Report No. CDOT-DTD-97-10

LOADING TEST OF GRS BRIDGE PIER AND ABUTMENT IN DENVER, COLORADO

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Final Report
October 1997

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REPORT DOCUMENTATION PAGE

1. AGENCY USE ONLY (Leave Blank)	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	October 1997	Final Report

4. TITLE AND SUBTITLE

Load Test of GRS Bridge Pier and Abutment in Denver, Colorado

5. FUNDING NUMBERS

6. AUTHORS(S)

Kanop Ketchart and Jonathan T.H. Wu

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(S)

Reinforced Soil Research Center

Department of Civil Engineering

University of Colorado at Denver

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(S)

Colorado Department of Transportation

4201 E. Arkansas Ave.

Denver, Colorado 80222

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

CDOT-DTD-97-10

11. SUPPLEMENTARY NOTES

Prepared in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

12a. DISTRIBUTION/AVAILABILITY STATEMENT

No Restrictions: This report is available to the public through the National Technical Information Service, Springfield, VA 22161

12b. DISTRIBUTION CODE

13. ABSTRACT (Maximum 200 words)

A GRS bridge abutment and two GRS bridge piers were constructed inside a 3.5-m deep pit in Denver, Colorado. The structures were constructed with a "road base" backfill and reinforced with layers of a woven geotextile. Dry-stacked hollow-cored concrete blocks were used as facing. One of the piers and the abutment, both 7.6 m in height, were load tested. The load was applied using concrete barriers stacked in seven layers over three steel bridge girders. A total load of 2,340 kN, corresponding to 232 kPa vertical pressure, was applied. The pier and the abutment were instrumented with metal pipes and elastic springs to monitor the vertical and lateral movement of the facing, and strain gages to monitor deformation of the reinforcement. This report describes the configuration of the structures, the material properties of the backfill and the geotextile reinforcement, the construction procedure, the loading schemes, and the instrumentation. The report also presents measured results and discussions of the measured results.

14. SUBJECT TERMS

Fortress, Reinforcement, Geosynthetic,, Soils,

15. NUMBER OF PAGES

62

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
Unclassified	Unclassified	Unclassified	

Technical Report Documentation Page

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Chapter 1 INTRODUCTION

Geosynthetic reinforced soil (GRS) technology has been widely used in the construction of retaining walls, embankments, slopes, and shallow foundations. In July 1996, a 5.5-m high prototype GRS bridge pier was constructed at Turner-Fairbank Highway Research Center, Federal Highway Administration. A series of loading tests have demonstrated that the GRS bridge pier has a very high load capacity and excellent performance characteristics under typical design loads (Adams, 1997). This study was undertaken as a continuation of the Turner-Fairbank's test.

The objectives of this study were three folds. The first objective was to investigate the performance of a GRS bridge support system, including an abutment and a pier, subject to design loads. The second objective was to investigate the long-term performance of such a bridge support system under a sustained design load. The third objective was to examine the performance of GRS bridge abutment and pier when constructed in a less stringent condition.

This project was a joint effort of the Colorado Department of Transportation, Reinforced Soil Research Center of the University of Colorado at Denver, and Turner-Fairbank Highway Research Center of the Federal Highway Administration. The project

site is near the intersection of Interstate Highway-70 and Havana street in Denver, Colorado.

Chapter 2 PROJECT DESCRIPTION

This project consisted of two piers and one abutment. These structures were situated in a 3.53-m deep pit as depicted in Figure 1. The outer pier and the abutment were 7.6 m tall. The center pier was 7.3 m tall, 0.3 m shorter than the outer pier and the abutment. The pier was made shorter for the purpose of a second-stage load test to be conducted at a later time.

The center pier and the abutment were of a rectangular shape and the outer pier was of an oval shape, as shown in Figure 2. The bases of the outer pier, the center pier, and the abutment were 2.4 m by 5.2 m (major and minor axes), 2.7 m by 5.4 m, and 4.6 m by 7.2 m, respectively. The tops of the outer pier, the center pier, and the abutment were, respectively, 1.8 m by 4.6 m (major and minor axes), 2.1 m by 4.8 m, and 3.6 m by 5.2 m. The edge to edge distance between the outer pier and the center pier and between the center pier and the abutment was 2.7 m.

At the bottom of the pit was a geosynthetic-reinforced soil foundation. The reinforced soil foundation comprised three layers of geotextile reinforcement with a constant vertical spacing of 0.3 m. The geotextile reinforcement was the same type as those used in the piers and the abutment.

CONCRETE BLOCKS

GIRDERS

0.74 m

2.44 m

STEPS

Reinforcement spacing 0.2 m

Reinforcement spacing 0.2 m

Reinforcement spacing 0.2 m

7.62 m

7.31 m

7.62 m

5.61 m

3.53 m

5%   5%

4%   4%

3%   3%

3%

13%

Reinforcement spacing 0.3 m

OUTER PIER   CENTER PIER   ABUTMENT

0.9 m

4
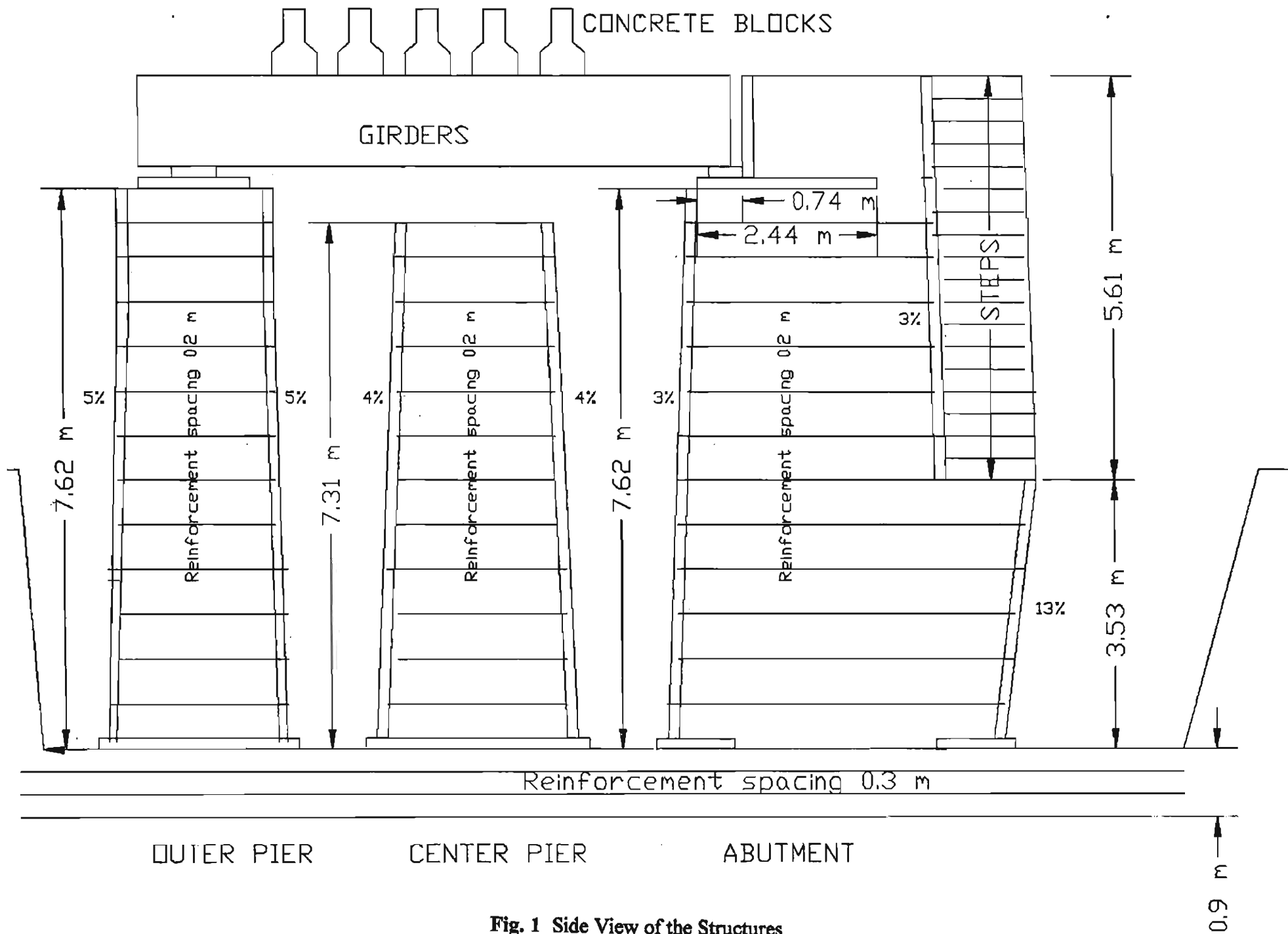
Fig. 1 Side View of the Structures

Fig. 2 Top View of the Structures

Outer Pier

2.45 m — 2.74 m — 2.74 m — 2.74 m — 4.57 m

0.91 m

concrete pad

4.57 m

3.05 m

5.18 m

top

base

1.82 m

N

W — E

S

Center Pier

2.10 m

top

base

4.78 m

5.42 m

Abutment

top

base

concrete pad

7.16 m

5.18 m

3.66 m

2.44 m

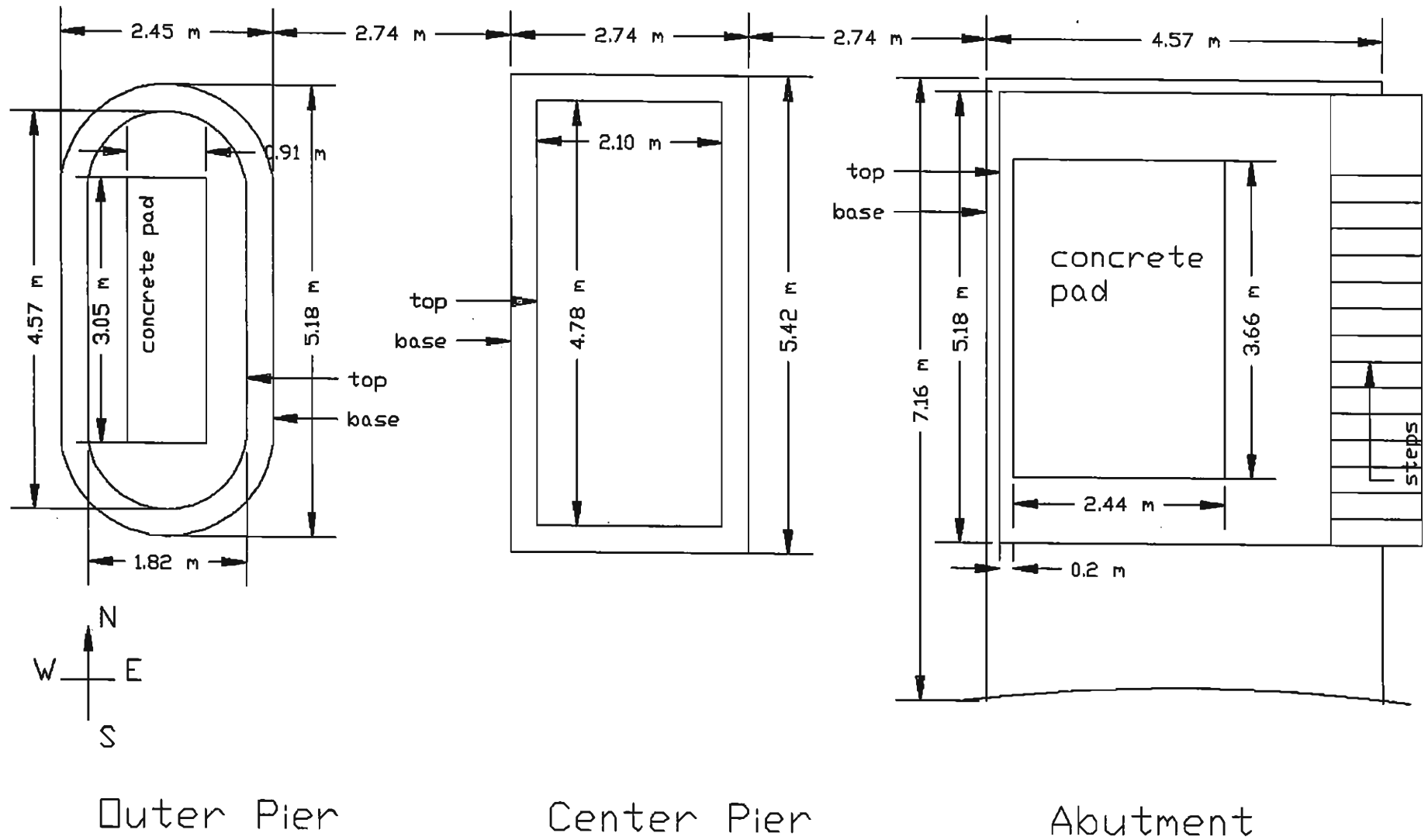0.2 m

steps

5

The piers and the abutment were constructed on a 0.15-m thick concrete pad placed over the reinforced soil foundation (see Figure 1). The vertical spacing of the geotextile reinforcement in all three structures was 0.2 m. The reinforcement covered the entire top surface area of backfill and facing blocks at each construction lift. The top four layers of the reinforcement in the abutment employed a wrapped-around procedure behind the facing block. A geotextile "tail", 1.2 m in length, was placed between each of these four layers to connect the backfill to the facing blocks. Modular blocks, 0.2 m in height, were used as the facing element for all three structures. Compaction of the backfill was conducted at each course of the facing blocks. The facing element was made to incline from the base to the top of approximately 5% in outer pier, 4% in the center pier and 3% in the abutment. On the east side of the abutment, the facing assumed a 13% negative batter up to a height of 3.5 m. From 3.5 m to the top of the abutment were walking steps as shown in Figure 1. The negative batter was made to examine the feasibility and stability of such a facing configuration.

On top of the piers and the abutment were 0.3 m-thick concrete pads to support steel bridge girders. The concrete pads were 0.9 m wide and 3.1 m long for the piers and 2.4 m wide and 3.7 m long for the abutment, as shown in Figure 2. It is to be noted that the clearance of the concrete pad was only about 0.02 m behind the back face of the abutment facing blocks (see Figure 1).

Chapter 3 MATERIALS

3.1 Backfill

The backfill was a "road base" material classified as A-1-A(0) according to AASHTO. It has 13% of fine particles (passing sieve #200). The gradation curve is shown in Figure 3. The maximum dry unit weight, per AASHTO T180 method D, is 21.2 kN/m^3. The optimum moisture is 6.7 %.

