Performance of a 41-Foot High Geotextile Wall

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Final Report
March 1992

Washington State Department of Transportation
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16. ABSTRACT

In order to provide for a preload fill in an area of limited right-of-way, the Washington State Department of Transportation designed and supervised the construction of a geotextile reinforced retaining wall 12.6 m (41.3 ft.) high. Because the wall supported a surcharge fill more than 5 m (16 ft.) in height and was significantly higher than any previously constructed wall of its type, an extensive program of instrumentation of the geotextile reinforcement and measurement of the wall movements was instituted. The paper describes the wall design and construction, purpose and objectives of the instrumentation program, instrumentation selection and installation, and results of the monitoring. The measured deflections and reinforcement strain were low, and overall wall performance was excellent.

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PERFORMANCE OF A 41 FOOT HIGH GEOTEXTILE WALL

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DISCLAIMER

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CHAPTER 1.0

INTRODUCTION

1.1 BACKGROUND

The Washington State Department of Transportation designed and supervised the construction of a series of geotextile walls in Seattle, one of which had a maximum height of 12.6 m (41.3 ft.) and which supported a 5.3 m (17.4 ft.) surcharge of fill above the top of the wall. Since the wall was higher than any geotextile reinforced wall built previously, it was considered to be experimental, and an extensive program of instrumentation and construction monitoring was developed to evaluate the performance of the wall. A summary of the project and some preliminary results have been reported previously by Christopher, et.al., (1990) and Holtz, et.al., (1991). This paper updates those preliminary results and provides additional information not previously published.

1.2 LITERATURE REVIEW

From the FHWA geotextile test walls in Glenwood Canyon, Colorado (1983), we know that present geotextile wall design procedures are very conservative, at least for low walls. However, without performance data on higher walls, it would be imprudent to modify existing design procedures. A review by Yako and Christopher (1987) identified approximately 200 walls and slopes which have been constructed in North America with polymeric reinforcement. The majority of these structures were designed using a conventional tieback wedge technique, similar to that utilized in the design of the wall described in this paper. It has been suggested that this method is quite conservative, as the actual strain and corresponding stress levels typically measured in the field are much less than those predicted by the design method.

Another question concerns the design strength of the geosynthetic. Conventionally, it is empirically selected as a percentage of its ultimate unconfined strength which is usually below the creep limit of the polymer. The relatively large strains, especially in geotextiles, at which the design strength would typically be developed has often been ignored. More recent design methods in use, such as recommended by Task Force No. 27 (1989), include a strain limit criterion which usually results in even lower geotextile design strengths. In practice, this strain limit is determined based on unconfined geotextile strength test results. Yet, it is well known (McGown, et.al., 1982) that the apparent stress-strain characteristics of a geotextile are improved when confined in soil. Thus, on the one hand, if strains are not considered, the design procedures do not predict performance; on the other hand, if low design strength levels to accommodate strains are chosen, the design procedures appear to be very conservative and uneconomical.

Unfortunately, much of the field experience to date has provided only a qualitative assessment of the design variables, and quantitative information is needed to substantiate design modifications. Of the projects reviewed by Yako and Christopher (1987), only 13 were found to have well-documented instrumentation. Of those, only five provided significant stress-strain information, and they were all geogrid reinforced structures. Horizontal movements reported to date have been very small, but most of these measurements have been made after construction of the wall was completed.

1.3 OBJECTIVES

The goal of the present project was to define the actual stress and strain distribution within the geotextile wall, both during and after construction, through an instrumentation and measurement program, in order to monitor and evaluate the performance of the wall. The results obtained should make it possible to determine if modifications to present design procedures are appropriate, especially for very high walls.

CHAPTER 2.0

PROJECT AND SITE DESCRIPTION

2.1 PROJECT LOCATION

The project was located at a major motorway interchange in Seattle, Washington. Six geotextile walls (Figure 1) were used to temporarily retain preload fills for the bridge abutments. The wall described in this paper, the SE wall, was the largest of the six walls. The walls were partially demolished and buried after about one year of service, at which point bridge construction began.

2.2 SITE DESCRIPTION

Geotechnical investigations indicated an upper layer about 6 m (20 ft.) thick of fairly dense granular materials (deltaic deposits and recent fills) overlying soft lacustrine silty clays and clayey silts which vary from 0 to 15 m(50 ft.) thick at the project site. Very dense glacially consolidated soils underlie the lacustrine deposits. Although these soft deposits are slightly overconsolidated, they are sufficiently compressible to yield settlements of up to 400 mm (16 in.) in the abutment; thus, a temporary surcharge fill was required.

Traffic and site geometric constraints dictated that some type of retaining structure would be required to support the surcharge fill. A geotextile reinforced retaining wall was chosen for this purpose because of its economy and ease of construction. The overall wall site plan is presented in Figure 1. A profile of the SE wall and surcharge is shown in Figure 2.

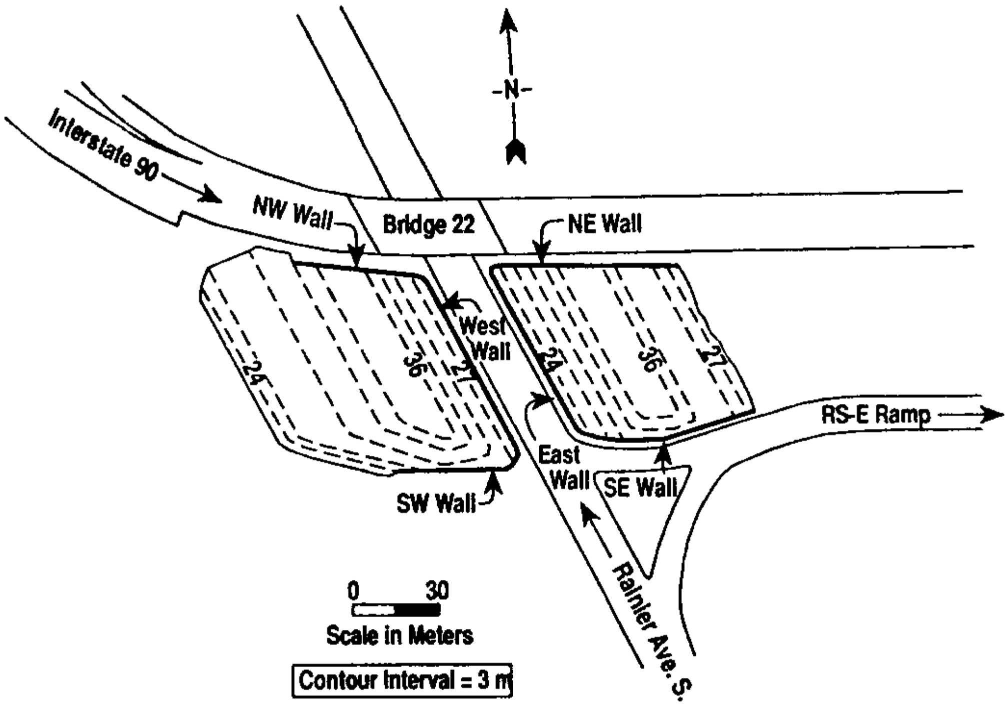

Figure 1. Plan view of geotextile wall and preload site.

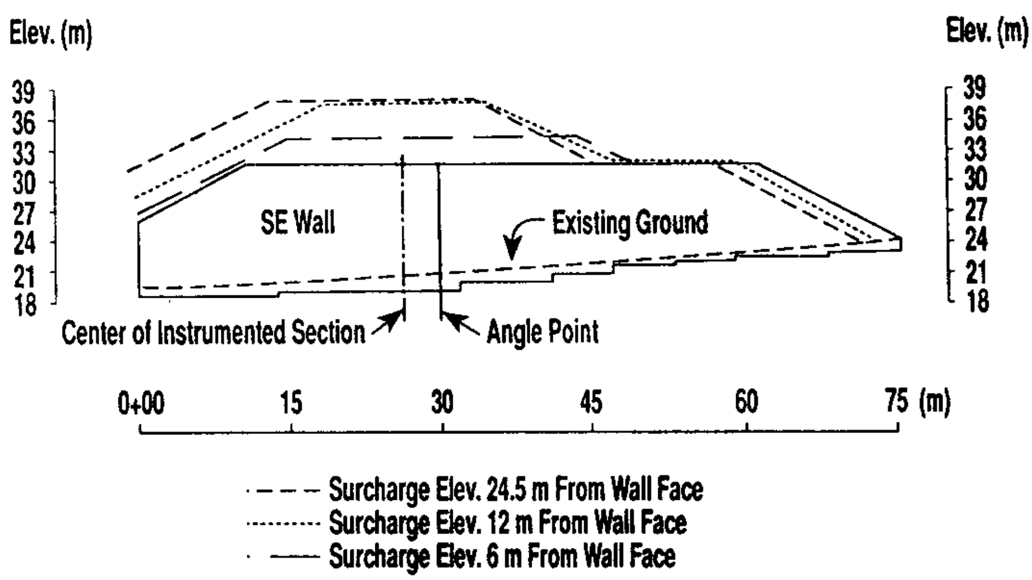

Figure 2. Profile of the SE wall and soil surcharge.

CHAPTER 3.0

WALL DESIGN

3.1 DESIGN PARAMETERS AND METHODOLOGY

A conventional tieback wedge analysis was performed by WSDOT engineers to establish the reinforcing requirements for the wall. Because the specific backfill to be utilized was not known during design, the analysis assumed a soil friction angle of 36° and a unit weight of 20.4 kn./m^3 (130 pcf). The active earth pressure coefficient was calculated assuming a 2:1 sloped surcharge and a wall face batter of 1:20.

