SOIL NAILING
OF A
BRIDGE EMBANKMENT

Report 2:
DESIGN AND FIELD PERFORMANCE REPORT

Experimental Features
Project OR 89-07

By

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ABSTRACT

Soil nailing has recently been introduced in Oregon as an alternative lateral earth support method. The first permanent soil nail wall on the state's highway system was used where an underpass was widened under the existing Oregon Slough Bridge in Portland, Oregon to provide for additional traveling lanes. The project required removal of the existing south end slope and the construction of a soil nail wall in front of the pile-supported end bent to permanently retain the existing bridge fill embankment. Construction and post-construction monitoring was performed to study the new wall's performance.

This report describes the design and the performance of the Interstate-5 soil nail wall. The instrumentation program implemented during the construction of the wall is discussed in detail. The instrumentation data at two vertical cross sections is presented and data interpretation is discussed. The performance predicted by the original design methodology is compared critically to the measured.

Based on the results of our study, it may be concluded that: a) the Interstate-5 Swift-Delta soil nail wall is performing well within structural safety limits for both the wall and the bridge abutment, b) tensile forces are maximum inside the soil nailed earth mass at some distance from the facing, c) a relative movement in the range of 1/8 to 1/4 inch (3.18 M M to 6.34 M M)is necessary to mobilize the tensile capacity of the soil nails, d) the Davis method overestimates the nail forces in the lower nails and underestimates the nail forces in the upper nails, and e) Terzaghi and Peck's braced cut empirical earth pressure diagram appears to be in reasonable agreement with measured loads to date.

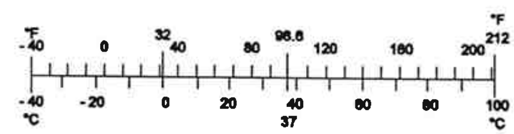
SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celsius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8 + 32	Fahrenheit	°F



* SI is the symbol for the International System of Measurement

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DISCLAIMER

SOIL NAILING OF A BRIDGE EMBANKMENT
DESIGN AND FIELD PERFORMANCE

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1.0 INTRODUCTION

1.1 BACKGROUND

Soil nailing, as an alternative lateral earth support system, has been used extensively in Europe since the early 1970's to stabilize soil cut slopes and to support temporary and permanent vertical soil cuts. The first soil nailing application was linked to the development of the New Austrian Tunneling Method which considers the ground as a carrying, rather than a "to be carried" element, when properly assisted or reinforced. Soil nailing was first used in the United States during the temporary excavation for the foundation of the Good Samaritan Hospital Expansion in Portland, Oregon in 1976.

Research carried out in the early 1980s led to a better understanding of the mechanisms of this technique. Since then it has been successfully used in the United States to support several temporary and permanent vertical earth cuts and, most recently, to stabilize high slopes. In 1985, the first highway application of a soil nail wall was used to temporarily support cuts up to 40-feet (12 m) high on the Federal Highway Administration's (FHWA) Cumberland Gap Tunnel project in Kentucky.

This embankment-support method consists of placing passive (unstressed) steel bars in the soil to improve its shear strength by limiting decompression and dilation immediately after excavation. The reinforced soil body, becomes the prime structural element and performs as a homogenous unit to support the unreinforced soil behind it, in a manner similar to a gravity wall.

Soil nailing is used in cut retention applications and consists of staged excavation from the top-down. The soil is reinforced with passive steel bars (the soil nail) placed in drilled bore holes which are then grouted along their total length. The soil nails are installed in a lift-by-lift sequence as the excavation progresses. The nails are spaced so the material between them arches to form a reinforced earth block. During construction the outside facing typically consists of a thin layer of shotcrete with wire mesh reinforcing. This acts to prevent relaxation or sloughing of the ground at the wall face. For permanent walls, a final permanent facing, consisting of either cast-in-place concrete or additional shotcrete, is placed over the initial construction facing.

Oregon's Interstate-5 Swift-Delta project (road widening under an existing bridge) required a top-down staged excavation. A conventional bottom-up wall would have required expensive temporary shoring to retain the existing bridge end embankment.