3.2 Geotextile Reinforcement

The reinforcement was a woven prolyproplylene geotextile. The geotextile reinforcement was the same type used in the Turner-Fairbank's load test.. The wide width tensile strength in both fill and warp directions of the geotextile is 70 kN/m. The tensile strengths at 5% strain of the fill and the warp directions are 38 kN/m and 21 kN/m, respectively. Some index properties of the geotextile reinforcement are shown in Table 1.
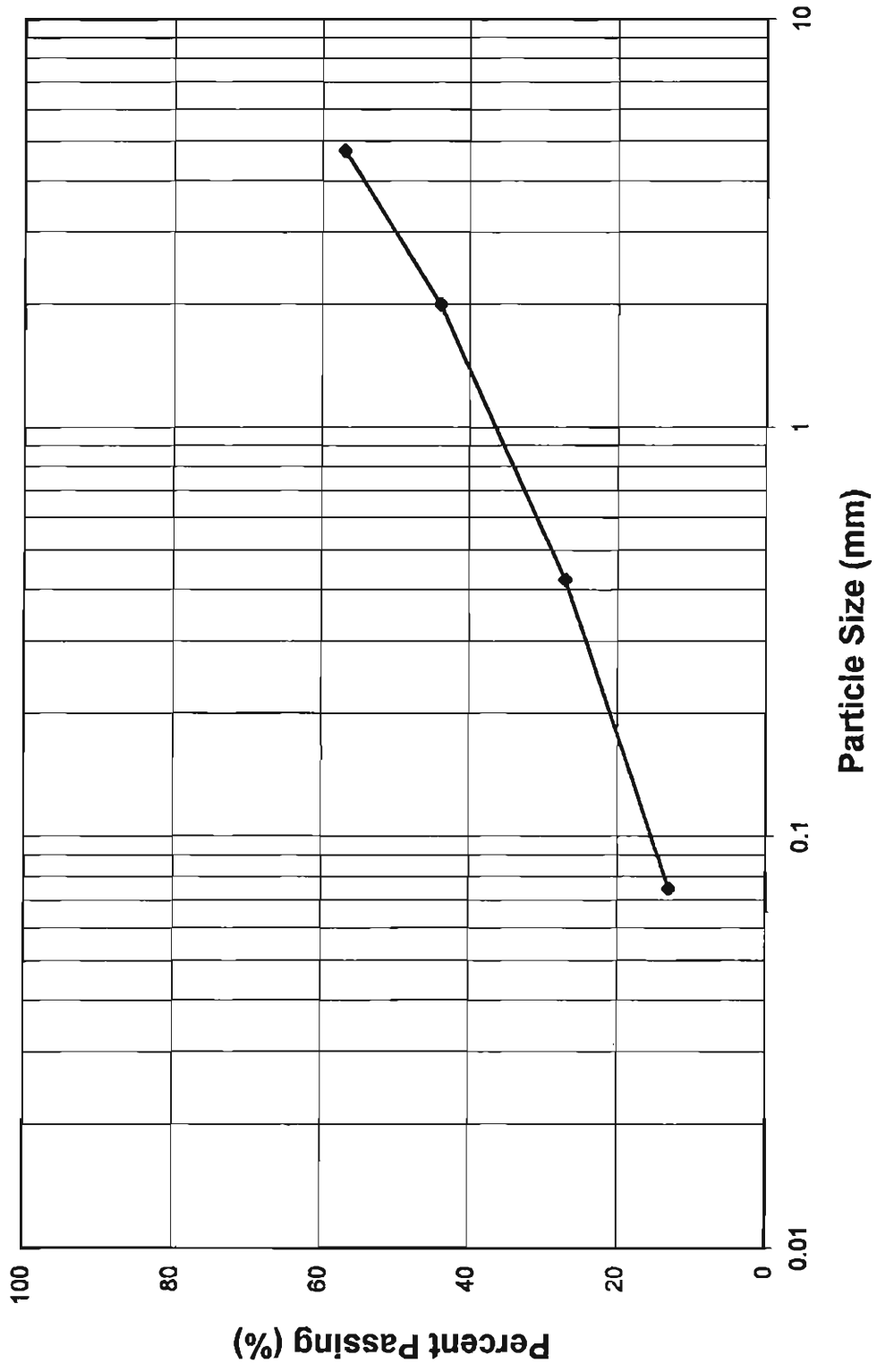
Fig. 3 Gradation Curve

Polymer Type	Structure	Wide Width Tensile Strength ASTM D 4595 (kN/m)				Grab Tensile Strength ASTM D4632-86 (kN)	
		Strength @ 5% Strain		Ultimate Strength (% @ Break)		(% @ Break)	
		Fill Direction	Warp Direction	Fill Direction	Warp Direction	Fill Direction	Warp Direction
Polypropylene	Woven	38	21	70 (18%)	70 (18%)	2.6 (20%)	2.2 (20%)

Table 1 Some Index Properties of the Reinforcement

Chapter 4 CONSTRUCTION

The construction procedure of the GRS pier and abutment is described in the following steps:

1. Excavate a 3.5-m deep test pit;

2. Prepare the geosynthetic-reinforced soil foundation;

3. Pour and level a 0.15-m thick concrete pad on top of the geosynthetic-reinforced soil foundation;

4. Lay a course of facing blocks conforming to the designed shape of the structure;

5. Backfill and compact in 0.2 m lifts;

6. Place a layer of geotextile reinforcement covering the entire top surface area of the compacted fill and the facing blocks;

7. Repeat steps 4, 5 and 6 until completion.

Selected photos taken during construction of the abutment and piers are shown in Plates 1 to 13.

Field density tests were performed on the center pier and the abutment during construction. The average dry unit weight of the center pier was 19.3 kN/m^3 (91% of the modified Procter relative compaction) with the average moisture of 2.5%. For the abutment, the average dry unit weight was 19.1 kN/m^3 (90% of the modified Procter

relative compaction) and the average moisture was 1.6%. The density of the outer pier was believed to be lower than these measured values as a lighter compaction plant was employed.

Plate 1 Excavation of the Construction Pit

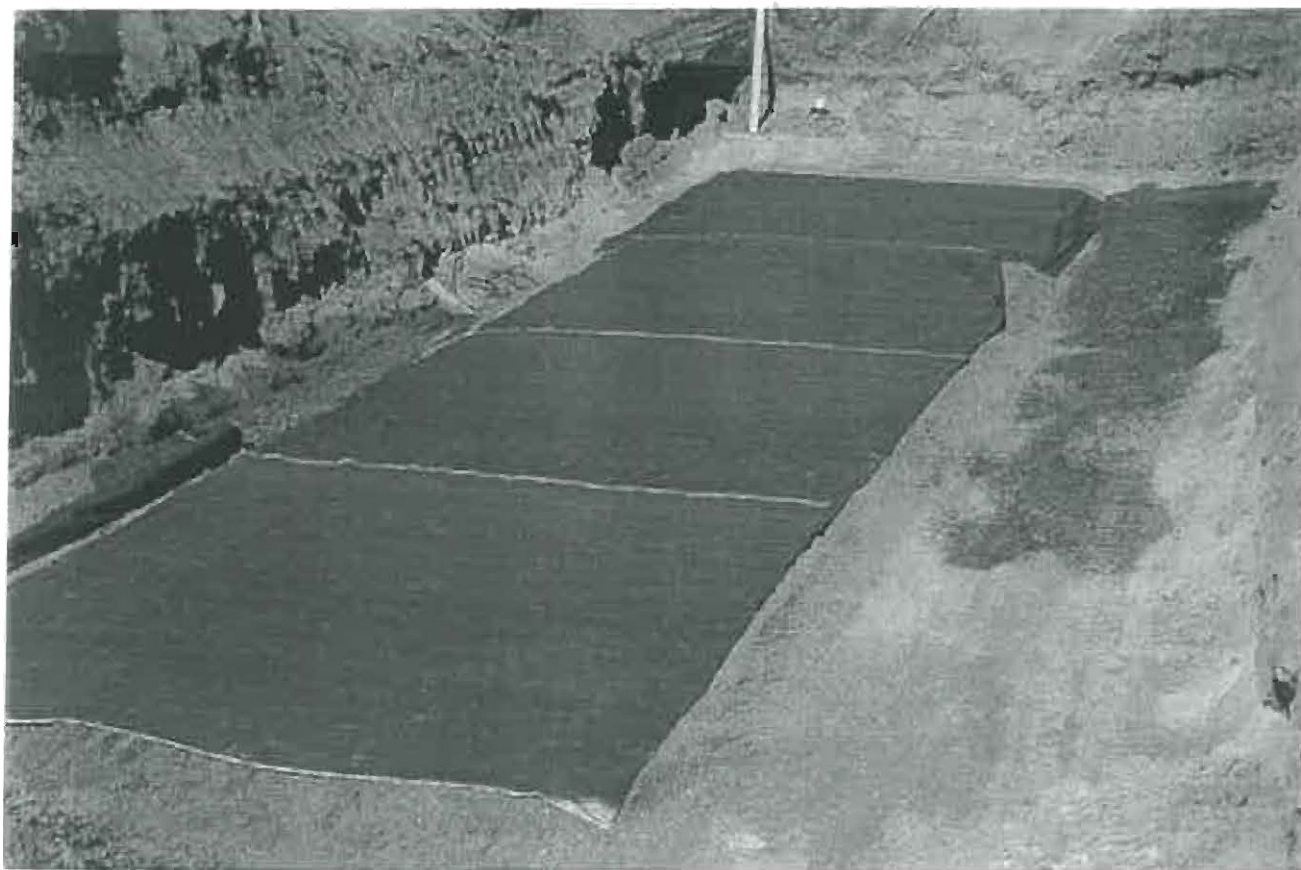Plate 2 Construction of the Reinforced Soil Foundation

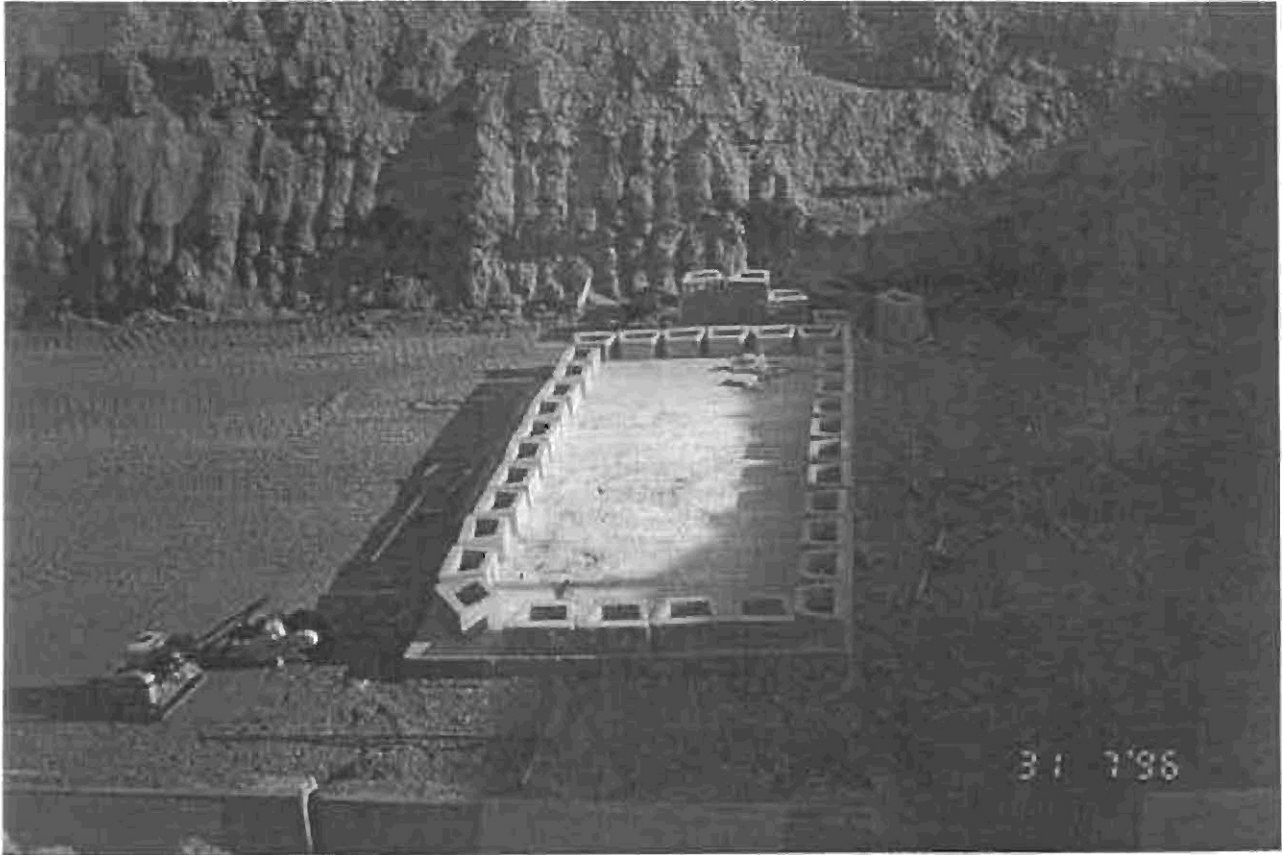Plate 3 Reinforced Soil Foundation with a Woven Geotextile Reinforcement

Plate 4 Laying the First Course Facing Block on a Concrete Pad
(Note the light-weight compaction plant at the lower left-hand corner)

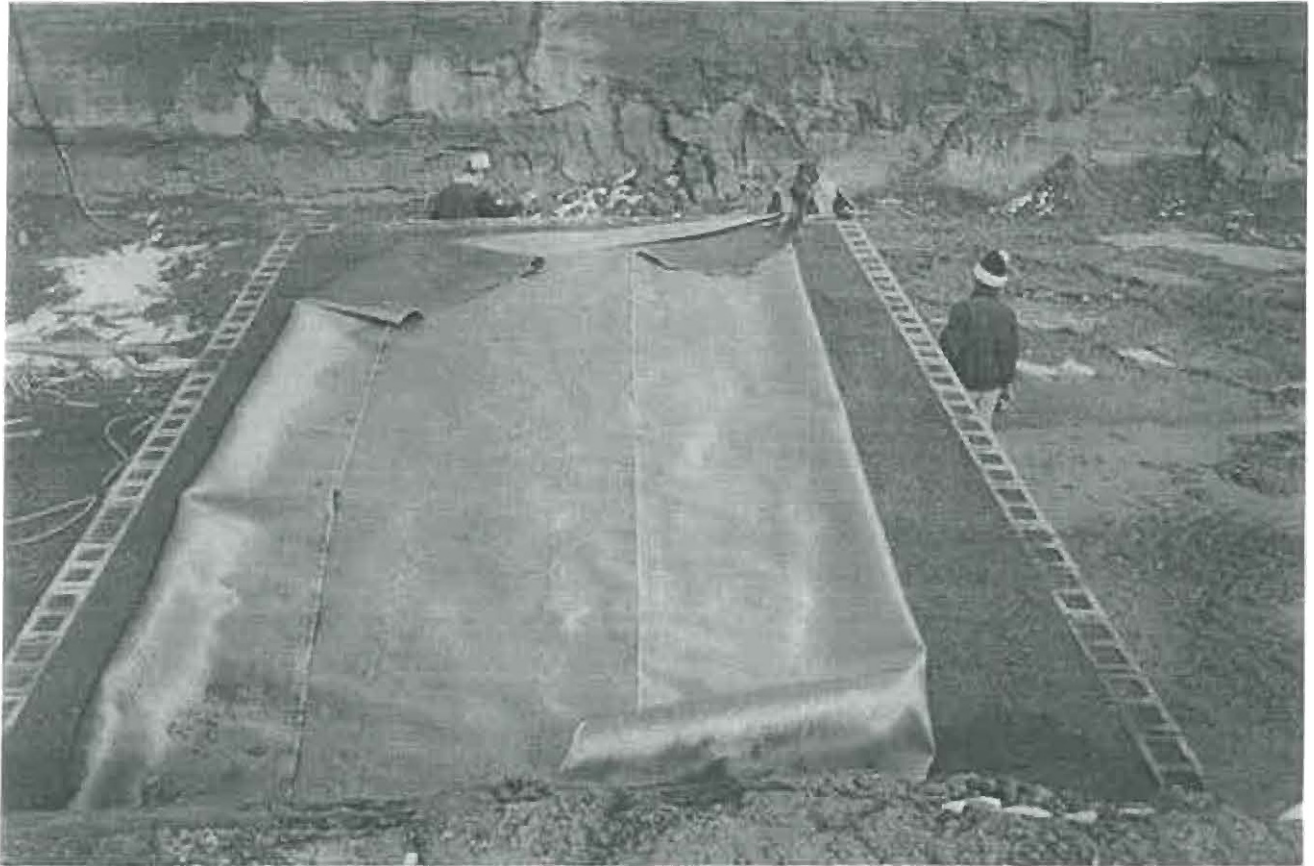Plate 5. Compaction of the Center Pier

Plate 6 Placement of a Layer of Geotextile Reinforcement
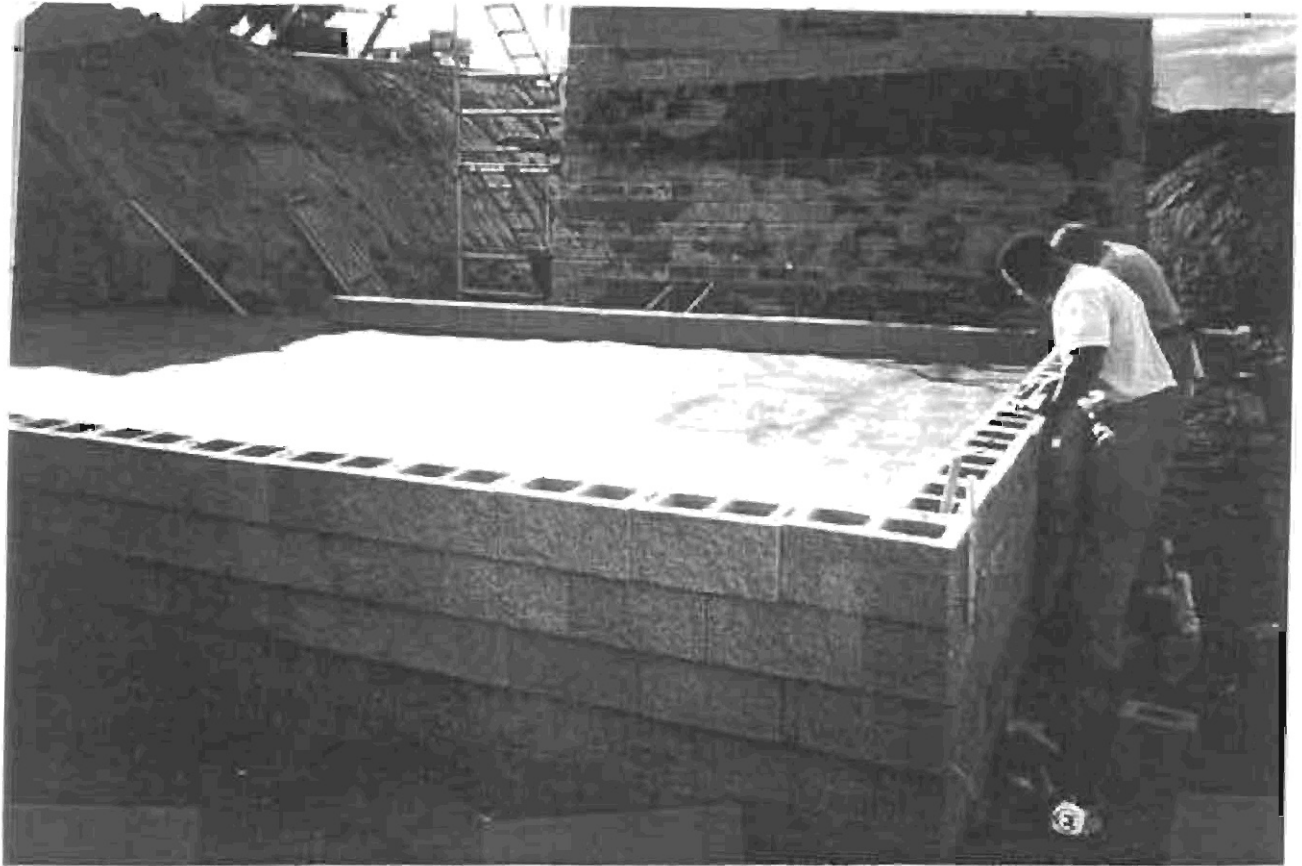
Plate 7 Alignment of Facing Blocks

