed beneath the fill. The chemical analysis indicated that chloride, iron and manganese were occasionally in excess of the RALs while background samples did not. Reported concentrations of sodium, and alkalinity exceeded the Ontario Drink Water Objectives. Analysis of variances for the measured elements indicated a strong time effect on the concentrations of magnesium, manganese, and sodium. Manganese had a significant positive correlation with time. Chemical analysis of various organic compounds was not performed.

Humphrey et al. (1997) investigated the impact on water quality from two field embankments constructed with tire chips which were placed above the groundwater table. Results of the chemical analysis indicated that tire chips did not increase the concentration levels of inorganic elements having a primary drinking water standard (RALs), such as barium (Ba), cadmium (Cd), chromium (Cr), lead (Pb), and selenium (Se). Some evidence suggested that iron (Fe) and manganese (Mn) may, under certain conditions, exceed their respective concentration limits. Any organics compounds leached from the tire chips were below detection limits. This investigation revealed no significant adverse effects to date (after 2.5 years) on the water quality from tire chips fills placed above the ground water table.

Lerner et al. (1993) investigated the potential use of tire chips as a drainage bed for domestic septic tanks. Laboratory water quality tests, in which tire chips were soaked in different environments (various pH and ionic strengths), indicated that typical rubber compounds such as zinc and benzothiazoles were leached from the tire chips regardless of environmental condition. Water samples taken from a pilot septic drainage bed revealed fewer contaminants. The authors recommended that prototype structures be constructed to ascertain any potential environmental impact on ground water quality; shredded tires should not be positioned which would offer any

leachate a clear path to the ground water. They also recommended soaking the tires in open lagoons for a period of six to twelve months, removing many potential toxic compounds, which would render them more acceptable for environmental uses.

Toxicity test were conducted by Abernethy (1994) on water samples in which an automobile tire had been submerged in 300 litres of water. The “tire water” cause 100% mortality of rainbow trout fry, usually within 48 hours. Lethality was not observed for three other test species (*Daphnia magna*, *Ceriodaphnia* and fathead minnows). The contaminated water samples were analyzed for 143 target compounds. Only zinc was found, and its concentration was well below the lethal level. Gas chromatography-mass spectrometry (GC-MS) analyses were performed to identify nontarget chemicals. Up to 62 organic compounds were detected, but less than half were identified (a known compound). Bench top treatments such as aeration, and the additions of acid, base, an anti-oxidant and a metal chelating agent did not reduce the toxicity of the tire water. However, the addition of activated carbon completely removed the lethal effects. The chemical cause(s) of the toxicity to trout remained unconfirmed.

A more recent study by Abernethy et al. (1996) revealed that tires placed in a tank of flowing water were nonlethal to trout so long as the flow rate was greater than 1.5 litres per minute per 600 litre of water volume. The rate of chemical release from tires decreased with each subsequent submersion period and which was attributed to a continuous leaching process which depleted the tires of leachable chemical substances. Chemical release may also be inhibited by bacterial growth and any compositional changes occurring at the surface of the tire. The study also revealed that tires collected from an artificial reef in Lake Erie were less toxic than scrap tires which were never exposed to an aquatic environment. The amount of chemical release from the reef tires was much less than those from recently discarded tires. A reduction in contaminant levels in water surrounding an aquatic tire structure may occur by such environmental processes as biodegradation, photolysis and particle-binding. The toxicant could not be identified, however, it was characterized as a non volatile mixture of polar and non-polar organic compounds (specific constituents were not identified) of which aromatic amines were the principal suspects.

7.2 Reported Exothermic Reactions in Tire Shred Fills

One of the legitimate concerns on the use of rubber products as fills is the potential for exothermic reaction. An exothermic reaction is one where heat is released as a result of chemical or biochemical reaction which causes the temperature to rise. In 1995, there were three cases of fills comprising of **tire chips** in the USA which were reported to be smouldering after construction (Humphrey, 1996). The three projects are SR 100 in Ilwaco, Washington, Fallings Spring Road fill in Garfield County, Washington, and a retaining wall project alongside Highway I-70 in Colorado. It is important to note that in all three cases, the fills were constructed entirely of tire chips, and where a high proportion of steel belt was exposed in a matrix, and was sometimes encountered in a matrix of crumb rubber. The upper surfaces of these fills were provided with a course of soil cover. Hot spots have also been reported in four stockpiles of tire chips in the USA.

In order for ignition to occur, it is essential that an initial exothermic reaction occurs which can raise the temperature above the ignition point. According to Humphrey (1996), the potential causes of initial exothermic reaction are: oxidation of exposed steel belts and wires, oxidation of rubber, microbes consuming exposed steel wires or generating acidic conditions, and microbes causing liquid petroleum products. Humphreys found that in all three cases where exothermic reaction was observed, appropriate conditions for oxidation existed. In some cases it was lack of adequate cover which would minimize availability of free oxygen, while in some cases, exothermic reaction occurred in areas which were exposed to fertilizer-rich soil, or where crumb rubber was found.

It is of interest to review the list of preliminary recommendations for construction of tire shred fills indicated by Humphrey:

- The amount of steel belt exposed at the cut edges of the tire sheds should be limited. Consideration should be given to using magnetic separation to limit the amount of exposed steel belt.

- Topsoil should not be placed directly on tire shreds.
- Contact between tire shreds and fertilizer should be prevented.
- Any tire shreds that have been contaminated by liquid petroleum products should be removed from the fill and disposed of in an environmentally acceptable manner.
- The tire shred fill should be covered by 1.2 m of mineral soil with a minimum of 25% fines to limit the contact of the fill with oxygen.
- Consideration should be given to using larger size tire shreds (200 mm. to 300 mm in maximum size) for thick tire shred fills as these would have fewer cut surfaces with exposed steel belt compared to smaller shreds.
- No crumb rubber should be allowed in tire shred fills. One source of crumb rubber is the material that accumulates around shredding machinery and associated conveyor belts. This material is typically composed of crumb rubber, fine steel cord wire, and soil that was brought in with the waste tires. Some tire shred producers dispose of this material by delivering it to the job site mixed with the tire shreds. This practice should not be allowed. Material accumulated during cleanup operations at the (tire chip) production facility should be banned from tire shred fills.

It will become obvious that the manner in which the test fills and retaining walls have been constructed are markedly different from the three cases where exothermic reaction has occurred. The use of whole tires, or tires with one side wall removed as a ring addresses all of the concerns raised in the above list of recommendations. Also, the tires as used in study are buried in a mass of compacted cohesive soil or sand, which significantly reduces, if not eliminates, the supply of free oxygen. No crumb rubber is generated in the slicing process. Certainly, the use of whole tires or those with one side wall removed appear to offer a significant advantage over the use of fills

comprising of tire chips alone. The prototype embankment constructed in this study has been hydro-seeded, with no sign of any exothermic activity.

7.3 Environmental Testing Program

Laboratory and field tests were carried out to identify any potential toxic substances leaching from buried tires. The purpose of the investigation was to gather information on the short term (with respect to the service life of these structures) chemical leaching trends, as a function of exposure time, soil type and environmental conditions, and also to evaluate in which medium (if any) they are produced, promoted or removed. Laboratory testing in lysimeter columns offers certain advantages over testing on samples obtained from the field due to the following reasons:

- (i) Due to low seepage, very limited volumes of samples could be obtained from the site over a short collection period.
- (ii) Due to financial constraints, no control structure was built. Hence, it is difficult to ascertain if the contaminants originated from the tires or from the fill or the construction site.
- (iii) Use of tire chips in lysimeters would provide a conservative assessment since the reinforcement is exposed.
- (iv) The lysimeters permit a controlled testing environment, in contrast to field scenarios. Tests could be conducted at various hydraulic and chemical conditions. In particular, the lysimeter columns permit the simulation of seepage over long periods of time, especially in clayey soils.

7.3.1 Field Monitoring

A 76 mm diameter perforated pipe was installed in the sand drainage blanket, under each section, to collect leachate samples as rain water percolates down through the tires. The drain pipe was wrapped in a geotextile to prevent clogging and was graded to flow towards the retaining wall

section. The effluent is collected in three separate wells consisting of a vertical PVC cylindrical container of 150 mm diameter and approximately 1 m in depth. The top of the well was fitted with a protective cap that could be easily removed during sampling. Epoxy was used to assemble all pipes and fittings. A schematic diagram of the field collection well is given in Figure 7.1.

Water samples were collected, periodically, using a hand pump. The water samples were stored in 1 litre HDPE bottles and were refrigerated to minimize degradation. It should be noted, that the wells remained dry during the summer months, and therefore most water samples could be collected only during the spring and fall months.

7.3.2 Laboratory Leaching Tests

Laboratory testing of lysimeter columns with whole or sliced tires is clearly impractical. The use of tire chips in lysimeters provides conservative test results compared to the use of whole tires since larger surface areas, and in particular, the metal reinforcement are exposed. Laboratory testing consisted of 12 large lysimeter columns of 150 mm diameter by 400 mm long that were filled with various mixtures of tire chips and soil, as illustrated in Figure 7.2. Soil and tire chip mixtures are divided into two main groups: sand - tire chips and pure kaolin clay - tire chips. Twelve tires collected from the Conroy Auto-Part Recycling site (field embankment test site) were cut into 50 mm transverse sections with a band saw and each of these sections was further cut into four separate pieces. Typical chip size was 50 mm by 50 mm. The tire chip content varied typically from 25 to 30% by volume of the lysimeter chamber. An outline of the laboratory environmental testing program is given in Table 7.1.

7.3.2.1 Sand and Tire Chips

The four lysimeter filled with sand and tire chips were exposed to three different initial environmental conditions: acidic, neutral and basic. Acidic condition was provided by a 0.1% solution of sulphuric acid (pH ~ 3.5) and attempted to simulate the effects of acid rain. The

neutral condition was provided by distilled water only and had an initial pH of about 6.5. The basic condition was provided by a 0.05% solution of sodium hydroxide (pH~9.5) to simulate the hydrolysis potential of groundwater. Tire chips and sand were initially washed with distilled water; then the tire chips and sand were placed in the appropriate testing environment. The water was continually circulated by chemical feed pumps in order to assure proper mixing. Continuous circulation of the effluent is conservative since it represents batch test conditions and does not represent boundary conditions in tire reinforced structures constructed above the groundwater table. Water samples were collected in 1 litre HDPE bottles at the termination of the test and were refrigerated to minimize degradation of the sample. The exposure time was varied from 90 to 180 days. The testing procedure was repeated for each different exposure times using the same tire chips.

The above four tests represent cases where relatively stagnant groundwater (ponding of rain or snow melts in surface aquitards) is exposed to automobile tires for long periods of time.

7.3.2.2 Clay and Tire Chips

The eight remaining lysimeters are divided into three groups (Table 7.1). Each group was composed of a standard (no tire chips) and 1 or 2 clay and tire chips columns. Again, each group was subjected to a different environment (acidic, neutral, basic). The tire chips at known weight/volume of soil were randomly placed within the lysimeter cylinder with a mixture of pure kaolin clay (obtained from Fisher Company Ltd., Colorado). The kaolin sample was fabricated by mixing dry clay with distilled water. Flushing of the pore water (in the pore space between the clay particles and tire chips which is occupied by water) was achieved by placing a source reservoir over the top of the soil/tire chip column and applying a pneumatic pressure of 200 kPa. The source reservoirs were filled with the appropriate initial environmental solution and were as follows: 2 (numbers in each group includes the standard) with distilled water; 3 with a 0.1% solution of sulphuric solution; 3 with a 0.05% solution of sodium hydroxide. The flushed pore water was collected into 11 graduated cylinders at the base of the lysimeter cell. Samples were analysed after passing of one pore volume (total volume of voids in the soil/tire chip sample). A

maximum of 5 pore volumes for each environmental cell was tested (5 “tire water” samples were collected from each environmental cell).

7.3.3 Chemical Compounds of Interest

Water samples from field and those collected from lysimeters were analyzed for selected inorganic elements to compare them with the requirements of the Ontario Drinking Water Objectives (ODWO). Based upon published environmental assessments on tire chip fills, particular attention was given to iron (Fe), manganese (Mn) and zinc (Zn). The organic chemical compounds leached from scrap tires can be classified as arylamines or phenols and alylated forms. Organic analysis of “tire water” performed by Abernethy (1994) identified four compounds which were found in all samples: aniline, 4-(1-methyl-1-phenylethyl)-phenol, benzothiazole, and 4-(2-benzothiazolythio)-morpholine. The most prevalent compounds were 2(3H)-benzothiazolone, benzothiazole which are the probable stable end products of mercaptobenzothiazole (MBT) formed by methylation, photolysis and oxidation (Brownlee et al. 1992) and 4-(2-benzothiazolythio)-morpholine which is a common compound used as a delayed-action accelerator in rubber processing (Taylor and Son 1982). Therefore, these constituents were also selected as representative compounds to establish any chemical leaching trends. Collected water samples were sent for chemical analysis to the Center for Analytical and Environmental Chemistry at Carleton University. A list of the chemical compounds and parameters of interest for both field and laboratory tests is given in Table 7.2.

7.4 Results

7.4.1 Field monitoring

A background concentration of the targeted compounds in the pore water of both backfill material was performed. The results are presented in Table 7.3. Several samples of both soil types were taken from their respective stockpiles at various times during the construction of the prototype embankment. All soil samples collected on a specified day were properly mixed together to produce a representative sample. 100 ml of distilled water was added to 50 g of dry soil. The mixture was vigorously stirred for a period of 24 hours. The supernatant was filtered

and analysed for the appropriate chemical compounds. The reported concentrations were adjusted to the respective in-situ water contents of both backfill materials. Selenium was the only compound of interest which exceeded the Ontario Drinking Water Objectives. Reported values were between 0.012 and 0.014 mg/l. All other compounds of interest were either below detection limits or below their respective allowable concentration limits.

Results of the chemical analysis performed on water samples collected from each well at the test fill site are given in Tables 7.4 to 7.6, respectively. It should be noted that the well located to collect effluent from the cohesive soil Section C, was dry most of the time. Indeed, all wells remained dry during the summer months. Also, the presence of a final 2 m surcharge of clay fill acts as a hydraulic barrier and therefore limits infiltration into the structure. The amount of seepage through the test fill was not monitored. The results indicate that barium (Ba), chromium (Cr), selenium (Se), and fluoride (F) elements having Maximum Acceptable Concentration limits, were present in trace amounts or were below the detection limits. The results were similar to those reported in the background analysis (Table 7.3). The detection limits of lead (Pb), chromium (Cr) and cadmium (Cd) in Section A, and lead and chromium in Section B were above their respective drinking water concentration limits for the water samples collected three months after completion of the test embankment. The high concentrations of lead, cadmium and chromium suggest that these are anomalous results and are believed to be caused by testing inaccuracies since lead, chromium and cadmium were below their detection limits in all the other samples tested. Also, investigations by Eldin and Senouci (1992) and Humphrey et al. (1997) on the water quality from tire chip fills placed above the groundwater table reported that substances having a MAC designation did not exceed the regulatory limits. Most of these elements were below their respective detection limits. Therefore, based on this study (including laboratory lysimeter tests, discussed in detail below) and others, the water medium was unable to leach heavy metals from the tire substance or they are present in very minuscule amounts, if at all, within the tire materials.

Inorganic elements having an Aesthetic Objective, including iron (Fe), manganese (Mn), zinc (Zn), copper (Cu), sodium (Na), chloride (Cl), and sulphate (SO₄) were all below the applicable

limits except for iron, manganese and aluminum concentrations for one sampling period collected 3 months after the beginning of field monitoring. Concentrations of all three metals (Fe, Mn, Al) were above the ODWO. limits in both sand sections (Sections A and B). The well located in the cohesive soil section was dry, and therefore, a comparison is not possible. Again, these anomalous results are believed to arise from testing difficulties. Based upon past research, the presence of tire chips in road embankments were responsible for the increase in iron, manganese and aluminum concentrations in the surrounding groundwater, and on several testing periods exceeded the ODWO. limits (Eldin and Senouci 1992, and Humphrey et al. 1997). The increase of such metals was attributed to the large amounts of exposed steel reinforcements as result of tire shredding. In contrast, engineered fills using whole or cut tires would severally limit the amount of exposed steel reinforcement. Also, the tire rubber would protect the mild steel reinforcement from corroding (AB-Malek and Stevenson 1986). This is evident by the low concentrations levels of metals, with the exception of observation in one collection period, that were similar or below the measured background levels for both sand and cohesive soils. It is interesting to note that higher levels of manganese and aluminum concentrations were found in the porewaters of both clean backfills i.e. in soil not exposed to tires. Thus, for future investigations, it may be critical to establish control wells in order to distinguish background levels with those obtained from leachate from buried tire structures. In general, the amount of inorganic elements capable of being leached out of engineered soil structures using whole tires or cut tires will be insignificant.

None of the elements surpassed the Ontario Drinking Water Objectives in the limited number of samples obtained from the cohesive fill section (Section C). The mixture of cut tires placed horizontally and in-filled with a cohesive or clayey soil produces a composite material of low permeability. The infiltration and the migration of rain water (the structure was constructed above the groundwater table) through the fill is inhibited, as indicated by only two samples obtained over the last two years. These two samples were obtained during spring time when the surface runoff was the highest due to snow melt. Therefore, it is possible that the water collected in this well may also originate from the surface spring runoff rather than water which has percolated through the test fill. The movement of contaminants through soils of low permeability

is strongly time dependent, and thus, a testing period of two years may not be sufficient to determine any possible environmental impact on the surrounding groundwater. However, the amount of chemical flux entering the surrounding environment, at a given time, would be severely restricted in this type of soil. The migration of inorganic elements in clayey soils can be highly retarded, i.e. chemical compounds are absorbed by the clay minerals, and is usually dominated by diffusion (Garga and O'Shaughnessy 1994). The coupling of a low hydraulic potential (placement of the structure above the groundwater table) and a minimum of leachable compounds (protection of the steel reinforcement by the tire rubber) may render any potential contaminants leached from buried tires immobile (considering the life span of the structure) when placed in a cohesive or clayey backfill. It is relevant to note that clay is used both as a capping layer to reduce leachate production, and as a hydraulic barrier to reduce contaminant migration in modern landfill sites.

The concentration levels of the targeted organic compounds from all field monitoring wells were below the test method detection limits with the exception of one compound. The exception being 4-(2-benzothiazolythio)morpholine which was detected twice in the well located in section B (full tires in sand). These concentrations were, however, significantly below levels measured in the laboratory lysimeter tests. The presence of 4-(2-benzothiazolythio)morpholine indicates that leaching of some organic compounds from tires placed above the water table can occur. The presence of this compound could have resulted from the exposed tire face of the retaining wall which was subject to ultra-violet breakdown. The placement of a protective facing, such as shotcrete or panel facing, could eliminate this potential problem. No organic compounds were detected in the cohesive soil section. Humphrey et al. (1997) also reported that no measurable levels of organic compounds were found in tire chip fills placed above the groundwater table. These fills were monitored for a period of over two years. The environmental impact of leachable organics from tires is still unknown.

7.4.2 Laboratory Tests on Tire Chips Embedded in Sand

All elements having a MAC designation were present in similar amounts to those from the control test or were below the detection levels. The variation in iron concentration with exposure

time (note: each exposure time represents one single batch test) for the four different simulated environment and testing conditions is shown in Figure 7.3. The iron concentration generally exceeded the drinking standard limits. However, Fe levels fell below the designated AO limit for all test environments after a cumulated exposure time of 400 days. The decrease in soluble iron concentration levels is associated with the formation of rust (Fe_2O_3) which precipitated out of solution. A high amount of a rust aggregate, a mixture of rust and sand particles, had form around the exposed steel reinforcing elements which may have also slowed the leaching process. The acidic environment promoted the oxidation of the exposed steel elements and thus decreasing the available iron for each subsequent batch test. This may explain why soluble iron was only found in the basic environment after the second batch test (130 days). It should be noted that in each consecutive batch test, the leaching medium was totally replaced, thereby removing all soluble elements in that test. In addition, cutting of a whole tire by a band saw instead of a shredder produced tire chips having a smaller number of exposed steel reinforcement wires.

A trend similar to iron was also observed for aluminum levels (Figure 7.4). After an exposure time of 130 days, the tire chips were clean using distilled water and the rust aggregates were removed. This cleaning process exposed fresh steel reinforcements may have produced the increase in Al concentrations in the neutral and acidic conditions for the final batch test of 180 days.. Manganese concentration levels were generally above the acceptable concentration limits accept for tire chips placed in the basic environment (Figure 7.5). Indicating that most of the manganese leached from the steel reinforcement remained in solution. The concentration levels of zinc in the water environment was below the AO designated limit. Zinc was easier to leach out of the tire chips under a basic condition (Figure 7.6). The long term potential for zinc in leaching out of the tire rubber is a concern, since zinc oxide is used in the vulcanization process to produce tire rubber and is responsible for approximately 2% of the total weight of the tire (Table 2.4). Lysimeter test results indicate that it may be beneficial to mix the tire chips and the soil together as to form a composite material. The presence of a soil within the tire chip matrix may reduce leaching of inorganic elements.

The tire chips used in earth fills, under appropriate conditions, would increase the concentration of iron, aluminum, zinc, and manganese in the surrounding groundwater. This finding is consistent with studies of water quality monitoring of tire chip embankment placed above the ground water table (Eldin and Senouci 1992, Humphrey et al. 1997). As indicated in the field study, this increase is generally associated with the high number of exposed steel reinforcement wires. However, engineered soil structures using full tires would have lower impact on the water quality since the steel reinforcement is not exposed, and in addition, the rubber layer would protect the mild steel reinforcement from corroding (AB-Malek and Stevenson 1986). This analogy is consistent with results obtained from the field monitoring program.

Benzothiazole concentration decreased with the number of washes and exposure time except for test series 2 in which tire sidewalls were placed in an acidic environment (Figure 7.7). Minimum leaching of benzothiazole occurred in the neutral environment. The basic environment promoted the release of this organic compound, however, there was a mark decrease in concentration after the second wash and exposure time. The 2(3H) benzothiazolone and phenol compounds disappeared after the second wash and exposure time, as shown in Figure 7.8 and 7.9, respectively. Again, the release of the phenol compound was promoted under basic conditions. The 2(3H) benzothiazolone was not leached for the tire chips which were subjected to a neutral environment. 4-(2-benothiazolythio)morpholine was the only target compound to show a steady increase in concentration levels, in all the 4 test series, with increasing exposure time (Figure 7.10). This trend may indicate that 4-(2-benothiazolythio)morpholine is a more stable end by-product of the leaching process. This may also explain why it was the only organic compound to be detected during field monitoring.