A top down staged excavation can also be accomplished by using a conventional tied-back wall. The soil nailing technique was chosen for the following reasons:

1. Soil nailing, unlike tied-back walls, requires no soldier pile installation; therefore, holes do not have to be made through the existing bridge deck to drive the pilings, and bridge traffic is not disrupted.

2. A tied-back wall face must be designed to resist full design earth pressure. A soil nail wall face is designed for significantly less than full earth pressure. It prevents local failure of the soil between the nails and is not required to provide the major structural stability of the wall.

3. Soil nails are not prestressed, and are installed at a closer spacing than prestressed tie-back anchors; the consequences of a single soil nail failure are not necessarily severe.

4. Construction equipment is relatively small, mobile and quiet.

5. In ground well suited to soil nailing, construction cost is typically 10 to 30 percent less than that of a tied-back wall.

1.2 OBJECTIVES AND SCOPE

The proposed Interstate-5 Swift-Delta soil nail wall called for the removal of the existing south approach embankment end slope of the Oregon Slough Bridge and the construction of a soil nail wall to permanently retain the fill behind the pile-supported abutment. This was the first permanent nailed wall used on a highway project within Oregon. Since this technique is relatively new, the project was defined as experimental. Construction and post-construction monitoring was used to study the performance of the new wall.

The purpose of this project was:

1. to implement an instrumentation program to improve our understanding of soil nail wall performance under service conditions. This is not well predicted by the limit equilibrium analysis (at failure conditions) currently used to design soil nail walls. The instrumentation data has been analyzed and will be discussed in this report.

2. to evaluate the influence of the existing piles on the behavior of the soil nail wall, and to monitor the performance of the piles as a result of the excavation and soil nailing.

To accomplish the project objectives, the wall was monitored for the first three years after wall construction was completed. It will be monitored for the next two years to further study its long-term performance.

The project report is divided into three documents for purposes of organization:

Report 1 (Soil nailing of a Bridge Fill Embankment - Construction Report) covered the construction and the short-term performance of the new soil nail wall and discussed construction problems. Conclusions and recommendations to update the standard specifications were presented.

Report 2 (Soil nailing of a Bridge Fill Embankment - Design and Field Performance Report) discusses the instrumentation program implemented during the construction of the wall and the data taken at two vertical cross sections of the wall. Short-term (ending March 16, 1991) and long-term (ending October 21, 1993) performances are analyzed. A preliminary understanding of the influence of the existing pile bent on the behavior of the soil nails is presented. The performance predicted by the original design methodology is compared critically to the measured performance. Conclusions and recommendations are presented.

Report 3 will discuss how the staged excavation in front of the pile-supported abutment was analyzed using a computer model to predict the stress variation in the existing piles. The predicted results will be compared to the measured performance. The influence of the existing pile bent on the behavior of the soil nail wall will be evaluated to better our understanding of soil-structural interaction in the soil nail retaining wall and pile foundation systems.

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2.0 PROJECT LOCATION

This project is located in North Portland, Oregon on Interstate-5 at MP 307.46. (See Figure 1). The purpose of this project was to widen and lower the grade of the Swift Highway under the south end of the existing Oregon Slough Bridge.

The project was part of the Swift Interchange - Delta Park Interchange reconstruction scheme jointly funded by the State of Oregon and the Federal Highway Administration (FHWA).

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3.0 PROJECT DESCRIPTION

This construction project involved the widening of the Swift Highway under the Oregon Slough Bridge. The widening required the removal of the existing south embankment end slope in front of the pile-supported bridge abutment at pier 10. A permanent soil nail wall was constructed to retain the bridge embankment (See Figure 2).

The total wall length was 256 feet (78 m), of which 165 feet (50 m) was located under the Oregon Slough Bridge. The maximum wall height was 19 feet (6 m); total surface area was 4,105 square feet (581 m^2). For contract plans and general specifications, see Appendix B.