Plate 8 Construction of the GRS Abutment with a Negative Batter

Plate 9 Cutting Concrete Blocks to Conform to the Designed Configuration
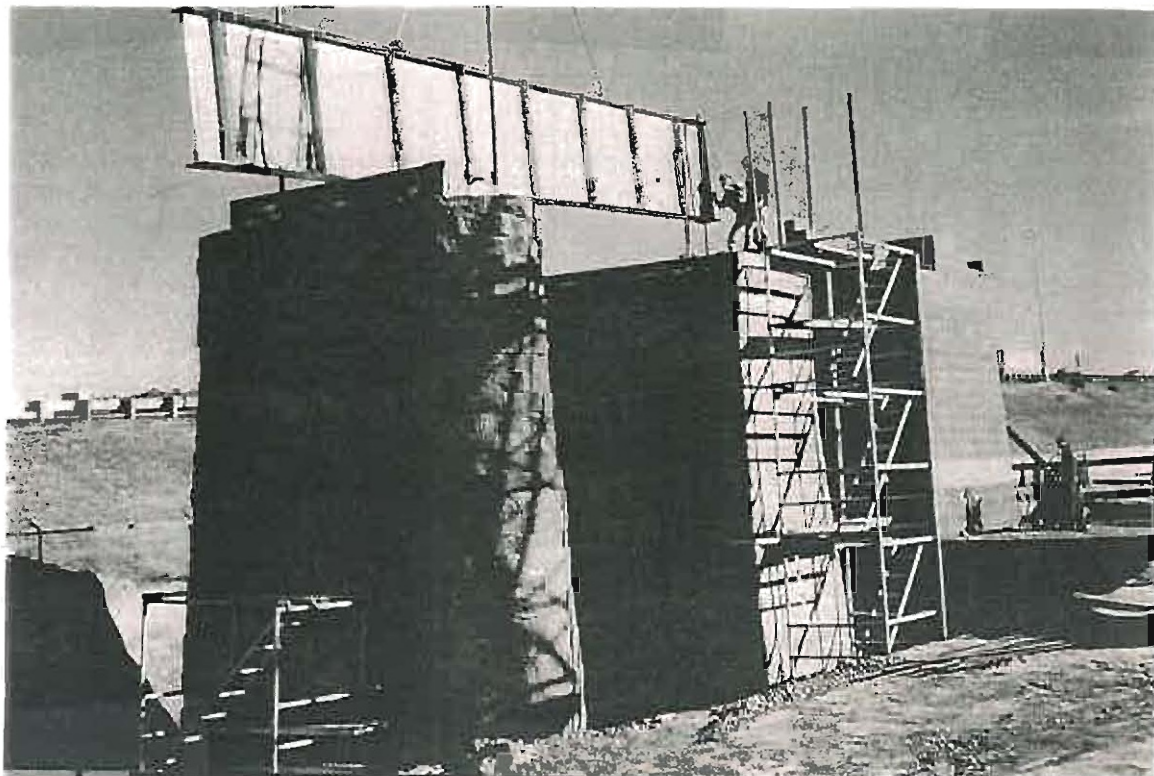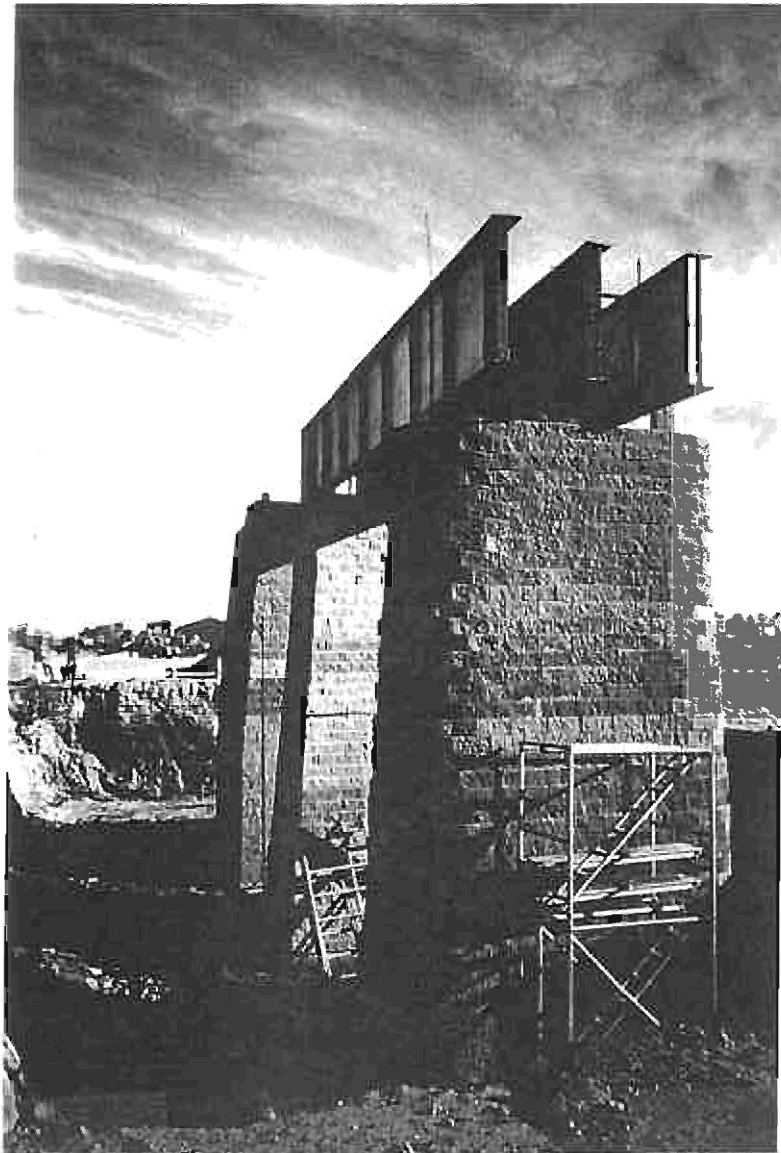
Plate 10 Placement of a Steel Bridge Girder

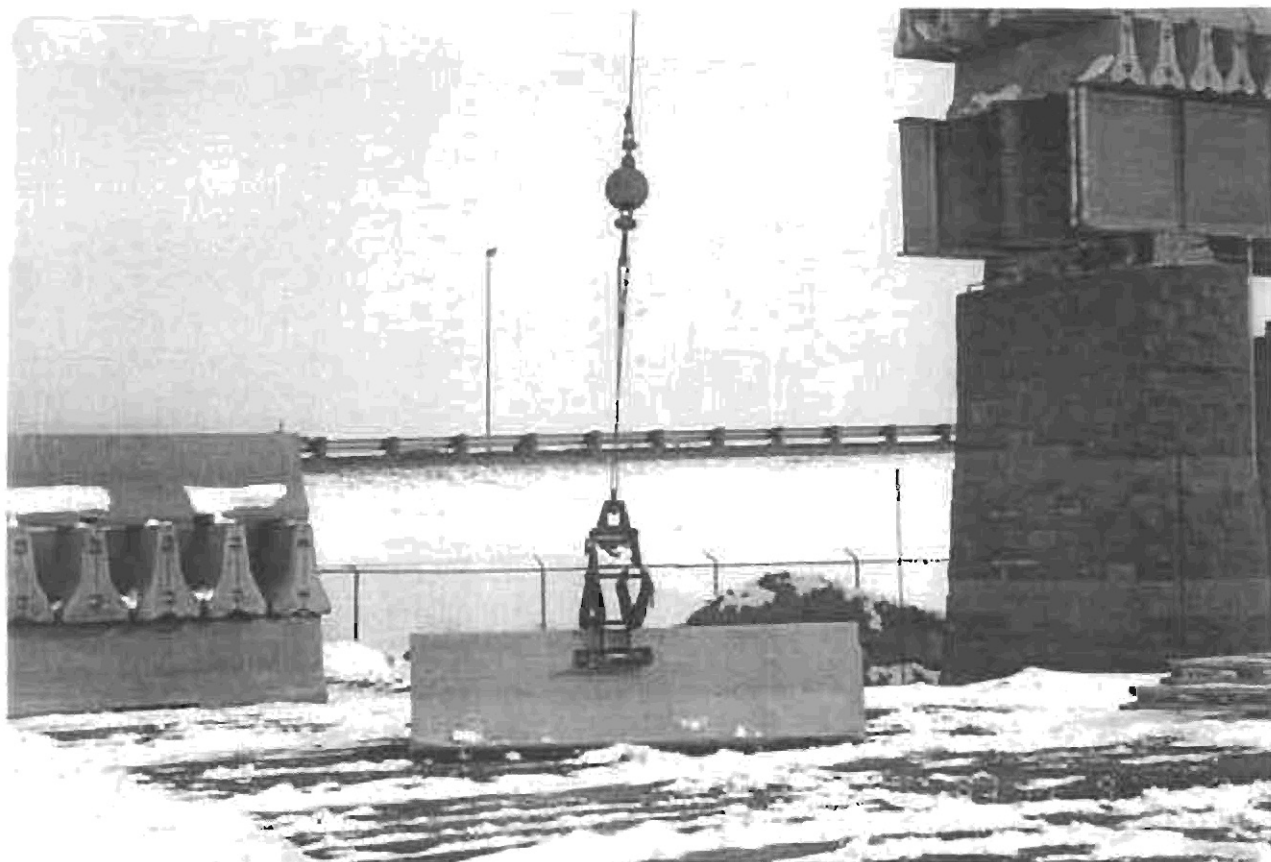Plate 11 Three Steel Bridge Girders were Welded Together

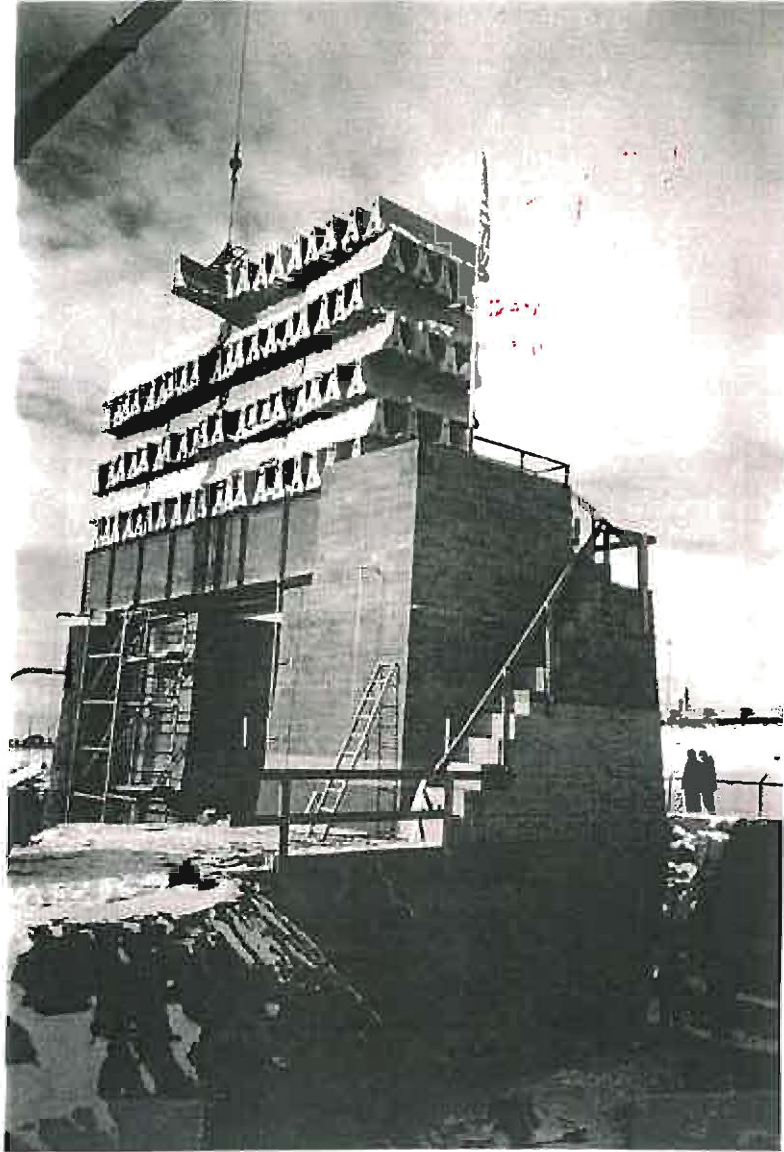Plate 12 A Clamp of a Forklift was Used to Lift a Jersey Barrier

Plate 13 Placement of Jersey Barriers

Chapter 5 LOADING SCHEME

There are two stages in the load test. The first stage was conducted on the outer

pier and the abutment. The second stage will be conducted on the center pier.

5.1 The First-Stage Load Test

Three steel bridge girders were placed over the top concrete pads of the outer pier

and the abutment. Each girder was supported by steel bearing plates resting on the concrete

pads. The steel bearing plates were located along the center line of the top concrete pad of

the outer pier, and a 0.3 m offset from the back face of the abutment facing blocks. The

span of the girders was 10.4 m. A total of 124 concrete blocks ("Jersey Barriers") was

placed on the girders in seven layers as shown in Figure 4. The total load was 2,340 kN

which corresponded to an applied pressure of 232 kPa on the outer pier. Note that such a

pressure is slightly higher than the 200 kPa maximum pressure suggested in the FHWA

Demo 82 (1996) for mechanically stabilized bridge abutments. Also note that the

maximum applied pressure of the Turner-Fairbank's test was 900 kPa which was about four

times higher than the suggested maximum pressure.

Fig. 4 First-Stage Load Test

5.2 The Second-Stage Load Test

The loading scheme has yet to be determined.

Chapter 6 INSTRUMENTATION

The focus of this project was on the second-stage load test on the center pier. Simple devices were used in the first-stage load test to obtain some quantitative measure of the lateral and vertical movements of the outer pier and the abutment.

6.1 Vertical Movement

A leveling rod was attached to a metal pipe, as shown in Figure 5, with one end fixed to either the top or the base concrete pads. The vertical movement of the concrete pads was measured by a precision survey transition of the leveling rod. Two fixed posts for survey transit were installed outside the test pit to the north and south of the outer pier and the abutment. The locations of the leveling rods are shown in Figure 6.