The total amount of chemicals released from the tire chips, independent of the leaching environment, decreased with the number of washes and increased exposure time. Abernethy et al. (1996) also reported that the rate of chemical release decreased during each tire submersion period. The decrease in the rate of chemical release could be attributed to several factors. There is only a limited amount of leachable material found at the surface of the tire rubber. Leaching of the contaminants is probably limited to the tire surface, since water (the leaching medium) is

essentially unable to penetrate the tire rubber (rubber is very impermeable material), and therefore, depletion of the available compounds would occur only at the initial stages and over a short time period. AB-Malek and Stevenson (1986) studied the physical condition of vulcanized natural rubber submerged in 24 m of sea water for a period of 42 years. The amount of water absorbed into the tire rubber was minimal, only 4.7%. The limited amount of water adsorption was attributed to the formation of a thin surface layer (0.05 mm) of an iron base material. This thin layer may inhibit the release of organic compound. Also, the growth of bacteria on the surface of submerged tires may also reduce the amount of chemicals released (Arbermethy et al. 1996). Alternatively, the chemical composition at the tire surface can be altered over time which could modify chemical release.

7.4.3 Laboratory Tests on Tire Chips Embedded in Clay

All test series demonstrated that the inorganic target elements were either below their respective concentration limits or below the concentration levels found in the control samples. The variation in the organic compound concentrations for tire chips embedded in the kaolin clay subject to a neutral environment is shown in Figure 7.11. The release of benzothiazole and 2(3H)-benzothiazolone was minimal and fell to zero after 2 pore volumes of flushing. The 4-(2-benzothiazolythio)morpholine had the highest peak concentration after 2 pore volumes but disappeared after 3 pore volumes. The phenol compound showed an increase in concentration with an increase in the number of pore volumes of flushing. However, the amount of chemical release was much less than that associated with tire chips embedded in sand. The chemical release trend from tests performed under acidic conditions are presented in Figures 7.12 and 7.13, and for tire chips and clay subjected to a basic environment are shown in Figures 7.14 and 7.15, respectively. In all tests, a consistent chemical trend was observed. The concentration of the target compound would reach a maximum level and then would begin to decrease with increasing number of pore volumes, or would disappear altogether. The chemical release trends observed for tire chips placed within the clay medium are consistent with the test series 1 to 4, performed in the sand. However, the amount of chemicals leached for the clay embedded tire chips was generally less than that associated with tire chips embedded in sand. This leaching trend could be the result of the clay soil covering the tire chip surface which would limit the

amount of surface area exposed to the leaching medium. Also, these chemical compounds could be absorbed by the clay particles, restricting their movement. The release of organic compounds from a saturated clay medium will be strongly time dependent. However, as mentioned before, the amount of chemical flux entering the surrounding environment, at a given time, would be severely restricted in clays.

Table 7.1. Outline of laboratory environmental testing program.

Test Series	Environmental Cell Composition	Environmental Conditions
1	Sand + 30% (by volume) Tire Chips	Distilled Water, pH ~ 6.5 (neutral environment)
2	Sand + 25% Tire Chips (sidewalls only)	Acidic Condition, pH ~ 3.5 (H ₂ SO ₄ + NaHCO ₃)
3	Sand + 30% Tire Chips	Basic Condition, pH ~ 9.5 (NaOH + NaHCO ₃)
4	Sand + 30% Tire Chips	Acidic Condition, pH ~ 3.5 (H ₂ SO ₄ + NaHCO ₃)
5	Kaolinite (Clay) Water Content (w) = 38.5%	Distilled Water, pH ~ 6.5 (Control Sample)
6	Kaolinite, w = 37% + 20% Tire Chips	Distilled Water, pH ~ 6.5
7	Kaolinite, w = 38.5%	Acidic Condition, pH ~ 3.5 (H ₂ SO ₄ + NaHCO ₃) (Control Sample)
8	Kaolinite, w = 37.8% + 25% Tire Chips	Acidic Condition, pH ~ 3.5 (H ₂ SO ₄ + NaHCO ₃)
9	Kaolinite, w = 37.8% + 25% Tire Chips (Top Half Only)	Acidic Condition, pH ~ 3.5 (H ₂ SO ₄ + NaHCO ₃)
10	Kaolinite, w = 37.5%	Basic Condition, pH ~ 9.5 (NaOH + NaHCO ₃) (Control Sample)
11	Kaolinite, w = 37.5% + 30% Tire Chips	Basic Condition, pH ~ 9.5 (NaOH + NaHCO ₃)
12	Kaolinite, w = 37.5% + 25% Tire Chips (Top Half Only)	Basic Condition, pH ~ 9.5 (NaOH + NaHCO ₃)

Table 7.2 A list of chemical compounds and parameters of interest.

Compound of Interest	Ontario Drinking Water Objectives	
	Type	Concentration Limits (mg/l)
<u>Inorganic</u>		
Alkalinity (as CaCO ₃)	OG	500
Aluminum (Al)	OG	0.1
Barium (Ba)	MAC	1
Cadmium (Cd)	MAC	0.005
Calcium (Ca)		N.A.
Chromium (Cr)	MAC	0.05
Chloride (Cl)	AO	250
Conductivity (as µS)		N.A.
Copper (Cu)	AO	1
Fluoride (F)	MAC	1.5
Iron (Fe)	AO	0.3
Lead (Pb)	MAC	0.01
Magnesium (Mg)		N.A.
Manganese (Mn)	AO	0.05
pH	AO	6.5-8.5 (no units)
Potassium (K)		N.A.
Selenium (Se)	MAC	0.01
Sodium (Na)	AO	200
Sulphate (SO ₄)	AO	500
Zinc (Zn)	AO	5
<u>Organic</u>		
Benzothiazole		P.N.L.
(1,1-dimethylethyl)-2-methoxyphenol		P.N.L.
2,5-dibutylthiophene		P.N.L.
4-(2,2,4-trimethylpentyl)phenol		P.N.L.
2(3H)-Benzothiazolone		P.N.L.
4-(2-benzothiazolythio)morpholine		P.N.L.

MAC = Maximum Acceptable Concentration

AO = Aesthetic Objective

OG = Operational Guidelines

N.A. = Not Applicable

P.N.L. = Parameter Not Listed

Table 7.3 The chemical background concentrations of the porewater for field test soils.

Inorganic Element and Organic Compounds	Detection Limits mg/L	Sand mg/L	Cohesive Soil mg/L
Barium (Ba)	0.001	0.021	0.009
Cadmium (Cd)	0.0005	<0.0005	<0.0005
Chromium (Cr)	0.004	<0.004	<0.004
Lead (Pb)	0.004	<0.004	<0.004
Selenium (Se)	0.01	0.012	0.014
Fluoride (F)	0.5	<0.5	<0.5
Manganese (Mn)	0.004	0.054	0.014
Iron (Fe)	0.003	<0.003	<0.003
Zinc (Zn)	0.04	<0.04	<0.04
Copper (Cu)	0.001	<0.001	<0.001
Sodium (Na)	0.02	10.5	13.6
Chloride (Cl)		1.5	2.1
Sulphate (SO ₄)		14.3	18.8
pH (no units)		N.D.	N.D.
Aluminum (Al)	0.002	0.064	0.049
Calcium (Ca)	0.005	18.7	31.8
Magnesium (Mg)	0.002	3.2	15.7
Potassium (K)	0.02	3.3	4.1
Alkalinity (as CaCO ₃)		N.D.	N.D.
Conductivity (as μ S)		N.D.	N.D.
Benzothiazole	0.001	Z.D.	Z.D.
(1,1-dimethylethyl)-2-methoxyphenol	0.001	Z.D.	Z.D.
2,5-dibutylthiophene	0.001	Z.D.	Z.D.
4-(2,2,4-trimethylpentyl)phenol	0.001	Z.D.	Z.D.
2(3H)-Benzothiazolone	0.001	Z.D.	Z.D.
4-(2-benzothiazolythio)morpholine	0.001	Z.D.	Z.D.

N.A. = Not Applicable

N.D. = Not Determined

N.R. = Not Reported

P.N.L. = Parameter Not Listed

Z.D. = Zero Detection

Table 7.4 Water quality results of the effluent collected from section A

Inorganic Element and Organic Compounds	Detection Limits mg/L	Regulatory Limits mg/L (O.D.W.O.)	Sampling Date							
			9/16/94	11/20/94	6/08/95	10/03/95	11/06/95	5/04/96	6/17/96	8/14/96
Concentrations In mg/L or Otherwise Stated										
Barium (Ba)	0.001	1	0.041	0.086	0.028	0.055	0.037	0.018	0.015	0.035
Cadmium (Cd)	0.0005	0.005	<0.0005	0.02	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Chromium (Cr)	0.004	0.05	N.D.	0.13	0.0003	<0.004	<0.004	<0.004	<0.004	<0.004
Lead (Pb)	0.004	0.01	<0.004	1.3	<0.004	<0.004	<0.004	<0.004	<0.004	<0.004
Selenium (Se)	0.01	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	0.01	0.01
Fluoride (F)	0.5	1.5	0.177	N.D.	0.287	<0.5	<0.5	<0.5	<0.5	<0.5
Manganese (Mn)	0.004	0.05	<0.004	0.18	<0.004	<0.004	<0.004	<0.004	<0.004	<0.004
Iron (Fe)	0.003	0.3	<0.003	0.93	<0.003	<0.003	<0.003	0.05	0.044	0.067
Zinc (Zn)	0.04	5	0.013	0.44	0.002	0.086	<0.04	0.13	0.14	0.14
Copper (Cu)	0.001	1	0.003	0.34	0.007	0.11	0.052	0.003	<0.001	<0.001
Sodium (Na)	0.02	200	13	24	8.4	7.8	6.6	5	1.4	<0.02
Chloride (Cl)		250	3.1	N.D.	1.8	N.R.	N.R.	9.3	1.9	3.3
Sulphate (SO4)		500	60.9	N.D.	16.3	N.R.	N.R.	4.8	4.5	11.2
pH (no units)		6.5-8.5	7.1	6.7	6.3	6.8	7.1	7.4	7.2	7.2
Aluminum (Al)	0.002	0.1	0.007	0.76	0.002	0.021	0.03	0.005	0.063	0.025
Calcium (Ca)	0.005	N.A.	87	82	53	81	62	53	53	63
Magnesium (Mg)	0.002	N.A.	18	16	18	20	15	10	9	11
Potassium (K)	0.02	N.A.	2.4	2.7	1.9	2.9	1.1	7	7.2	8.9
Alkalinity (as CaCO3)		500	246	323	246	340	262	154	354	92
Conductivity (as µS)		N.A.	460	600	480	626	340	258	340	390
Benzothiazole	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.
(1,1-dimethylethyl)-2-methoxyphenol	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.
2,5-dibutylthiophene	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.
4-(2,2,4-trimethylpentyl)phenol	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.
2(3H)-Benzothiazolone	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.
4-(2-benzothiazolylthio)morpholine	0.001	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.

N.A. = Not Applicable P.N.L. = Parameter Not Listed N.R. = Not Reported

N.D. = Not Determined Z.D. = Zero Detection

O.D.W.O. = Ontario Drinking Water Objectives

Table 7.5 Water quality results of the effluent collected from section B

Inorganic Element and Organic Compounds		Detection Limits mg/L	Regulatory Limits mg/L (O.D.W.O.)	Concentrations In mg/L or Otherwise Stated							Sampling Date			
				9/16/94	11/20/94	6/08/95	10/03/95	11/06/95	5/04/96	6/17/96	8/14/96			
Barium (Ba)	0.001	0.005	1	0.019	0.057	0.022	0.032	0.023	0.017					
Cadmium (Cd)	0.0005	0.005	0.005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005			0.021		
Chromium (Cr)	0.004	0.05	0.05	N.D.	0.012	<0.004	<0.004	<0.004	<0.004			<0.0005		
Lead (Pb)	0.004	0.01	0.01	<0.004	0.022	<0.004	<0.004	<0.004	<0.004		D	<0.004		
Selenium (Se)	0.01	0.01	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		R	<0.004		
Fluoride (F)	0.5	1.5	1.5	0.84	N.D.	0.287	<0.5	<0.5	0.57		Y	0.019		
Manganese (Mn)	0.004	0.05	0.05	0.004	0.078	<0.004	<0.004	<0.004	<0.004			<0.5		
Iron (Fe)	0.003	0.3	0.3	<0.003	2.7	<0.003	<0.003	<0.003	<0.003		W	<0.004		
Zinc (Zn)	0.04	5	5	0.008	0.15	<0.04	<0.04	<0.04	0.036		E	0.037		
Copper (Cu)	0.001	1	1	0.003	0.11	0.005	0.012	0.023	0.014		L	0.19		
Sodium (Na)	0.02	200	200	17	32	26	23	17	15		L	0.038		
Chloride (Cl)		250	250	3.2	N.D.	2.5	N.R.	N.R.	5.5			13		
Sulphate (SO4)		500	500	100	N.D.	94.5	N.R.	N.R.	22.1			3.1		
pH (no units)		6.5-8.5	6.5-8.5	6.8	7.4	6.8	7.2	7.4	7.6			13.9		
Aluminum (Al)	0.002	0.1	0.1	0.019	1.6	0.002	0.017	0.026	0.013			7.4		
Calcium (Ca)	0.005	N.A.	N.A.	63	75	52	79	60	79			0.015		
Magnesium (Mg)	0.002	N.A.	N.A.	24	36	45	35	28	34			81		
Potassium (K)	0.02	N.A.	N.A.	1.9	3.7	2.2	2.9	1.3	7			34		
Alkalinity (as CaCO3)		500	500	184	308	354	400	308	262			7		
Conductivity (as µS)		N.A.	N.A.	450	740	775	711	498	758			400		
Benzothiazole	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			588		
(1,1-dimethylethyl)-2-methoxyphenol	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			Z.D.		
2,5-dibutylthiophene	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			Z.D.		
4-(2,2,4-trimethylpentyl)phenol	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			Z.D.		
2(3H)-Benzothiazolone	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			Z.D.		
4-(2-benzothiazolylthio)morpholine	0.001	P.N.L.	P.N.L.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.	Z.D.			Z.D.		
				Z.D.	Z.D.	Z.D.	0.017	Z.D.	0.002			Z.D.		
N.A. = Not Applicable				P.N.L. = Parameter Not Listed				N.R. = Not Reported						

N.A. = Not Applicable

N.D. = Not Determined

O.D.W.O. = Ontario Drinking Water Objectives

P.N.L. = Parameter Not Listed

N.R. = Not Reported

Table 7.6 Water quality results of the effluent collected from section C.

Inorganic Element and Organic Compounds	Detection Limits mg/L	Regulatory Limits mg/L (O.D.W.O)	Sampling Date							
			9/16/94	11/20/94	6/08/95	10/03/95	11/06/95	5/04/96	6/17/96	8/14/96
Concentrations In mg/L or Otherwise Stated										
Barium (Ba)	0.001	1	0.031		0.086			0.024		
Cadmium (Cd)	0.0005	0.0005	<0.0005		<0.0005			<0.0005		
Chromium (Cr)	0.004	0.05	N.D.	D	<0.004	D	D	<0.004	D	D
Lead (Pb)	0.004	0.01	<0.004	R	<0.004	R	R	<0.004	R	R
Selenium (Se)	0.01	0.01	<0.01	Y	<0.01	Y	Y	<0.01	Y	Y
Fluoride (F)	0.5	1.5	0.64		N.D			0.59		
Manganese (Mn)	0.004	0.05	>0.2		0.036			<0.004		
Iron (Fe)	0.003	0.3	<0.003		<0.003			<0.003		
Zinc (Zn)	0.04	5	0.038	W	<0.04	W	W	0.045	W	W
Copper (Cu)	0.001	1	0.004	E	0.005	E	E	0.016	E	E
Sodium (Na)	0.02	200	33	L	32	L	L	31	L	L
Chloride (Cl)		250	22.6	L	N.D	L	L	8.4	L	L
Sulphate (SO4)		500	106		N.D			47		
pH (no units)		6.5-8.5	6.6		6			7.6		
Aluminum (Al)	0.002	0.1	0.007		0.002			0.002		
Calcium (Ca)	0.005	N.A.	78		71			80		
Magnesium (Mg)	0.002	N.A.	24		36			35		
Potassium (K)	0.02	N.A.	2.3		6.3			7		
Alkalinity (as CaCO ₃)		500	292		231			310		
Conductivity (as µS)		N.A.	550		220			540		
Benzothiazole	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		
(1,1-dimethylethyl)-2-methoxyphenol	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		
2,5-dibutylthiophene	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		
4-(2,2,4-trimethylpentyl)phenol	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		
2(3H)-Benzothiazolone	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		
4-(2-benzothiazolylthio)morpholine	0.001	P.N.L.	Z.D.	Z.D.	Z.D.			Z.D.		

N.A. = Not Applicable

N.D. = Not Determined

O.D.W.O. = Ontario Drinking Water Objectives

P.N.L. = Parameter Not Listed

Z.D. = Zero Detection

N.R. = Not Reported

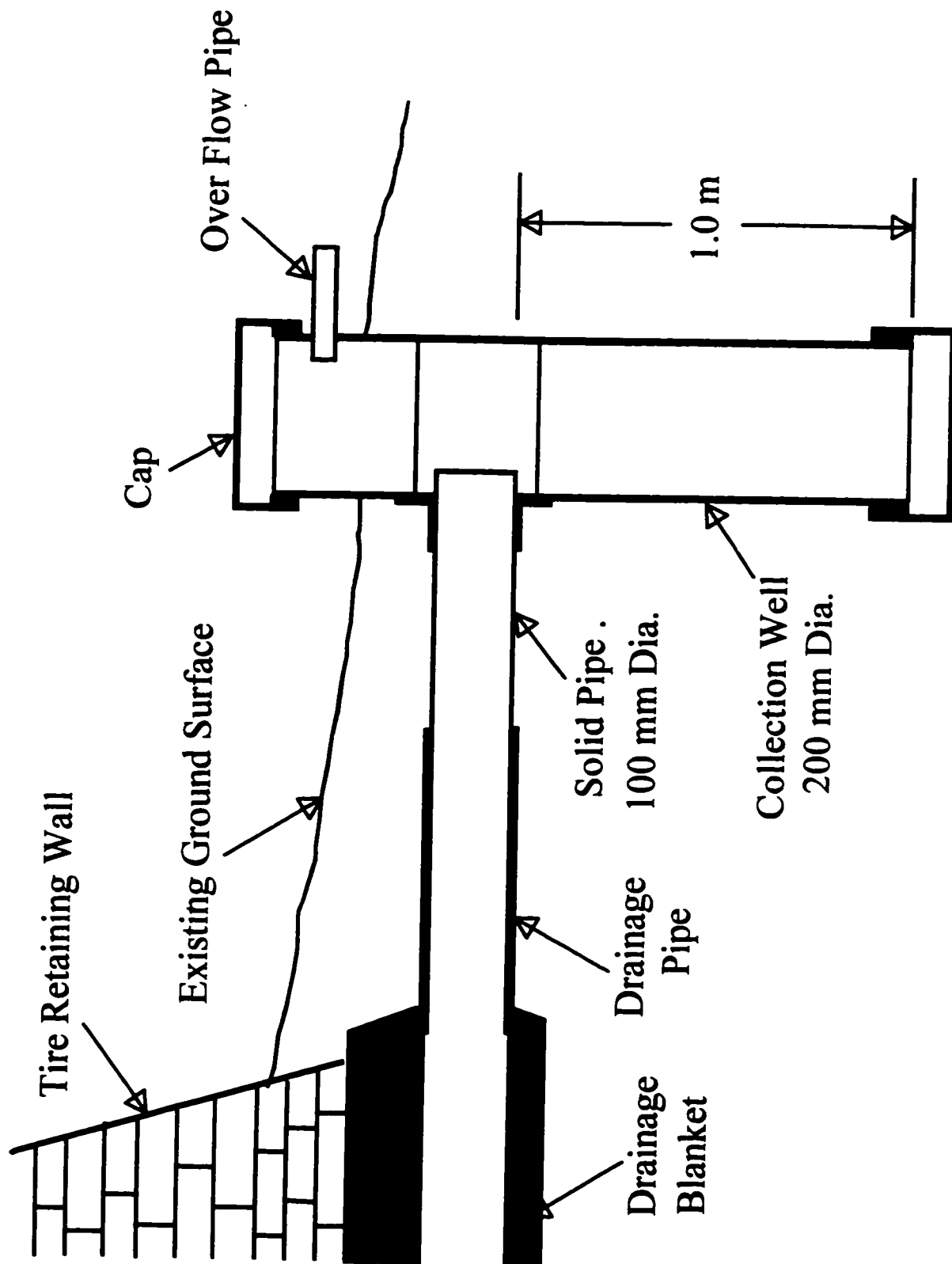


Figure 7.1. Schematic diagram of the field effluent collection well.