3.2 DESIGN TENSILE STRENGTH

The design tensile strength of the geotextile reinforcement was determined using a partial factor of safety approach to account for the various geosynthetic strength loss mechanisms (Allen, 1991). The degradation mechanisms considered include installation damage, creep, and chemical aging. Due to the temporary nature of the wall, chemical aging was expected to be negligible. Furthermore, it was expected that relatively high strength woven geotextiles which were expected to be highly resistant to installation damage would be selected. Therefore, only creep would require a partial factor of safety greater than 1.0. The partial factor of safety for creep was set at 2.5 for polypropylene (PP) and 1.7 for polyester (PET) geotextiles for the wall design. The creep limits found in the literature for these two types of geotextiles would result in partial safety factors of 3.3 to 5.0 for PP and 2.0 to 2.5 for PET geotextiles for long-term applications (Allen, 1991). It was felt that lower partial factors of safety were acceptable in this case due to the short wall design life and the creep reducing effect of soil confinement observed by others (Bell, et.al., 1983, and McGown et.al., 1982).

The partial factor of safety for each polymer type was applied to the ultimate wide width tensile strength of the geotextile (ASTM D-4595) to obtain the design tensile strength. The design tensile strength was then equated to the calculated force per reinforcement layer based on the tieback wedge method utilizing a factor of safety for internal stability of 1.2 (Allen and Holtz, 1991).

3.3 REINFORCEMENT SPACING

The wall was designed for a constant reinforcement vertical spacing of 0.38 m (1.25 ft.). The specified geotextile strength was varied with the height of the wall to more closely match the theoretical design strength requirements. Four 3 m (10 ft.) vertical zones of constant required strength were utilized with one geotextile type for each zone. The minimum ultimate wide width tensile strength requirements for the wall are included in Table 1.

Table 1. Geotextile Characteristics.

Distance From Top of Wall (m)	Min. Wide Width Tensile Strength Required (kN/m) PP	PET	Geotextile Type Selected by Contractor	Wide Width Tensile Strength (kN/m)	Elongation at Peak Tension %	Modulus at 5% Strain (kN/m)
0 - 3	26	18	PP slit film woven	31	21	198
3 - 6	53	35	PP stitch-bonded (2 layers) slit film woven	62	16	453
6 - 9	79	53	PP stitch-bonded (3 layers) slit film woven	92	17	662
9 - 12	105	70	PET multi-filament woven	186	18	1068

Note: 1 kN/m = 5.71 lbs/in.

3.4 EXTERNAL STABILITY REQUIREMENTS

External stability evaluation included bearing capacity, sliding, and overturning of the reinforced wall section. Factors of safety of 2.0, 1.5, and 2.0, respectively, were used for the analysis. The sliding stability was found to control the wall width rather than reinforcement pullout based on a pullout safety factor of 2.0. This resulted in a minimum wall width requirement of 80 percent of the wall height.

CHAPTER 4.0

INSTRUMENTATION PROGRAM

4.1 INSTRUMENTATION OBJECTIVES

The instrumentation and monitoring program had the following objectives:

1. Observe the stress and strain distribution within the reinforced soil wall;
2. Evaluate the change in stress distribution due to an inclined surcharge;
3. Confirm internal and external design stress levels;
4. Observe the deformation response of the structure;
5. Provide a reference base for future designs with the possibility of improving design procedures and/or reducing costs.

4.2 PARAMETER SELECTION

Parameters selected for monitoring were as follows:

1. Local and overall, or global, stress and strain states in the geotextile, with special attention to the magnitude and location of maximum stress;
2. Vertical and horizontal movement of the instrumented section;
3. Lateral earth pressure at the back of the reinforced section;
4. Stress distribution at the base of the reinforced section;

5. Stress distribution in the reinforcement due to the surcharge load;
6. Stress relaxation or creep in the reinforcement with time;
7. Changes in temperature, rainfall, humidity, snowfall, barometric pressure, and wind which may affect instrument readings.

4.3 SITE SELECTION

One section of the SE wall was selected to be instrumented and monitored. The location of the center of the instrumented section is shown in Figure 2.

The instrumented section was considered to be representative of the behavior of the entire wall. Since most instruments measure conditions at only one point, a large number of measurement points was required to evaluate parameters of interest over an entire section of the structure. Instrumented geotextile layers were installed at 1.1, 3.1, 6.1, and 9.5 m (3.8, 10.0, 20.0, and 31.3 ft.) from the bottom of the wall; these instrumented layers were near the bottom of each of the four geotextile strength zones. Figure 3 shows the type and location of instruments installed.

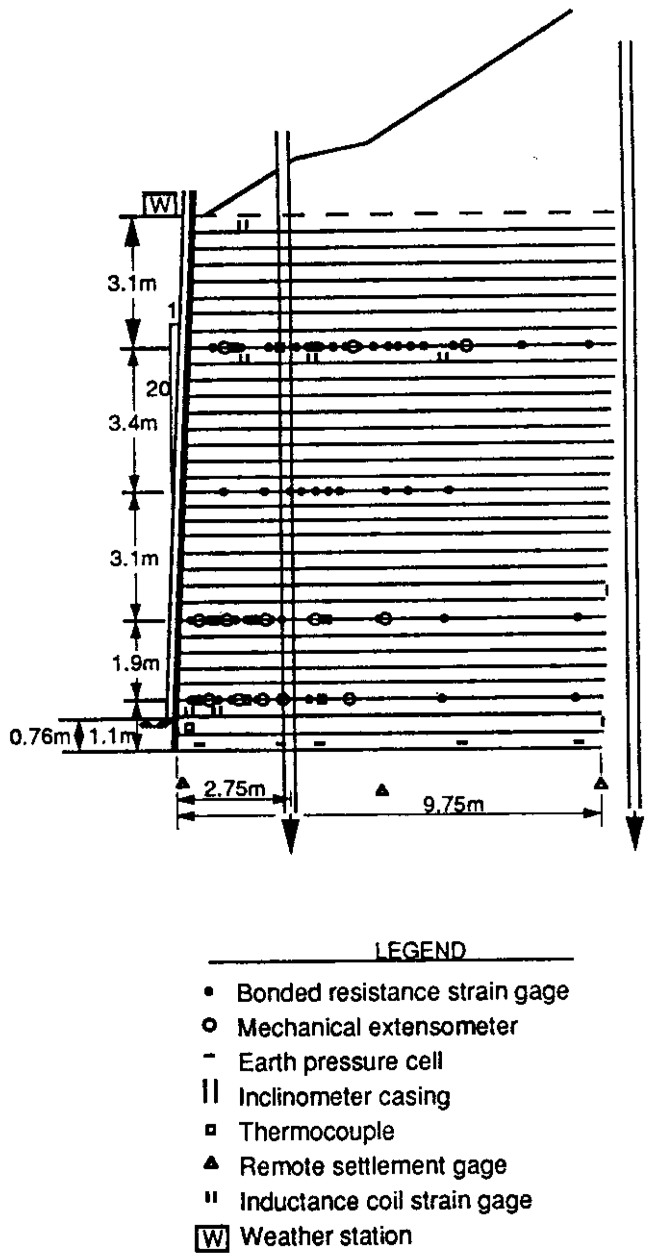

Figure 3. Instrumented wall section.

4.4 STRAIN GAGE INSTALLATION

Bonded resistance strain gage sets were mounted on four layers of the geotextile reinforcement as indicated in Figure

3 to measure local reinforcement strain. The gages were concentrated in the area of the theoretical Rankine and the coherent gravity failure surfaces of the reinforced structure with spacings along the reinforcement layer as close as 0.3 m (1.0 ft.) between gage sets. Each set consisted of a strain gage on both the upper and lower surfaces of the geotextile to eliminate any effects of local bending and to provide redundancy. Each gage could also be read independently to help explain any anomalous data obtained.

Based on an extensive calibration program using both unconfined wide width and in-soil pullout tests, high elongation, post yield gages capable of measuring in excess of 10% strain were selected and attached to the geotextile with high elongation, low creep adhesives. BLH Inc. SR-4 type PA3 gages, with dimensions of 50 x 14 mm (2.0 x 0.55 in.), were bonded with SR-4 adhesive to the PET geotextile. Tokyo Lokki Kenkyojo Ltd. YL-20 gages, with dimensions of 31 x 7 mm (1.2 x 0.3 in.), were attached with CN adhesive to the PP geotextile layers. The area affected by the adhesive was slightly larger than the area of the gage backing. The gages were connected in a three wire quarter bridge at the monitoring point and hooked in series to the readout unit to electronically eliminate bending strains. All gages were insulated with Micromeasurements Inc. GA2 coating and waterproofed with a T.B. Moore Inc. polysulfite sealant on the geotextile. All gages were installed in the laboratory and then shipped to the project site. A total of 45 bonded resistance strain gage points were utilized. The gages as installed on the second from the bottom geotextile layer are shown in Figure 4.

Figure 4. Installation of strain gages and extensometers.

4.5 STRAIN GAGE PERFORMANCE

Only three gages were lost during construction. Throughout the life of the wall, 69% of the gages were apparently still working, although a few gave questionable results.