6.2 Lateral Movement

The term "lateral movement," unless otherwise specified, was referred to the total expansion along the perimeter of the structure. Elastic springs were wrapped around the circumference of the outer pier and three sides (north, west and south) of the abutment at selected heights. By measuring the elongation of the elastic spring, the lateral movement of the outer pier and the abutment was obtained. The heights at which the elastic springs were installed on the outer pier were 2.1 m, 4.6 m, and 6.6 m from the base. For the abutment, the elastic springs were located at 5.2 m, 6.0 m and 6.4 m from the base. The locations of
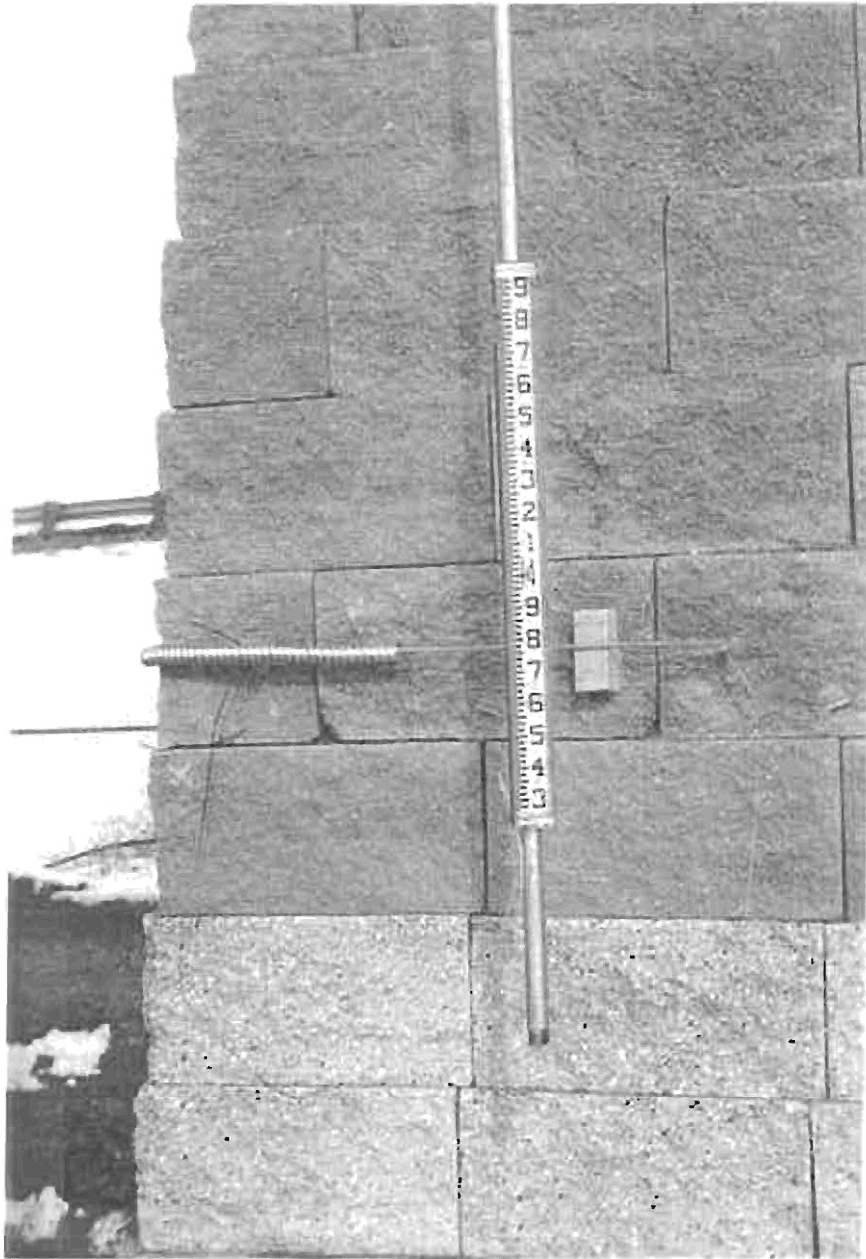
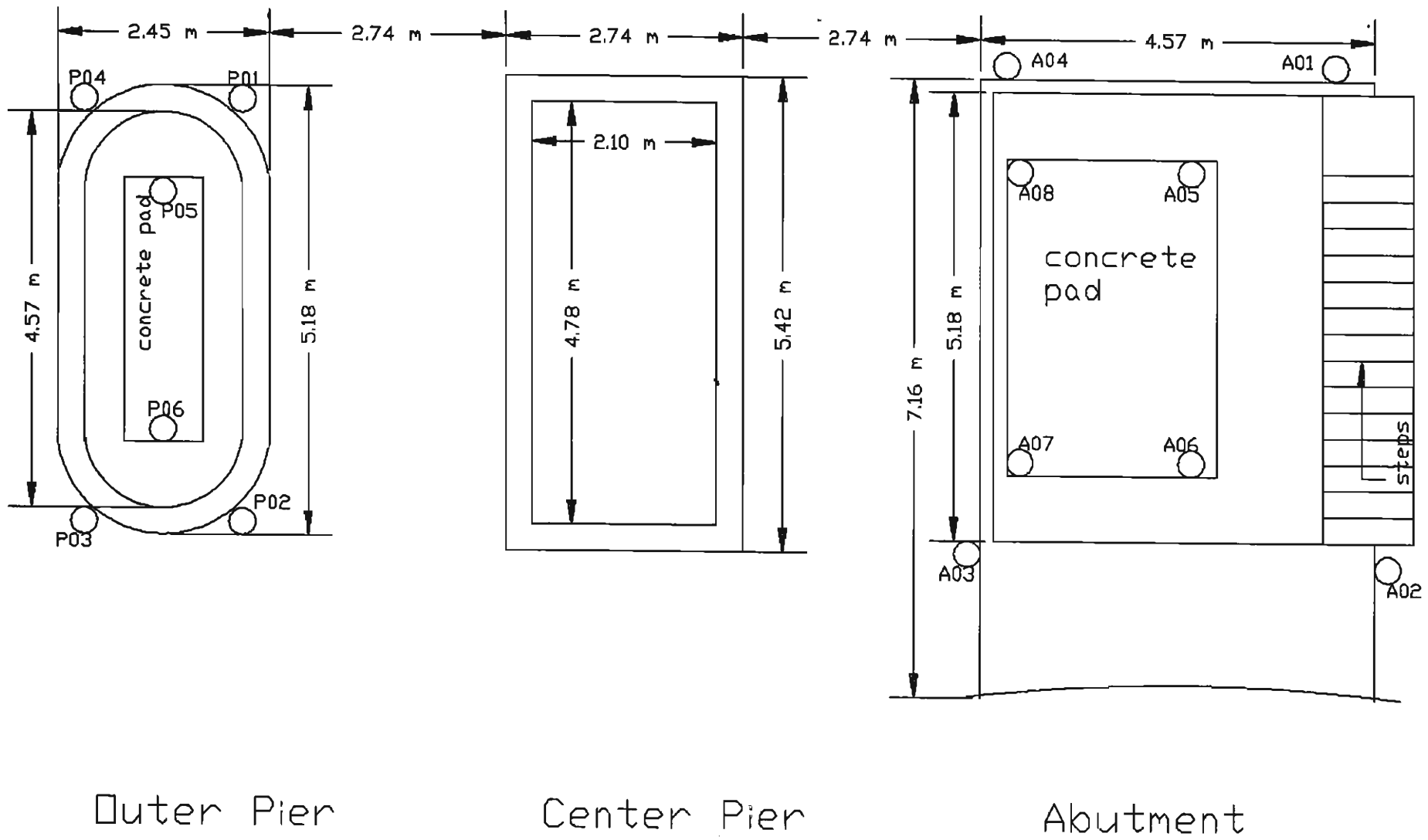Fig. 5 Instrumentation for Vertical and Lateral Movements

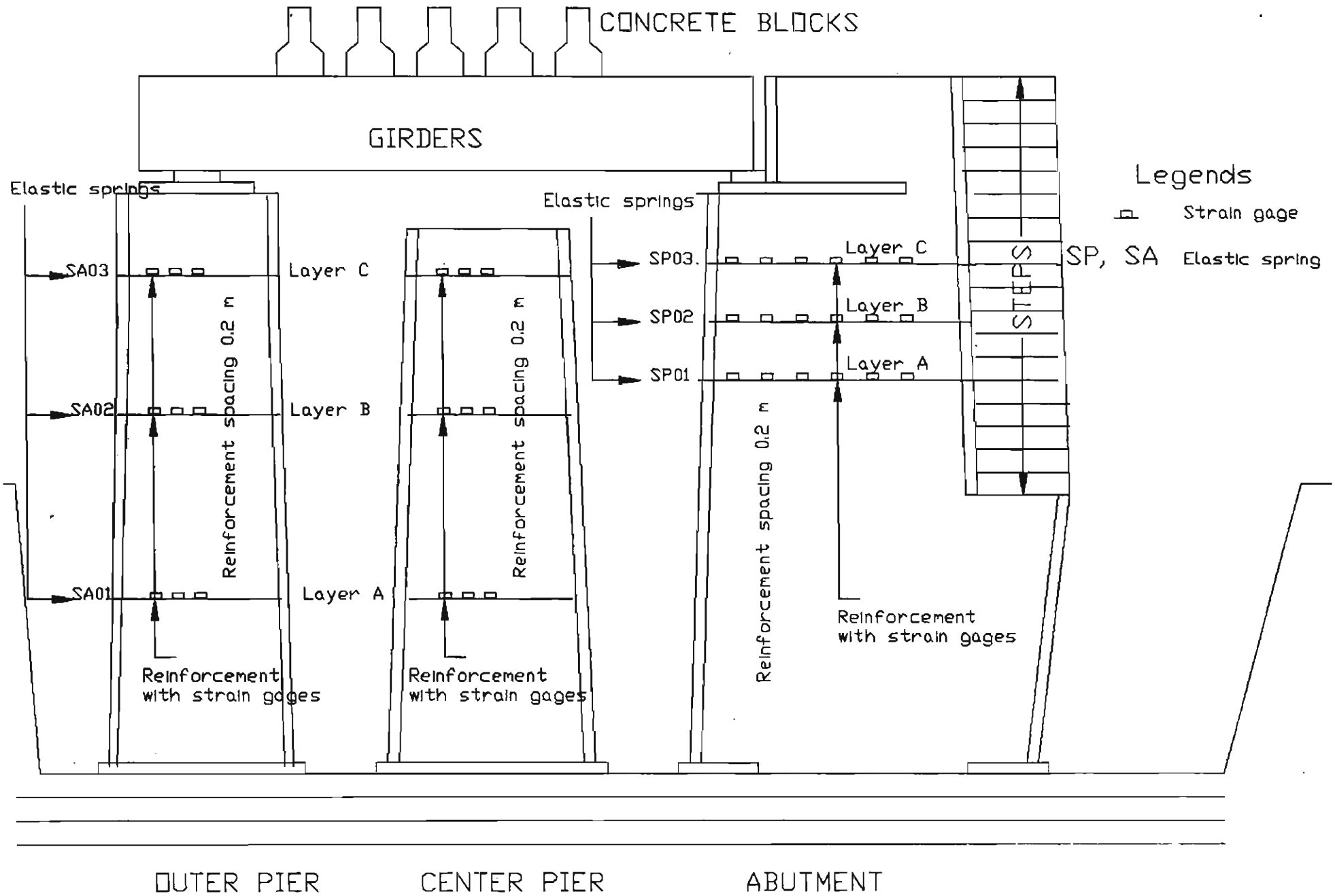**Fig. 6** Locations of the Leveling Rods

Fig. 7 Locations of the Reinforcements with Strain Gages and the Elastic Springs

the elastic springs (denoted as SP for the outer pier and SA for the abutment) are shown in Figure 7.

6.3 Strains in Reinforcement

High elongation strain gages, manufactured by Measurements Group, Inc. (type EP-08-250BG-120), were used to measure the strain distribution in the geotextile reinforcement. A total of six strain gages were mounted along the fill and the warp directions of each instrumented sheet of reinforcement for the outer and center piers. There were three sheets of instrumented geotextile reinforcement located at 2.0 m, 4.5 m, and 6.5 m from the base in the outer pier and 1.9 m, 4.3 m, and 6.5 m from the base in the center pier. The abutment had three sheets of instrumented geotextile reinforcement with six strain gages along the fill direction on each sheet. They were located at carrying 5.1 m, 5.9 m, and 6.5 m from the base. The locations of the reinforcement sheets with strain gages are shown in Figure 7.

Each strain gage was glued to the geotextile only at the two ends to avoid inconsistent local stiffening of the geotextile due to the adhesive. The strain gage attachment technique was developed at the Reinforced Soil Research Center of the University of Colorado at Denver. The gage was first mounted on a 25 mm by 76 mm patch of a light weight nonwoven geotextile. The light-weight geotextile patch (with a strain gage and wax) was then attached to the woven geotextile reinforcement at selected locations. A microcrystalline wax material was applied over the gage to protect it from soil moisture. Figure 8 shows a strain gage mounted on a light-weight nonwoven geotextile

patch and attached to the woven reinforcement.

Due to the presence of the light-weight geotextile patch, calibration is needed. Calibration tests were performed to relate the strain obtained from the attached strain gage to the actual strain of the reinforcement. The calibration curves along the fill and the warp directions of the woven geotextile reinforcement used in the load test are shown in Figure 9 and Figure 10, respectively.
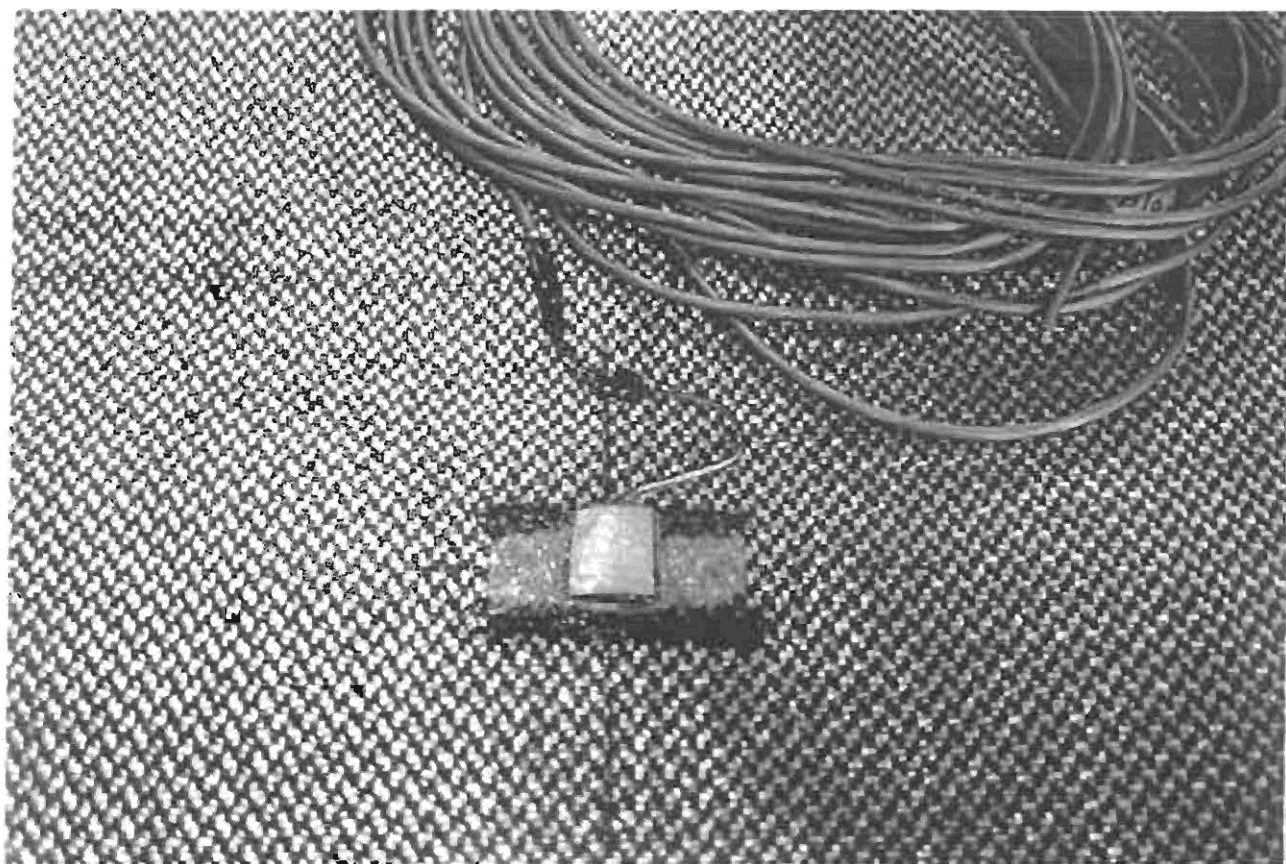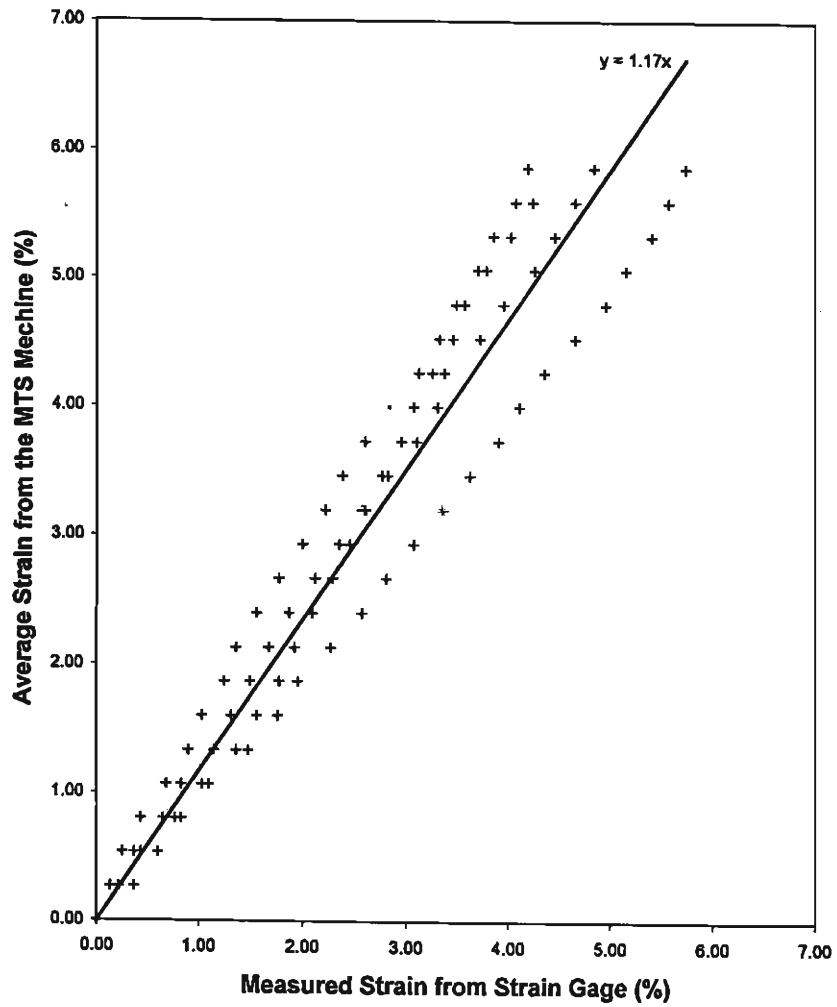
Fig. 8   Strain Gage Attachment