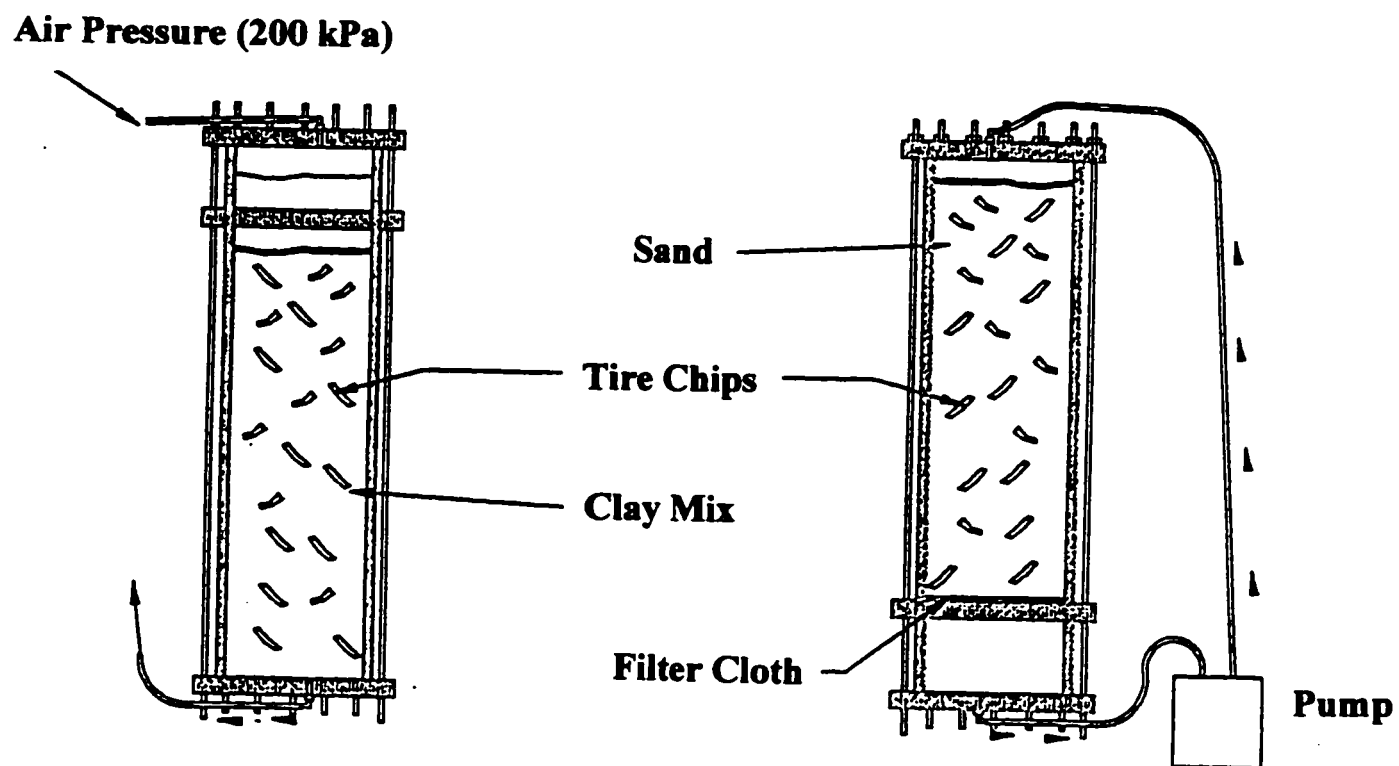


Figure 7.2. Schematic diagram of the laboratory lysimeter column tests.

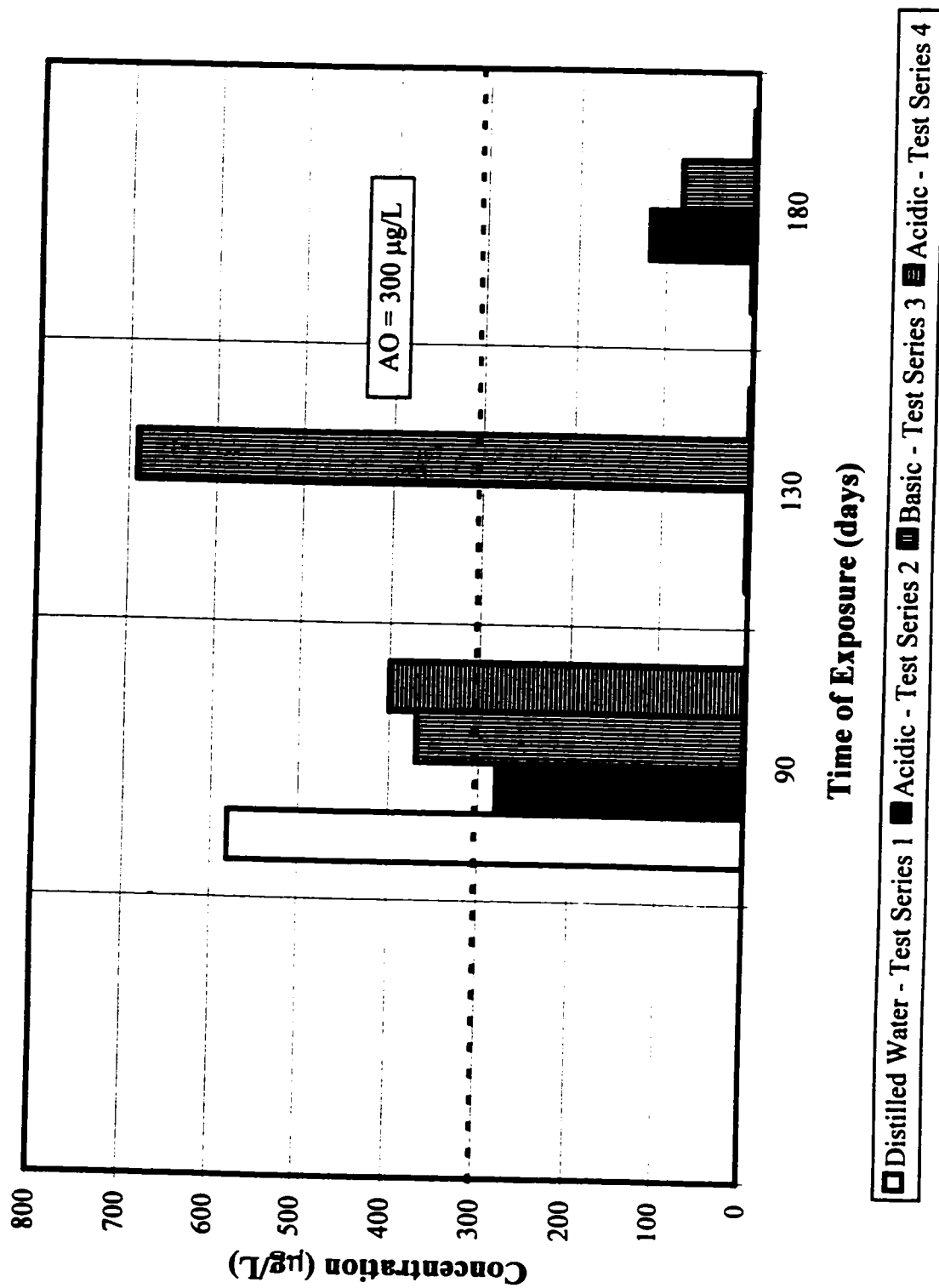


Figure 7.3. The variation in iron (Fe) concentration as a function of the leaching environment and subsequent time of exposure.

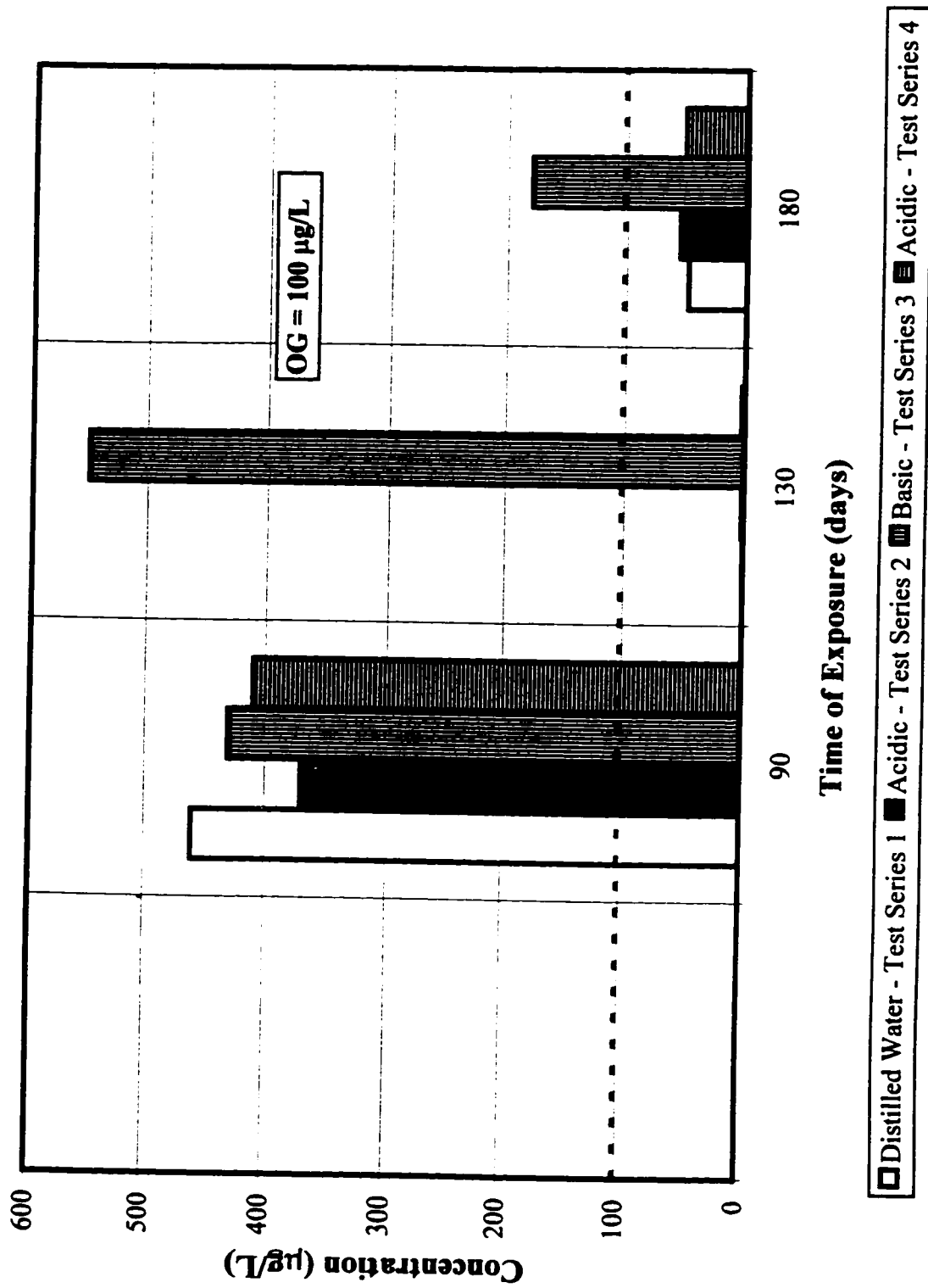


Figure 7.4. The variation in aluminum (Al) concentration as a function of the leaching environment and subsequent time of exposure.

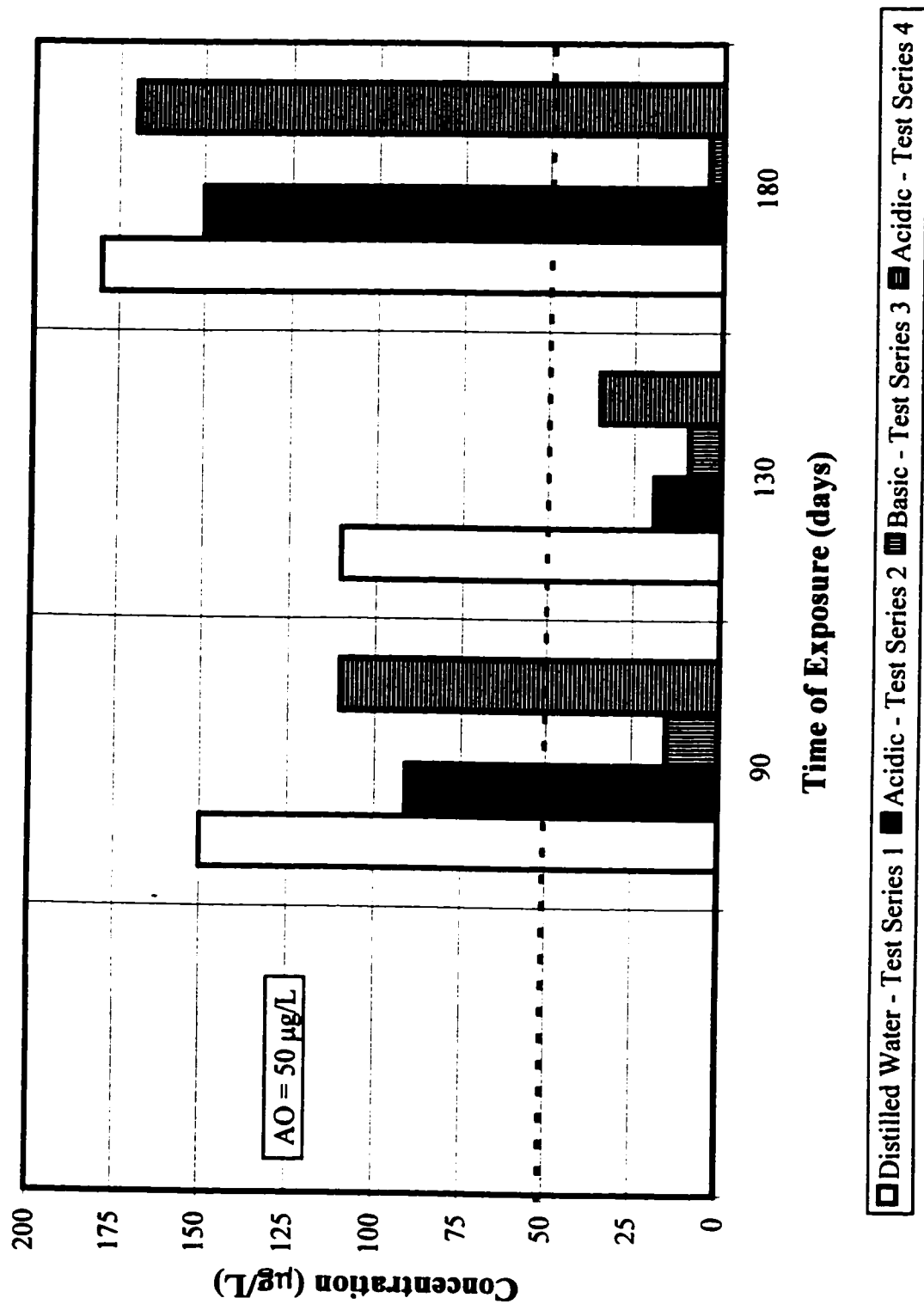


Figure 7.5. The variation in manganese (Mn) concentration as a function of the leaching environment and subsequent time of exposure.

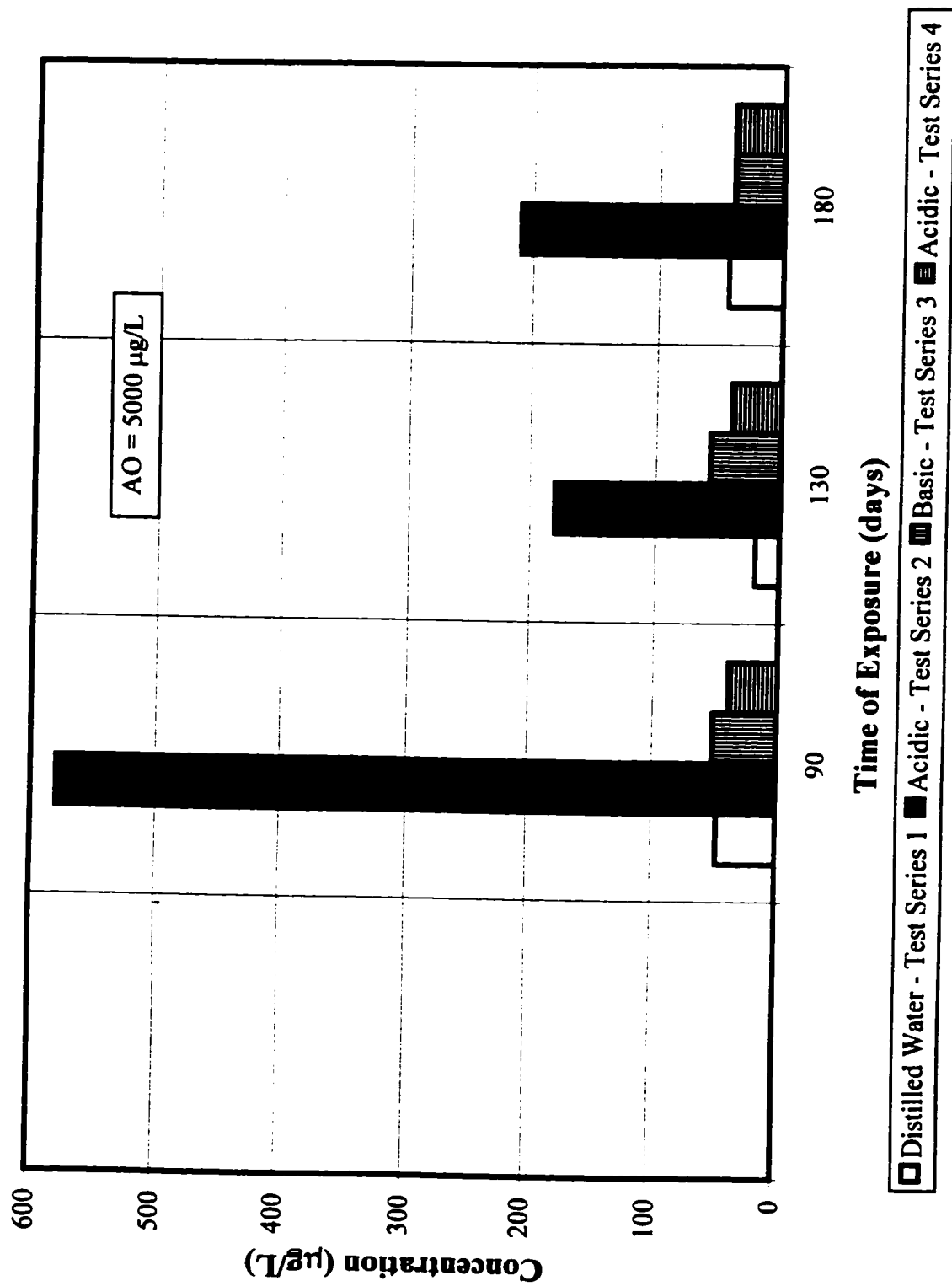


Figure 7.6. The variation in zinc (Zn) concentration as a function of the leaching environment and subsequent time of exposure.

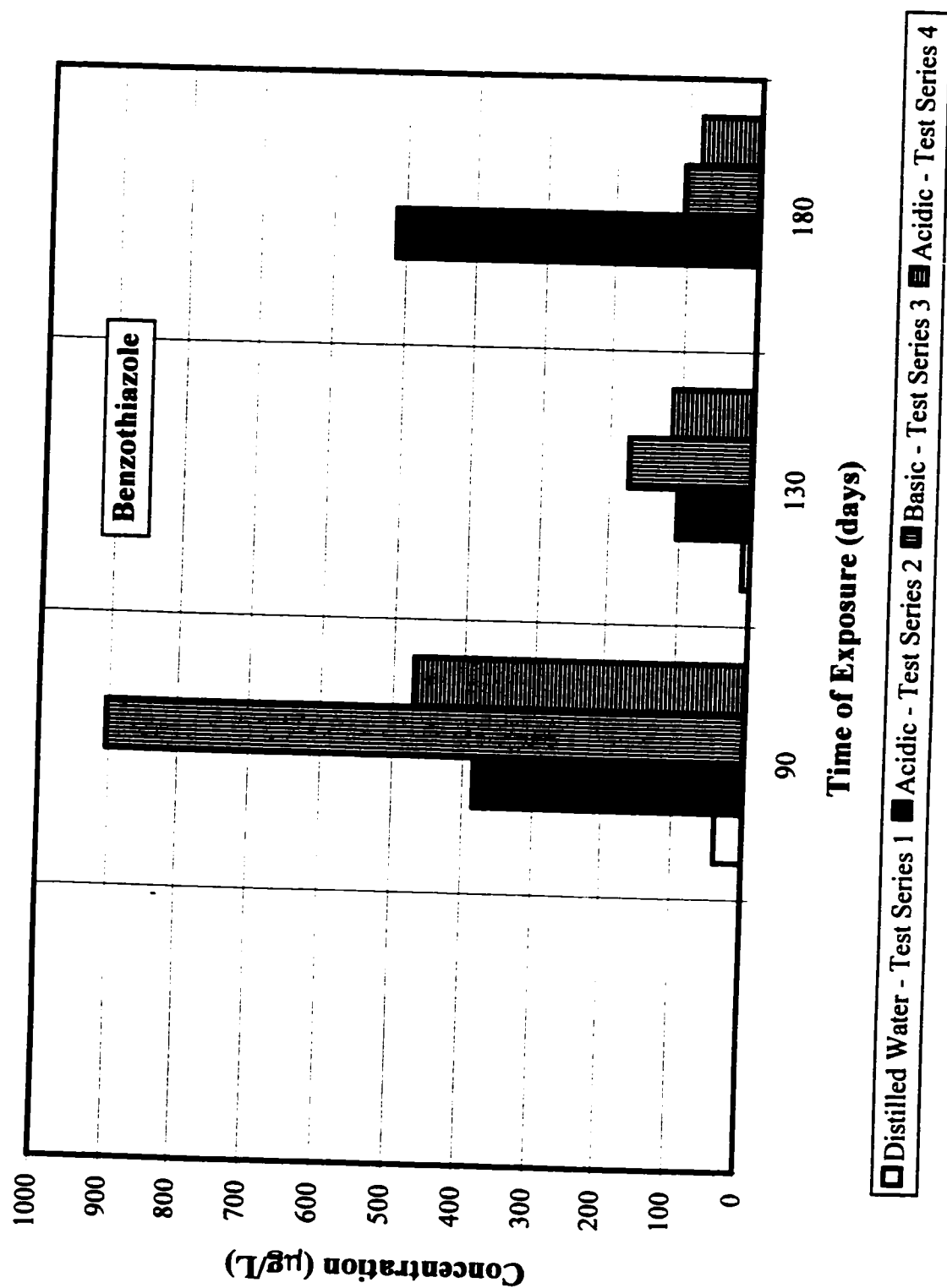


Figure 7.7. The variation in Benzothiazole concentration as a function of the leaching environment and subsequent time of exposure.