4.6 EXTENSOMETER INSTALLATION AND PERFORMANCE

To evaluate the global strain and stress state in the geotextile and provide additional redundancy for the bonded strain gage data, 14 mechanical extensometers were mounted on reinforcement layers 1, 2, and 4 (Figure 3). These gages were specially made by the Glötzl Instrument Co., and they consisted of a flexible fiberglass rod with an anchor for attaching it to the geotextile. A hardened PVC monitoring head and micrometer permitted measurements to 0.1 mm (0.004 in). Gage lengths varied from 0.5 m to 1.5 m (1.5 to 5.0 ft.). The extensometers are being installed as shown in Figure 4. The extensometers on the lowest two instrumented reinforcement layers appeared to be working adequately throughout the life of the wall. One of the extensometers on the highest instrumented layer was broken during wall construction, and the remaining two extensometers on that layer did not appear to be working properly.

4.7 BISON COIL GAGE INSTALLATION

Five pairs of Bison inductance coil soil strain gages were placed in a vertical arrangement in the backfill soil between reinforcing layers (Figure 3) to evaluate lateral strains within the reinforced soil mass. The global state of stress behind and below the reinforced soil mass was evaluated by seven Glötzl earth pressure cells placed as shown in Figure 3. Calibration of the earth pressure cells was based on laboratory in-soil compression tests and field loading during the initial stages of construction.

4.8 INCLINOMETER INSTALLATION AND OPTICAL SURVEYING

To evaluate the horizontal movement of the wall, inclinometer tubes (Figure 3) were installed at the face of the wall, within the reinforced soil section 2.75 m (9.0 ft.) behind the wall face as measured at the toe, and behind the reinforced section (as a reference point). The two inclinometers in the backfill were embedded 17 m (55 ft.) below the base of the wall into dense soils. The inclinometer at the face of the wall was not embedded, but its base was surveyed at each reading so that any lateral movement occurring between readings could be determined. To provide additional redundancy, 18 survey points located in three vertical rows in the vicinity of the instruments were established on the face of the wall during construction. Each survey point consisted of a short piece of No. 8 rebar embedded between two reinforcing layers.

4.9 PHOTOGRAMMETRIC MONITORING

Periodic photogrammetric evaluation of the wall face was also undertaken to provide redundancy for the optical surveys and inclinometer measurements. Eleven photo targets were affixed to the geotextile face in the vicinity of the optical survey targets and instrumented geotextile section for the photogrammetric monitoring. Three permanent camera stations were established directly across an on-ramp from the wall. The photographs were taken with a Hasselblad MK70 metric camera, and image coordinates of each target on film negatives were observed using a Kern Monocomparator. One set of photographs taken during the early stages of construction was considered to be the undeformed reference position. Subsequent deformations were computed by comparing the coordinates of the photo targets on the subsequent photographs to the reference spatial positions of the same targets. Average computed error in the photogrammetric measurements was about 15 mm (0.59 in.) with a maximum error of 26 mm (1.0 in.) for one set of observations. Details of the photography, corrections, coordinate transformation, and error analysis are given by Kim (1990).

4.10 OTHER DATA COLLECTION MODES

Vertical movements of the instrumented all section were monitored by conventional optical surveys and three liquid settlement sensing devices installed at the base of the wall as shown on Figure 3.

A weather station at the site monitored ambient temperature, humidity, rain and snowfall, barometric pressure and wind. Twelve thermistors were attached to the reinforcement (Figure 3) for measuring internal temperatures of the soil and the reinforcement layers.

4.11 DATA COLLECTION SYSTEM

A field compatible fast data acquisition system, developed by Brewer-Teledyne, Inc., was installed at the site for recording the bonded resistance strain gage data, as this data was considered to be of primary importance. Readings of the strain gages were taken three times per day by the acquisition system during wall construction and afterwards by telephone access. Monitoring continued until the wall was demolished in April, 1990 after the surcharge loading of the foundation was complete.

CHAPTER 5.0

MATERIAL PROPERTIES AND TESTING DURING WALL CONSTRUCTION

5.1 INITIAL MATERIAL PROPERTIES

The contractor selected a relatively clean subrounded to subangular gravelly sand for the retaining wall backfill and preload fill. Typical soil gradation curves are shown in Figure 5. The soil had an average moist unit weight of 21.1 kn./m^3 (134 pcf), as determined by field nuclear density testing, and a moisture content of 7.5%, which is only 0.5% over optimum. Two consolidated drained triaxial test series were performed on backfill obtained from the wall. The triaxial samples were compacted as closely as practical to field density and moisture content. The triaxial sample size was 200 mm (8.0 in.) long by 100 mm (4.0 in.) in diameter. The test method used was in accordance with AASHTO T234, using a testing rate of 0.08%/minute. Samples were tested at four confining pressures for each test series which simulated the pressures anticipated in the wall. The triaxial testing indicated that the actual soil friction angle in the wall backfill varied from 43° to 47°, which is considerably higher than the 36° friction angle assumed for design.

The characteristics of the geotextiles selected by the contractor are summarized in Table 1. Note that in some cases the geotextiles selected were considerably stronger than required by the design, apparently due to material availability and possibly other considerations. This is not a unique situation based on the authors' experience and certainly doesn't hurt the wall from the engineer's standpoint.

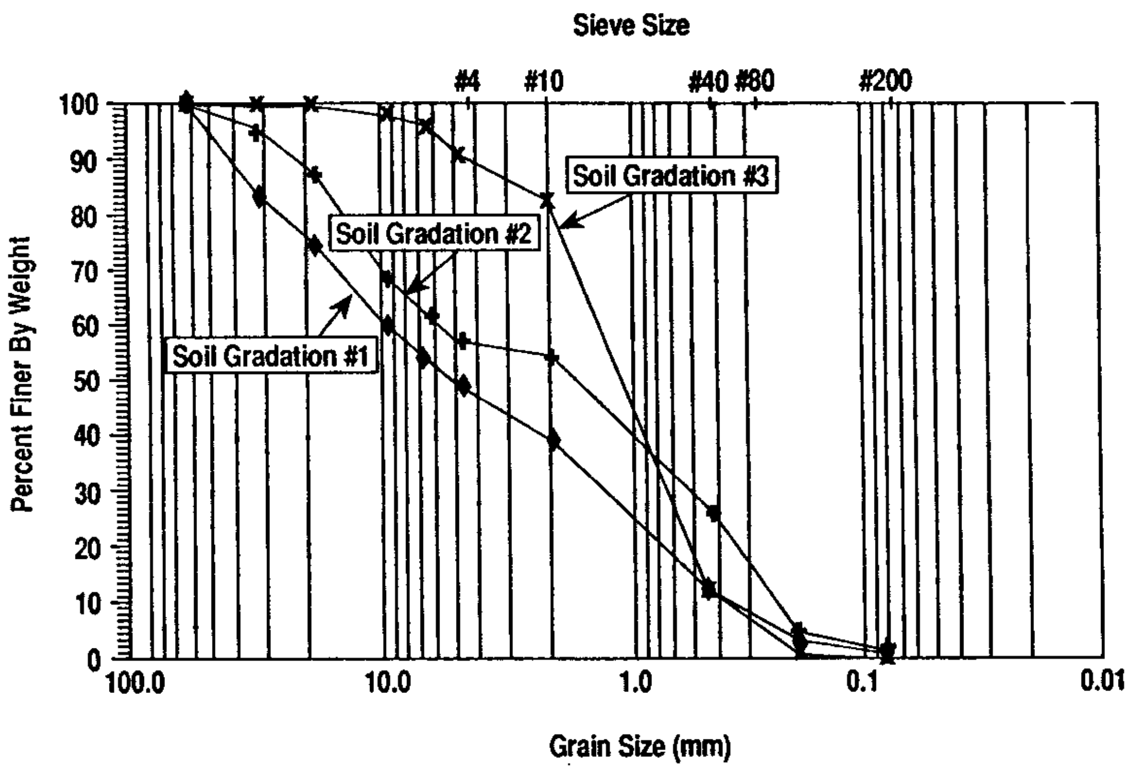

Figure 5. Typical soil gradational characteristics for wall backfill and preload material.

5.2 PROPERTIES AFTER INSTALLATION

Tests to determine the effect of installation damage were performed on the selected geotextiles both before and during retaining wall construction. Samples obtained before wall construction were subjected to the worst case installation conditions allowed by the construction specifications, i.e., a 150 mm (6.0 in.) initial lift over the geotextile and vibratory compaction. Samples obtained within the wall during construction were subjected to a 200 mm (8.0 in.) initial lift over the geotextile and static compaction.

Both sets of samples were tested for wide width tensile strength (ASTM D-4595). The percent strength, strain, and modulus retained after installation for each of the geotextiles are summarized in Table 2. The test results showed that strength loss was significant, especially for the PP slit film woven and the PET multifilament woven geotextiles, where the strength retained was as little as 60 percent of the undamaged strength. The strength loss appears to be due to loss of strain at failure rather than loss of modulus. The results also show that the PET geotextile was more susceptible to strength loss due to installation damage than the PP geotextiles, when comparing geotextiles with similar weights and strengths.

Table 2. Geotextile Strength, Strain, and Modulus Retained After Installation.

Geotextile Type	Original Wide Width Tensile Width Strength (kN/m)	Case 1			Case 2		
		Percent Strength Retained	Percent Failure Strain Retained	Percent 5% Secant Modulus Retained	Percent Strength Retained	Percent Failure Strain Retained	Percent 5% Secant Modulus Retained
PP slit film woven	31	77%	83%	86%	60%	64%	97%
PP stitch-bonded (2 layers) slit film woven	52	92%	89%	98%	77%	83%	101%
PP stitch-bonded (3 layers) slit film woven	92	--	--	--	88%	75%	122%
PET multi-filament woven	186	71%	63%	91%	60%	61%	115%

Case 1: 200 mm (8.0 in) initial lift with static compaction.
Case 2: 150 mm (6.0 in) initial lift with vibratory compaction.
Note: 1 kN/m = 5.71 lbs/in.