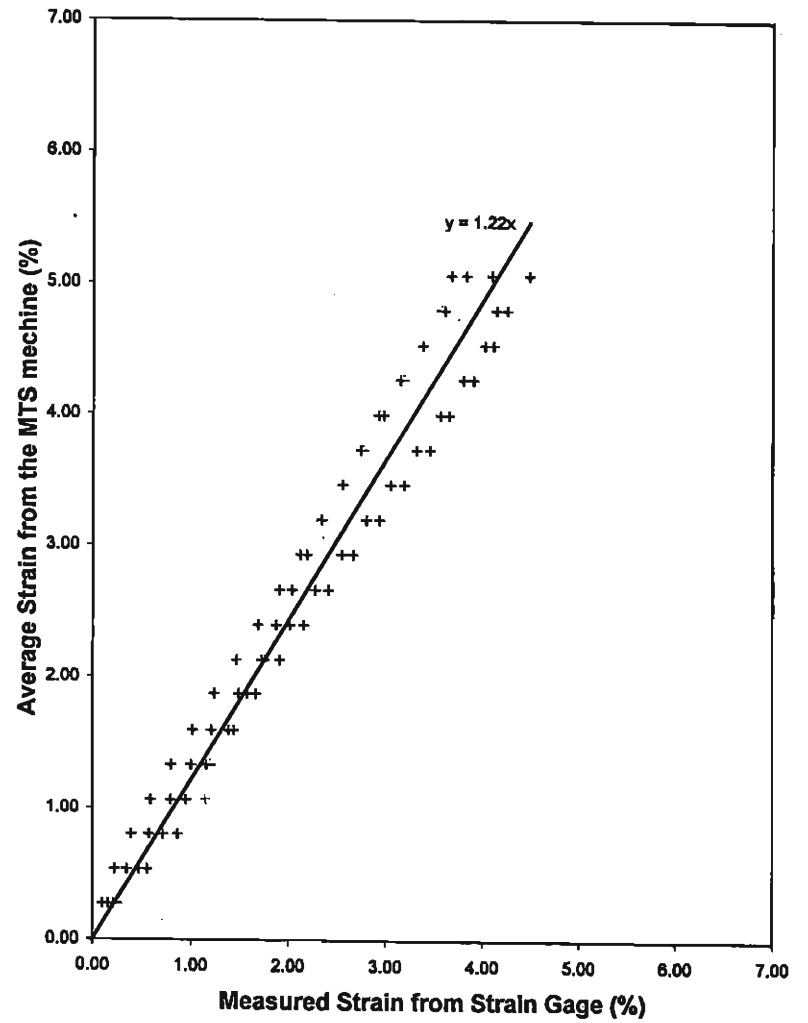**Fig. 9** Strain Gage Calibration in the Fill Direction of the Reinforcement

Fig. 10 Strain Gage Calibration in the Warp Direction of the Reinforcement

Chapter 7 RESULTS OF THE FIRST-STAGE LOAD TEST

7.1 Short-Term Behavior

The measured short-term vertical and lateral displacements of the abutment and the

pier, as well as the measured strains in the geotextile reinforcement soon after the load

application are presented in the following sections. In addition, discussions of the

measured results are presented.

7.1.1 Vertical and Lateral Displacements

Figure 11 and Figure 12 show the applied load versus displacement relationships of

the abutment in the vertical and lateral directions, respectively. The vertical displacements

were fairly uniform along the two axial directions. The maximum vertical displacements at

1,170 kN were 27.1 mm at the top and 5.2 mm at the base. The maximum lateral

movement at 5.2 m from the base was 14.3 mm.

Figure 13 and Figure 14 show the applied load versus displacement relationships of

the outer pier in the vertical and lateral directions, respectively. Similar to the abutment,

the vertical displacements were fairly uniform along the two axial directions. The

maximum vertical displacements at 1,170 kN (232 kPa pressure) were 36.6 mm at the top

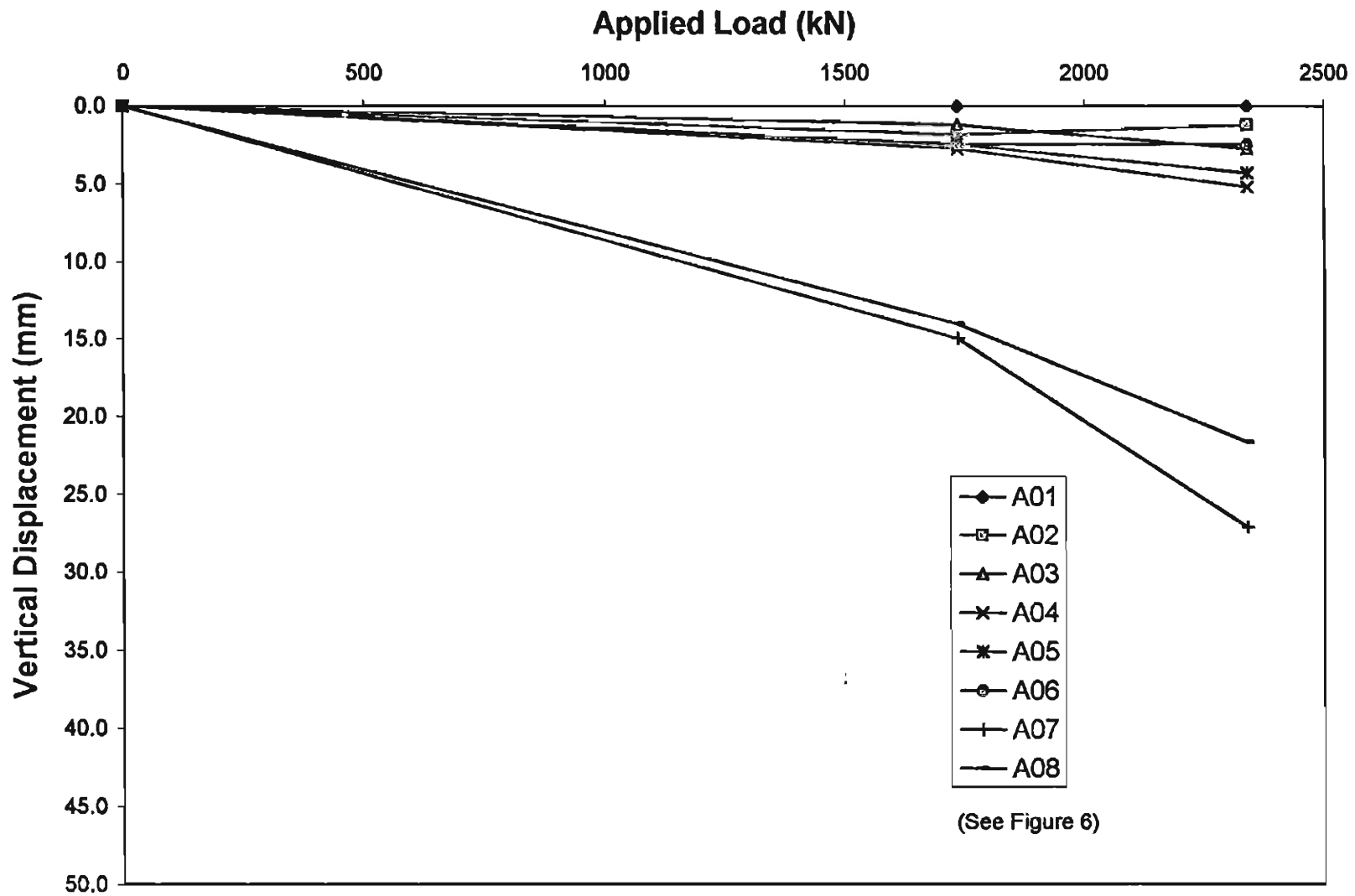and 6.1 mm at the base. The maximum lateral movement at 4.6 m from the base was

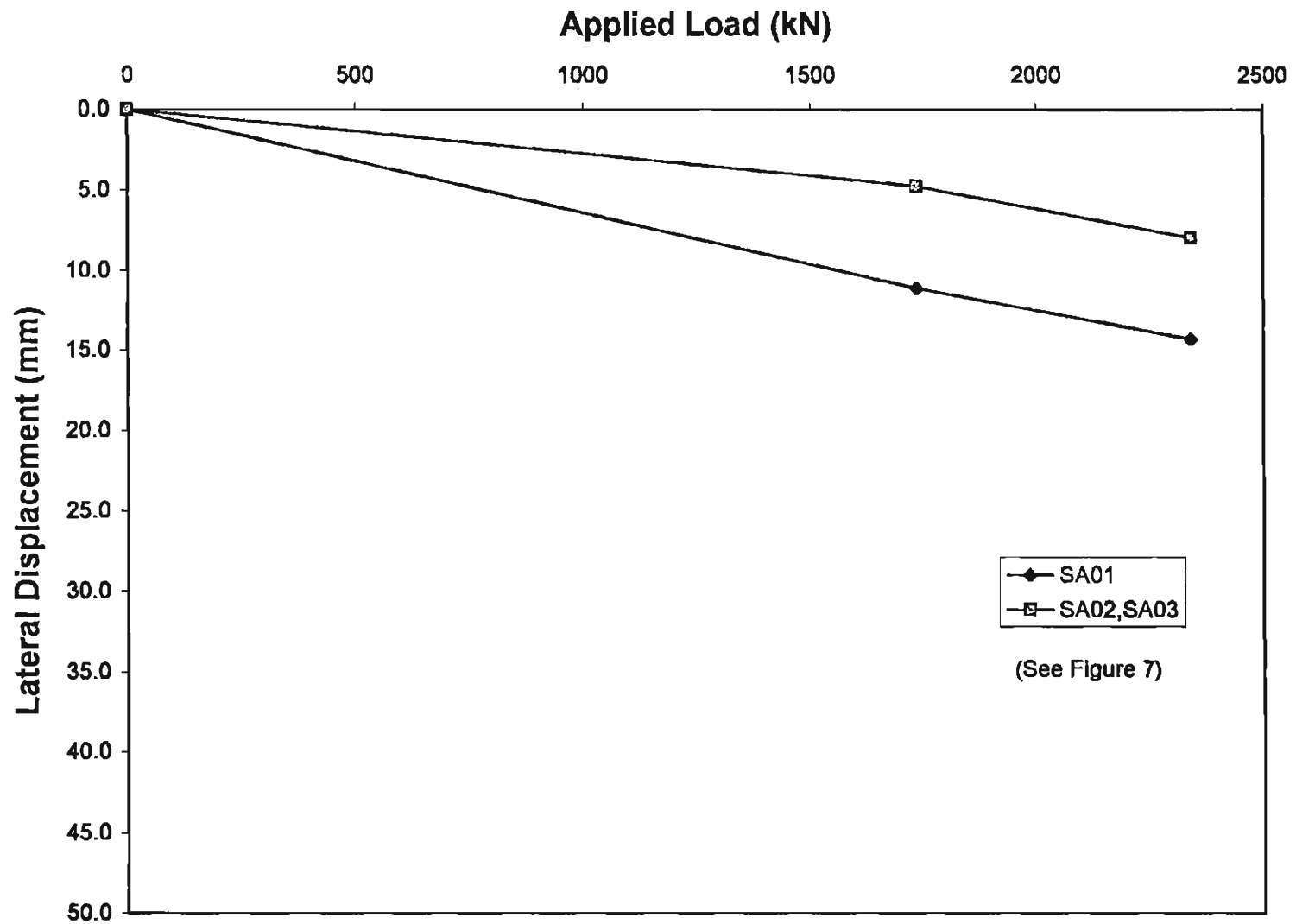**Fig. 11** Vertical Displacement versus Applied Load Relationships of the Abutment

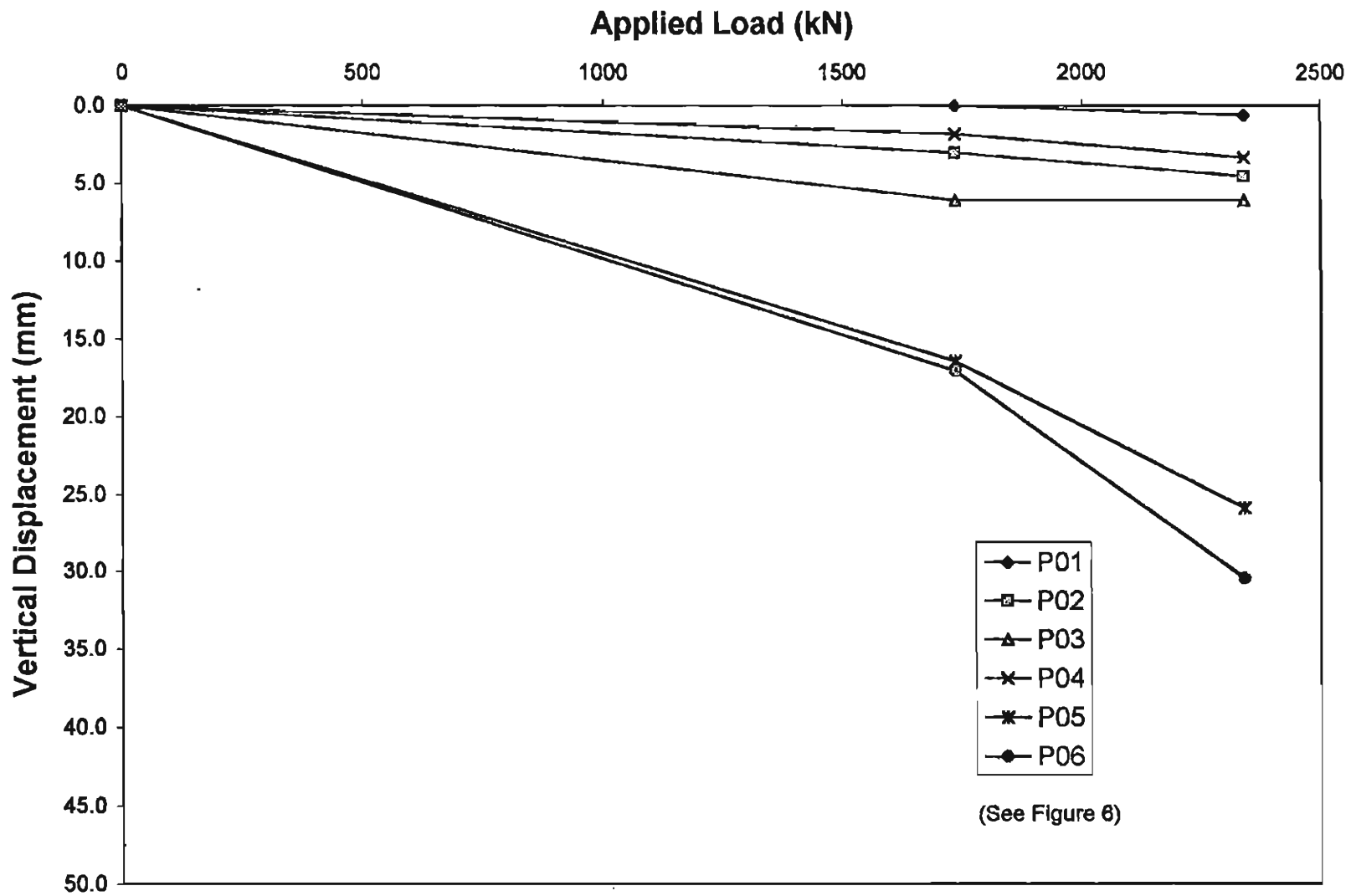**Fig. 12** Lateral Displacement versus Applied Load Relationships of the Abutment