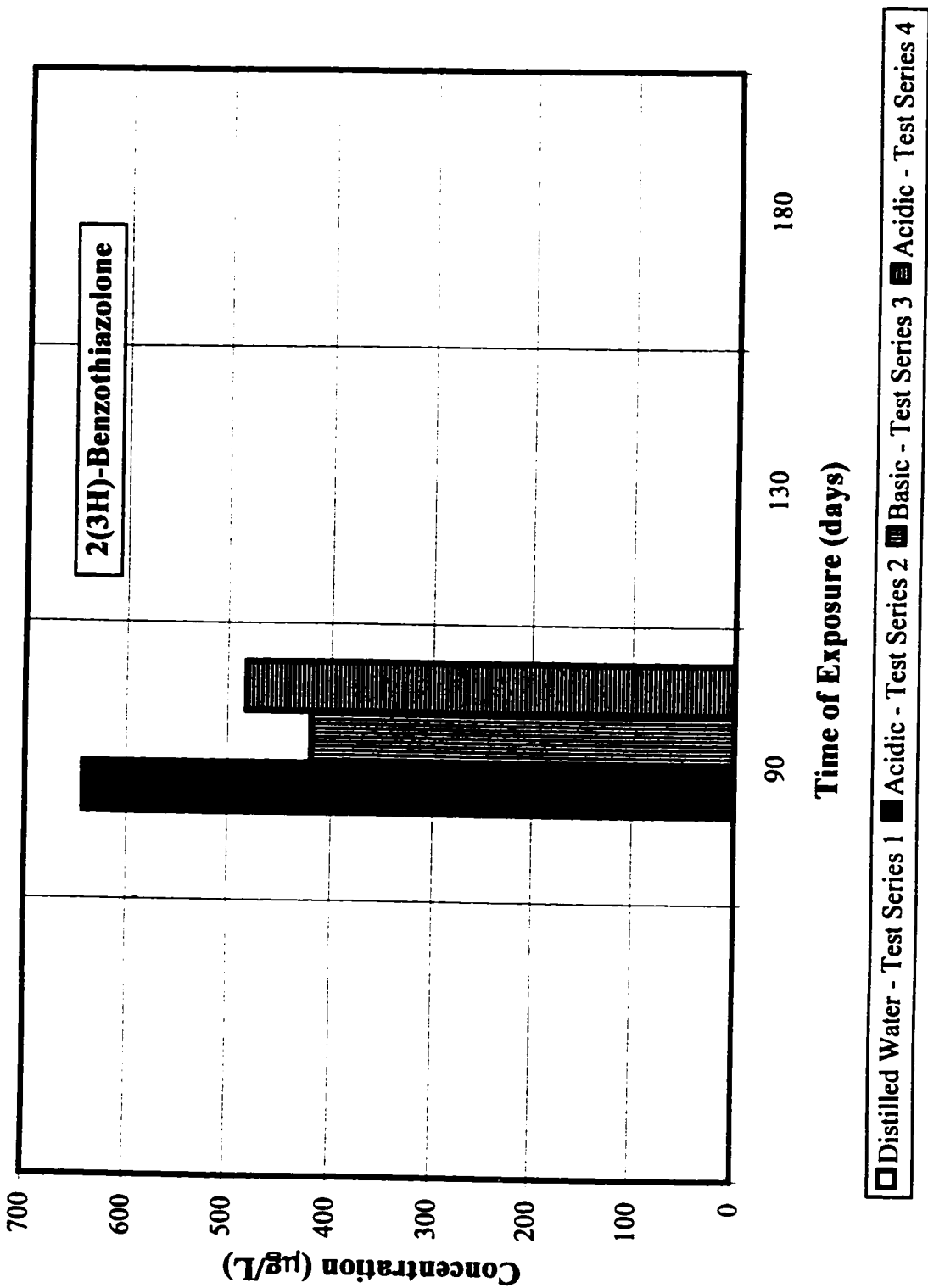


Figure 7.8. The variation in 2(3H)-Benzothiazolone concentration as a function of the leaching environment and subsequent time of exposure.

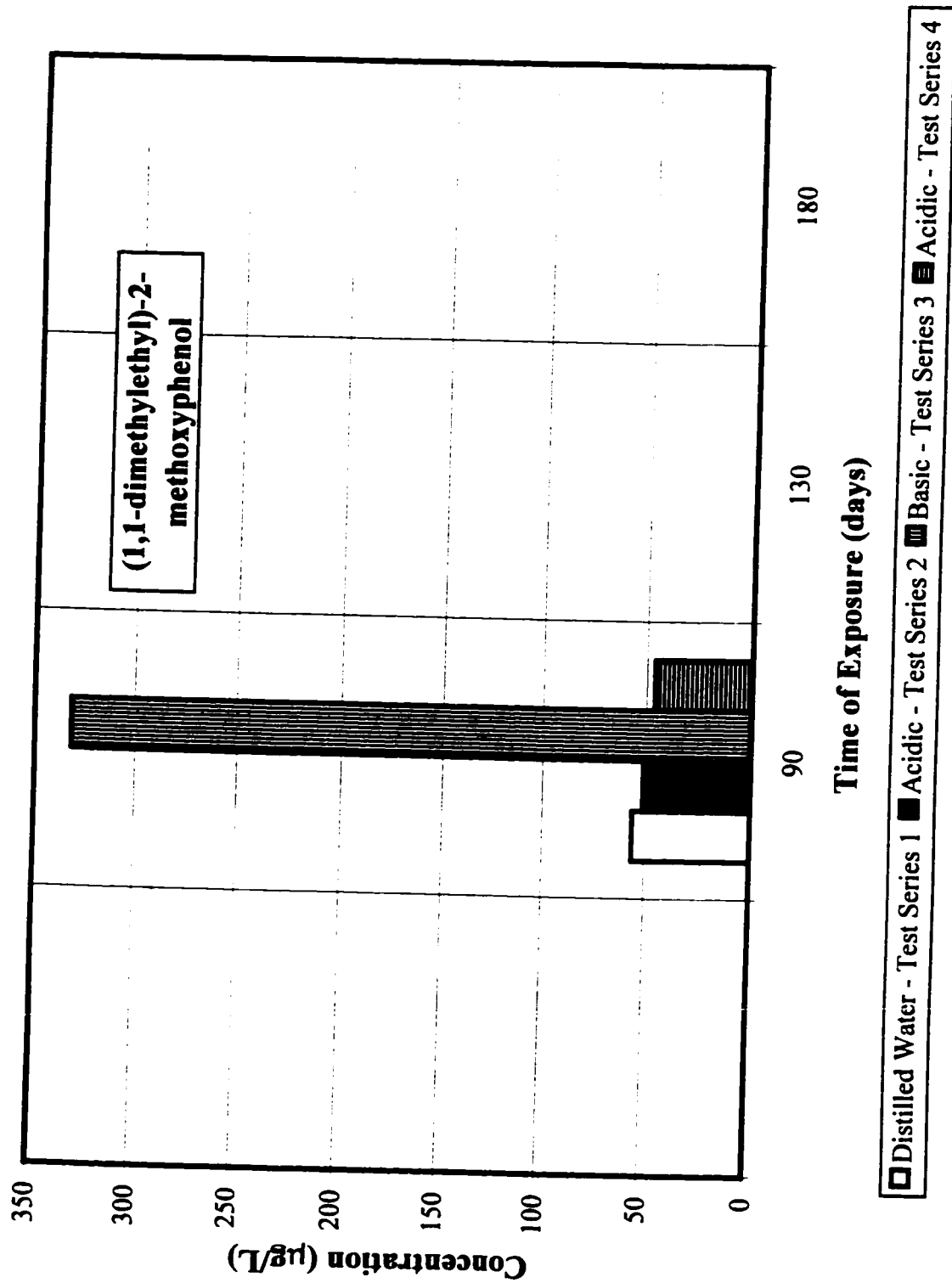


Figure 7.9. The variation in 3-(1,1-dimethylethyl)-2-methoxyphenol concentration as a function of the leaching environment and subsequent time of exposure.

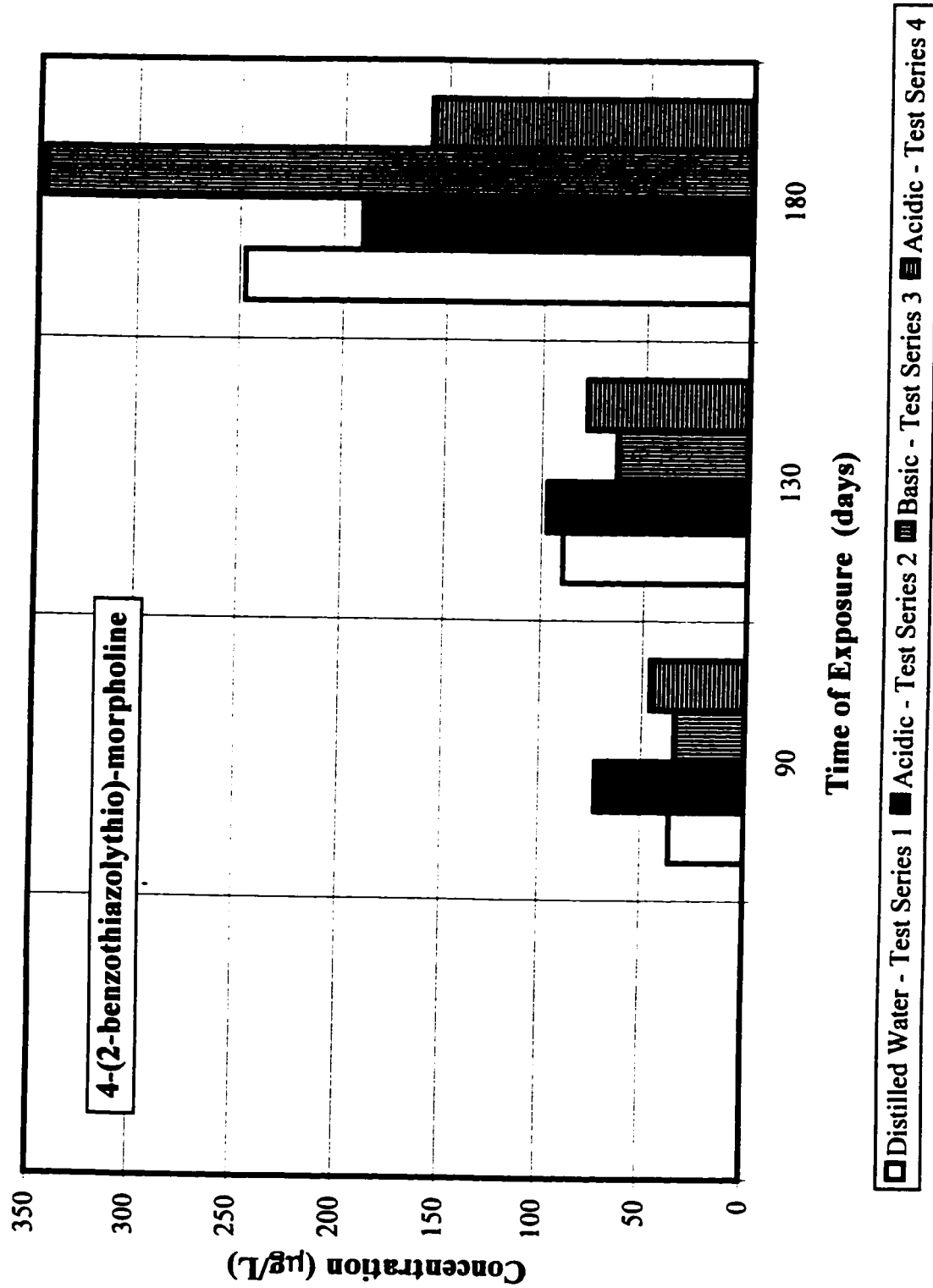


Figure 7.10. The variation in 4-(2-benzothiazolythio)-morpholine concentration as a function of the leaching environment and subsequent time of exposure.

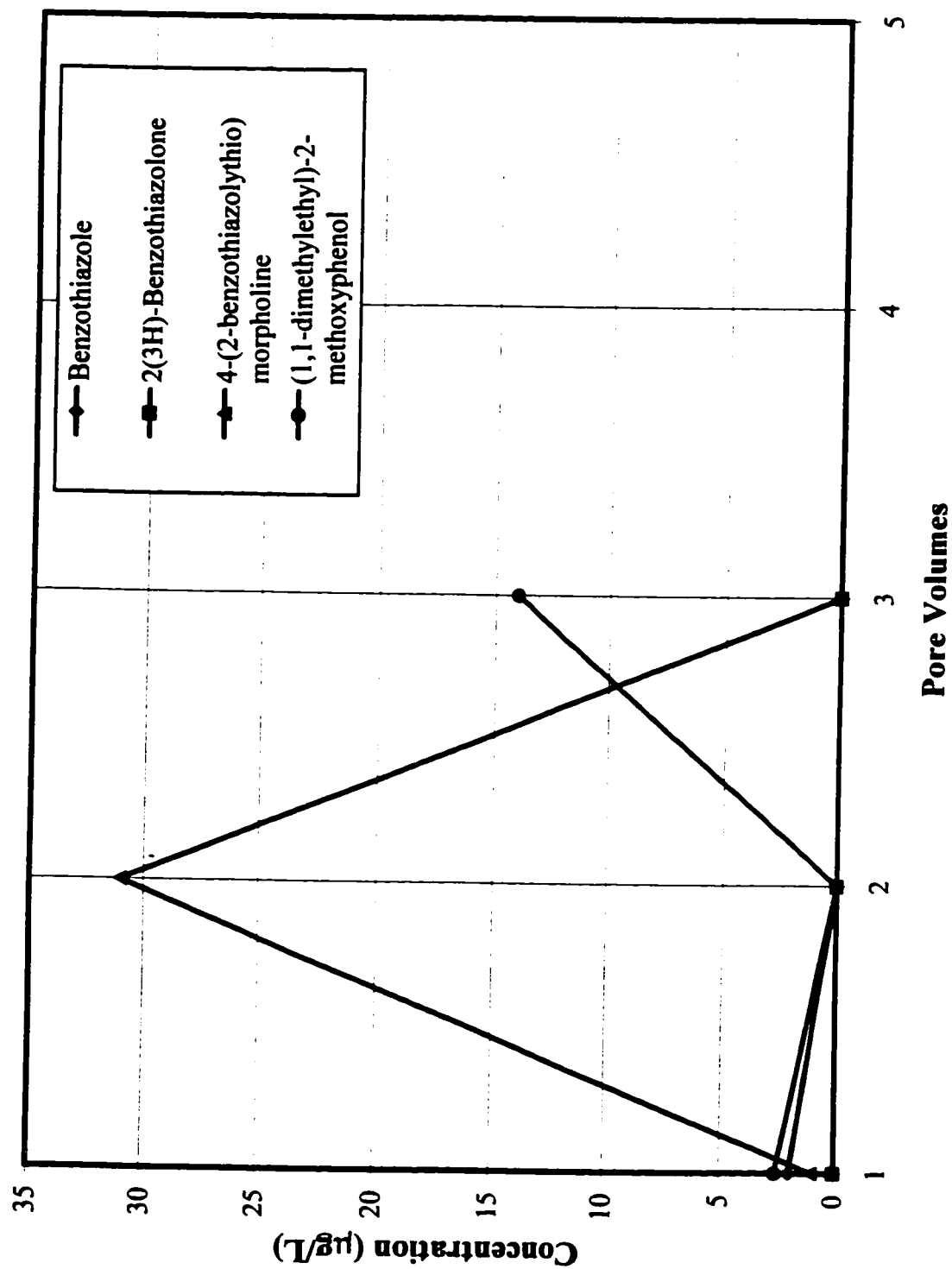


Figure 7.11. The variation in organic compound concentrations leached from tire chips embedded in a kaolin clay subjected to a neutral environment, Test Series 6.

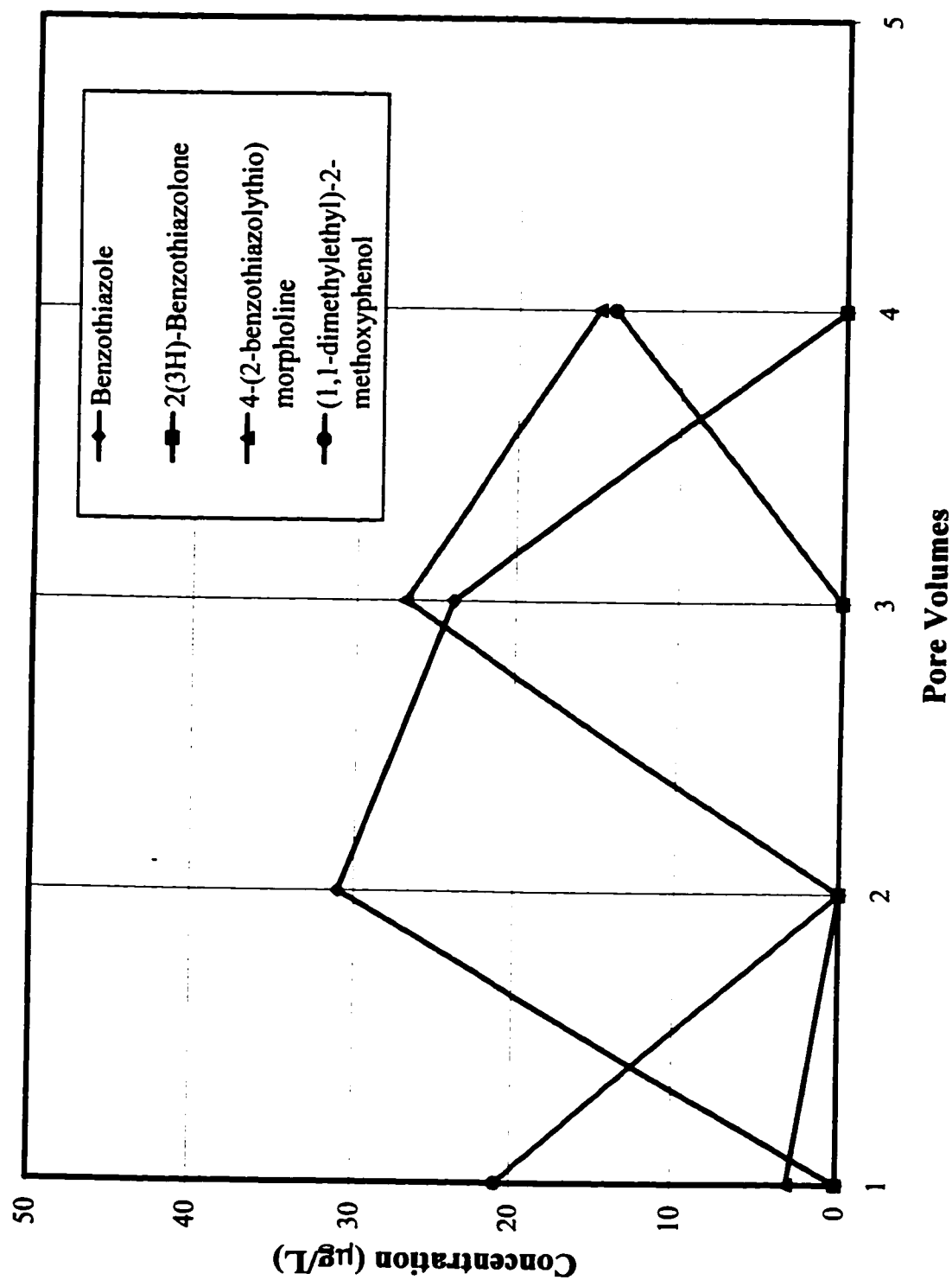


Figure 7.12. The variation in organic compound concentrations leached from tire chips embedded in a kaolin clay subjected to an acidic environment, Test Series 8.

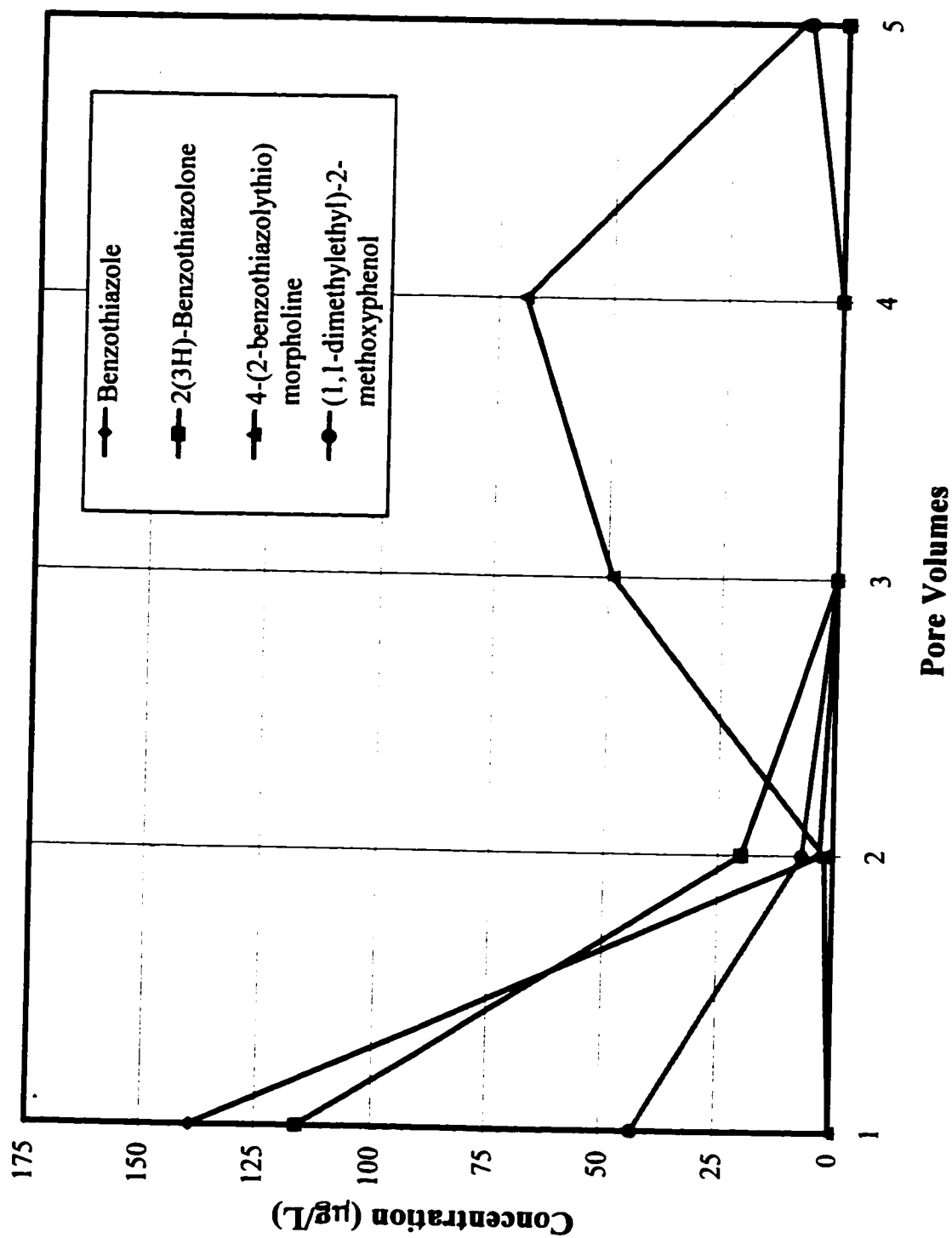


Figure 7.13. The variation in organic compound concentrations leached from tire chips embedded in a kaolin clay subjected to an acidic environment, Test Series 9.