3.1 SUBSURFACE CONDITIONS

An exploration and testing program was undertaken to evaluate subsurface materials and soil standup time along the proposed wall alignment. This consisted of rotary drilling; test pit excavation; field tests, including in-situ torvane testing; and laboratory soil tests. Exploration was limited to the wall portions located east and west of the Oregon Slough Bridge. The embankment foundation material properties were available from a foundation report prepared when the bridge was constructed.

The subsurface material consisted of approximately 35 feet (11 m) of approach fill material, overlying 115 feet (35 m) of unconsolidated alluvial sediments, overlying the cemented gravel Troutdale Formation.

The near surface fill material encountered in the borings and test pits consisted of two soil units identified from the existing ground surface downward as Units A1 and A2.

1. Soil Unit A1 - a silty fine sand with varying amounts of rock and debris. East of the bridge abutment, seven feet (2 m) of fine silty sand mixed with approximately 40 percent rounded gravel up to two inches (50 mm) in diameter was encountered. West of the bridge abutment, six feet (2 m) of fine silty sand was encountered, with concrete pieces, boulders up to two feet (0.6 m) in diameter and metal debris making up about 50 percent of the material. Soil Unit A1 was not continuous under the bridge abutment.

   East and west of the bridge abutment, the soil was generally loose with varying amounts of moisture. Sloughing of the top six feet (2 m) of the test pits occurred during excavation due to the looseness of the soil and the large concrete pieces and rock boulders that were dislodged by the backhoe.

2. Soil Unit A2 - a clayey silt (soil type - ML) layer overlying clean fine sand (soil type - SP). A four-foot thick silt layer was encountered west of the bridge abutment. Standard Penetration Test (SPT) blow counts ranged from four to six blow per foot (bpf). Torvane values in the silt ranged from 0.5 to 0.75 tons per square foot (5 to 7 tonnes/m^2). Moisture content ranged from 33 to 40 percent. The material was easily excavated and held a vertical slope for the three-day standup test period.

   The underlying clean fill sand was continuous under the bridge abutment. The sand was medium dense, damp to moist and poorly graded to uniform in size. SPT counts ranged from 11 to 28 bpf with a general increase in bpf with increased depth. Direct shear testing of the sand showed a phi angle of 32 degrees (0.6 rad). Minor sand sloughing occurred in the test pits after three days, but the sidewalls did not collapse.

Ground water was encountered in three borings and in one test pit. The elevation of the ground water was five to six feet (1.5 to 2 m) below the bottom of the wall base, and was not expected to be encountered during wall excavation. No evidence of a perched water condition was noted.

3.2 DESIGN METHODOLOGY

The design method used to dimension the wall was a force limit equilibrium analysis. It was an adaptation of the Davis method developed by C. K. Shen, et al *(1)*.

This limit force equilibrium procedure is based on conventional slope stability analysis. It assumes that the failure surface is represented by a parabolic curve passing through the toe of the wall. Two cases are typically investigated separately. The first case investigates a failure surface which lies entirely within the reinforced soil mass (See Figure 3). The second case investigates a failure surface which extends beyond the reinforced zone (See Figure 4). The overall stability of the excavation system is evaluated from the equilibrium of the driving forces and the resisting forces developed along the assumed failure surface.

The Davis method uses a computer code to calculate the overall safety factor *(2)*. For a given set of geometric and strength parameters, the computer code calculates the minimum safety factor by searching a series of potential failure surfaces passing through the toe of the wall.

Some of the assumptions in the original Davis method include:

1. The failure surface is a parabolic curve passing through the toe of the wall.
2. The face of the excavation is vertical.
3. The ground surface on top and at the bottom of the wall is horizontal.
4. Vertical nail spacing equals horizontal nail spacing.
5. The soil nails are loaded in tension. (The Davis method neglects shear force and bending moment.)

6. The strength of the soil nail system is mobilized by friction at the soil/reinforcement interface.
7. The effective length of the nail is the length of nail in the passive zone behind the theoretical failure surface.
8. The frictional pullout resistance of the nail is calculated only along the effective length of the nail.
9. The frictional resistance is the shear stress developed between the nail reinforcement grout column and the surrounding soil. This is calculated as a function of the normal overburden stress and soil shear strength, angle of internal friction, and cohesion properties (i.e., Mohr-Coulomb failure criterion).
10. The breakage strength of the nail reinforcement is equal to the structural yield strength of the nail reinforcement.
11. The maximum resistance of the nail utilized for design is the minimum of the frictional resistance or the yield strength of the nail reinforcement, factored by an appropriate safety factor.