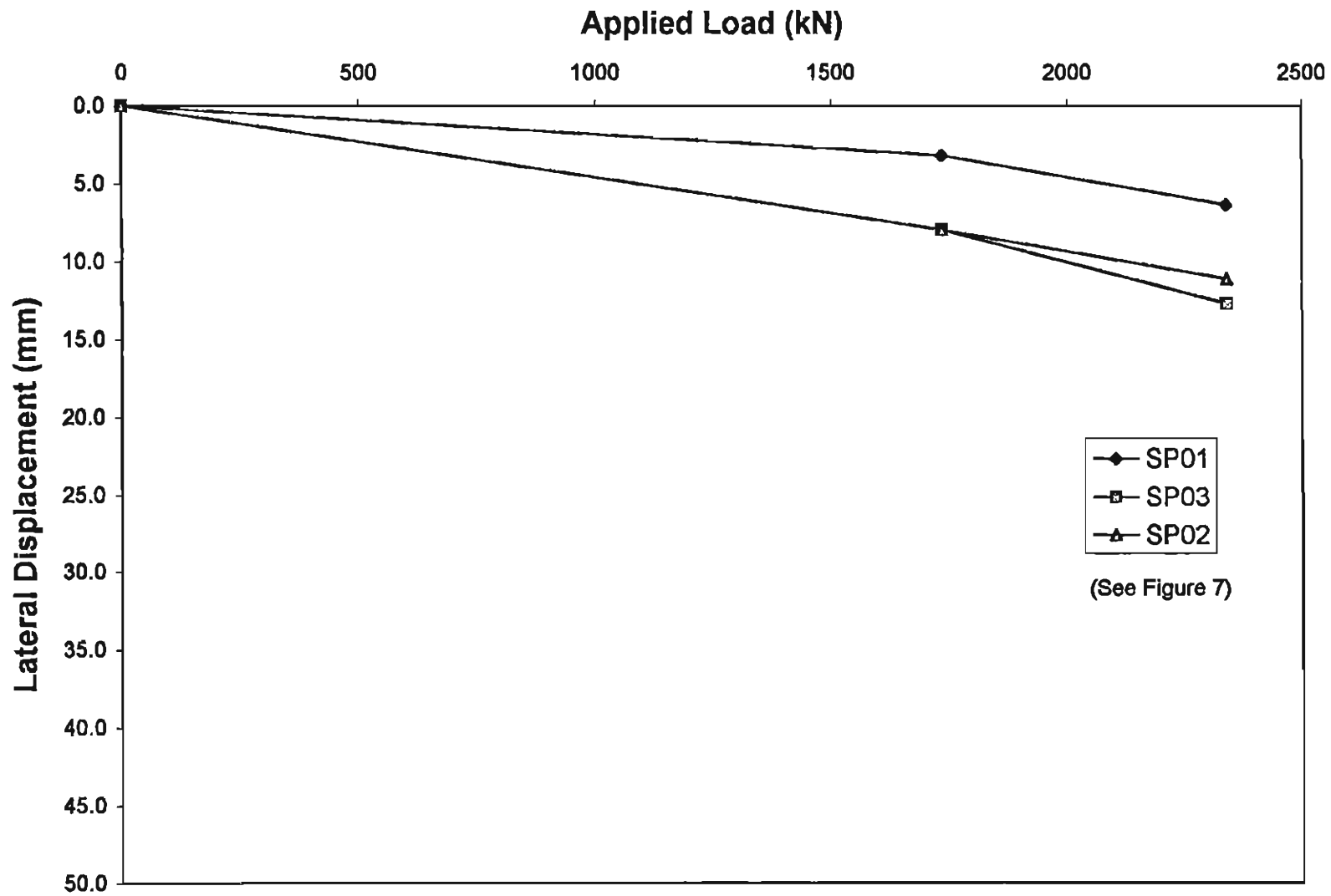**Fig. 13** Vertical Displacement versus Applied Load Relationships of the Outer Pier

Fig. 14 Lateral Displacement versus Applied Load Relationships of the Outer Pier

12.7 mm.

Figure 15 shows the vertical and lateral movements of the outer pier and the abutment at the applied load of 1,170 kN. Table 2 summarizes the maximum vertical and lateral movements of the outer pier and the abutment. It is shown that the vertical movement of the foundations for the outer pier and the abutment was about the same (6.1 mm for the pier, 5.2 mm for the abutment). The different magnitudes of the vertical movement on top of the pier and the abutment were, therefore, a result of the different amounts of vertical compression of the structures upon loading. The maximum vertical movement of the pier was 0.48% of its height. Such a value was higher than that of the Turner-Fairbank's load test which was 0.30% (without prestraining) at the same applied pressure. This may be attributed to the much lower compaction effort on the outer pier. The maximum lateral movements of the pier and the abutment were comparable (12.7 mm in the pier, 14.3 mm in the abutment).

The maximum differential settlements at the bases of the abutment and the pier were 5.2 mm and 5.5 mm, respectively. The 0.15-m thick concrete pads at the base and the geosynthetic-reinforced soil foundation appeared to be a competent platform for the abutment and the pier.

From the applied load versus displacement relationships of the abutment and the pier (Figures 11, 12, 13, and 14), the bearing capacity of the structures can be determined based on the tolerable vertical and lateral deformation upon loading. For example, the
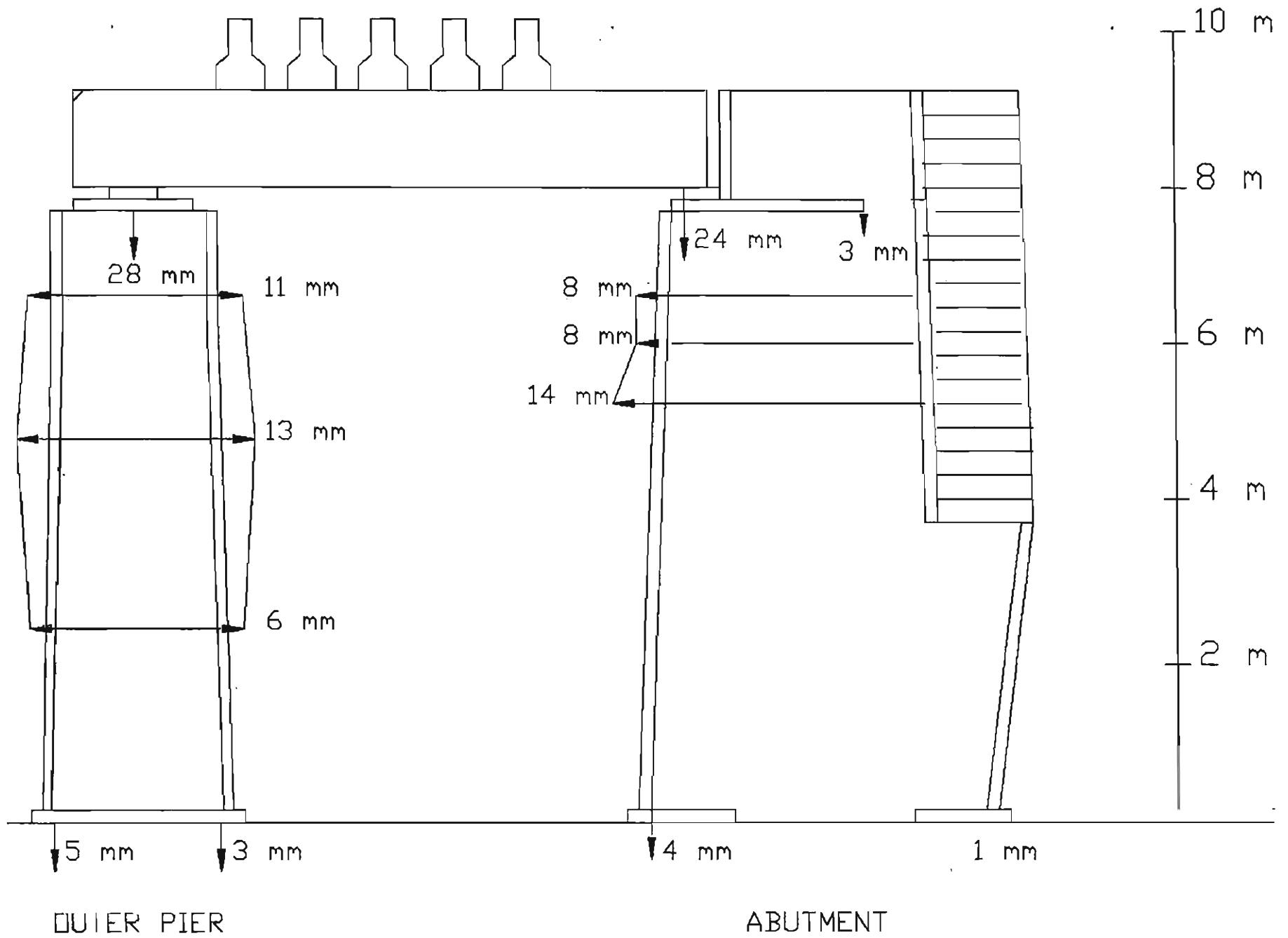
Fig. 15 Vertical and Lateral Movements of the Outer Pier and the Abutment at 1,170 kN

Structure	Maximum Vertical Movement (mm)			Maximum Lateral Movement (mm)
	Foundation	Structure	Total	
Outer Pier	6.1 mm	30.5 mm	36.6 mm	12.7 mm
Abutment	5.2 mm	21.9 mm	27.1 mm	14.3 mm

Table 2 Maximum Vertical and Lateral Movements of the Outer Pier and the Abutment at 1,170 kN

bearing capacity of the abutment at the tolerable vertical settlement of 12.7 mm (0.5 in.) is limited to 146 kPa (see Figure 11).

After the load of 1,170 kN was applied, predominantly vertical hairline cracks in the facing blocks were observed along the length and the width directions of the outer pier and the face of the abutment. It was not clear when was the first crack developed. With time, however, the cracks increased both in the number and in the width. It is recommended the geosynthetic reinforcement be wrapped around at the facing as in the last four layer of reinforcement of the abutment to prevent losing of the backfill through the cracks.
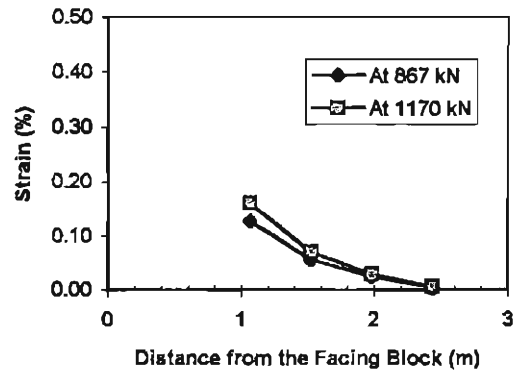
7.1.2 Strains in Reinforcement

Figure 16 shows the strain distributions in the fill direction along the length of the three instrumented sheets of reinforcement in the abutment. The largest strains were on the order of 0.15% to 0.18% at the applied load of 1,170 kN. The largest strains occurred adjacent to the facing (the center of bearing plates was only 0.3 m from the back face of the facing blocks) and decreased toward the other end. The strains were nearly zero at 2.5 m from the facing.

The reinforcement strains in the fill direction of the outer pier were on the order of 0.2% to 0.4% at the applied load of 1,170 kN (232 kPa pressure). Note that such a magnitude of reinforcement strain was similar to that measured in the Turner-Fairbank's load test at the same applied pressure. This implies that at the same applied pressure the lateral displacement of the outer pier and the Turner-Fairbank pier was comparable.

**Reinforcement Strain Distribution of
the Abutment, Layer A**
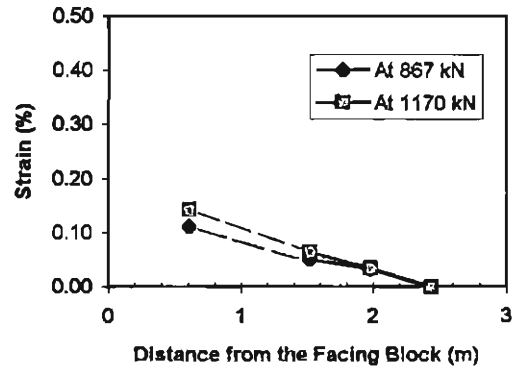
**Reinforcement Strain Distribution of
the Abutment, Layer B**

**Reinforcement Strain Distribution of
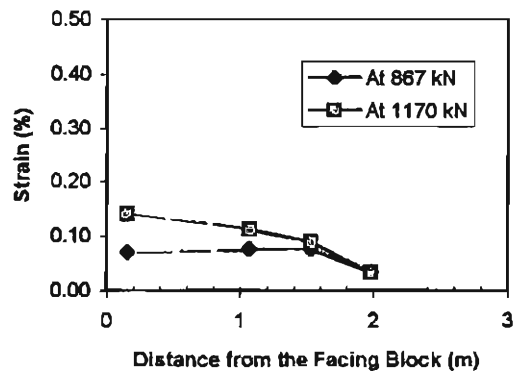the Abutment, Layer C**

Fig. 16 Reinforcement Strain Distributions in the Abutment

Compared the largest strains in the abutment and the outer pier (on the order of 0.2% to 0.4%) to the rupture strain (18%), the safety margin appeared to be very high against rupture failure of the reinforcement. However, the load carrying capacities of both structures may not be governed by the rupture failure of the reinforcement. Slippage between the backfill and the reinforcement or the shear failure of the backfill may occur first.

7.2 Long-Term Behavior under 1,170 kN Load

Time dependent behavior of the abutment and pier -- including vertical displacements, lateral displacements, and reinforcement strains -- under a sustained load of 1,170 kN is presented in the following sections. Discussion of the measured results are also presented.