CHAPTER 6.0

WALL CONSTRUCTION AND OVERALL PERFORMANCE

6.1 WALL CONSTRUCTION

Construction of the SE wall began in March, 1989 and took approximately two months to complete. The wall construction rate was relatively slow during construction of the first few lifts but gradually stabilized at a rate of approximately 30 m^2 (320 ft^2) of wall face per day. Due to the inexperience of most contractors in constructing these relatively new systems, a learning period is generally required to achieve construction efficiency and meet face batter tolerances. A single layer forming system and construction method similar to that reported by Bell, et. al., (1983) was used. The forming system for the previous layer installed was left in place while the next geotextile layer was placed as an aid to maintaining wall face alignment. The contractor was, for the most part, able to construct the wall within the $\pm$75 mm ($\pm$ 3.0 in.) tolerance required by the specifications.

6.2 CONSTRUCTION PROBLEMS

Wall backfill compaction was the only real difficulty experienced by the contractor during wall construction. The compaction difficulties appeared to be related to the soil gradation characteristics, which caused the contractor to expend considerable effort to obtain the desired level of compaction.

Both vibratory and static compaction methods were used. A small vibratory plate compactor was used within 0.9 m (3.0 ft.) of the wall face. It was found, however, that the use of a large vibratory roller created difficulties in maintaining wall face tolerances, even when the roller was kept at least 1.8 m (6.0 ft.) away from the wall face. The heavy vibration caused the sand backfill to shift, causing the geotextile face in the upper three layers to shift outward somewhat and the forming system for the current lift to move. Therefore, the use of the vibratory compaction method with the large roller was curtailed within the reinforced wall section, severely reducing the rate of compaction and reducing overall wall construction rates by 25 to 40 percent at times. The construction specifications actually did address limiting the use of large vibratory rollers within the wall, as the wall face alignment problem due to heavy vibration had occurred occasionally on previous projects where sandy backfills were used.

Other than the compaction difficulties, wall construction and instrumentation installation went smoothly. Figure 6 shows the SE wall after construction was complete. Figure 7 shows the face of the wall at the instrumented section.

6.3 WALL DEMOLITION

The wall was torn down after approximately one year of service, during which time the wall performed very well. The high strength geotextile was easily ripped and removed during wall demolition by a backhoe and scrapers. Some of the actual instrumented geotextile layers were obtained intact during wall demolition for future testing. Figure 8 shows the second from the bottom instrumented layer after it was uncovered.

Figure 6. The SE wall after completion.

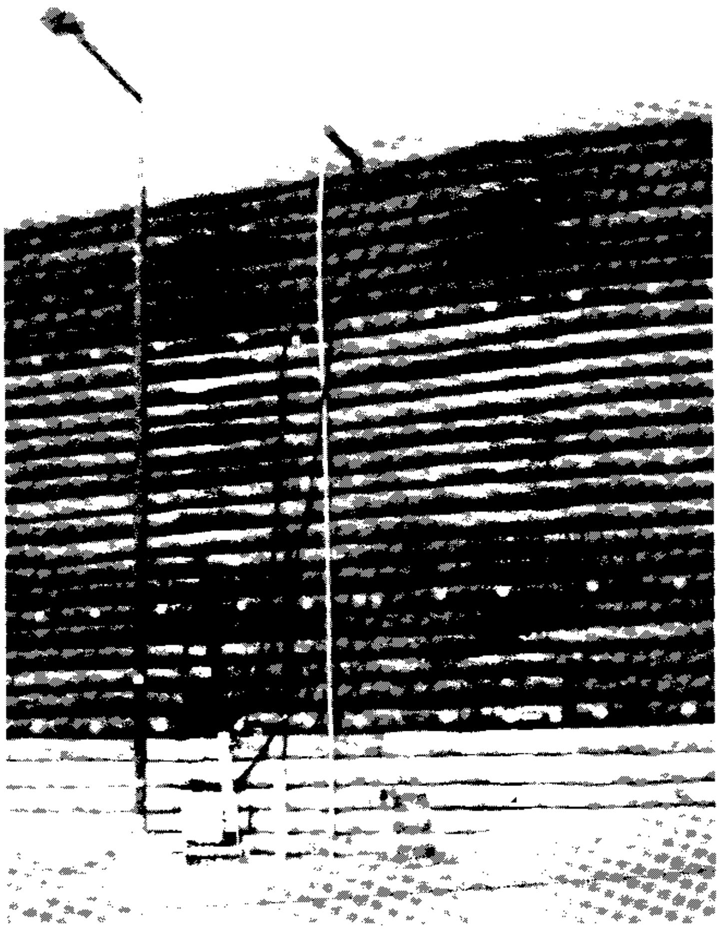

Figure 7. Front view of the SE wall and the instrumented section.

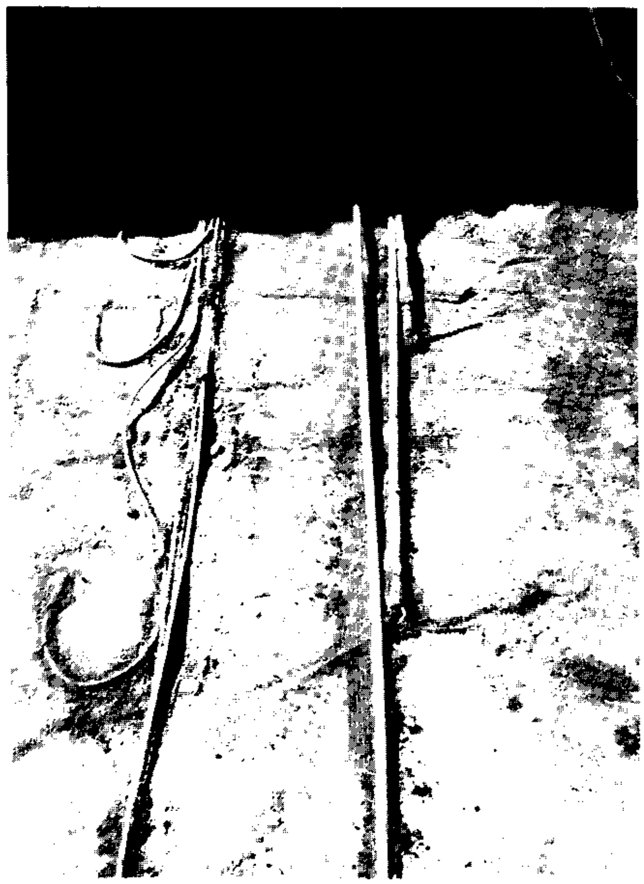

Figure 8. The second from the bottom instrumented layer after unearthing during wall demolition.

CHAPTER 7.0
MONITORING RESULTS

7.1 INTRODUCTION

Redundancy in the instrumentation program was used to verify the correctness of an individual measurement as well as to account for the possibility that some instruments would be damaged during construction. In this section, the data obtained from the instrumentation program is presented and its accuracy assessed in terms of whether a particular measurement is believable, or is the result of a malfunction or some inherent inaccuracy of the particular instrument type.

7.2 LOCAL STRAIN

Figure 9 shows a typical development of strains in the reinforcement during construction of the wall. The gage designations shown in the figure are explained as follows: the first number is the geotextile layer number relative to the bottom of the wall, and the second number is the distance in feet from the wall face (1 ft. = 0.305 m). Strains developed more gradually in the lower elevation geotextile layers than in the higher layers. The rate of increase in strain decreased as the fill elevation increased above the instrumented layer. This characteristic strain gage behavior was used to evaluate the reliability of each gage and to make adjustments in the strain readings where necessary. The measurements of strain from the extensometer data also exhibited this characteristic behavior.

A few gages indicated strains which for one reason or another were suspect. In such cases, the characteristic gage behavior mentioned above was used to extrapolate the strains as shown in Figure 9. One type of problem is illustrated by Gage 26-11 in Figure 9 in which the gage appeared to behave normally until a certain strain was reached; then the strain remained constant as additional fill was placed. Possibly the gage had become unbonded from the geotextile. The other type of gage problem is illustrated by Gage 9-2 in Figure 9. The shape of the curve appears reasonable, but the strain magnitudes are higher than expected with the gage eventually becoming unreadable or highly erratic. Such behavior may be caused by high local point loads or damage to gage connections and lead wires. Questionable strain gage readings are indicated in Figure 10. The behavior of gages 9-2 and 9-3 apparently explains why the peak strain as measured by the strain gages is in a different location than the peak shown by the extensometers for the second instrumented layer. Thus, the real peak strain is at a distance of 2 m (6.6 ft.) from the wall face in this layer, which fits well with the peak strains in the other layers.

Plots such as Figure 9 were also utilized to extrapolate the "initial" readings, some of which were taken with a significant amount of fill on the gages, back to a most probable initial reading when there was no fill on the gages.