The original Davis computer code was modified for use during the design of this project. The modifications and changes were dictated by the method's limitations, the physical constraint represented by the existing bridge piling system and modeling the ten feet of bridge embankment surcharge fill. The computer code was modified to:

1. allow different vertical and horizontal nail spacing.
2. incorporate field pullout test results which directly account for the shape of the bore hole, the drilling method, the method used to clean the nail bore hole, and the grouting method. This change allows the input of a site-relevant soil nail adhesion.
3. allow the modeling of up to 30 nails in one vertical row. An assumed soil nail adhesion can be specified at each nail level.
4. provide the effective nail length at each nail level. The original Davis method does not provide for this, however, the parabolic failure surface can be plotted and the effective length behind the failure surface can be obtained by scale.

3.3 DESIGN SOIL STRENGTH PARAMETERS

The following soil parameters were used for the design analysis:

Cohesion Strength	= 100 psf (500 kg/m^2)
Frictional Strength	= 32 degrees (0.6 radians)
Soil Unit Weight	= 115 pcf (1,800 kg/m^3)
Soil-Grout Adhesion	= 1000 psf (5,000 kg/m^2)
Stress Ratio	= 0.43

A literature search *(3, 4, 5, 6)* of pullout test results provided the following range of soil-grout interface frictional resistance: lower bound of 600 psf (3,000 kg/m^2) with the augured method of installation, and upper bound of 4000 psf (20,000 kg/m^2) with the rotary drill method of installation and low pressure grouting (less than 150 psi) of the bore hole. An assumed frictional resistance of 1000 psf (5,000 kg/m^2) was used to design the wall. The specified minimum drill hole diameter was 7 inches (180 mm).

A stress ratio (defined by the ratio of the horizontal stress to the vertical stress) corresponding to an active earth pressure condition was assumed. This assumption is consistent with the requirement of relative movement at the soil-grout interface to mobilize the tensile capacity of the nails. It is also consistent with observed maximum movements reported in the literature at the top of soil nail walls. This reported maximum horizontal movement varied from 0.1 to 0.3 percent of the wall height. This range is what would be required to mobilize active earth pressure in cohesionless soil *(7)*.

3.4 STABILITY ANALYSIS

The existing bridge pile system required locating the soil nails midway between adjacent piles at a horizontal spacing of 4.5 feet (1.4 m). The vertical spacing of the nails was 3 feet (0.9 m). The nail inclination was 15 degrees (.26 rad) from the horizontal at all rows of nails with the exception of the bottom row which was installed at 25 degrees (0.44 rad) from the horizontal (See Figure 5). The vertical number of nails varied from a minimum of two to a maximum of six nails.

For corrosion protection, the design assumed a seven-inch (180 mm) bore hole diameter to provide a three-inch (80 mm) clear grout cover all around the nail reinforcement. However, a five-inch (130 mm) drill hole diameter with pressure injected grout was constructed. Approval of the change was based on existing literature indicating that pressure injected grout increases the pullout capacity of tie-back anchors installed in porous cohesionless soil. Any theoretical decrease in the soil-grout bond stress, attributed to the use of the smaller five-inch (130 mm) drill hole diameter, was expected to be offset by the increase in the bond stress associated with pressure grouting.

A preliminary external stability analysis determined the reinforced wall width necessary to prevent the wall from sliding along its base and overturning about the toe. The earth pressure exerted by the unreinforced soil and resisted by the reinforced earth block required a minimum nail length of 15 feet (4.6m) to provide safety factors of 1.5 and 2.0 against sliding and overturning, respectively.

A soil-structure interaction analysis of the soil-piling foundation determined the maximum depth of the first excavation lift. The soil response was modeled by a family of curves that show the soil resistance P as a function of deflection Y, and depth below the ground surface. The analysis used the computer code COM624 *(8)* which generated P-Y curves representing the soil behavior under lateral loads. These curves are mathematical relationships of soil reaction per unit length of pile versus lateral pile deflection, and vary with position along the pile.