7.2.1 Vertical and Lateral Creep Displacements

Figure 17 and Figure 18 show, respectively, the vertical and lateral displacements versus time relationships of the abutment under the sustained load of 1,170 kN. The maximum vertical displacements at the top and the base after 70 days were 18.3 mm and 6.7 mm, respectively. The maximum lateral displacement was 14.3 mm after 70 days. Most of the maximum vertical and lateral displacements (12 mm and 13 mm, respectively) occurred in the first 15 days.
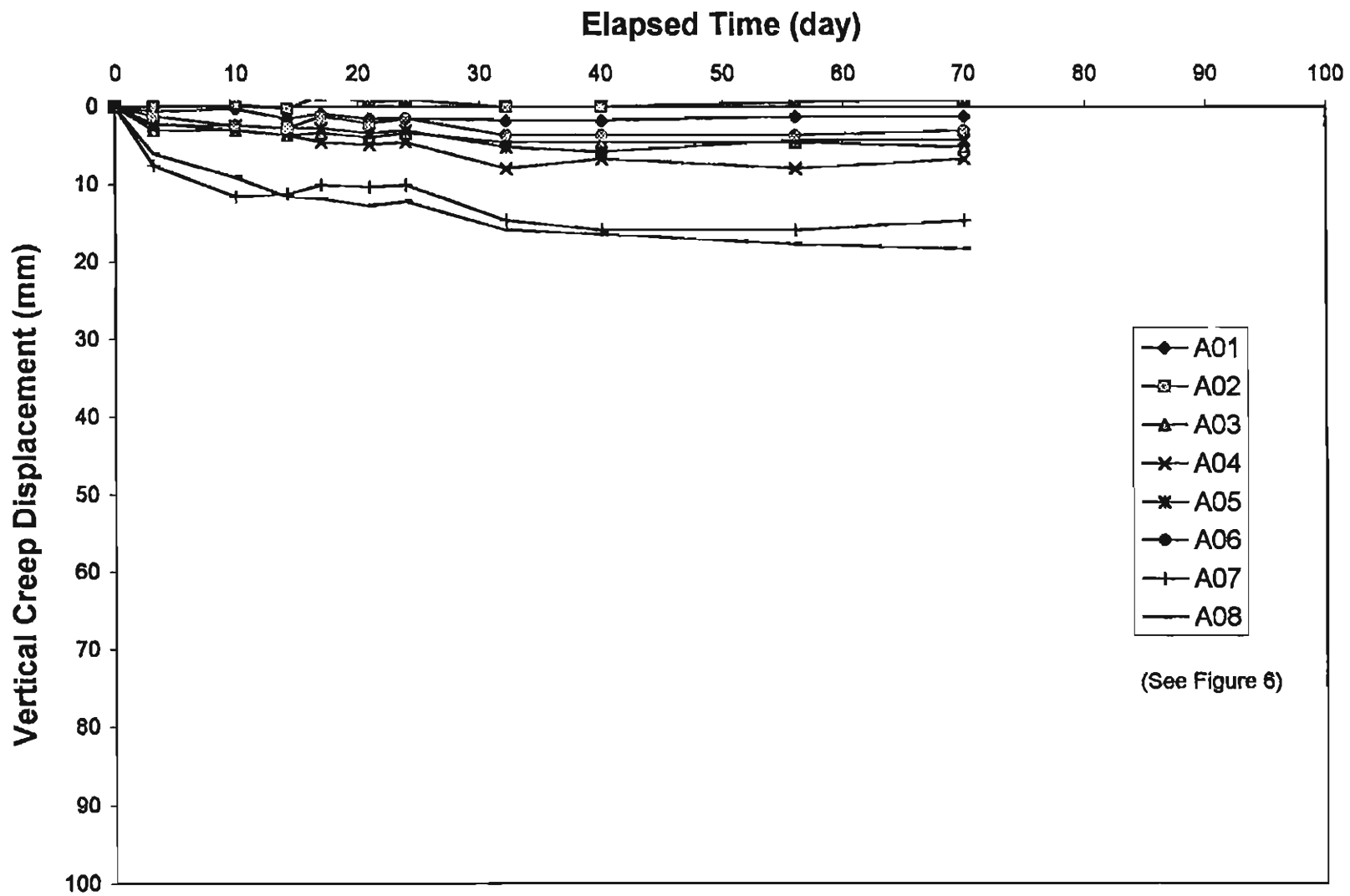
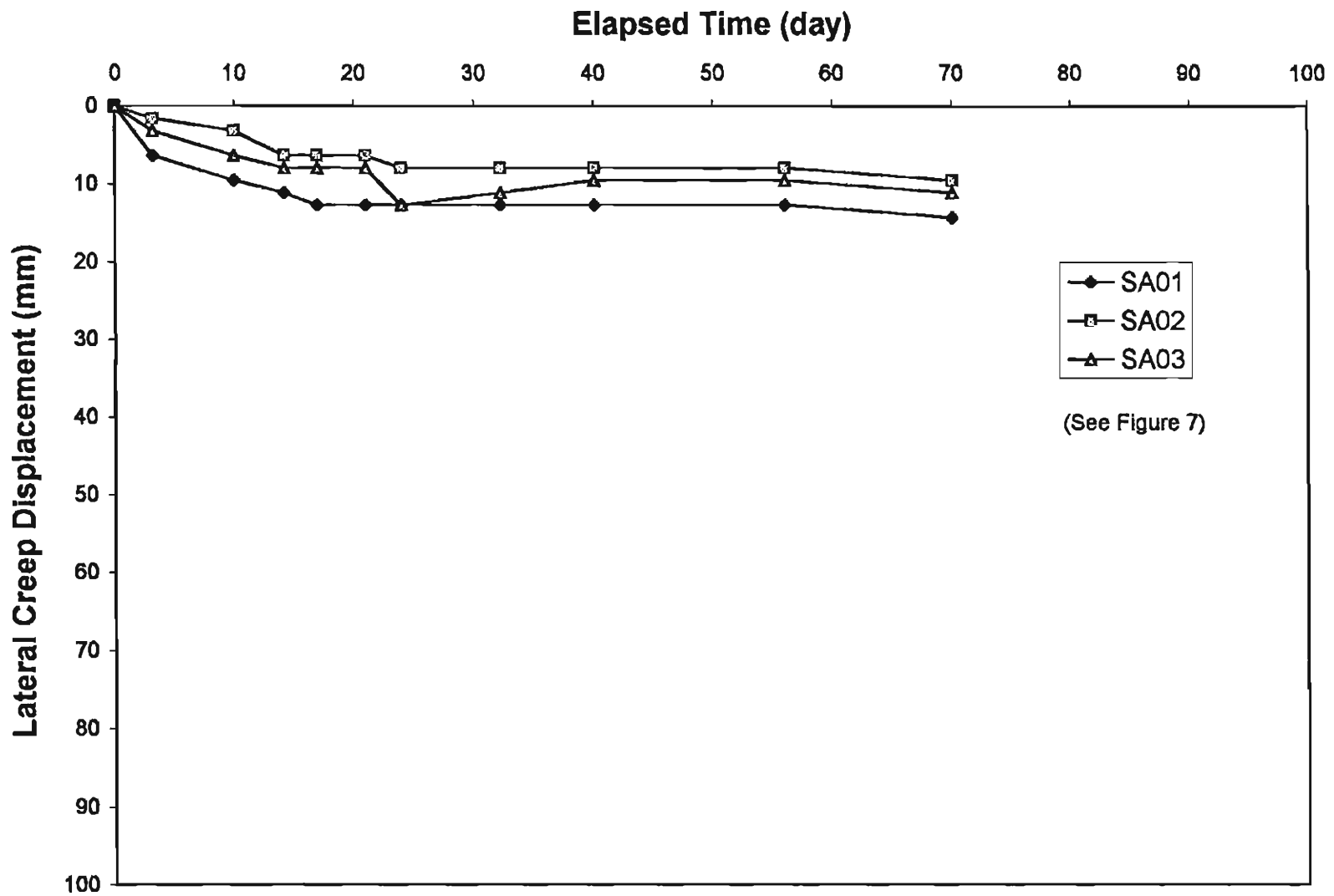Fig. 17 Vertical Creep Displacement versus Time Relationships of the Abutment

Fig. 18 Lateral Creep Displacement versus Time Relationships of the Abutment

Figure 19 and Figure 20 show, respectively, the vertical and lateral displacements versus time relationships of the outer pier under the sustained load of 1,170 kN. The maximum vertical displacements at the top and the base after 70 days were 61.6 mm and 5.2 mm, respectively. The maximum lateral displacement was 59.5 mm after 70 days. Similar to the abutment, a large portion of the maximum vertical and lateral displacements (48 mm and 46 mm, respectively) also occurred in the first 15 days. The maximum vertical and lateral displacements of the outer pier were about four times as large as those of the abutment. This is most likely due to poor compaction by a light weight vibrating plate used during construction.

Figure 21 shows the average vertical creep rate of the top loading pad versus time relationships of the outer pier and the abutment plotted on a log-log scale. It is shown that, for the most part, the vertical creep rate of both structures reduced nearly linearly (on a log-log scale) with time. The vertical creep rates in the abutment reduced from 2.2 mm/day after 3 days to 0.03 mm/day after 70 days in the abutment. During the same period of time, the creep rate reduced from 7.5 mm/day to 0.1 mm/day in the outer pier. At around 25 days, both creep rates of the pier and the abutment significantly increased as shown in Figure 21. This behavior is attributed to softening of the frozen backfill due to a temperature increase following an extended period of freezing temperatures. Extrapolations of the average vertical creep rates were drawn as the shaded areas in Figure 21. The extrapolation may be used to obtain approximate creep rates beyond the measurement period. For instance, after a year, the vertical creep rates of the abutment and the outer pier were in the ranges of 0.003 mm/day to 0.008 mm/day and 0.012 mm/day to
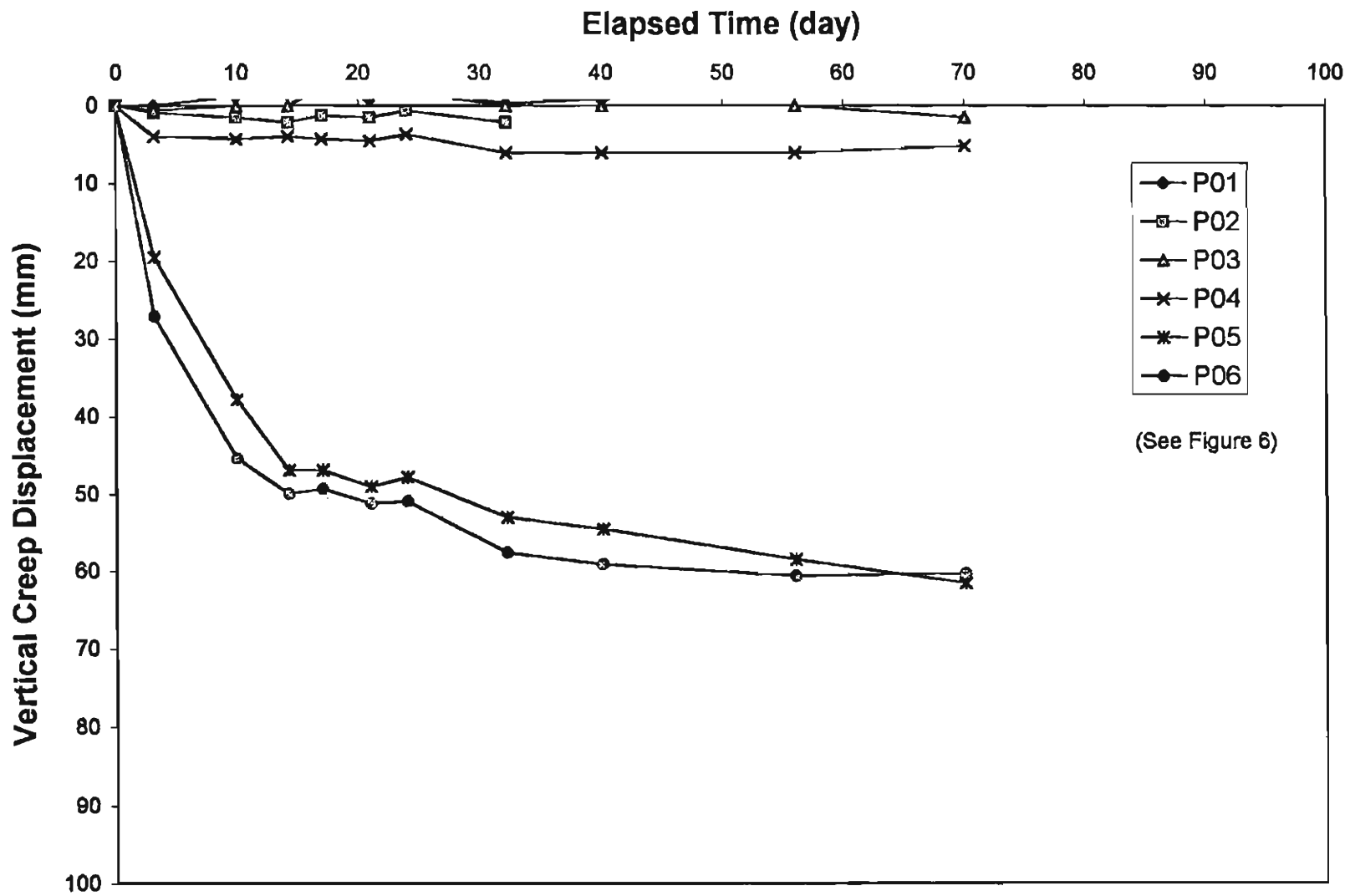
Fig. 19 Vertical Creep Displacement versus Time Relationships of the Outer Pier
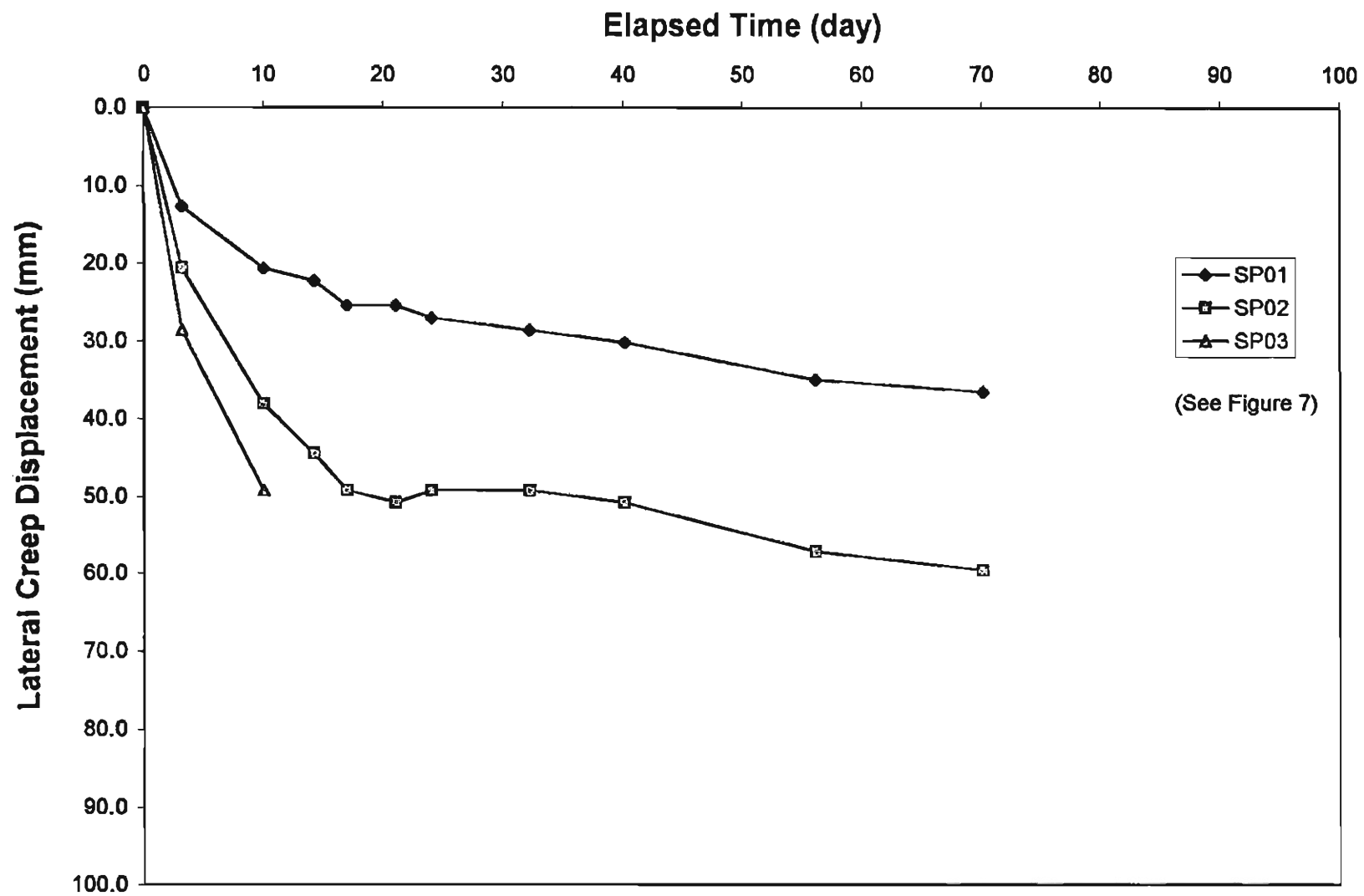
Fig. 20 Lateral Creep Displacement versus Time Relationships of the Outer Pier
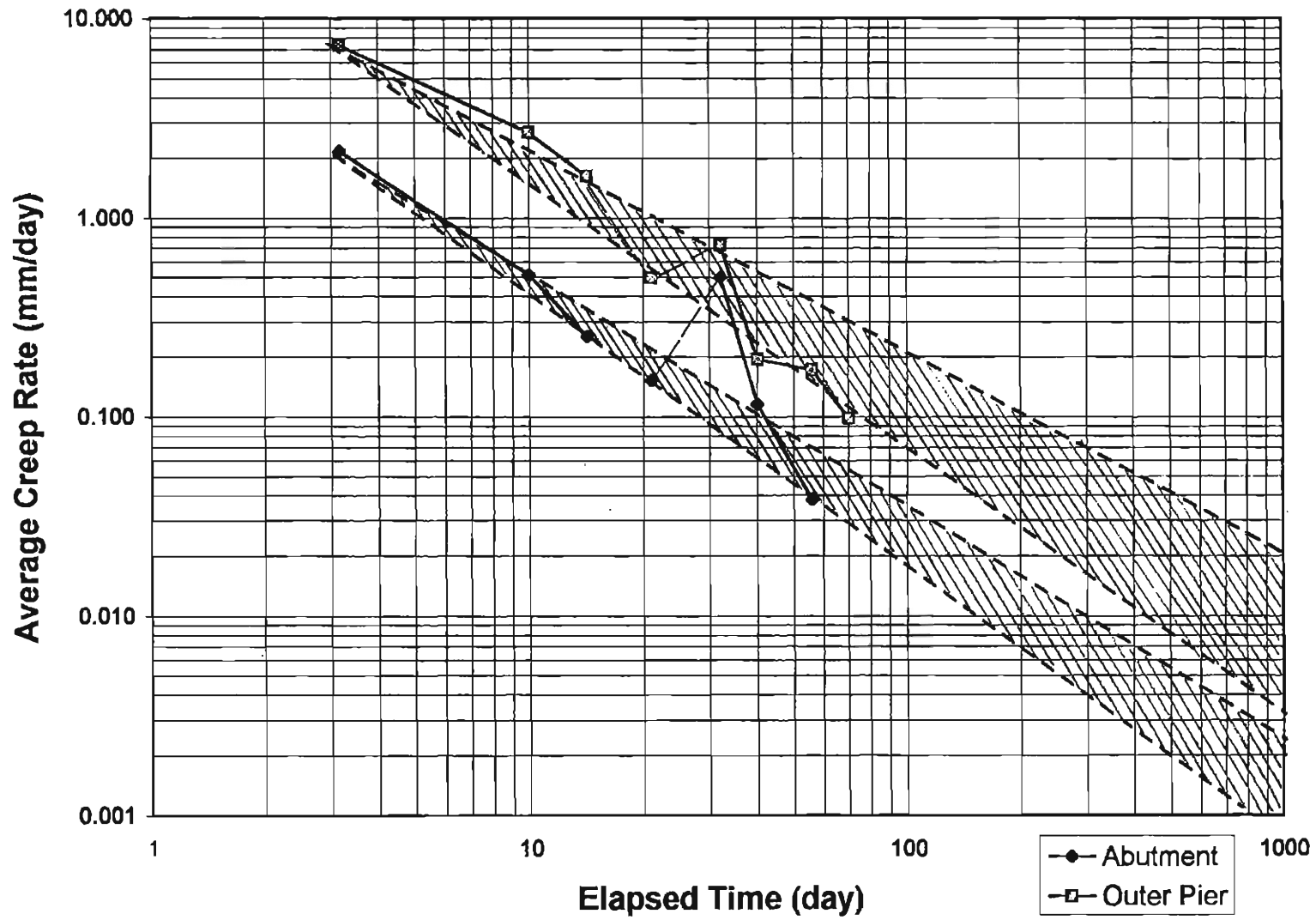
Fig. 21 Average Vertical Creep Rate at Top of the Outer Pier and the Abutment

0.06 mm/day, respectively.