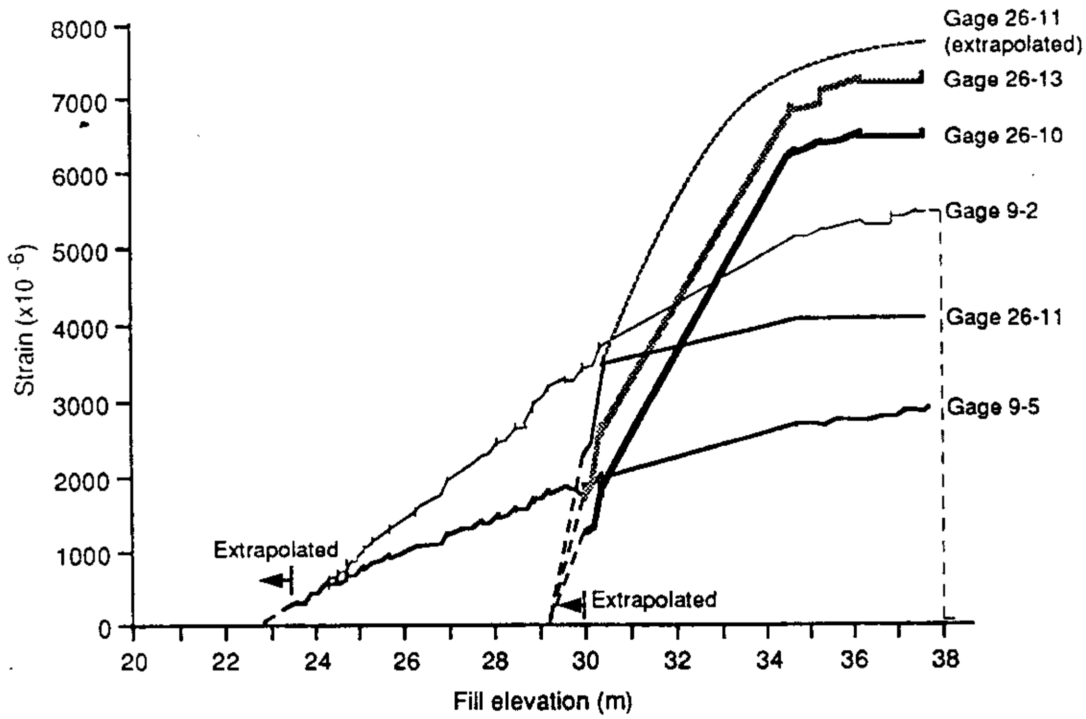

Figure 9. Typical development of geotextile strains with increase in fill height. Base of wall is at 19.7 m.

Gage readings before fill is placed are usually very erratic. Some fill is necessary to tension the geotextile to remove wrinkles that inadvertently occur. Some extrapolations are illustrated in Figure 9.

Figure 10 shows the final strain distribution in each instrumented reinforcement layer from the strain gage and the extensometer measurements taken at the end of construction, immediately after placement of the surcharge, and six months later. Based on the strain gages the maximum local strains in the reinforcement are on the order of 0.5%. Strains measured by the extensometers were on the order of 0.7 to 1.0%, which were somewhat higher than the strain gage results. The surcharge caused relatively small increases in strain (usually less than 0.05%) in the lower reinforcement layers, but relatively greater increases (0.1 to 0.2%) in the upper layers.

Bison coils were used to measure horizontal soil strain. Coils were located right at the face, near the theoretical Rankine failure surface, and away from the Rankine failure surface, but at different elevations in the wall (see Figure 3). Soil strains were, in general, greater than the geotextile strains. Soil strains were on the order of 1 to 2 % once the wall and surcharge were complete, and soil strain at the wall face was over 7 %.

Once the wall and surcharge were complete, strain gage and extensometer readings were continued during the next 10 months until the wall was torn down. Based upon the results obtained from the soil stress cells, inclinometers, and survey and photogrammetric measurements, it was determined that the relatively minor increases in strain measured by the strain gages and extensometers were due to creep and not load increases.

Typical creep curves measured for each geotextile layer near where strain was maximum are shown in Figure 11.

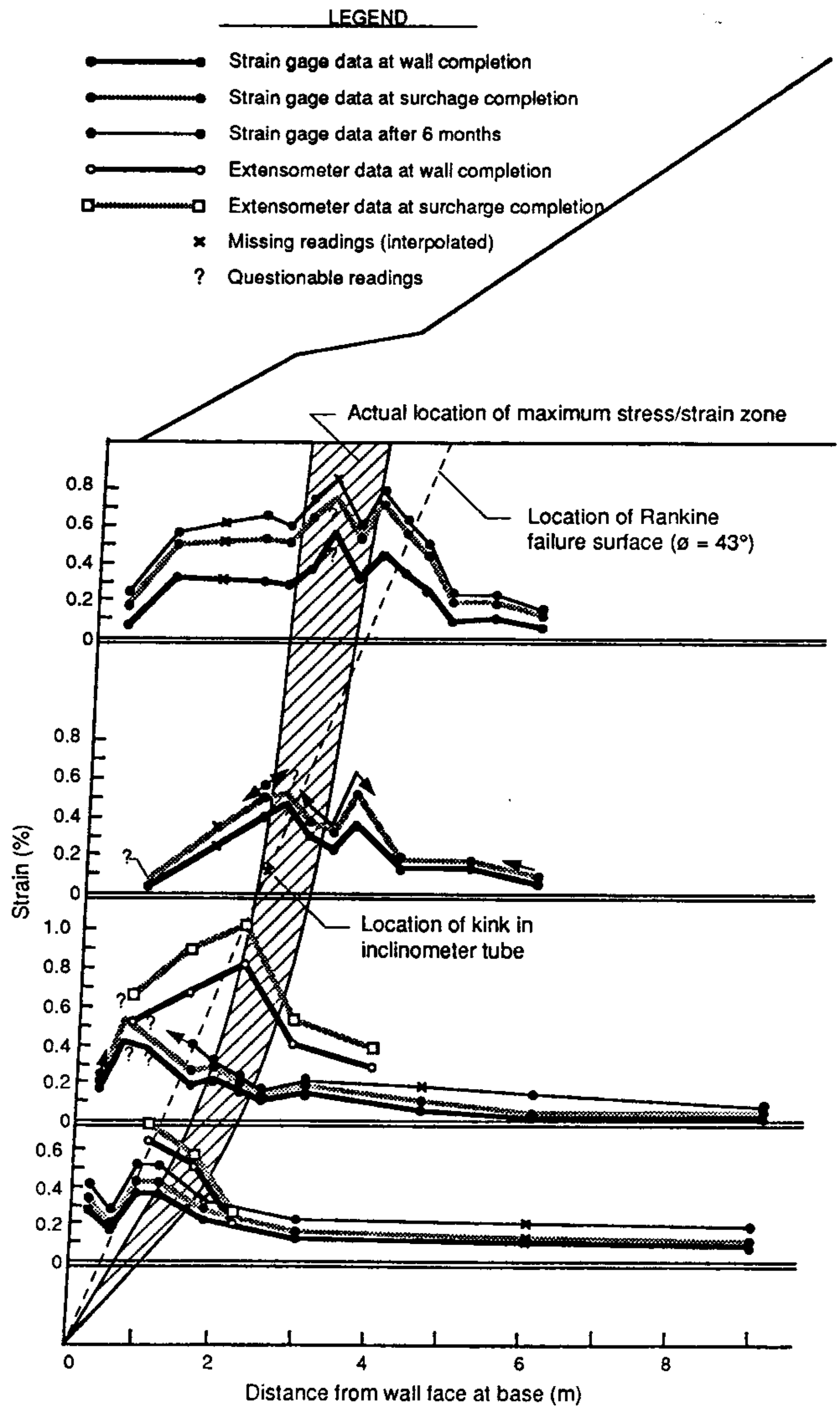

Figure 10. Distribution of strains in each instrumented layer at various times.

Measured creep rates were approximately 4.5 x 10-6 mm/mm/day one month after surcharge construction and 2.0 x 10-6 mm/mm/day ten months after surcharge construction. Only primary creep was observed, with stabilization beginning to occur.

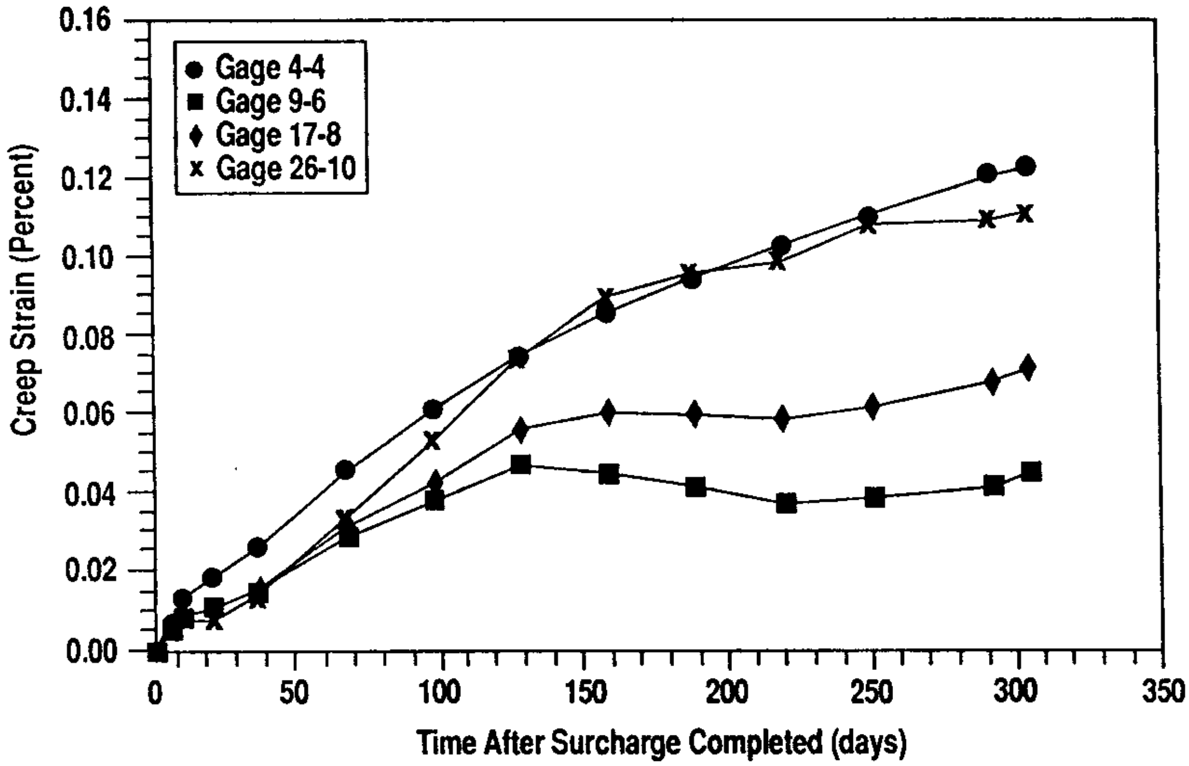

Figure 11. Geotextile creep after surcharge construction as measured by strain gages.