The pile model consisted of two segments. The top segment represented the concrete pile cap (Figure 6). Which was transformed into an equivalent steel section with the appropriate transformed section properties. The lower segment represented the 14-inch (360 mm) diameter concrete-filled steel pipe pile. The concrete in the piling was ignored in the analysis since it was not reinforced nor positively connected to the steel shell to allow load transfer.

The span-9 precast, prestressed beams are hinged to the top of the 3-foot (0.9 m) wide pile cap. One and one-quarter inch diameter dowels spaced at 12-inch (305 mm) centers and a continuous shear key transfer the acting horizontal loads from the superstructure to the substructure (Figure 6).

The model assumed the pile head was free to rotate and fixed against translation, and that a horizontal strut force acted at the top of the pile. The strut force counteracted the lateral earth pressure from the embankment fill and represented the effect of the superstructure in preventing lateral translation at the pile head. The validity of this assumption is discussed further when the extensometer data is presented.

The loading on the pile in the analytical model consisted of axial and lateral loads. The axial load represented the superstructure dead weight and traffic live load. The lateral load represented the lateral earth pressure imposed by the embankment fill. The computer code COM624 yielded the maximum total stress in the pile for the assumed excavation depth. The combined stress ratio (i.e., combined effect of axial and lateral loads) was computed next *(9)*. The depth of the first excavation lift was limited to a vertical distance of 3.5 feet (1.1 m) below the bottom of the existing bridge pile cap. This limitation ensured that the combined stress ratio in the pilings did not exceed 1.0.

Stability analyses using the modified Davis computer code were performed for various wall heights at typical wall cross sections. The ten feet (3 m) of bridge abutment surcharge was modeled as an additional wall height. Fictitious nails with frictional resistance equal to zero were specified within this additional height. Stability analyses were also performed for wall heights with the ten feet (3 m) of bridge abutment surcharge treated as an ordinary dead weight surcharge. Modeling the bridge abutment surcharge as additional wall height rather than an ordinary dead weight surcharge consistently gave a lower safety factor for a given nail length. The additional wall height locates the critical failure surface deeper into the zone behind the wall face than a lower wall height with an equivalent applied surcharge load. Two feet (0.6 m) of live load surcharge was used in the design of the wall sections retaining the bridge abutment fill.

The soil nails were sized and spaced to meet the following global stability safety factor (SF) criteria:

1. minimum SF of 1.25 after the excavation is completed to specified grade and the bottom row of nails not installed.

2. minimum SF of 1.50 after the excavation is completed and the bottom row of nails installed.

3.5 NAIL FORCES

The modified Davis method provided the effective length at each row of nails and the total frictional resistance required to maintain a safety factor of 1.5 assuming that the effective length of the nail is the length in the passive zone behind the theoretical failure surface. The frictional resistance at each nail is calculated along the effective length of the nail. The total frictional resistance at a given wall cross section is assumed to equal the sum of the frictional resistance among the individual nails. The major limitation of the Davis method as a design tool is that it does not provide insight on the tensile forces developed along the nails under service conditions or at failure conditions.

Research work by Juran and Elias *(10)* suggests that in homogenous granular soils, the maximum nail forces under service conditions may be estimated using the empirical diagrams of earth pressure distribution proposed by Terzaghi and Peck for the design of braced open cut excavations. Actual measured service loads are compared to the Terzaghi and Peck empirical diagram later in this report.

In the Swift-Delta project, the nail bars were sized to carry a minimum tensile load equivalent to the frictional resistance calculated along the effective length of the nails. An allowable tensile stress equal to 55 percent the yield strength of the steel bar was then used to size the bars, ensuring the bars would not fail by structural breakage during the production testing.

3.6 SHOTCRETE FACING DESIGN

In soil nailing, the nails are spaced so that the material between them will arch and form a reinforced earth block. The shotcrete facing resists some earth pressure, prevents relaxation or sloughing of the ground, and prevents a local failure of the soil between the nails.