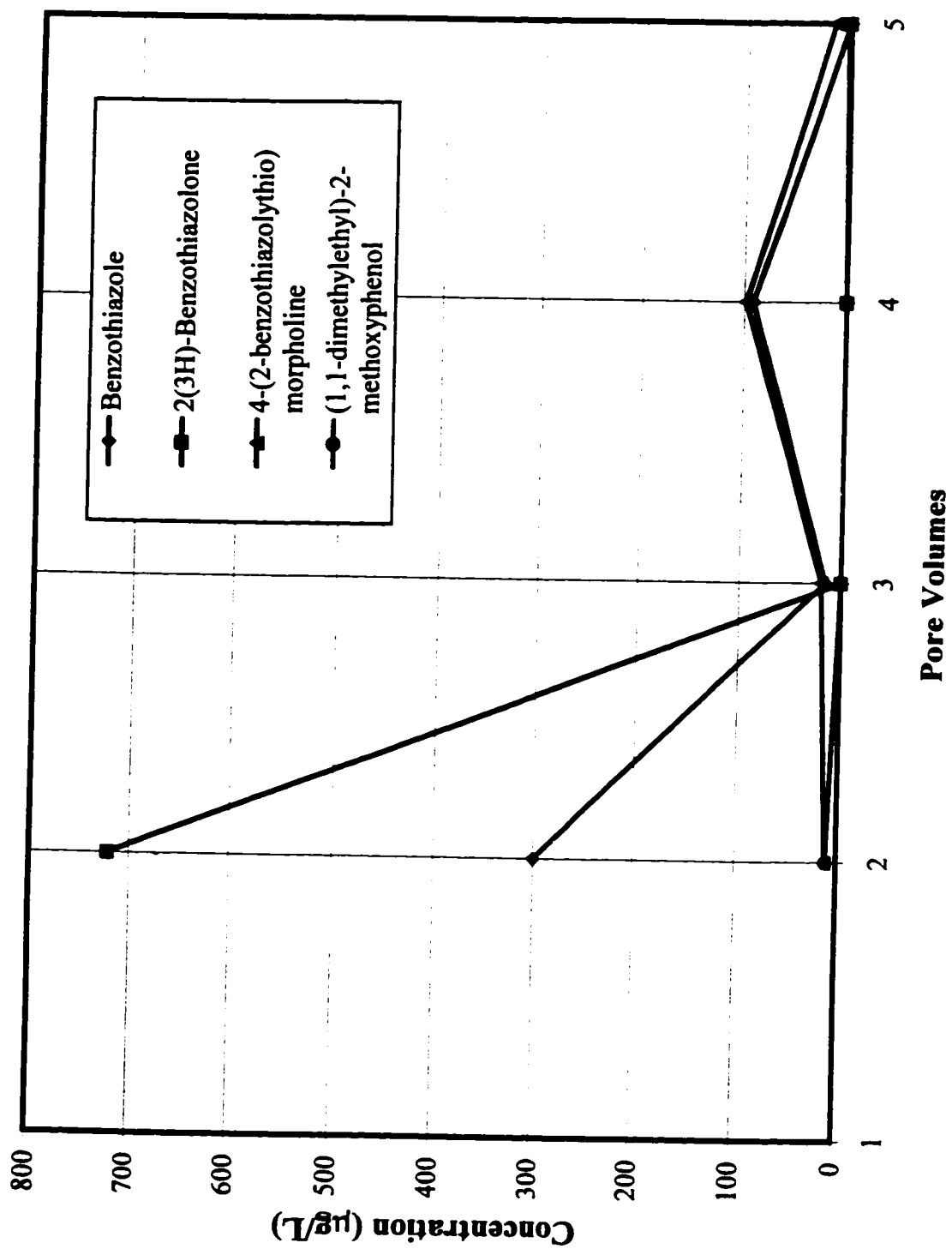


Figure 7.14. The variation in organic compound concentrations leached from tire chips embedded in a kaolin clay subjected to a basic environment, Test Series 11.

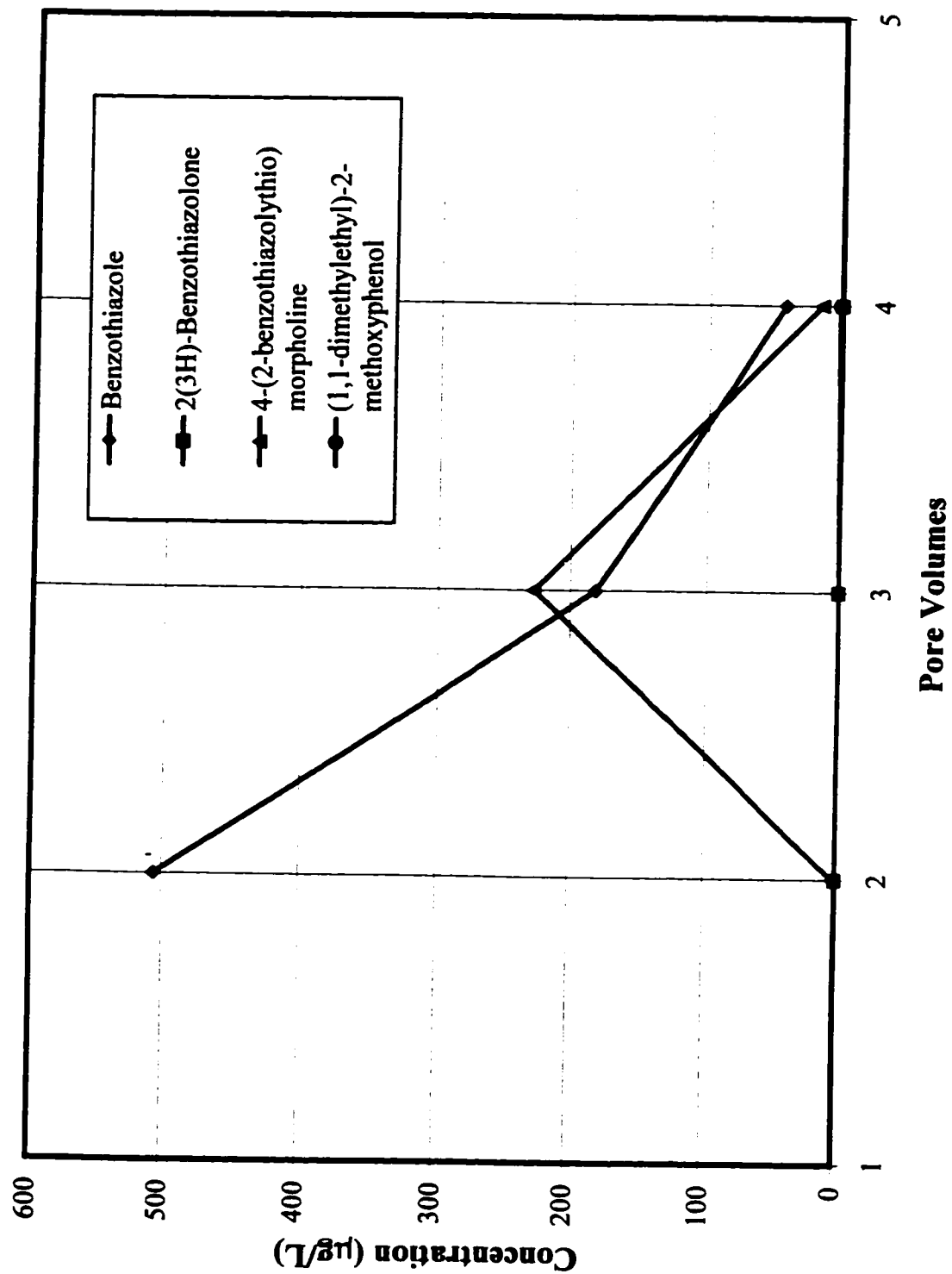


Figure 7.15. The variation in organic compound concentrations leached from tire chips embedded in a kaolin clay subjected to a basic environment, Test Series 12.

CHAPTER 8

RECOMMENDED CONSTRUCTION AND DESIGN GUIDELINES

8.1 Introduction

The following recommendations for the construction and the design of reinforced earth structures using scrap tires are based on the results of the current investigation. This investigation has provided a basic understanding of the principles factors governing tire reinforcement. The recommended guidelines try to cover all essential aspects of earth reinforcement related to scrap tires. However, improvements and additions to these basic guidelines will occur as more earth structures are constructed using scrap tire reinforcement.

8.2 Construction Guidelines

- The investigations have demonstrated the practical feasibility of constructing both tire reinforced fills and tire reinforced gravity retaining walls. These engineered structures can provide an economical and environmentally acceptable alternative for the use of waste tires. The structures can be constructed with conventional fill placement equipment. Virtually no damage was observed as the trucks and the lightweight compactors traversed over the tires.
- Tires used for such structures should preferably be either used or come from old stockpiles. The current available environmental data indicates that the use of new tires should be avoided.
- The structure is very amenable for construction with both cohesionless as well as cohesive soils. The quality of fill placed in such structures can be of an inferior quality in terms of their placement conditions (degree of compaction, water content etc.) than would be normally accepted in conventional earthfill.

- It is recommended that tires with one side wall removed should be used with clayey fills. For embankment constructed with a clayey backfill do require careful compaction, preferably with a sheep's foot type roller in lifts not exceeding 30 cm.
- Drainage measures should be provided in both retaining walls and fills constructed with cohesive soils. For the former, a semi -inclined pervious geotextile or sand drainage layer should be provided along the internal edge of the tires together with the associated base drainage measures. In the case of tire reinforced clay fill, it is recommended that horizontal drainage layers be provided at appropriate intervals. This would also assist in increasing the rate of consolidation of the clayey backfill so that the primary settlements would occur in a shorter period of time.
- The present research indicates that maximum batter angle for the retaining walls should not exceed 70 degrees when using a low quality backfill. Compaction behind the retaining wall should be carefully carried out in order to limit the development of high lateral stresses and also to reduce the outward lateral deformation. Overhang of tires must not be permitted.
- If the amount of lateral wall deformations is a concern, a trapezoidal reinforcement layout is recommended in which the reinforcement length is increased with the wall height.
- For retaining walls, the different tire layers should be staggered laterally in order to distribute voids uniformly across the structure. A complete infilling of the voids is desirable in order to develop a good interlocking friction between the different tire reinforcing layers and also to minimize wall settlement. Attachment of the front tire row to deeper embedded tire elements is required.
- Tires deteriorate under ultraviolet rays. In order to prevent the chemical breakdown of rubber, and also to prevent erosion of the face, a flexible protective facing must be provided after construction of both types of structures, preferably after most of the lateral deformation has occurred. The experience from the test fill indicates that the reinforced clay fill can be

satisfactorily vegetated, while tire reinforced retaining walls would require a facing. Shotcrete has been successfully used on a tire reinforced wall in Brazil (Garga, private Comm.).

- Tying of tires with polypropylene ropes was an acceptable method for creating tire mats. However it was a time consuming process. Other more convenient tire attachments, such as reinforcing steel clamps, rivets etc. may be considered in light of local regulations. On a larger construction project, it should also be possible to preassemble large lengths of tire mats. These mats can then be transported to the construction site, and will only require connecting the ends at the site.
- In order to reduce environmental impact, such structures should ideally be placed above permanent groundwater and should be constructed of old tires.

8.3 Design Guidelines for Retaining Walls

A general review of the design guidelines are provided in Chapter 5. It is emphasized that conventional design methodology for soil reinforcement can equally be applied to the design of reinforcement using tires. The following is a step-by step outline for the design of soils reinforced with used tires. The procedure outlined by Christopher et al. (1990) for conventional reinforced earth structures can be used after appropriate modifications for tire reinforcements, based on the result of the current investigations.

The following design guidelines are suggested for the construction of simple retaining wall structures with a face inclination of 70° to 90° (a near vertical face), a uniform width (rectangular geometry) and a height of up to 10 m. The measures would be applicable for structures higher than this height; however detailed analyses would be required, which are beyond the scope of the present recommendations. Further details of the procedure are outlined in section 8.3.2. Modification to these procedures for trapezoidal tire retaining walls are also provided. The design steps are:

Step 1: Establish design parameters, scope of the project, and external loads:

- External wall height, H .
- Wall face batter, ω .
- Total length of wall and variation in wall height along the length.
- Slope of the backfill, β .
- External loads and their locations.
- Type of tire reinforcement used (whole tires, cut tires). Note: it is dependent on the characteristics of the backfill (see section 8.1).
- Type of attachment.
- Design and service life periods.
- Environmental conditions such as frost action, drainage, seepage and chemical nature of the backfill.

Step 2: Determine the engineering properties of the foundation soil:

- foundation conditions.
- strength parameters of foundation (angle of internal friction ϕ'_f , cohesion c'_f) and unit weight (γ_f).
- consolidation parameters (C_c , C_r , c_v and σ'_{pmax}) for each foundation stratum.
- location of ground water table. Establish drainage requirements.

Step 3: Determine the backfill properties of both retaining wall section and retained backfill (see Chapter 3 for more details)

- water content, gradation, and plasticity index.
- compaction characteristics (dry unit weight γ_d , optimum water content w_{opt} , or relative density D_r)

- Peak angle of internal friction ϕ'_{re} , from drained direct shear tests for the material used in the retaining wall section and ϕ'_b from either drained direct shear or triaxial tests for the retained backfill (if different from that used in the retaining wall). Note that caution should be exercised in the use of the cohesion term.
- Deformation characteristics.
- Chemical and biological characteristics (pH, chloride, sulphides and other agents in soil or ground water that may affect the durability of the tire reinforcement, see sections 2.9.2, 2.13, and 3.2.4)

Step 4: Establish design factors of safety and construction criteria. The recommended minimum values are listed below. However local building codes may require higher values:

- Sliding: F.S. ≥ 1.5 .
- Overturning: F.S. ≥ 2.0 .
- Bearing Capacity: F.S. ≥ 2.0 .
- Overall Stability: F.S. ≥ 1.5 .
- Settlement: based on the project requirements.
- Lateral displacement: determine the tolerable total movement and the differential displacement based on wall batter, ω , and deformations due to construction activity.

Typical construction displacement provisions are:

- Coarse-grained backfill:

10 mm per metre of wall height for cut tire reinforcement.

15 mm per metre of wall height for full tire reinforcement.

- Cohesive backfill

40 mm per metre of wall height for cut tire reinforcement.

Step 5: Establish preliminary wall dimensions (see section 8.3.1).

- geometric configuration of the wall (tire reinforcement layout).
- preliminary reinforcement length L .

Step 6: Establish interface parameters between soil and tire reinforcement.

- The peak interface friction angle (δ) under drained conditions (see section 4.3)
- determine the direct sliding efficiency coefficient (α_{ds}) from Equation 4.3.

Step 7: Determine the lateral thrust at the back of the wall and the vertical stress distribution at the base of the wall taking into account surcharge loads (see section 8.3.1), and relative compressibility of wall and backfill.

Step 8: Check wall stability (see section 8.3.1)

- Sliding resistance.
- Overturning of the wall.
- Bearing capacity.
- Overall stability.
- Compound failure.

Adjust preliminary tire reinforcement length as necessary.

Step 9: Estimate the settlement of the tire retaining wall using conventional settlement analysis.

Step 10: Evaluate anticipated lateral displacement (see sections 6.4 and 8.3.1.5).

8.3.1 Detailed Procedure

A more detailed design procedure for steps 5, 6, 7, 8 and 10 are given below.

8.3.1.1 Preliminary Wall Dimensions

The geometric configuration of the retaining wall and reinforcement length are governed by the strength and deformability of the backfill used. For a good quality backfill, a permeable coarse-grained soil, a uniform reinforcement layout can be employed (the reinforcement length remains constant throughout the height of the structure). A reduction in the reinforcement length at the upper levels of the retaining wall should be avoided. However, depending on the external loading conditions, wall batter and stiffness of the backfill, a shorter reinforcement length can be used near the top of the wall so long as it satisfies both stability and lateral deformation requirements. For retaining wall constructed on a firm foundation and using a coarse-grained backfill, a preliminary reinforcement length of $0.85H$ should satisfy all requirements. Structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcements for stability, often on the order of $0.95H$ to as high as $1.3H$.

The different retaining wall geometries analysed in Chapter 6 and the corresponding wall deformation results can be used as a preliminary estimate of the reinforcement layout requirements when using a low quality backfill such as silty clay or any fine-grained soil. The required reinforcement length may be governed by the tolerable lateral displacement. If so, in order to lower the reinforcement length requirements, and therefore, to minimize the costs, it is suggested that the wall be sloped at 70 degrees and a trapezoidal reinforcement layout be employed.

8.3.1.2 Interface Parameters

The interface friction angle δ between the tire reinforcement and the soil in contact should be determined for:

1. The backfill material used (δ_b).

2. The foundation soil (δ_f), if its shear strength is less than that of the backfill material.

The soil-tire rubber interface friction angle δ should be determined, preferably by means of an interface direct shear test under drained conditions. It can also be indirectly evaluated from pull-out tests, based upon the bond efficiency coefficient (α_b).

If the interface friction angle can not be evaluated from a direct shear test apparatus, the following relationships from the present research are suggested:

- Coarse-grain soils: $\delta \cong 0.7\phi'$
- Fine-grain soils (cohesive soils): $\delta \cong 0.8\phi'$

where ϕ' is the effective internal friction angle of the soil.

A coefficient value lower than 0.7 can be used, if a more conservative estimate is warranted.

The frictional resistance developed at the base of the retaining wall is reduced by the weaker contact between tire sidewall and soil. The coefficient of resistance to direct sliding in this case is determined by:

$$\alpha_{ds} = 1 - f \left(1 - \frac{\tan \delta}{\tan \phi'} \right) \quad (8.1)$$

The f factor represents the fraction of the sidewall surface area in a unit area of a tire mat reinforcement. On average, sidewalls contribute to approximately 25% of the gross tire mat area, and therefore, the f factor can be assumed in most cases to be equal to 0.25. (Note: for pull-out resistance calculations where α_b is assumed to equal α_{ds} , $f \approx 0.5$ for whole tire reinforcement with two sidewalls, since the failure surface develops at both upper and lower surfaces).

8.3.1.3 Lateral Earth Pressures and Vertical Stresses for Retaining Walls

A. Lateral Earth Pressure

The lateral thrust at the back of a compressible retaining wall with a sloping surcharge is shown in Figure 8.1. The thrust due to lateral earth pressure is assumed to be inclined upwards relative to the horizontal by an inclination angle λ , where $\lambda = (\theta + \delta) - 90^\circ$. This thrust angle takes into account the wall batter and the wall friction (relative downward movement of the tire retaining wall with respect to the backfill). The thrust (P_b) at the back of the wall is equal to:

$$P_b = \frac{1}{2} K_a \gamma_b H'^2 \quad (8.2)$$

$$\text{where,} \quad H' = H + \frac{L \sin \beta}{\sin(\theta - \beta)} \cos \omega \quad (8.3)$$

The influences of other forces such as surcharge or concentrated loads must be added. Based on the current investigation, it is advisable to use Coulomb's active earth pressure coefficient (K_a) which takes into account wall batter, negative wall friction, and sloping backfill. For complex retaining wall problems, a graphical technique or a computer program is required for applying Coulomb's method to design.

$$K_a = \left[\frac{\csc \theta \sin(\theta - \phi'_b)}{\sqrt{\sin(\theta - \delta)} + \sqrt{\frac{\sin(\phi'_b - \delta) \sin(\phi'_b - \beta)}{\sin(\phi'_b - \beta)}}} \right]^2 \quad (8.4)$$

B. Vertical Stress Distribution

The vertical stress distribution at the base of the wall is shown in Figure 8.1. The vertical stress acting at the base of the wall is estimated using the method proposed by Meyerhof (1953). It is

assumed that the eccentric loading condition results in a uniform redistribution of pressure over a reduced area at the base of the wall. The length of the wall is reduced by twice the eccentricity as shown in Figure 8.1. It is important to note that the actual stress distribution at the base of a retaining wall constructed with tire mat reinforcement is unknown, since, stresses at the base of the wall were not measured in the test embankment. The steps for estimating the equivalent uniform vertical stress are:

1. Determine λ , where $\lambda = (\theta + \delta) - 90^\circ$.
2. Determine wall batter ω , where $\omega = 90^\circ - \theta$.
3. Calculate P_b , see equation 8.2.
4. Estimate the unit weight of the tire wall (γ_{re})

The unit weight of the tire wall (γ_{re}) can be related to the unit weight of the backfill (γ_b) by the following relationships:

Coarse-grained backfills:

$$\text{Cut tire reinforcement, } \gamma_{re} \approx 0.95 \times \gamma_b$$

$$\text{Full tire reinforcement, } \gamma_{re} \approx 0.85 \times \gamma_b$$

Fine-grained (cohesive) backfills:

$$\text{Cut tire reinforcement, } \gamma_{re} \approx 0.80 \times \gamma_b$$

5. Calculate the eccentricity of the resultant force, e , by considering moment equilibrium at the center of the retaining wall (point O in Figure 8.1); i.e. $\sum M_o = 0$, and where V represents the sum of the vertical forces. This condition yields:

$$e = \frac{P_b (\cos \lambda) \left(\frac{H'}{3} \right) + P_b (\sin \lambda) \left(\frac{L}{2} + \frac{H'}{3 \tan \theta} \right) - W' \left(d - \frac{L}{2} \right)}{\gamma_{re} HL + W' - P_b \sin \lambda} \quad (8.5)$$

5. Calculate the equivalent uniform vertical stress on the base (σ_v):

$$\sigma_v = \frac{\gamma_{re}HL + W' - P_b \sin \lambda}{L - 2e} \quad (8.6)$$

6. Add the influence of surcharge and concentrated loads to σ_v .

8.3.1.4 Stability Analysis

A. Sliding Resistance

Sliding along the base of the tire wall occurs when the driving force is greater than the frictional resistance. The resisting force at the base of the tire wall must be large enough to withstand horizontal sliding forces applied to the back of the wall. It is required that:

$$F.S_{\text{sliding}} = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Driving Forces}} \geq 1.5 \quad (8.7)$$

Figure 8.1 shows the calculation of the sliding stability for an inclined compressible tire wall retaining with a sloping backfill. Reference to the Canadian Foundation Engineering Manual should be made for additional surcharge loads such as wheel loads (point loads) and traffic barriers (line loads). The calculation steps are:

1. Calculate the thrust P_b (equation 8.2)
2. Calculate the sum of the driving forces (F_D): $P_b \cos \lambda$
3. Determine the most critical interaction properties (see section 8.3.1.2) at the base. Choose the minimum friction properties of: the backfill ϕ'_b , the foundation ϕ'_f or the soil fill used in the reinforcement zone, ϕ'_{re} .
4. Determine the coefficient of resistance to direct sliding (α_{ds}), equation 8.1.
5. Calculate the sum the resisting forces (R_s):

$$R_s = (\gamma_{re}HL + W' - P_b \sin \lambda) \alpha_{ds} \tan \phi_{(b,f,re)} \quad (8.8)$$

Only permanent external loads should be included in the calculation of the resisting force (R_s). For instance, live traffic loads should not be included.