7.2.2 Creep Strains in Reinforcement

Figure 22 shows the reinforcement creep strain distributions in layers A, B, and C (see Figure 7) of the abutment after 10, 25, and 70 days. The creep strain distributions were somewhat more uniform than the short-term reinforcement strain distribution. After 70 days, the maximum creep strains in layers A, B, and C were 0.30%, 0.75% and 0.38%, respectively. Note that such maximum creep strains were also far from the rupture strain (18%) of the reinforcement.

Figure 23 shows the reinforcement creep strain distributions in layer A (both the warp and fill directions) and layer C (the warp direction) of the outer pier after 10, 25, and 70 days. A uniform strain distribution was assumed in the reinforcement of the outer pier. The average strain of each layer was shown in Figure 23. After 70 days the average creep strains of layer A were 0.20% and 0.46% in the fill and warp directions, respectively, and 0.53% in the warp direction of the layer C. The higher creep strains in the warp direction was mainly due to the fact that the geotextile is nearly twice as likely to creep in its warp direction (Ketchart and Wu, 1996).

From the average strains in layer A of the outer pier, the lateral creep displacements were calculated and compared to the measured lateral creep displacements. The comparison is shown in Figure 24. It is seen that the calculated lateral creep displacements

(from the strain distributions) were in very good agreement with the measured displacements.
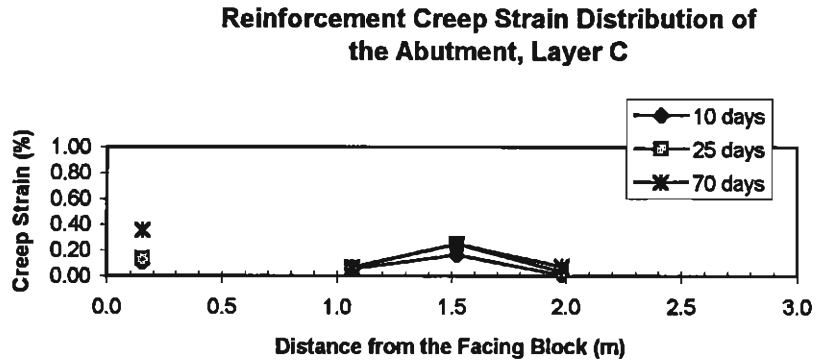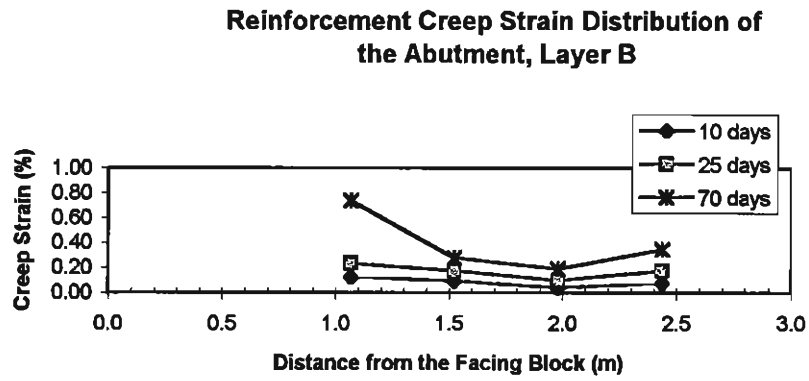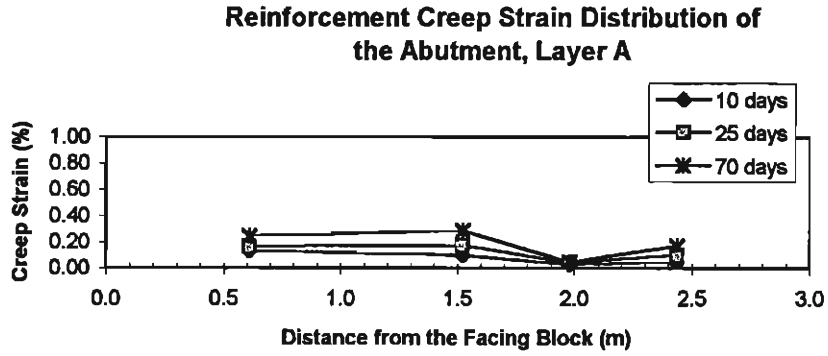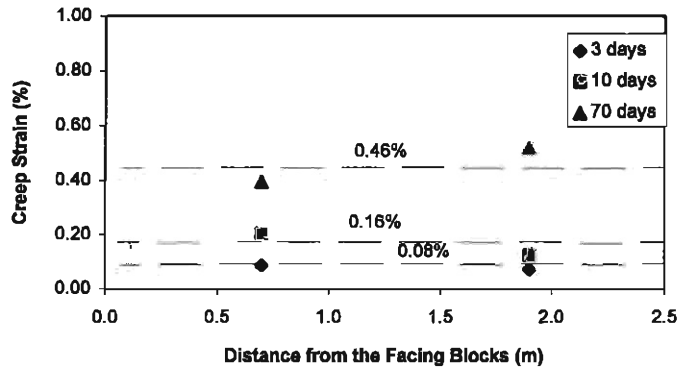
**Reinforcement Creep Strain Distribution of
the Abutment, Layer A**

**Reinforcement Creep Strain Distribution of
the Abutment, Layer B**

**Reinforcement Creep Strain Distribution of
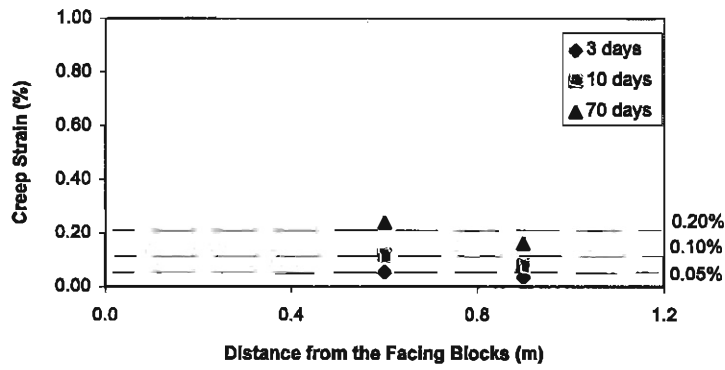the Abutment, Layer C**

Fig. 22 Reinforcement Creep Strain Distributions in the Abutment

**Reinforcement Creep Strain Distributions of
the Outer Pier, Layer A, Warp Direction**

**Reinforcement Creep Strain Distributions of
the Outer Pier, Layer A, Fill Direction**

**Reinforcement Creep Strain Distributions of
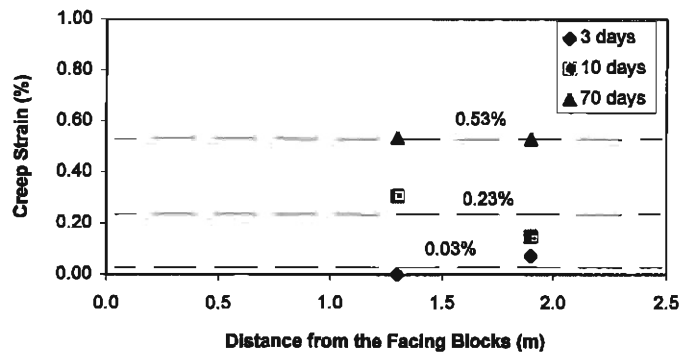the Outer Pier, Layer C, Warp Direction**

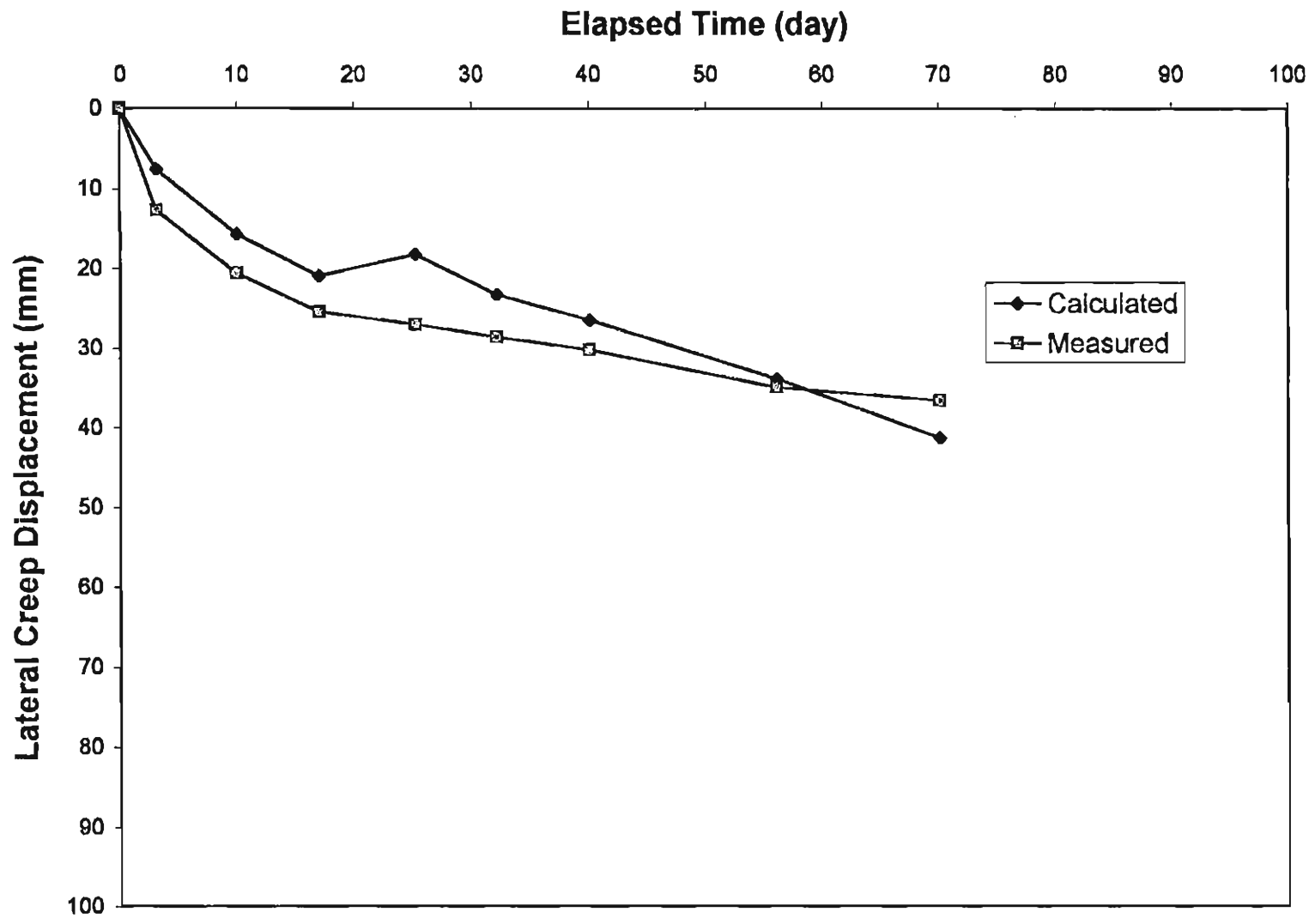**Fig. 23** Reinforcement Creep Strain Distributions in the Outer Pier

Fig. 24 Calculated versus Measured Lateral Creep Displacements of the Outer Pier

Chpater 8 SUMMARY AND CONCLUSIONS

A GRS bridge abutment and two GRS bridge piers were constructed inside a 3.5-m deep pit. The structures were constructed with a "road base" backfill reinforced with layers of a woven geotextile. Hollow-cored concrete blocks were used as facing. One of the piers (i.e. the outer pier) and the abutment, both 7.6 m in height, were load tested. The load was applied using concrete barriers stacked in seven layers over three steel bridge girders. A total load of 2,340 kN, corresponding to 232 kPa pressure, was applied. The pier and the abutment were instrumented with metal pipes and elastic springs to monitor the vertical and lateral movement of the facing, and strain gages to monitor deformation of the reinforcement. The findings are summarized as follows:

1. Construction of the GRS pier and abutment is indeed rapid and simple.

2. Load carrying capacities of the pier and the abutment were higher than the 200 kPa maximum pressure suggested by the FHWA Demo 82 (1996).

3. The displacements at 1,170 kN of the pier and the abutment were comparable. The maximum vertical displacement was slightly higher in the outer pier than in the abutment. The maximum vertical displacements were 27.1 mm in the abutment and 36.6 mm in the outer pier, corresponding, respectively, to 0.35% and 0.48% of the structure height. The maximum lateral displacement in the abutment was somewhat higher than that in the outer pier. The maximum lateral elongation of the perimeter were 4.3 mm in the abutment and 12.7 mm in the outer pier.

4. The ratio of the vertical movement to the structure height at 232 kPa of the outer pier (0.48%) was higher than that of the Turner-Fairbank pier (0.30%) at the same applied pressure. This may be attributed to the much lower compaction effort on the outer pier. The reinforcement strains in the fill direction of the outer pier and the Turner-Fairbank pier, however, were on the same order of magnitude (0.2% to 0.4%). This implies that the lateral movements of both piers are comparable.

5. Under a sustained load of 1,170 kN for 70 days, the creep displacements in both vertical and lateral directions of the outer pier were about four times larger than those in the abutment, due to lower compaction effort of the outer pier. The maximum vertical creep displacement was 61.6 mm in the outer pier, and 18.3 mm in the abutment. The maximum lateral creep displacement was 59.5 mm in the outer pier and 14.3 mm in the abutment.

6. A significant portion of the maximum vertical and lateral creep displacements of the pier and the abutment occurred in the first 15 days. At 15 days, the maximum vertical and lateral creep displacements were about 70% to 75% of the creep displacements at 70 days in respective directions.

7. Creep deformation of the structures decreased with time. The vertical creeprates reduced nearly linearly (on log-log scale) with time. The creep rates of the outer pier (7.5 mm/day after 3 days and 0.1 mm/day after 70 days) were higher than those of the abutment (2.2 mm/day after 3 days and 0.03 mm/day after 70 days).

8. Hairline cracks of the facing blocks occurred in the outer pier and the abutment due to the lateral bulging and the downdrag force due to the friction between the backfill and the facing blocks. This may be alleviated by providing flexible cushions between

vertically adjacent blocks.

9.   The maximum strains in the reinforcement were less than 1.0%. Compared to the rupture strain of the reinforcement of 18%, the safety margin against rupture of reinforcement appeared to be very high.

10.   The calculated lateral displacements from the reinforcement strain distribution were in very good agreement with the measured lateral displacements.

11.   With the less stringent construction condition (using a light-weight vibrating compaction plate), the outer pier showed about 1.5 times larger vertical displacement-to- height ratio than the Turner-Fairbank pier; whereas the lateral displacements were similar.

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Ketchart, K. and Wu, J.T.H. (1996), "Long-Term Performance Tests of Soil-Geosynthetic Composites," Report No. CDOT-CTI-96-1, Colorado Department of Transportation.

ACKNOWLEDGMENTS