7.3 GLOBAL DEFORMATION

Global deformation was determined using inclinometers, photogrammetric and survey measurements, and liquid settlement devices. Vertical movements of the instrumented wall section were monitored by conventional optical surveys and three liquid settlement gages installed at the base of the wall as shown in Figure 3.

7.4 SETTLEMENT

The settlement distribution along the base of the wall is shown in Figure 12. As shown in this figure, the maximum total settlement occurred near the wall face and was lowest near the middle of the reinforced section. The liquid settlement devices indicated maximum settlements at the wall face of 88 mm (3.5 in.) at the end of surcharge construction and 102 mm (4.0 in.) when settlement was complete. However, the vertical settlement as determined from the optical surveys and photogrammetric measurements near the bottom of the wall face, which agreed well with one another, was considerably lower than the settlement measured by the liquid settlement devices. Vertical settlement determined from optical survey and photogrammetric measurements was on the order of 30 to 35 mm (1.2 to 1.4 in.) at the end of surcharge construction and 35 to 40 mm (1.4 to 1.6 in.) when settlement was complete.

7.5 COMPARISONS OF INCLINOMETER & OPTICAL SURVEY MEASUREMENTS

Figure 13 shows the deflection perpendicular to the wall face as measured by the inclinometer in the middle of the reinforced section. The kink at elevation 26 m (85 ft.)

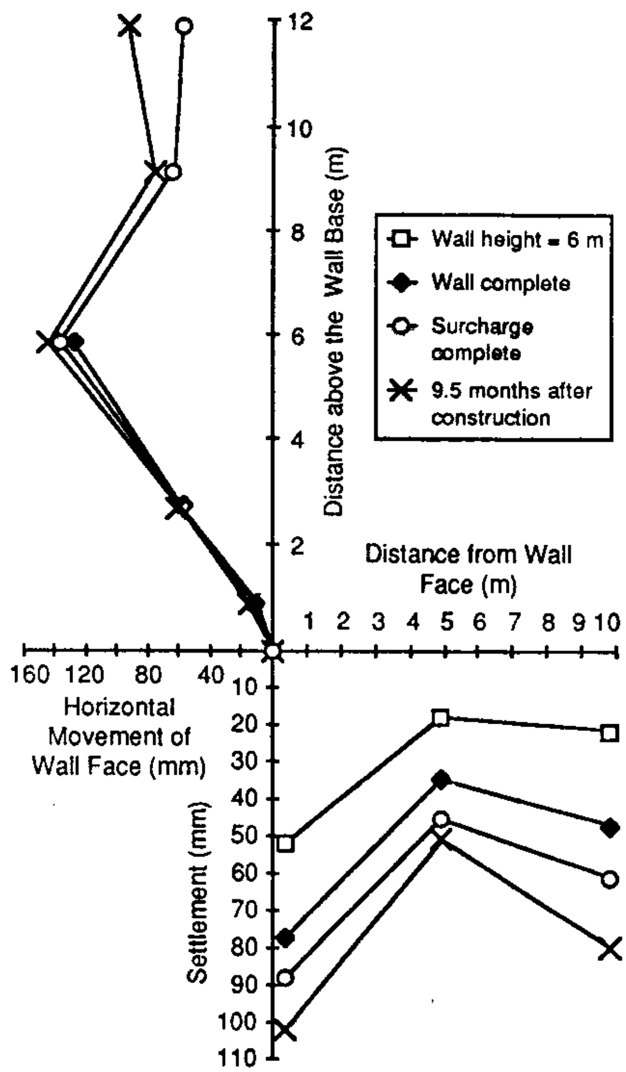

Figure 12. Settlement of the wall based on liquid settlement devices and lateral movement of the wall face as determined by optical surveys.

corresponds quite well to the location of the measured peak strains in the instrumented geotextile layers as shown in Figure 10. Deflection of the inclinometer at the back of the reinforced section was small, i.e., less than 4 mm (0.2 in.) maximum at all elevations. The inclinometers measured the total movement relative to the bottom of the inclinometer casing, which is a fixed reference. The optical survey readings shown in Figure 12 are incremental, in that the measured deflections are relative to the initial survey readings taken for each specific optical target. This is the basic reason for the different shapes of the curves in Figures 12 and 13. Such incremental deflections can be compared with measured geotextile strains, while total deflections from inclinometer measurements can be used to estimate overall wall movements.

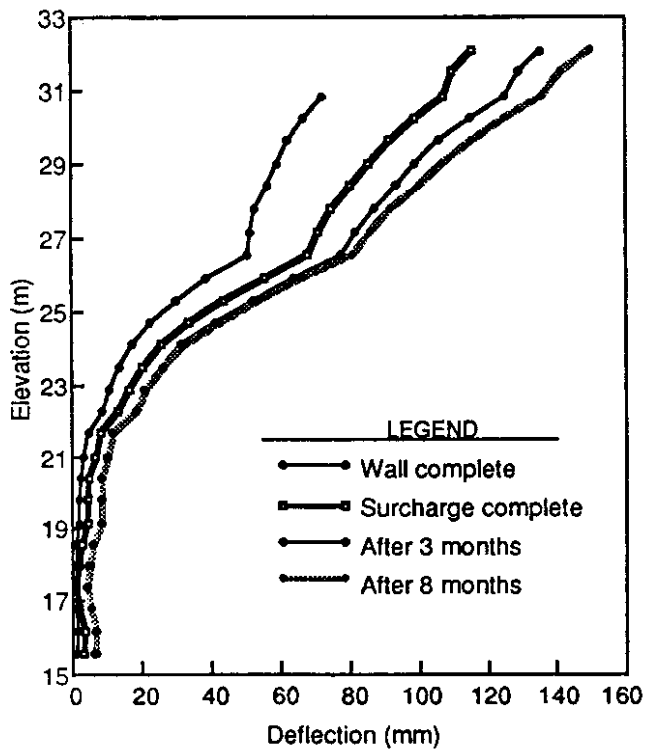

Figure 13. Deflection perpendicular to the wall face obtained from the inclinometer in the middle of the reinforced backfill. Base of wall is at 19.7 m.

The inclinometer tube on the wall face was attached when the wall was already nearly 10 m (33 ft.) high. Therefore, any wall face movements that occurred up to that elevation were not observed by this method. However, these observations are still valuable as a check of the other observations of the face movements.

7.6 COMPARISONS OF OPTICAL SURVEY & PHOTOGRAMMETRIC MEASUREMENTS

The optical survey readings as well as the photogrammetric measurements are relative to the initial readings taken for each specific optical or photogrammetric target. Comparison of the optical and photogrammetric face deflections obtained during construction up to completion of the wall and surcharge is shown in Figure 14. Both the shapes of the wall face and the magnitude of the deflections indicated by the two methods are generally similar. Optical survey measurements of the extensometer face plates, as well as the summation of strain measurements integrated over the layer length for both strain gages and extensometers, are also shown in Figure 14.

A complicating factor for the photogrammetric measurements was the lack of a suitable fixed reference target for the top of the wall, especially as the wall reached its design height. The top of a nearby lamp post was used for this purpose. However, it was impossible to attach a proper photo target to it, and the post was not very stable due to wind. This geometric weakness caused the photogrammetric measurements to have less accuracy, especially toward the top of the wall. The effect of this geometric weakness is analyzed in detail by Kim (1990).

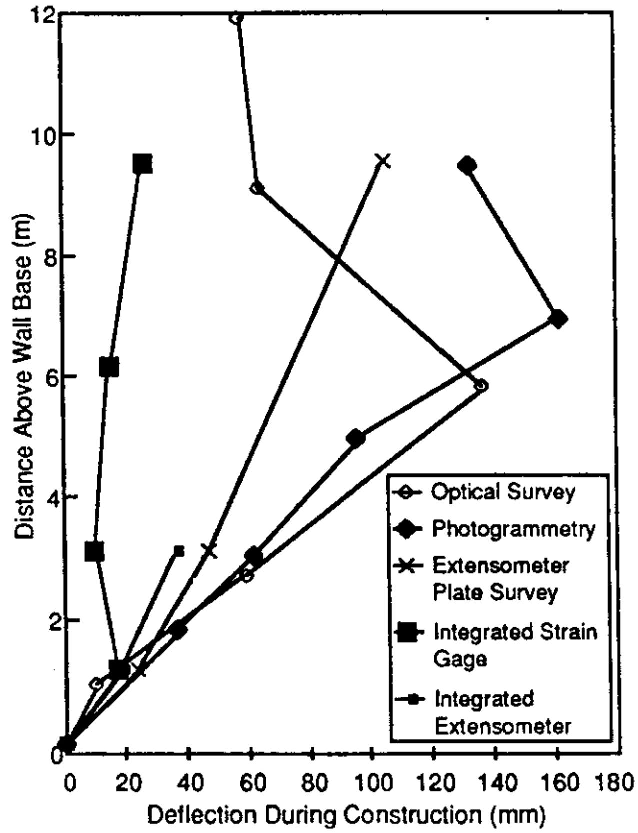

Figure 14. Wall face deflection during construction as determined from various methods.