6. Calculate the factor of safety with respect to sliding:

$$F.S_{\text{sliding}} = \frac{R_s}{F_D} = \frac{((\gamma_{re} HL + W') - P_b \sin \lambda) \alpha_{ds} \tan \phi_{(b,f,re)}}{P_b \cos \lambda} \quad (8.9)$$

7. If the value does not meet the specified requirements:

- Increase reinforcement length, L .
- Increase wall batter, ω .
- Decrease backfill slope angle β .

Repeat the calculations.

B. Overturning

The tire wall is assumed to behave as a monolithic block of material. However, due to the flexibility of the structure, failure in this manner is questionable. Nonetheless, it is recommended that an adequate factor of safety should be used to limit excessive outward tilting and distortion of a suitably designed wall (Christopher et al. 1990). Overturning stability is analyzed by considering rotation about the toe requiring that the sum of the resisting moments divided by the sum of the driving moments should be greater than or equal to the prescribe factor of safety:

$$F.S_{\text{overturning}} = \frac{\sum \text{Resisting Moments}}{\sum \text{Driving Moments}} \geq 2.0 \quad (8.10)$$

The calculation steps for a compressible reinforced tire wall with a sloping backfill and batter (Figure 8.1) are:

1. Calculate the sum of the driving moments, M_d , including all live loads:

$$M_d = (P_b \cos \lambda) \left(\frac{H'}{3} \right) \quad (8.11)$$

2. Calculate the sum of the resisting moments, M_r , excluding live loads:

$$M_r = (\gamma_{re} HL) \left(\frac{L}{2} + \frac{H}{2 \tan \theta} \right) + W'd - (P_b \sin \lambda) \left(L + \frac{H'}{3 \tan \theta} \right) \quad (8.12)$$

3. Calculate the factor of safety with respect to overturning:

$$F.S_{\text{overturning}} = \frac{M_r}{M_d} = \frac{(\gamma_{re} HL) \left(\frac{L}{2} + \frac{H}{2 \tan \theta} \right) + W'd - (P_b \sin \lambda) \left(L + \frac{H'}{3 \tan \theta} \right)}{(P_b \cos \lambda) \left(\frac{H'}{3} \right)} \quad (8.13)$$

4. If the value does not meet the specified requirements, increase the reinforcement length, L .

5. Calculate the eccentricity, e , equation 8.5, and verify that the eccentricity does not exceed $L/6$.

If $e > L/6$, increase the reinforcement length.

C. Bearing Capacity

Stresses imposed by the self weight of the tire wall and surcharge should not exceed the bearing capacity of the foundation soil. The vertical stress at the base is estimated using the Meyerhof distribution (1953). A minimum safety factor of 2 is generally accepted for reinforced soil walls. The flexible nature of the tire wall and the ability to accommodate large differential settlement reduces the required minimum factor of safety when compared to conventional reinforced concrete retaining walls ($FS \geq 3$). This stability check requires that:

$$F.S_{\text{Bearing Capacity}} = \frac{\text{Bearing Capacity of Foundation}}{\text{Vertical Stress Distribution at the Base}} = \frac{q_{ult}}{\sigma_v} \geq 2.0 \quad (8.14)$$

The calculation steps for an inclined compressible retaining wall with a sloping surcharge (Figure 8.1) are:

1. Calculate the eccentricity, e , of the resultant force at the base of the wall, equation 8.5.
2. Calculate the vertical stress distribution, σ_v , assuming Meyerhof distribution, equation 8.6.
3. Determine the ultimate bearing capacity q_{ult} using methods outlined in the CFEM (1985), e.g.:

$$q_{ult} = cN_c(\phi) + \frac{1}{2}(L - 2e)\gamma_f N_\gamma(\phi) \quad (8.15)$$

where N_c and N_γ are dimensionless bearing capacity coefficients.

4. Verify that: $\sigma_v \leq \frac{q_{ult}}{2}$.
5. The vertical stress (σ_v) will decrease and the bearing capacity (q_{ult}) will be increased by lengthening the tire mat reinforcement. However, an excessive reinforcement length will result in a massive tire wall which may not be economical, and therefore, an improvement of the foundation soil will be more beneficial.

D. Overall Stability

The tire retaining walls must also satisfy overall slope stability. The overall stability is determined using an appropriate classical slope stability analysis method. Computer programs are available for this type of analysis. For simple structures with a rectangular geometry and constructed with a good quality backfill, analysis of compound failure surfaces passing through both the backfill and the reinforced wall will not be generally necessary. In this case, the retaining structure can be considered as rigid body and only failure planes passing completely outside the reinforced zone are considered. The stability analysis should include compound failure surfaces when the retaining structure is more complex, and where use of a cohesive backfill, changes in length of tire reinforcement, or high surcharge loads may occur. The integrity of the structure can be verified by analyzing various failure planes and ensuring that there is sufficient resistance along each critical plane.

8.3.1.5 Lateral Wall Displacement Evaluation

Unfortunately, there is no simple procedure available to estimate the amount of lateral deformations which would occur in a tire reinforced retaining wall. The primary factors governing wall deformations are the compaction effort, reinforcement length, reinforcement layout, shear strength and deformability of backfill, compressibility of the foundation, and external loadings. Provisions for construction movements are outlined in section 8.3, Step 4. Post construction movements, including surcharge loads, of simple tire wall structures can be evaluated by the results of the finite element analyses presented in Chapter 6. An estimate of the lateral wall movement can be obtained by simply multiplying the deformation of the standard wall used in the analysis by the appropriate deformation indices, provided in Figure 6.8 and Figures 6.10 to 6.16. The combined effects of wall geometry, backfill strength and deformation characteristics, and external loading conditions, such as reinforcement length, batter, and uniform surcharge can be estimated by multiplying the deformation indices in series. A finite element analysis should be performed to evaluate the lateral deformation for more complex structures than those outlined in Chapter 6 or for critical structures requiring precise tolerances.

If the anticipated lateral displacement is excessive, adjust the preliminary wall geometry by one or a combination of:

- increase wall batter, ω .
- increase reinforcement length, L .
- a trapezoidal reinforcement layout.
- extend the tire reinforcement beyond the stable slope of the backfill (ϕ').

8.3.2 Tire Walls Having a Trapezoidal Geometry

The design of trapezoidal walls requires two analyses (Christopher et al. 1990):

1. Standard stability analysis in which the wall geometry is simplified.

- The tire wall with a trapezoidal distribution of length of reinforcement is represented by an equivalent rectangular block (L_1 , H') having the same total height and the same cross-sectional area (Figure 8.2).
- The thrust (P_b) at the back of the is inclined at an angle equaled to λ , where $\lambda = (\theta + \delta) - 90^\circ$.

2. Undertake a global stability analysis.

8.4 Design Guidelines for Reinforced Slopes

As stated in Chapter 5, the stability analysis of reinforced slopes and embankments are performed using modified versions of the classical limit equilibrium slope stability methods. The reinforcement increases the resisting force or moment by providing a tensile force, of specified amount, acting at their installed location within the embankment or slope. Usually the shear and bending strengths of the tire reinforcement are not considered and therefore the analysis is somewhat conservative. Generally, all methods provided essentially the same factor of safety, provided that the reinforcement forces are included in the equations of horizontal, vertical and moment equilibrium (Wright and Duncan 1991).

The following design procedures are recommended for slopes which are to be constructed over a stable foundation. It does not include cases where the foundation soils may fail (deep seated failure). A good engineering judgment in the selection of the appropriate design parameters is required. The design steps for reinforcing a steep slope are:

Step 1: Establish the geometric and loading conditions:

- slope height, H .
- slope angle, θ .
- external (surcharge) loads.

Step 2: Determine the engineering properties of the natural soils in the slope:

- determine the foundation and retained soil profiles.
- strength parameters of the foundation (c_u , ϕ_u , c' , ϕ') and unit weight (γ_f).
- consolidation parameters (C_c , C_r , c_v and σ'_{pmax}) for each foundation stratum.
- location of ground water table. Establish drainage requirements.

Step 3: Determine the backfill properties (see Chapter 3 for more details):

- water content, gradation, and plasticity index.
- compaction characteristics (dry unit weight γ_d , optimum water content w_{opt} , or relative density D_r)
- Peak shear strength parameters, (c_u , ϕ_u , c' , ϕ')
- Chemical and biological characteristics (pH, chloride, sulphides and other agents that may affect the durability of the tire reinforcement, see sections 2.9.2, 2.13, and 3.2.4)

Step 4: Establish tire reinforcement requirement:

- Type of tire reinforcement used (whole tires, cut tires, sidewalls only...). Note: it is dependent on the characteristics of the backfill (see section 8.1).
- Type of attachment and strength characteristics.
- Design and service life periods.
- Determine the allowable tensile force per attachment with respect to service life and durability requirements (see Chapter 2, section 2.9):

-

-For steel attachment: $T_a = 0.48\sigma_v A_c$ (8.16)

where A_c is the cross-sectional area minus corrosion loss.

-For polymeric attachment: $T_a = \frac{T_{ult} (CRF)}{(FD)(FC)(FS)}$ (8.17)

where CRF, FD, FC, and FS are strength reduction factors as described in section 2.9.1.2.

Step 5: Establish performance requirements. The recommended minimum values are listed below; however local building codes may require higher values:

1. External Stability:

- Sliding: F.S. = 1.5 for granular soils. Use a F.S. = 2.0 for cohesive soils where the permanence of the cohesive component is questionable.
- Overall Stability (Deep Seated): F.S. = 1.3.
- Compound Failure (through the reinforced zone): F.O.S. = 1.3.
- Settlement: site specific.

2. Internal Stability:

- Slope Stability: F.S. = 1.3 or greater.
- Pull-out Resistance: FS_p of 2.5 is recommended which incorporates an allowance for deformability.

Step 6: Determine internal stability requirements:

Internal failure occurs when the critical failure surface passes through the reinforced zone. The slope fails either by pull-out of the reinforcement or rupture of the reinforcing elements. The stability of the reinforced slope can be verified by several simplified approaches. The method illustrated in Figure 8.3 (same as Figure 5.5), showing conventional rotational slip surface methods, was recommended by Christopher and Holtz (1984). This approach can accommodate a variety of conditions depending on the analytical method used (e.g. Bishop, Janbu, etc.). To determine the necessary tire reinforcement layout, it is necessary to find the most critical failure plane having the largest tensile force per unit width of reinforcement (T_s) requirements. The essential design procedure is:

- a) Check unreinforced stability: Perform a stability analysis without reinforcement using conventional methods to determine safety factors and driving moments for potential failure surfaces for both short- and long-term conditions. Potential failure surfaces include both circular and wedge-type passing through the toe, the face (at any elevation) and the foundation below the toe. Determine the zone to be reinforced, by plotting the various failure surfaces on a cross-section of the slope. The reinforcing limits are equal to the surfaces which just meet the target factor of safety (FS_r).
- b) For each of the potential circular failure planes inside the reinforcing limits in step (a), calculate the total required reinforcing tension T_s using the following equation (see Figure 8.3 for more details):

$$T_s = (FS_r - FS_u) \frac{M_D}{D} \quad (8.18)$$

Where:

T_s = sum of the required tensile force per unit width of reinforcement (considering rupture and pull-out) in all reinforcement layers intersecting the failure surface.

M_D = driving moment about the center of the failure circle.

D = the moment arm of T_s about the center of the failure circle.
 = vertical distance, Y , to the centroid of T_s . Assume $H/3$ above the base of the slope for preliminary calculations. The tire reinforcement layer is fairly stiff in the horizontal direction (perpendicular to the failure surface), and therefore, the tensile force T_s is assumed to act parallel to the reinforcement layer.

FS_r = target minimum slope safety factor.

FS_u = unreinforced slope safety factor.

The largest calculated T_s value establishes the required reinforcement design tension, T_{max} . For cohesive soils, establish both short- and long-term maximum reinforcement tensile requirements.

c) Calculate the redistributed maximum tensile force within the slope:

The slope is divided into three zones of equal height. The total required tension in each zone are found from (Christopher et al. 1990):

$$T_{bottom} = (1/2) T_{max} \quad (8.19)$$

$$T_{middle} = (1/3) T_{max} \quad (8.20)$$

$$T_{top} = (1/6) T_{max} \quad (8.21)$$

d) Determine the allowable design tensile capacity of the tire reinforcement (T_{dc}) for each zone:

The allowable design tensile capacity of the tire reinforcement T_{dc} (considers rupture, pull-out and deformations) is calculated in the middle of each zone by:

$$T_{dc} = \frac{P_d}{FS_p} \leq (T_a)(R_a) \leq T_r \quad (8.22)$$

Where:

T_{dc} = the design tensile capacity of the tire reinforcement which considers rupture and pull-out.

P_d = the design pull-out capacity of the reinforcement per unit width (see below).

FS_p = the factor of safety for pull-out. A value of 2.5 is recommended for the factored pull out capacity of the reinforcement. This factor takes in to account the allowable

deformation of approx. 5% strain in such engineered structures. A higher factor may be used to limit the allowable strains further.

T_a = the allowable tensile strength of the attachment (see step 4).

R_a = attachment coverage factor, relates the number of attachments per unit width of reinforcement.

T_r = the tire tensile strength per unit width of reinforcement. For tire mat reinforcement, a conservative value of 80 kN/m can be assumed (see section 2.14).

The design pull-out capacity (see Chapter 4, sections 4.4.4 and 4.5) per unit width of reinforcement (using passenger size tires, average diameter = 0.6 m) can be determined by

$$P_d = \left(\frac{5}{3}\right)(\alpha_b)(\tan \phi')(\sigma'_v)(L_d) \quad (8.23)$$

Where:

α_b = the bond efficiency coefficient. For tire mat reinforcement, it is very similar to α_{ds} and therefore can be estimated by equation 8.1 (section 8.3.1.2). In coarse grained backfills and long-term conditions in cohesive soils, a reasonable estimate of the bond efficiency coefficient is between 0.8 (full tires) and 0.9 (a cut tire with one sidewall removed).

σ'_v = the effective overburden stress determined at the mid-point of each zone.

L_d = the design embedment length of the tire reinforcement. To calculate the design pull-out capacity, an L_d of two passenger tire widths should be used (approx. 1.2 m). This restriction aims to limit outward deformations. The minimum embedment length is 1 m.

For cohesive soils, the ultimate pull-out capacity under short term (undrained) conditions can be estimated by (section 4.5):

$$P_d = \left(\frac{5}{3}\right)(\alpha_b)(c_u + \sigma_v \tan \phi_u)(L_d) \quad (8.24)$$

Where α_b is between 0.3 and 0.4, a value of 0.3 is recommended. In practice, pull-out tests should be performed to measure the actual bond resistance.

The design tensile capacity of the tire reinforcement (T_{dc}) will be governed by the allowable tensile strength (T_a) in the lower sections of the slope (at greater depths). Therefore, it may be beneficial to increase the attachment strength of these lower reinforcing layers. This would increase the required minimum reinforcement spacing (S_v). The design pull-out capacity (P_d) will control the required reinforcement spacing in the top portions of the slope. As a result, the allowable attachment strength can be reduced in proportion to (P_d/FS_p).

e) Determine the tire reinforcement vertical spacing, S_v , in each of the three zones:

Calculate the minimum number of reinforcing layers N required for each zone based on:

$$N = \frac{T_{\text{zone}}}{T_{dc(\text{zone})}} \quad (8.25)$$

where:

T_{zone} = maximum reinforcement tension required for each zone.

For cohesive soils, the internal stability will be governed by the condition (short- or long-term) which requires the greater number of reinforcing layers.

The vertical spacing of the tire reinforcement (S_v) is determined by:

$$S_v = \frac{H_{\text{zone}}}{N} \quad (8.26)$$

where:

H_{zone} = height of zone.

- The length of the reinforcement behind the critical failure surface must extend to the limits of the critical area to be reinforced, determined in step 6-a. The embedment length L_e must equal or be greater than the design length used in equations 8.23 and 8.24, $L_e \geq L_d$. The reinforcement in the upper levels can be shorted in certain cases, so long as sufficient reinforcement exists in the lower levels to provide the necessary stability requirements (FS_r) for all failure surfaces within the reinforced area and $L_e \geq L_d$ is maintained. However, in most cases, the reinforcement length will be governed by the external stability requirements.
- Simplify the reinforcement layout by providing two to three sections of equal lengths of tire mat reinforcement. Note that the length of the tire mat reinforcement is set by the diameter of the tire. The length of one passenger tire is approximately 0.6 m, so for example, if the minimum required length was 7 m, the number of tire elements needed would be 12 ($7/0.6 = 11.6$, say 12 tires)
- Use short secondary tire reinforcement layers, extending 1.2 to 1.8 m into the slope, to maintain a maximum vertical spacing of 0.6 m for face stability and compaction quality.

Step 7: Determine external stability requirements

The external stability analysis of the reinforced slope assumes that the reinforced soil mass acts as a monolithic block of material which is able to withstand all external loads without failure by one of the following mechanisms: sliding, deep seated instability, compound failure. Depending on the use of the structure, and the type of backfill used (sandy or clayey soil), the external stability analysis may be required for both the short term (undrained) and the long-term (drained) conditions.

a) Sliding Stability

The reinforcing layers must be sufficiently long at any level to resist sliding along the reinforcement. The wedge type failure surface analysis, performed in step 6-a, which just satisfies the stability requirements defines the critical limits of the reinforcement. As a second approach, the stability requirements can be verified by analysing the base sliding resistance of an equivalent rigid structure. The sliding resistance is determined using the same procedure outlined for retaining wall (see section 8.3.1.4).

b) Global Stability

An analysis should be performed to determine the potential for a deep seated failure. This failure surface passes completely behind the reinforced soil, and may extend below the toe. The analysis in step 6-a should provided this information. Improvement of the foundation may be required.