Field test results from several full-scale soil nail walls *(11, 12)* suggest that the maximum tensile force in a nail occurs at a certain distance behind the shotcrete facing, and that the earth pressure reaching the face is significantly less than full active earth pressure. Test results also suggest *(11)* that the earth pressure distribution behind the facing is closer to uniform than triangular.

In this project, the maximum nail forces predicted by Terzaghi and Peck's rectangular lateral earth pressure distribution for the design of braced open cut excavations were assumed to act at the shotcrete facing. No water pressure loading was assumed to act on the soil nail wall since ground water levels were deeper than excavation levels. In addition, vertical drainage fabric, centered between each vertical nail column, was placed behind the shotcrete facing to drain any water that might collect behind the wall.

The Load Factor method was used to design the structural shotcrete facing. The structural design was conducted by considering a horizontal beam strip with a span length of 4.5 feet (1.4 m) and a vertical width of one foot (0.30 m). The beam strip was conservatively assumed to be simply supported between two adjacent nails. The face thickness was designed to resist one-way shear, two-way shear (i.e., punching shear) and excessive bearing stresses.

3.7 CONSTRUCTION METHOD

The soil nail wall construction sequence (Figures 7 through 10) was as follows:

1. Preproduction nail testing to verify that grout-soil adhesion greater than 1000 psf (5,000 kg/m^2) (assumed in the model) would be produced by a combination of the contractor's proposed drilling procedure, the drill hole diameter of five inches (130 mm), and the grouting method, thus, giving as a minimum, the same pullout capacity per lineal foot of nail length predicted by the model.

2. Stage excavation of the existing end berm from the top down to the layer limits is shown in the plans. Due to some localized sloughing of the sand fill material immediately after excavation, the open cut was stabilized to neat line with a thin ± one-inch thick flashcoat of sacrificial shotcrete.

3. Erection of the steel welded wire mesh.

4. Application of 3.5 inch (90 mm) of air-blown structural shotcrete using the wet-mix method.

5. Drill the bore holes with a Krupp crawler drill through block-outs in the shotcrete at predetermined locations as shown in the plans. The cased hole drilling method was used, in which an outer casing and an inner drill steel advance simultaneously. The outer casing prevents the drill hole from collapsing on the completion of drilling.

6. Placement of the epoxy-coated Dywidag nails and low pressure grouting (150 to 200 psi) into the drill holes as the drill casing was withdrawn.

7. Placement of the 6-inch (150 mm) by 6-inch (150 mm) by 5/8 inch (16 mm) epoxy-coated steel bearing plate and fastening it with a nut which was secured wrench tight.

8. Repeat the process for all subsequent lifts until the bottom grade was reached.

9. Application of a second 4.5 inch (110 mm) thick, layer of structural shotcrete, for a total shotcrete thickness of 8 inches (200 mm). The facing was hand finished and was scored vertically and horizontally by temporary wood scoring strips to match the architectural finish of other walls in the project area.

For a detailed description of the construction process and related photographs, refer to the Construction Report *(13)*.

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4.0 INSTRUMENTATION PLAN

4.1 BACKGROUND

The Swift-Delta soil nail wall was the first permanent one used on a highway project in Oregon. Because this technique was relatively new, the wall was considered experimental, and construction and post-construction monitoring was required to study its performance. The specifications for the instrumentation plan are in Appendix C.

Some changes were made in the instrumentation program prior to soil nailing; additional instrumentation was added prior to construction completion. A description of the instrumentation plan (Figures 11 and 12) and associated revisions follow.

4.2 INSTRUMENTATION DESCRIPTION

Two wall sections were selected for the performance study. Section 1 was in front of the existing bridge abutment and consisted of five rows of nails (Figure 13). Section 2 was 17 feet west of the bridge abutment and also consisted of five rows of nails (Figure 14). The excavation progress at both sections is shown in Figures 15 and 16. The installation schedule is described in Table 1. The instrumentation installation is shown in Figures 17 through 28.