7.7 COMPARISON OF ALL DEFLECTION READING MODES

The optical surveys of the extensometer face plates, as shown in Figure 14, generally agree with the other optical and photogrammetric data as well as the integrated extensometer strain readings. However, the total deflections obtained by integrating the strain gage readings over the length of the geotextile layer are considerably lower than both the optical survey and photogrammetric measurements for the upper three instrumented layers; total deflections are somewhat higher for the lowest instrumented layer.

7.8 POST-CONSTRUCTION DEFORMATION MEASUREMENTS

Figure 15 shows post-construction deformation measured at the wall face after the surcharge was complete using various methods. Long-term deformations were quite small, at least based on the results of the optical survey data and integrated strains.

The photogrammetric measurements showed somewhat greater post-construction deformation, especially toward the top of the wall. This may be the result of the inaccuracy caused by the geometric weakness discussed previously. Continued settlement of the wall backfill, which resulted in face bulging of the wrapped section between reinforcement layers, may also explain the difference in the deformation measured. The photo targets were painted on the geotextile in the middle of the wrapped face, whereas the optical survey rebar targets and the extensometer face plates were located between geotextile layers. Therefore, the photogrammetry targets would be more affected by face bulging than the other targets (face bulging will be more fully discussed later).

considering that all of the other measurements made as shown in Figure 15 agree quite well with one another both in magnitude and in the shape of the curve, the post-construction deformation measured by the photogrammetric method is probably not indicative of the actual overall wall behavior.

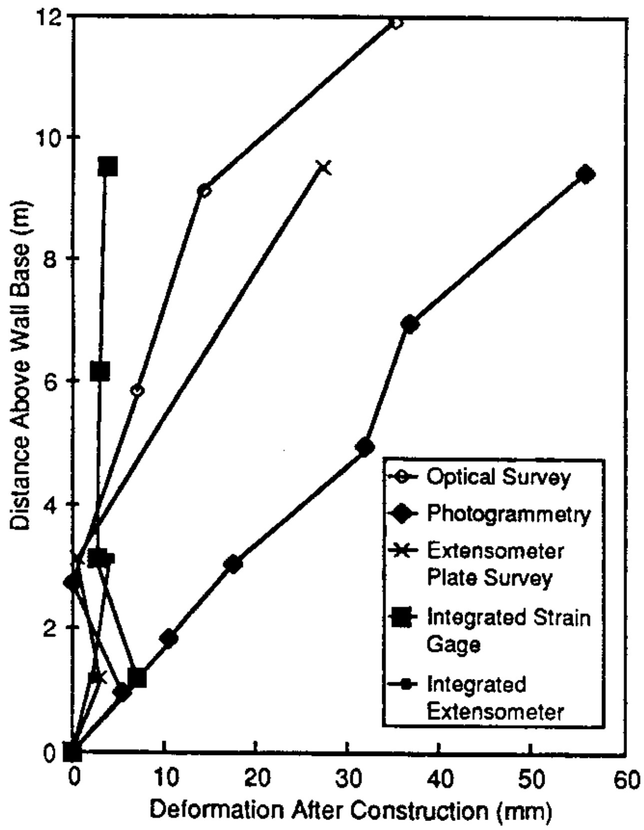

Figure 15. Deformation of the wall face after construction as determined by various methods.

7.9 VERTICAL SOIL STRESS

Vertical stresses beneath the wall were measured using Glötzl stress cells. The measured stresses are summarized in

Figure 16, which shows that toe stresses were approximately 20 % higher than in the middle or back of the wall. This corresponds well to the settlement profile at the base of the wall as shown in Figure 12. Such a profile is typical of flexible footings on granular soils (Perloff and Baron, 1976). Higher toe stresses could result from the tendency of the reinforced section to rotate outward about its toe. Vertical stresses were lower than the stresses predicted by overall wall weight and overturning calculations, and this point is still under investigation. Furthermore, vertical stresses remained constant after the surcharge was complete, providing additional proof that long-term changes in geotextile strain were due to creep and not due to changes in soil pressures.

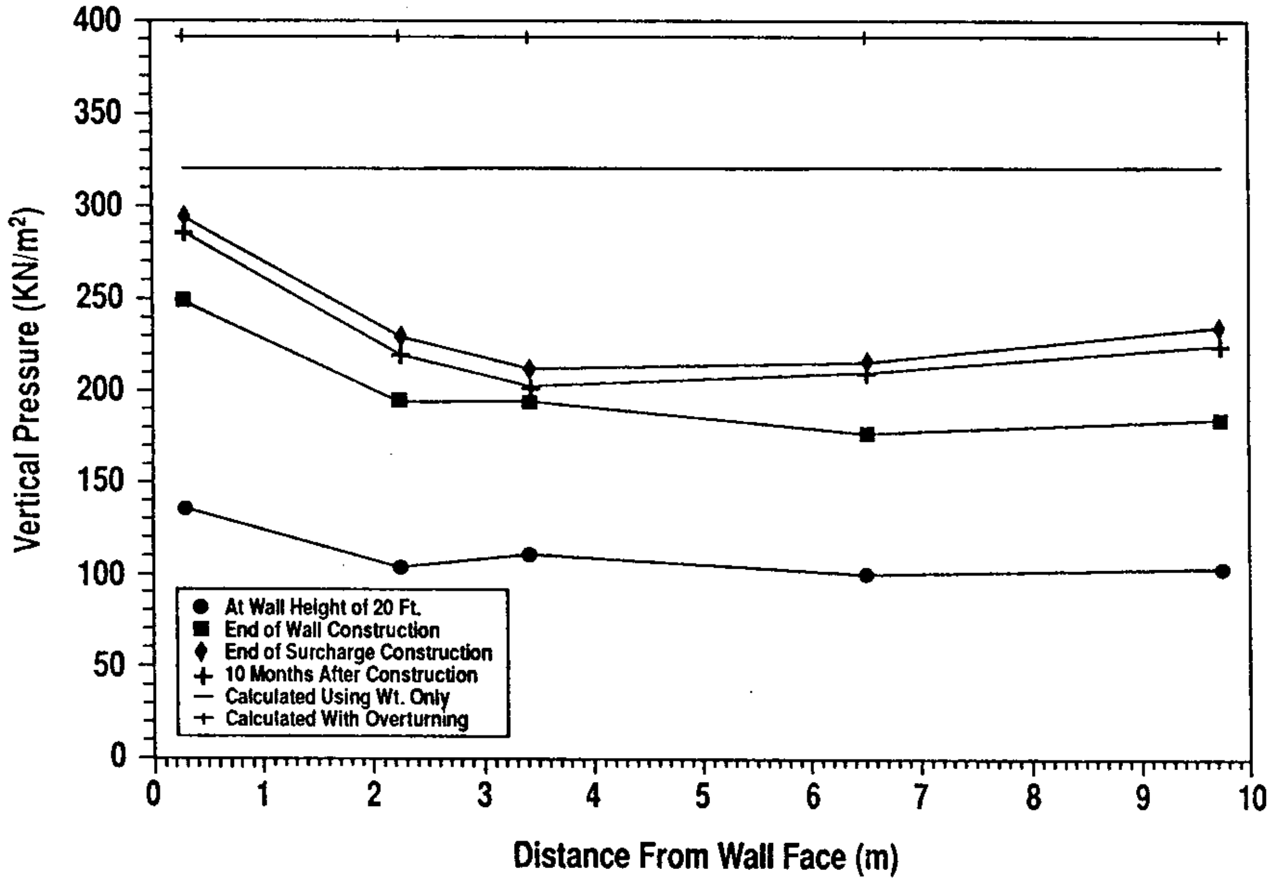

Figure 16. Vertical pressure beneath the wall.

CHAPTER 8.0

DISCUSSION

8.1 COMPONENTS CONTRIBUTING TO STRAIN

The differences observed in the strains as measured by the various types of instrumentation indicate the wall components which are contributing to the overall strain in the system. The extensometers have longer gage lengths than the strain gages. Therefore, the extensometers incorporate strain occurring in the geotextile macrostructure, including local effects such as creases and folds inadvertently placed in the geotextiles during shipping. Examples of such folds and creases are shown in Figure 8. These macrostructure effects appear to be the main reason that the strain gages recorded lower strains than the extensometers.

Since the extensometers incorporate strain due to folds and creases, extensometer strain is most appropriate for comparison to overall wall deflections. Strain gages, which do not incorporate these macrostructure strains, are most appropriate for determining stress level in the reinforcement, provided that the strain gages are properly calibrated.