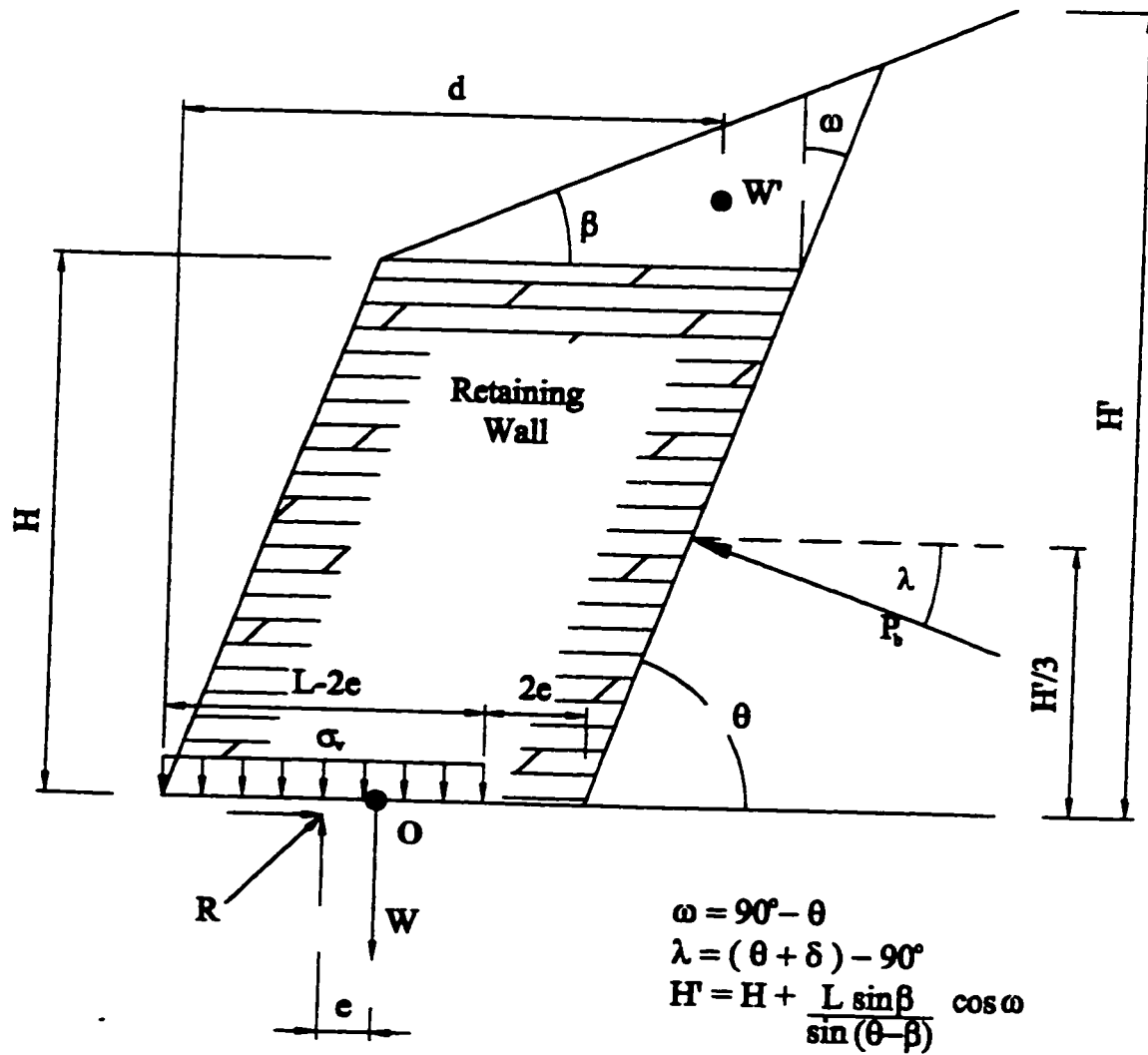
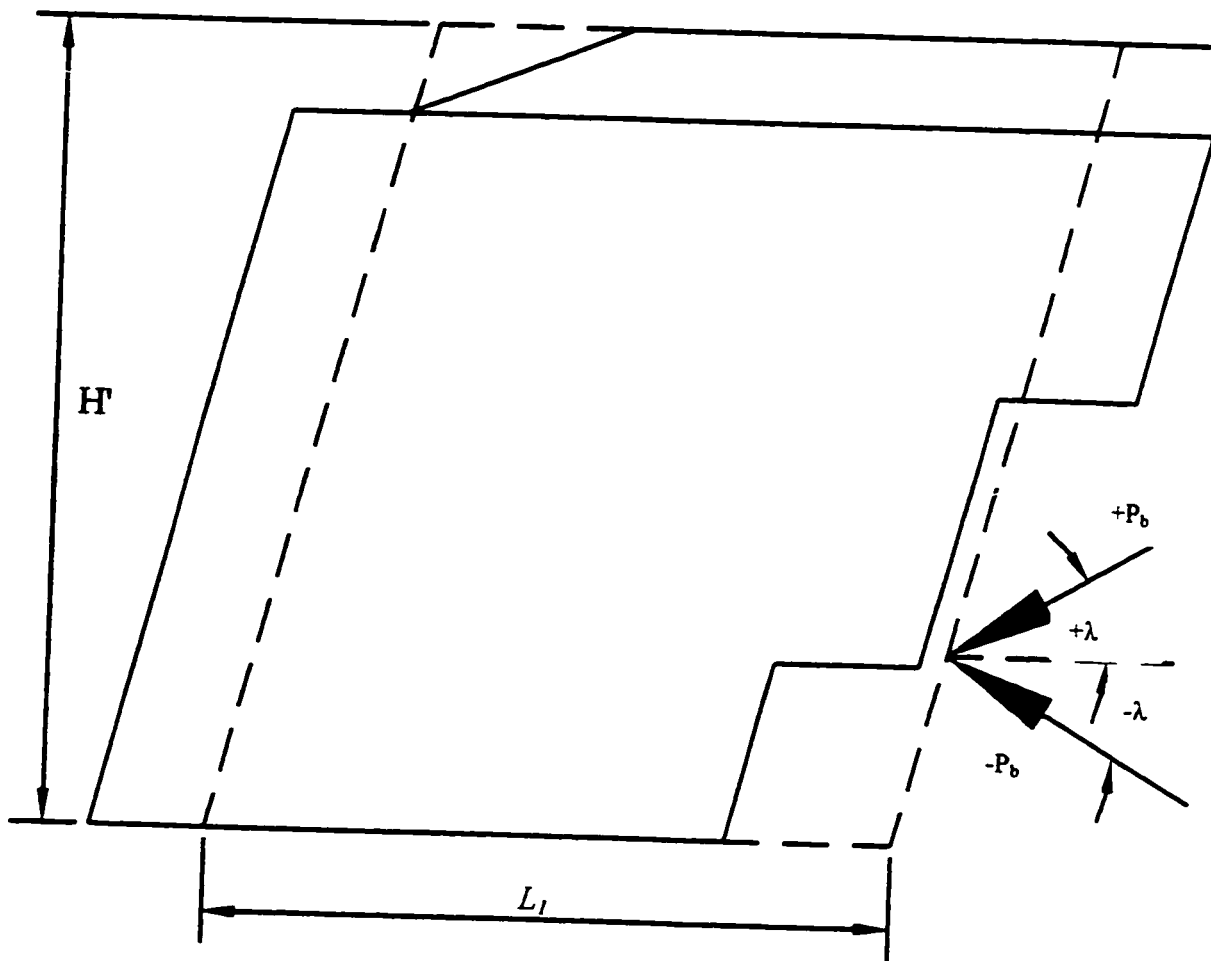
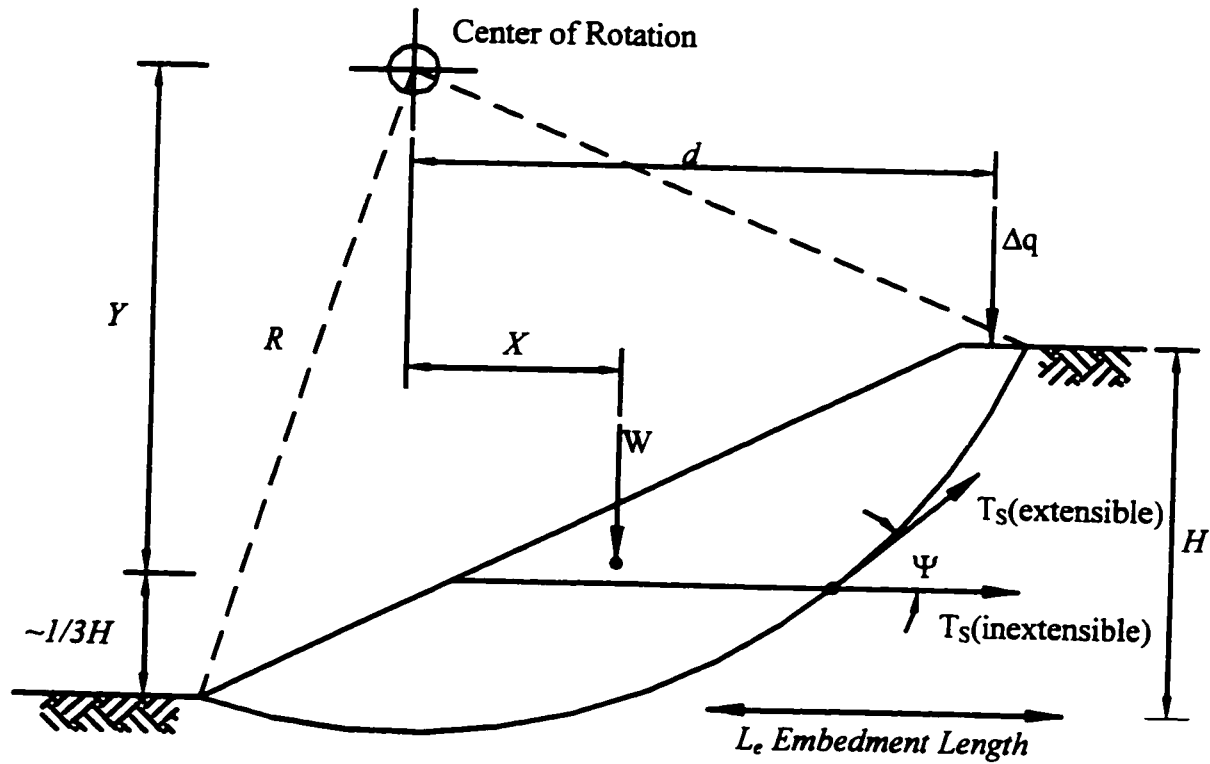


Figure 8.1. The resultant lateral active thrust and vertical stress distribution for a compressible tire retaining wall with a sloping surcharge.



$H'L_1 = \text{AREA OF THE ORIGINAL TIRE WALL}$

Figure 8.2. Thrust at the back of a trapezoidal tire retaining wall.



Factor of Safety of Unreinforced Slope:

$$F.S_u = \frac{\text{Resisting Moments } (M_R)}{\text{Driving Moments } (M_D)} = \frac{\int_0^{L_{sp}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)}$$

where: W = weight of sliding earth mass

L_{sp} = length of slip plane

Δq = surcharge

τ_f = shear strength of soil

Factor of Safety of Reinforced Slope:

$$F.S_r = F.S_u + \frac{T_s \cdot D}{M_D}$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers

D = moment arm of T_s about center of rotation

= R for extensible reinforcement

= Y for inextensible reinforcement

Figure 8.3. The rotational shear approach to determine the required strength of reinforcement (Elias et al. 1996).

CHAPTER 9

CONCLUSIONS AND SUGGESTED FUTURE RESEARCH

9.1 Conclusions

- (1) The ease of construction of the prototype embankment demonstrated the practical feasibility of using scrap tires as a soil reinforcement technique for both tire reinforced fills and tire reinforced gravity retaining walls. These structures can be constructed with conventional fill placement equipment. Virtually no damage was observed as the trucks and the lightweight compactors traversed over the tires.
- (2) Reinforced earth structures using discarded tires can be constructed with both cohesionless as well as cohesive soils. However, it is recommended that only tires with one side wall removed should be used with cohesive backfills. These fills do require careful compaction to insure proper infilling of the tire reinforcement.
- (3) The reported high values of α_{ds} , efficiency coefficient of resistance to direct sliding, indicate that the sliding resistance of a tire mat reinforcement is predominantly governed by the shear strength of the backfill used. Since the tire mat reinforcement geometry is able to fully capitalize on the shear strength provided by the soil, it provides an efficient means of reinforcing the soil.
- (4) The present research shows that the ultimate pull out capacity of tire mats is governed by the effective internal angle of friction of the soil. Pull out tests with different tire configurations show that the ultimate pull out capacity per unit tire width can be estimated by Equation 4.13. The bond efficiency (α_b) is close to the direct sliding resistance efficiency coefficient (α_{ds}), and therefore, can be estimated by Equation 4.3.

- (5) Due to the large displacements required to fully mobilized the ultimate pull-out resistance of tire reinforcement, it is recommended that the required embedment length of tire mat behind the potential failure zone need not be greater than two tire lengths (~ 1.2 m). Pull-out tests indicate that the use of additional tire elements would not provide additional benefits. The estimated ultimate pull-out resistance should be divided by a safety factor of 2.5 to provide the allowable pull-out resistance which would limit acceptable deformations to 5% strain. This level of strain is generally the maximum strain permitted in earth structures.
- (6) The higher compressibility of tire retaining walls with respect to backfills results in the development of negative wall friction. The development of high negative wall friction values significantly increase the active pressure acting on the back of the wall. A reduction in the active thrust can be achieved by providing a batter.
- (7) The present research indicates that maximum batter angle for the retaining walls should not exceed 70 degrees when using a low quality backfill that are highly deformable. Compaction behind the retaining wall should be carefully carried out in order to limit the development of high lateral stresses and also to reduce the outward lateral deformation. Overhang of tires must not be permitted.
- (8) For retaining walls, the different tire layers should be staggered laterally in order to distribute voids uniformly across the structure. A complete infilling of the voids is desirable in order to develop a good interlocking friction between the different tire reinforcing layers and also to limit settlement. Attachment of the frontal tire row to deeper embedded tire elements is required.
- (9) Results from the finite element analyses indicated that providing a wall batter combined with a trapezoidal reinforcement layout (the reinforcement length is increased with the wall height) will minimize lateral displacements in the tire reinforced wall.
- (10) Plate load test indicate that the tire reinforcement reduced the bearing capacity of the soils. These tire reinforced soils did not fail in shear during plate loading tests. As expected, higher

compressibility was observed in the cohesive tire-reinforced fill. The settlements in the two sections constructed with sand were in the conventionally acceptable limit (25 mm) under the maximum imposed stress of 225 kPa.

(11) Field monitoring of the prototype test embankment constructed with tires above the water table indicates that insignificant adverse effects on groundwater quality had occurred over a period of 2 years. Targeted inorganic elements having a MAC designation, which include barium (Ba), cadmium (Cd), chromium (Cr), lead (Pb), selenium (Se) and fluoride (F), were all well below their respective concentration limits. The analysis for elements having a designated Aesthetic Objectives, which include manganese (Mn), copper (Cu), iron (Fe), zinc (Zn), sodium (Na), chloride (Cl), and sulphate (SO_4), were again well below their respective concentration limits. However, some organic compounds can be leached out of the tire reinforced structure.

(12) There is no evidence of an exothermal reaction having taken place in the prototype embankment.

(13) Laboratory batch tests performed on tire chips embedded in sand provided evidence of an increase in aluminum (Al), iron (Fe), zinc (Zn) and manganese (Mn) concentrations which exceed their respective drinking water standards with the exception of zinc. The increase in aluminum, iron, and manganese was attributed to the exposed steel reinforcements in the tire chips. The use of tire chips provides a conservative estimate for fills reinforced with whole tires since the latter case provides limited exposure of steel reinforcement and affords additional protection by the rubberized cover. All inorganic target elements were below detection limits or background levels for tire chips embedded in a kaolin clay.

(14) The amount of organic compounds leached from the tire chips decreased with the number of exposure periods or pore volumes flushed through the soil. It is recommended that tire reinforced earth structures should be built using scrap tires which have been stockpiled or have been used for some time, and should ideally be placed above permanent ground water table.

9.2 Suggested Future Research

A totally functional tire reinforced structure should be built where the following aspects could be investigated:

1. The behaviour of pavements for trafficability.
2. The use of various types of facings to protect the exposed tires from erosion and ultraviolet light. Examples of such facings include shotcrete, concrete blocks etc.
3. Long term monitoring of deformations of the structure with particular reference to creep effects under sustained loading.
4. Consideration of other acceptable alternatives for tire attachments, and their durability including that of polypropylene.
5. Environmental impact of such a functional structure should include long term monitoring of effluent quality. Also, any differences in water quality from using old scrap tires, and the more recent scrap tires should be assessed.
6. Feasibility study should be undertaken to explore construction techniques to minimise cost of construction tire reinforced structures. Cost comparisons should allow for the cost of disposal of tires in engineered landfill sites.
7. Field monitoring of the prototype embankment constructed during this investigation should be continued. The feasibility of deliberately loading this structure to failure should be explored. Such a test will provide invaluable data on the ultimate strength of tire reinforced fills.

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APPENDIX A

WORKED EXAMPLES

A.1 Retaining Wall

Data

An 8 m high wall supporting a surcharge load of 40 kPa is to be constructed at an inclination angle of 80 degrees, using scrap tire reinforcement. The maximum allowable width of the wall is 15 m. A granular fill is available on site. Tests performed on the fill indicated a maximum dry density of 2000 kg/m^3 , strength parameters of $c' = 0$ and $\phi' = 35^\circ$ and an elastic modulus of 60 MPa. A field investigation revealed that the foundation was composed of a thick layer of silty sand, having the following properties: $\gamma_d = 17 \text{ kN/m}^3$, $c' = 0$ and $\phi' = 32^\circ$. The ground water table is located at 3 m below existing ground surface. The maximum permissible lateral movement of the wall is 0.7 m.

Design Steps

Step 1: Establish design limits, scope of the project, and external loads.

- External wall height, $H = 8 \text{ m}$.
- Wall face batter, $\omega = 10^\circ$.
- Maximum width of wall: 15 m.
- External loads and their locations: uniform surcharge 40 kN/m^2 .
- Type of tire reinforcement used: granular backfill, full tire reinforcement will be used.
- Type of attachment: 3 mm dia. polypropylene.

Step 2: Determine the engineering properties of the foundation soil. The foundation soil has the following engineering properties.

$$\gamma_d = 17 \text{ kN/m}^3, c' = 0 \text{ and } \phi' = 32^\circ.$$

The ground water table is located at 3 m below existing ground surface.

Step 3: Determine the backfill properties of both retaining wall section and retained backfill.

The same backfill material will be used for retaining wall section and retained backfill.
They are:

$$\gamma_d = 20 \text{ kN/m}^3, c' = 0, \phi' = 35^\circ$$

elastic modulus of 60 MPa.

Step 4: Establish design factors of safety and performance criteria.

- Sliding: F.S. ≥ 1.5 .
- Overturning: F.S. ≥ 2.0 .
- Bearing Capacity: F.S. ≥ 2.0 .
- Overall Stability: F.S. ≥ 1.5 .
- Settlement: $\leq 50 \text{ mm}$
- Lateral displacement:

the tolerable total movement 0.7 m.

estimated displacements due to construction activities 0.12 m.

permissible post-construction movement $(0.7 - 0.12) = 0.58 \text{ m}$.

Step 5: Establish preliminary wall dimensions.

A good quality granular backfill is to be used, and therefore, a uniform reinforcement layout (a rectangular retaining wall) will be initially tried and with a corresponding preliminary reinforcement length of $0.95H$ taking surcharge into account, $0.95 \times 8 \text{ m} = 7.6 \text{ m}$. Note: the length of the reinforcement is set by the diameter of the tire (avg. 0.6 m for passenger tires):

$$7.6 / 0.6 = 12.6, \text{ use 13 tires: } L = 7.8 \text{ m.}$$

Step 6: Establish interface parameters between soil and tire reinforcement.

The interface friction angle (δ) for both foundation and backfill materials are approximated by $0.7\phi'$. The interface friction angle (δ) for the two soils are:

- 1) the granular backfill, $0.7 \times 35^\circ = 24.5^\circ$, use 24° .
- 2) the foundation soil, $0.7 \times 32^\circ = 22.4^\circ$, use 22° .

The foundation soil offers the least resistance for sliding along the base and therefore the direct sliding efficiency coefficient (α_{ds}) is determine by:

$$\alpha_{ds} = 1 - f \left(1 - \frac{\tan \delta}{\tan \phi} \right)$$
$$\alpha_{ds} = 1 - 0.25 \left(1 - \frac{\tan 22^\circ}{\tan 32^\circ} \right) = 0.91$$

Step 7: Determine the lateral earth pressure coefficient and the vertical stress distribution at the base of the wall. The tire wall is assumed to move downward relative to the backfill and the uniform surcharge load is considered by increasing the design wall height by 2 m.

A. Lateral Earth Pressure Coefficient

Calculate the lateral earth pressure coefficient, K_a , by:

$$K_a = \left[\frac{\csc \theta \sin(\theta - \phi'_b)}{\sqrt{\sin(\theta - \delta)} + \sqrt{\frac{\sin(\phi'_b - \delta) \sin(\phi'_b - \beta)}{\sin(\phi'_b - \beta)}}} \right]^2$$
$$K_a = \left[\frac{\csc(80^\circ) \sin(80^\circ - 35^\circ)}{\sqrt{\sin(80^\circ - 25^\circ)} + \sqrt{\frac{\sin(35^\circ - 25^\circ) \sin(35^\circ)}{\sin(35^\circ)}}} \right]^2 = 0.35$$

B. Vertical Stress Distribution

1. Determine λ , $\lambda = (\theta + \delta) - 90^\circ = (80^\circ + 24^\circ) - 90^\circ = 14^\circ$.
2. Determine wall batter ω , $\omega = 90^\circ - \theta = 90^\circ - 80^\circ = 10^\circ$.
3. Calculate P_b .

$$P_b = \frac{1}{2} K_o \gamma_b H^2 = (0.5)(0.35)(20)(10)^2 = 350 \text{ kN/m}$$

4. Estimate the unit weight of the retaining wall (γ_{re})

Full tire reinforcement in a granular backfill

$$\gamma_{re} \approx 0.85\gamma_b = 0.85 \times 20 = 17 \text{ kN/m}^3$$

5. Calculate the eccentricity of the resultant force on the base as follows:

$$e = \frac{(P_b)(\cos \lambda)\left(\frac{H}{3}\right) + (P_b)(\sin \lambda)\left(\frac{L}{2} + \frac{H}{3 \tan \theta}\right)}{\gamma_{re}HL - P_b \sin \lambda}$$

$$e = \frac{(350)(\cos 14^\circ)\left(\frac{10}{3}\right) + (350)(\sin 14^\circ)\left(3.95 + \frac{10}{3 \tan 80^\circ}\right)}{(17)(10)(7.8) + (350)(\sin 14^\circ)}$$

$$e = 1.2 \text{ m}$$

6. Calculate the equivalent uniform vertical stress on the base (σ_v):

$$\sigma_v = \frac{\gamma_{re}HL + P_b \sin \lambda}{L - 2e}$$

$$\sigma_v = \frac{(17)(10)(7.8) - (350)(\sin 14^\circ)}{7.8 - (2)(1.2)} = 230 \text{ kN/m}^2$$

Step 8: Check wall stability:

A. Sliding resistance.

$$F.S_{\text{sliding}} = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Driving Forces}} \geq 1.5$$

1. Calculate the thrust: $P_b = 350 \text{ kN/m}$

2. Calculate the sum of the driving forces (F_D):

$$(P_b)(\cos \lambda) = (350)(\cos 14^\circ) = 340 \text{ kN/m}$$

3. The foundation soil provides the lowest resistance to sliding, use ϕ'_f .

4. The coefficient of resistance to sliding, $\alpha_{ds} = 0.91$ (see Step 6)

5. Calculate the sum of the resisting forces (R_s):

$$R_s = \{\gamma_{re}HL - (P_b)(\sin \lambda)\}\alpha_{ds} \tan \phi'_f$$

$$R_s = \{(17)(10)(7.8) - (350)(\sin 14^\circ)\} 0.91 \tan 32^\circ = 705 \text{ kN/m}$$

6. Calculate the factor of safety with respect to base sliding.

$$F.S_{\text{sliding}} = \frac{R_s}{F_D} = \frac{705}{340} = 2.07 > 1.5 \text{ OK.}$$

B. Overturning

$$F.S_{\text{overturning}} = \frac{\sum \text{Resisting Moments}}{\sum \text{Driving Moments}} \geq 2.0$$

(Note: Moments about the toe)

1. Calculate the sum of the driving moments, M_d .

$$M_d = (P_b)(\cos \lambda) \left(\frac{H}{3} \right)$$

$$M_d = (350)(\cos 14^\circ) \left(\frac{10}{3} \right) = 1132 \text{ kN} \cdot \text{m}$$

2. Calculate the sum of the resisting moments, M_r .

$$M_r = (\gamma_{re}HL) \left(\frac{L}{2} + \frac{H}{2 \tan \theta} \right) - (P_b \sin \lambda) \left(L + \frac{H}{3 \tan \theta} \right)$$

$$M_r = (17)(10)(7.8)(4.8) - (350)(\sin 14^\circ)(8.4) = 5690 \text{ kN} \cdot \text{m}$$

3. Calculate the factor of safety with respect to overturning:

$$F.O.S_{\text{overturning}} = \frac{M_r}{M_d} = \frac{5690}{1132} = 5.0 > 2.0 \text{ OK.}$$

4. Calculate the eccentricity, e , and verify that the eccentricity does not exceed $L/6$.

$$e = 1.2 \text{ (from Step 7-5)}$$

$$e = 1.2 < L/6 = 1.3 \text{ OK.}$$

C. Bearing capacity.

$$F.S_{\text{Bearing Capacity}} = \frac{\text{Bearing Capacity of Foundation}}{\text{Vertical Stress Distribution at the Base}} = \frac{q_{ult}}{\sigma_v} \geq 2.0$$

1. Calculate the vertical stress distribution, σ_v , assuming a Meyerhof distribution.

$$\sigma_v = 230 \text{ kN/m}^2, \text{ as determine from Step 7-6.}$$

2. Determine the ultimate bearing capacity q_{ult} using methods outlined in the CFEM (2nd Edition).

$$q_{ult} = \frac{1}{2}(L - 2e)\gamma_f N\gamma(\phi)$$

where $N_\gamma = 32$ for $\phi' = 32^\circ$ (CFEM, 2nd Edition)

$$q_{ult} = \frac{1}{2}(7.8 - 2(1.2))(17)(32) = 1470 \text{ kN/m}^2$$

4. Verify that: $\sigma_v \leq \frac{q_{ult}}{2}$, $\sigma_v \leq \frac{1470}{2} = 735 \text{ kN/m}^2$ OK.

D. Overall stability.

The overall stability of the structure was analyzed using the commercially available computer program SLOPE/W developed by GEO-SLOPE International. Since the structure is relatively simple and constructed with a granular backfill, the tire wall was considered in the analysis as a rigid body. The factor of safety for failure passing outside the reinforced zone were greater than 1.5.

Step 9: Estimate the settlement of the tire retaining wall.

The settlement of the tire wall was evaluated to be less than the performance requirements of the project (< 50 mm).

Step 10: Evaluate anticipated lateral displacement.

An estimate of the post construction movements of the tire wall structures was evaluated by the results of the finite analysis presented in Chapter 6. The wall batter was neglected, a conservative approach.

Anticipated post construction movement (Δ):

$$\Delta = 0.007H \times DI(\phi) \times DI(E) \times DI(q)$$

$DI(\phi)$ determined from Figure 6.12 ≈ 1.5 .

$DI(E)$ determined from Figure 6.13 ≈ 1.2 .

$DI(q)$ determined from Figure 6.12 ≈ 2.0 .

$$\Delta = 0.007(8)(1.5)(1.2)(2) \approx 225 \text{ mm}$$

225 mm < permissible post-construction movement (Step 4) = 580 mm. OK.

A.2 Reinforced Slope

Data

An embankment will be constructed using scrap tire reinforcement to elevate an existing roadway. The height of the proposed embankment will be 18 m and a slope angle of 50 degrees. A uniform surcharge of 12 kN/m^2 will represent the traffic loading condition. A field investigation revealed that the natural soil has the following properties: $c' = 15 \text{ kN/m}^2$ and $\phi' = 34^\circ$. The ground water table is located at 2 m below existing ground surface. Tests performed on the fill indicated a maximum dry density of 19 kN/m^3 , and strength parameters of $c' = 0$ and $\phi' = 34^\circ$. The minimum design life of the new embankment is 75 years.

Design Steps

Step 1: Establish the geometric and loading conditions:

- slope height, $H = 18 \text{ m}$.
- slope angle, $\theta = 50 \text{ degrees}$.
- external loading, surcharge load, $q = 12 \text{ kN/m}^2$.

Step 2: Determine the engineering properties of the natural soils in the slope:

For this project, the foundation and the existing embankment have the following strength properties:

$$c' = 15 \text{ kN/m}^2, \phi' = 34^\circ$$

Depth of the water table is 2 m.

Step 3: Determine the backfill properties.

The backfill material has the following properties.

$$\gamma_d = 19 \text{ kN/m}^3, c' = 0, \phi' = 34^\circ$$

Step 4: Establish tire reinforcement requirement:

- Type of tire reinforcement used: cut tires
- Type of attachment: 12.5 mm dia. polypropylene rope

The ultimate tensile strength was evaluated at 17 kN per wrap.

- Design and service life period is 75 years.
- Determine the allowable tensile force per attachment with respect to service life and durability requirements:

$$\text{For polypropylene rope: } T_a = \frac{T_{ult} (CRF)}{(FD)(FC)(FS)}$$

For this project, the following design factors are used:

$$FS = 1.5$$

$$FD = \text{durability factor of safety} = 1.25$$

$$FC = \text{construction damage factor of safety} = 1.1$$

$$CRF = \text{creep reduction factor} = 0.5$$

Therefore:

$$T_a = \frac{T_{ult} (CRF)}{(FD)(FC)(FS)} = \frac{(17)(0.5)}{(1.25)(1.1)(1.5)} = 4.1 \text{ kN per wrap}$$

Step 5: Establish performance requirements.

1. External Stability:

- Sliding: F.S. = 1.5.
- Overall Stability (Deep Seated): F.S. = 1.3.

2. Internal Stability:

- Slope Stability: F.S. = 1.5.
- Pull-out Resistance: FS_p of 2.5.

Step 6: Determine internal stability requirements:

The internal stability is analyzed using rotational slip surface method, as well as the wedge shape failure method to determine the required tire reinforcement tension satisfying the prescribed factor of safety of 1.5. The following design procedures were used:

a) Check unreinforced stability:

The new slope is first analyzed without reinforcement using the computer program such as SLOPE/W developed by GEO-SLOPE International. The factors of safety (FS_u) for circular failure surfaces are analyzed, in this example, by the Modified Bishop Method. Failure is considered passing through the toe of the slope as shown in Figure A.2. Several failure surfaces are evaluated until a safety factor of 1.5 or more is obtained.

Next, wedge shaped failure surface analysis is performed to establish the required area to be reinforced which satisfies the prescribe safety factor of 1.5 against a sliding failure. The result of the wedge type failure analysis (Figure A.2) indicate that the

area to be reinforced (the critical limits) can be reduced to 14 m at the top and 17.5 m at the bottom, for the required factor of safety.

- b) For each of the potential failure surfaces inside the reinforcing limits in step (a), calculate the total required reinforcing tension T_s using a safety factor (FS_r) of 1.5 and the following equation:

$$T_s = (FS_r - FS_u) \frac{M_D}{Y}$$

The maximum reinforced tension requirement is for a failure surface having:

$FS_u = 0.935$, (determined by the computer program)

$M_D = 65.9 \text{ MN} \cdot \text{m}$ per metre width (as determined by the computer program)

$Y = \text{the moment arm} = 32.3 \text{ m}$

$$T_s = T_{\max} = (1.5 - 0.935) \frac{65.9}{32.3} = 1150 \text{ kN/m}$$

- c) Calculate the distributed maximum tensile force within the slope:

Divided the slope into three reinforcement zones of equal height:

$$T_{\text{bottom}} = (1/2) T_{\max} = (1/2)(1150) = 575 \text{ kN/m}$$

$$T_{\text{middle}} = (1/3) T_{\max} = (1/3)(1150) = 383 \text{ kN/m}$$

$$T_{\text{top}} = (1/6) T_{\max} = (1/6)(1150) = 192 \text{ kN/m}$$

Calculate the allowable design tensile capacity of the tire reinforcement (T_{dc}) for each zone:

T_{dc} is calculated in the middle of each zone by:

$$T_{dc} = \frac{P_d}{FS_p} \leq (T_a)(R_a) \leq T_r$$

Where:

T_r = assumed to equal 80 kN/m.

$T_a \times R_a = 4.1 \text{ kN (determined in step 4)} \times 10/\text{m} = 41 \text{ kN/m.}$

$R_a = 2$ attachment points per tire, using 3 wraps of polypropylene rope, and the average diameter of the tires is 0.6 m $= (2)(3)/(0.6) = 10/\text{m}.$

Determine the design pull-out capacity per unit width of reinforcement at the mid-point of each zone by:

$$P_d = \left(\frac{5}{3}\right)(\alpha_b)(\tan \phi')(\sigma'_v)(L_d)$$

Where:

$\alpha_b = 0.9$ (one sidewall removed, a cut tire).

$L_d =$ two passenger tire widths (approx. 1.2 m).

Calculate P_d for the top zone (Note: each zone is equal to $18\text{m}/3 = 6 \text{ m}$ in height)

σ'_v is calculated for a depth of 3 m ($6\text{m}/2 = 3\text{m}$)

$$P_d(\text{Top Zone}) = \left(\frac{5}{3}\right)(0.9)(\tan 34^\circ)(3 \times 19)(1.2) = 68.5 \text{ kN/m}$$

Calculate T_{dc} for the top zone.

$$T_{dc}(\text{Top Zone}) = \frac{P_d}{FS_p} = \frac{68.5}{2.5} = 27.4 \leq 41$$

$$T_{dc}(\text{Top Zone}) = 27.4 \text{ kN/m}$$

For the middle and bottom zones, the T_{dc} is controlled by the strength of the attachment ($T_a \bullet R_a = 41 \text{ kN/m}$), since the factored design pull-out capacity of 55 kN/m at a depth of 6 m is greater than 41 kN/m.

e) Determine the tire reinforcement vertical spacing S_v :

Calculate the minimum number of reinforcing layers N required for each zone.

For the bottom 1/3 of slope:

$$N_b = \frac{T_{\text{zone}}}{T_{\text{dc}(\text{zone})}} = \frac{575}{41} = 14$$

For the middle 1/3 of slope:

$$N_m = \frac{383}{41} = 9.3$$

For the upper 1/3 of slope:

$$N_t = \frac{192}{27} = 7.1$$

Calculate the required vertical spacing of the tire reinforcement (S_v)

For the bottom 1/3 of slope:

$$S_v = \frac{H_{\text{zone}}}{N} = \frac{6}{14} = 0.428 \quad \text{use 15 reinforcing layers at 0.4 m spacing.}$$

For the middle 1/3 of slope:

$$S_v = \frac{6}{9.3} = 0.645 \quad \text{use 10 reinforcing layers at 0.6 m spacing.}$$

For the upper 1/3 of slope:

$$S_v = \frac{6}{7.1} = 0.845 \quad \text{use 7 reinforcing layers at 0.85 m spacing.}$$

- Provide short secondary tire reinforcement layers of 1.8 m in the upper 1/3 of the slope.,
- f) To verify that sufficient reinforcement is provided in the middle and upper third of the slope, the required reinforcement tension within these zones is calculated by repeating Step 6-a and b. The results are shown in Figure A.2.

Top 2/3 of slope, $T_{\text{req}} = 535 \text{ kN/m} < T_{\text{available}} = 600 \text{ kN/m}$ OK.

Top 1/3 of slope, $T_{\text{req}} = 171 \text{ kN/m} < T_{\text{available}} = 190 \text{ kN/m}$ OK.

g) Check that the length of the reinforcement provided behind the T_{max} failure surface (L_e) is greater than the design length (L_d). The extent of the required reinforcement is determined by the sliding wedge analysis. The area to be reinforced is defined by a length of 14 m at the top and a length of 17.5 m at the bottom. Thus:

$Z \geq 0.85 \text{ m}$, available length, $L_e = 4.85 \text{ m} > \text{design length, } L_d = 1.2 \text{ m}$ OK.

h) The final design was checked for pull-out using the slope stability program for surfaces extending behind the T_{max} failure surface. All surfaces reported a factor of safety greater than 1.5.

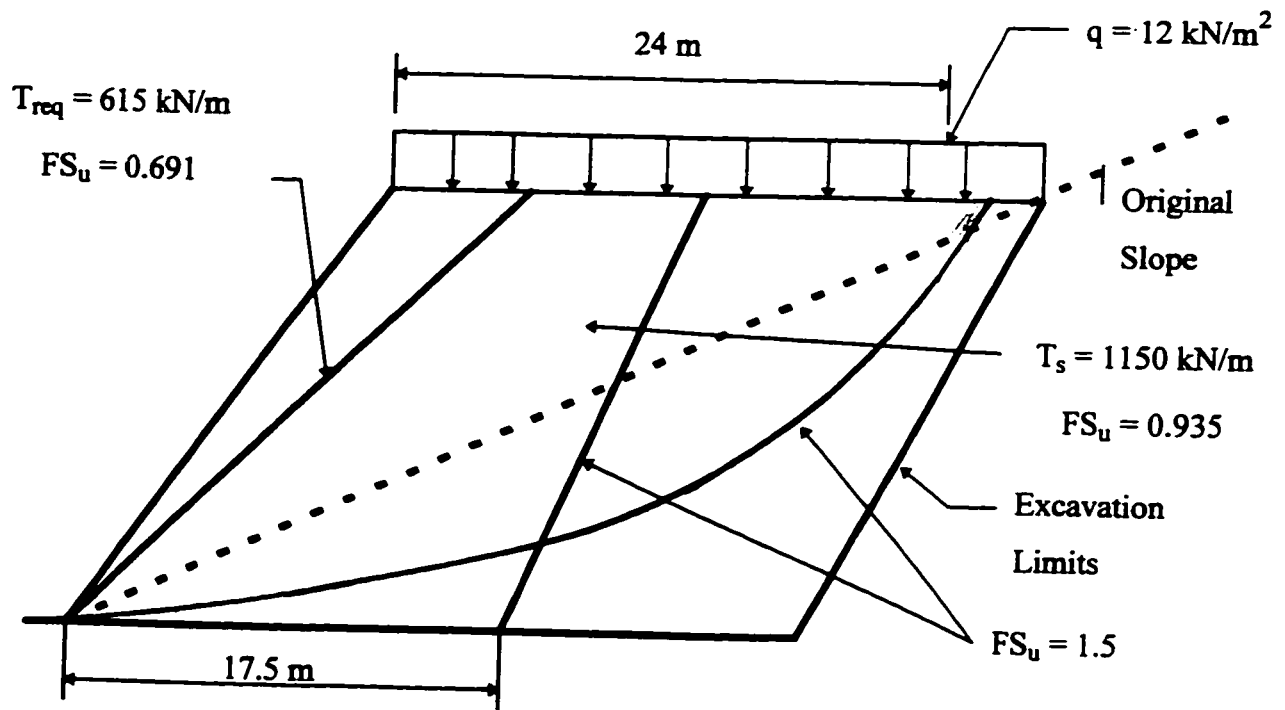
Step 7: Determine external stability requirements

a) Sliding Stability

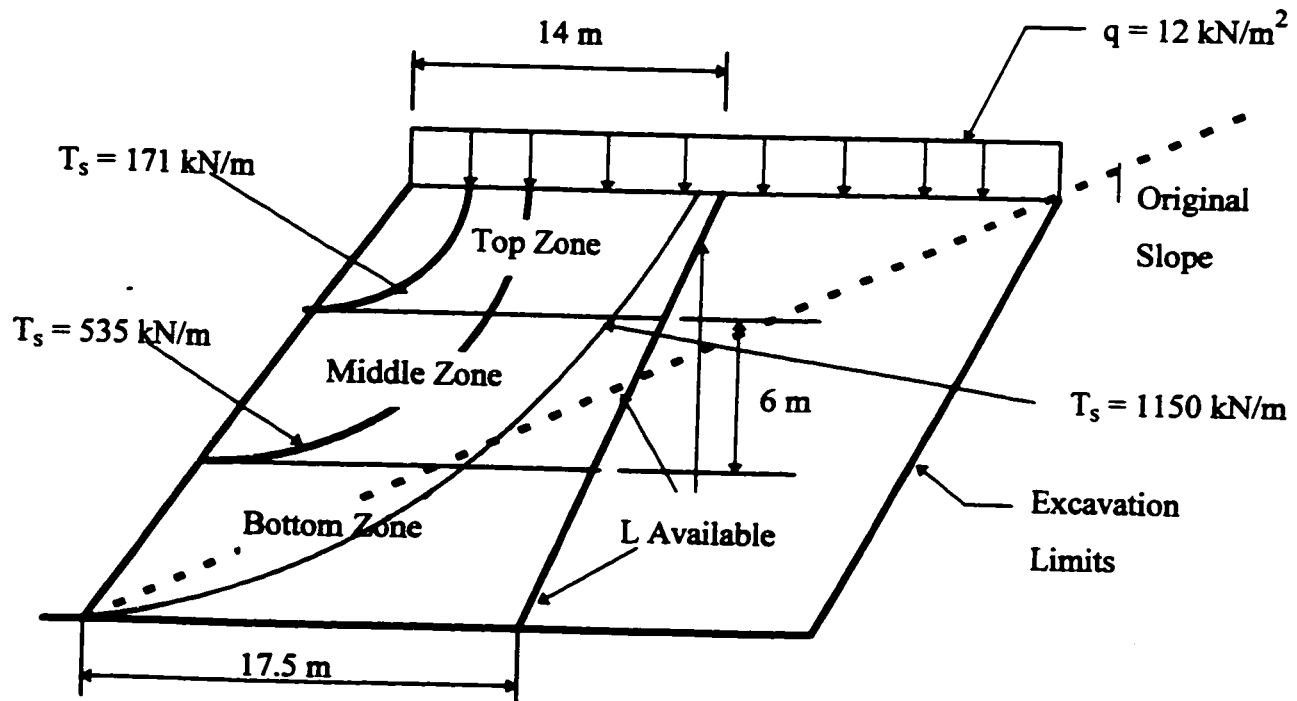
The external stability analysis was verified using SLOPE/W for wedge shape failure surface. The factor of safety of sliding stability was 1.5, as determined in step 6-a, for the failure surface passing outside the reinforced section.

b) Global Stability

A global stability analysis reported a factor of safety of 1.35 for failure surfaces passing completely beyond the reinforced section. Although, the prescribed global stability requirements were met, it may be prudent to improve the foundation or to construct the new slope at a flatter angle.



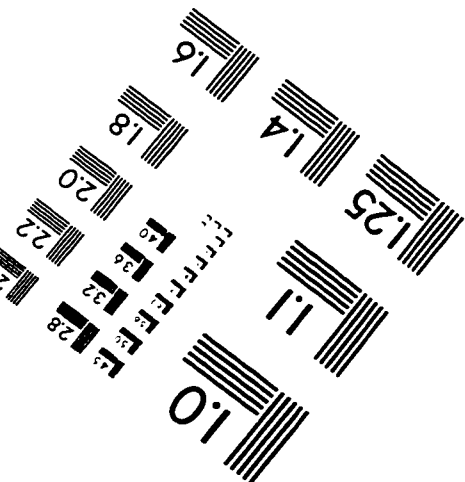
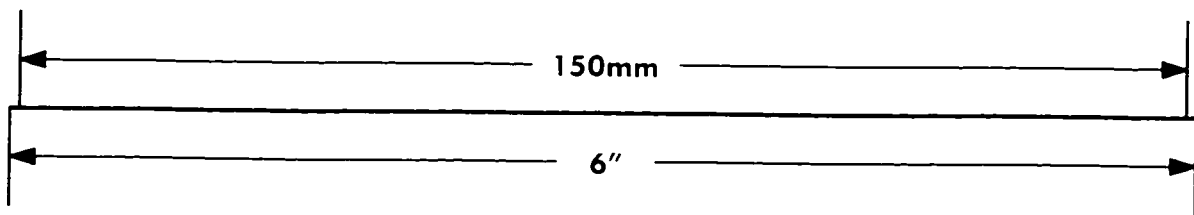
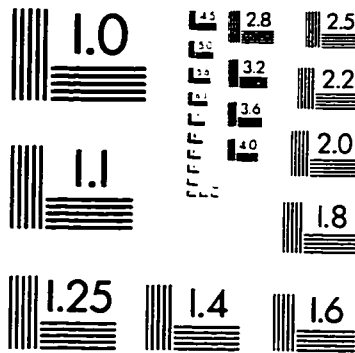
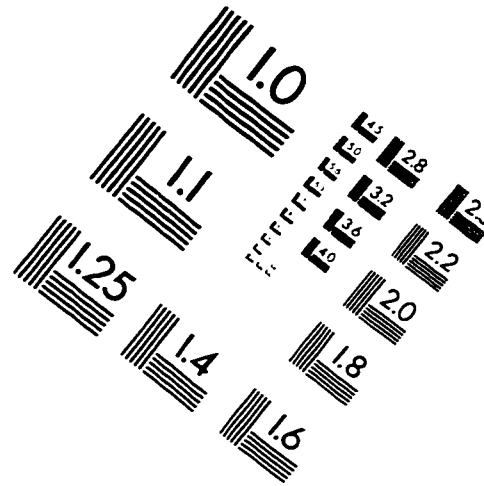
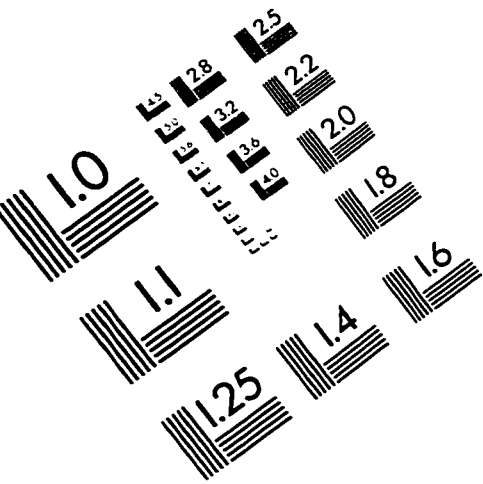
Step 6-a: Preliminary design length. Step 6-b: Determine $T_{\max}$.



Step 6-b: Determine $T_{\max}$. Step 6-f: check reinforcement in upper 2/3 and 1/3 of slope

Figure A.2 The tire reinforced slope design example.

IMAGE EVALUATION TEST TARGET (QA-3)



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