This difference also shows up in the deformations obtained by integrating the strain gage readings as shown in Figures 14 and 15. The only exception to this is the relatively high integrated deformation from the strain gages in the lowest instrumented layer, which is still under investigation. Additionally, since the extensometers were not rigidly affixed to the geotextile reinforcement but were only wired to the geotextile, it was possible for the extensometer to move relative to the geotextile (if the soil was moving relative to the geotextile layer near the front of the wall). This could result in shoving of the front extensometers closer to the wall face, increasing the extensometer strains.

Because the Bison coils indicated greater soil strains than the extensometer strains, some slippage may have occurred between the soil and the geotextile, especially near the wall face. The wrinkling of the geotextile at the extensometer anchor points, as shown in Figure 8, also appears to support this conclusion. The difference in strain readings between the Bison coils and the extensometers may also be the result of variation in the strain field from the geotextile to the center of the soil mass between the layers. The strain in the soil between geotextile layers may be increased due to consolidation and lateral compression of the soil. This varying strain field between reinforcement layers contributes to the characteristic face bulging which occurs between every two reinforcement layers. It is not clear at this time how far back from the face any soil slippage and varying soil strain fields occurs, but it appears to be concentrated relatively close to the face.

The difference in measured strains in the various components of the reinforced wall system must be considered when evaluating overall face deformations, as is implied by the results in Figures 14 and 15. The evaluation of the load distribution among the various reinforcing layers must also consider the variations in measured strains between the various wall components.

Overall, measured strains in the geotextile reinforcement were significantly lower than the strains which would be expected based on the measured soil properties (i.e., $\phi = 43°$ to $47°$, $\gamma = 21.1$ kn./m^3 or 134 pcf), a factor of safety of 1.0, and the in-isolation moduli of the selected geotextiles. Based on these measured soil and geotextile properties, the expected maximum reinforcement load level would be approximately 20 % of ultimate. Strains of 2.5 to 3.5 % would be expected at this load level for all four geotextile types in the wall. Even if the strains measured by the extensometers are considered, the measured strains are less than one-third of the expected strains.

8.2 CREEP RATES

Furthermore, the measured creep rates from the strain gages were an order of magnitude lower than in-isolation creep rates obtained by others at similar load levels (Still and Williams, 1987). This can be partially explained by the fact that the strain gages do not fully account for macro-structure creep, which can be significant. The effect of soil confinement appears to be the main reason for the difference in the strain rates. Possible differences between the calculated and the actual load level may be a contributing factor as well.

8.3 STRENGTH LOSS DURING INSTALLATION: EFFECT ON PERFORMANCE

Since the observed installation damage did not appear to affect the modulus of the reinforcement, the significant strength loss which occurred during installation (Table 2) was probably not a factor in the measurements obtained. Other studies have also shown that creep rates and modulus are not affected by installation damage if the damage is not severe (Allen, 1991, and Viezee, et.al., 1990). Therefore, the significant strength loss observed which was not accounted for by the design apparently did not adversely impact the safety of the wall, considering the low strain levels measured.

It is not clear at this time whether the lower strains and creep rates observed are the result of lower than expected load levels in the geotextile, or due to a poorly understood interaction between the geotextile and the soil resulting from differences in the stress-strain and creep behavior of these two materials. Limited in-soil tensile testing conducted for this project indicates that the geotextile tension is slightly less than or equal to the tension predicted by Rankine theory. Additional research on this point is required.

8.4 PEAK STRAIN MEASUREMENTS

The locations of the peak strains measured in the geotextile reinforcement consistently follow a curved surface, as shown in Figure 10. A Rankine surface for ϕ = 43° would be slightly closer to the wall face in the lower half of the wall and slightly farther from the wall face in the upper half of the wall. Interestingly, the kink in the inclinometer profile mentioned previously provides an excellent verification of the potential failure surface (see Figures 10 and 13).

8.5 WALL FACE DEFLECTIONS

Wall face deflections as measured by optical survey and photogrammetric methods were on the order of 140 to 160 mm (5.5 to 6.3 in.) during construction, and this maximum occurred near the mid-height of the wall as shown in Figure 14. It is interesting to note that the region of maximum horizontal movement, i.e., at one third to one half the height of the wall, corresponds quite well to the region of maximum horizontal strain observed by others (Christopher, et.al., 1989). The magnitude of the horizontal wall face displacement was estimated using an empirical relationship suggested by Christopher, et. al. (1989), which was developed from finite element analyses, small scale model tests, and very limited field evidence on walls up to 6 m high. Without a surcharge, this relationship predicts horizontal face movements between 130 and 140 mm (5.1 and 5.5 in.), or slightly less than the observed maximum deflection. Thus, for design purposes, the Christopher, et. al., (1989) recommendation appears to be satisfactory, even for walls over 12 m (40 ft.) high.

Post construction wall face movements (Figure 15) were very small near the bottom of the wall but generally increased to a maximum amount of approximately 30 mm (1.2 in.) near the top of the wall. Post-construction deformation measured by photogrammetry was greater than this, but was not considered an accurate measure of the true creep deformation, as discussed previously. The greater creep measured at the top of the wall appears to be due to the upper geotextile layers being stressed more than the lower layers; i.e., higher creep deformations indicate higher stresses relative to the ultimate strength of the geotextile. The reinforced soil-geotextile system is more flexible at the top. Because the confining stress is less and the modulus of the upper geotextile is lower, the top part of the wall may tend to yield more under sustained loading.

8.6 SETTLEMENT

Maximum total settlement of the preloaded area was predicted in advance of construction to be about 25 mm (1.0 in.) in the vicinity of the wall. This settlement estimate was made assuming that there were little or no compressible silts and clays directly beneath the wall at this location, as the test section was near the edge of the lacustrine deposit. Subsurface information obtained during installation of the inclinometers verified this assumption. The optical survey and photogrammetric measurements indicated that actual settlement was close to that predicted (i.e., approximately 40 mm or 1.6 in.), but the liquid settlement devices indicated unexpectedly high settlements (i.e., approximately 100 mm or 4.0 in.). Due to the close proximity of the freeway on-ramp in front of the wall, the reservoir for the liquid settlement devices had to be located beyond the east end of the preload fill where the ground surface was approximately 5.5 m (18 ft.) higher in elevation than the devices themselves. This elevation difference is near the limit of the fluid pressure that the devices can handle. Furthermore, the tubes for the devices are filled with an ethylene-glycol/water mixture which may tend to separate with time. Since the two liquids have different specific gravities, separation of the liquids could change the readings obtained. Considering all this, we feel that the true total settlement is likely closer to the 40 mm (1.6 in.) value measured by the optical survey and photogrammetric methods.

8.7 SURCHARGE EFFECTS

The soil surcharge at the top of the wall had only a small effect on the strains and deformations measured in the lower half of the wall, but had a more significant effect on the upper half of the wall, especially at the wall top. This was true for both the strains measured during construction as well as those measured after construction. Current design methodology assumes that the soil surcharge equally affects the wall throughout its height. Based on the observed wall deformations, this assumption may be overly conservative, especially in the lower half of the wall. The effect of the soil surcharge will become much clearer once the actual load levels in the wall are better defined through future research.

8.8 WALL FACE MOVEMENTS: IMPLICATIONS FOR DESIGN

Previous studies of geosynthetic reinforced walls have reported very small horizontal deflection of the wall face. Most of those measurements were made after construction was completed. The present study has shown that significant wall face movements occurred during construction. Thus, it is important to consider the wall face movements occurring during construction in order to maintain proper wall batter and alignment of the face. Furthermore, large face deflections have serious implications with regard to the installation of shotcrete or rigid face panels. If the latter are to be used, it may be advisable to attach the panels to the wall face after it has been constructed to full design height.

CHAPTER 9.0

CONCLUSIONS

Overall, the instrumentation program was successful in that most instruments survived construction, and they appeared to provide consistent results. Further analysis of the strain distribution data to determine the stress state in the reinforcement will be valuable for refinement and modification of existing design methods so that more reliable factors of safety can be used in future designs.

Several lessons were learned from the instrumentation program regarding instrument selection, installation, calibration, and monitoring. Probably the most important was the task of strain gaging the geotextile reinforcement. For future projects we recommend:

1. The instrumentation should be sensitive over a wide range of strains (large during construction to very small following construction).
2. The gages and attachment methods must be compatible with the type of reinforcement material.
3. Sufficient redundancy to explain anomalous data should be provided.
4. Sufficient number of instruments along with preferential spacing to identify areas of high stress should be provided.

5. Measurement of both local (micro) and global (macro) strains is desirable.
6. Calibration of samples of gaged reinforcement using both unconfined tension and confined in-soil tension or pullout tests should be performed.
7. Strain gages should be placed on both top and bottom of the reinforcement to identify bending stresses and other localized high stress areas.
8. Temperature effects should be evaluated to insure that the instruments are not affected.
9. Great care should be taken during initial placement of soil cover.
10. Continuous monitoring during construction (not just at its beginning and end) is highly desirable.
11. Future instrumented walls should include a more detailed evaluation of soil strain in comparison to reinforcement strain.

Total strains and long-term creep rates were much lower than expected, showing that current design methodologies are conservative, even for very high walls. A better understanding of in-soil load-strain and creep behavior of geotextiles, especially soil-geotextile interface creep, is needed if more accurate design methodologies are to be developed.

Based on the measurements, most of the movement in the geotextile wall occurred during construction. This movement was significant, on the order of 1 % of the height of the structure. Post-construction creep deformations were small but were greatest at the top of the wall. Thus, facing systems on permanent walls of this height must consider all movements of the wall, especially near its top and if large surcharges are likely to be present.

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