At each monitoring station, the following instruments were installed:

1. **Vibrating Wire Strain Gauges - Nails:**Geo Kon Model VK 4100 strain gauges were mounted on the nails to evaluate the load transfer with time along the length of the nails as well as the load distribution along the nails.

   The original instrumentation plan consisted of mounting five pairs of strain gauges to each of three nails at Section 1, and four pairs to each of three nails at Section 2. The instrumentation plan was revised and all five rows of nails at both sections were instrumented with vibrating wire strain gauges (Figures 13 and 14).

   The strain gauges were mounted in pairs opposite each other and aligned vertically and then spot welded along each Dywidag bar. This allowed the measurement of bending, and the calculation of average axial strain. The gauges were spaced equally along each nail (Figures 13 and 14). The spacing from the shotcrete facing to the upper pair of gauges varied due to drill hole length. A mechanical coupler was placed behind the shotcrete face to splice the instrumented nails corresponding to Section 2, Row 3 and Row 5, and Section 1, Row 3. Wire leads from each instrumented nail were installed in PVC tubing for protection within the shotcrete wall surface and brought to a common monitoring control panel.

In general, the strain gauges performed quite well during construction and continue to perform satisfactorily, nearly four years into the monitoring program. Only one strain gauge, B7 at Section 1 (on top of the nail, Figure 13) failed during the installation.

2. **Vibrating Wire Strain Gauges - Pile:** Geo Kon Model VK 4100 strain gauges were also mounted on two existing steel pipe piles to evaluate possible load transfer to the piles during staged excavation.

   Prior to excavation and soil nailing, the contractor hand excavated a shored pit (9 feet long by 4 feet wide (3 m by 1 m) under the bridge abutment, exposing two adjacent pipe piles to which four strain gauges (two to each pile) were spot welded. The gauges were mounted 5 feet (1.5 m) and 10 feet (3 m) below the bottom front face of the existing pile cap. Following the installation of the strain gauges, the pit was backfilled with a lean-mix concrete.

3. **Electric Load Cells:** Three Carlson/RST model SGA-100-1.5 cells were installed at each station to monitor load buildup at the nail anchorage. The load cells were located in Rows 1, 3 and 5. Each was placed between the bearing plate and the nut at the head of each instrumented nail. The nut was then secured wrench tight following the grouting and prior to excavating the next lift. The base reading was taken immediately following the fastening of the nut and is referred to as the "lock-off load".

4. **Pneumatic Earth Pressure Cells:** SINCO model S1482 cells were installed to monitor earth pressure buildup on the shotcrete facing.

   The original pressure cells instrumentation scheme consisted of six pressure cells at each test section for a total of twelve. Two cells were to be located at nail levels corresponding to Rows 1, 3 and 5. One pressure cell was to be placed near the instrumented nail and the second cell midway between the instrumented nail and an adjacent nail.

   Subsequent to bid letting, the reliability of the pneumatic earth pressure cells was judged to be questionable based on preliminary information gathered from other projects around the country. Therefore, the instrumentation plan was revised and only two earth pressure cells were installed. One cell was located under the bridge immediately west of Section 1 and approximately 1.5 feet (0.5 m) below Row 2. The other cell was placed outside the bridge embankment influence immediately east of Section 2 and 1.5 feet (0.5 m) below Row 2. The cells were placed against the excavated open face and loosely covered with fill to prevent damage during the application of the shotcrete face. Wire leads were enclosed in a PVC conduit within the shotcrete face and brought to a junction box for monitoring.

. **Single Point Extensometer:** A Carlson/RST model EX-1 extensometer was installed to monitor the horizontal deflection of the pile cap as excavation progressed.

The contractor drilled a hole through the existing pile cap and installed the single point extensometer at the center of the cap (Figure 12). The anchor depth of the extensometer was 61.0 feet (19 m) behind the bridge pile cap. The reference head of the extensometer was mounted in the pile cap. Readings are made using a manual digital readout micrometer which measures the distance between the reference plate and the stainless steel rod tip, located inside a small diameter hole in the reference plate.

6. **Ceramic Tiltmeter Plates:** Three SINCO model 50302300 tiltmeters were installed on the exposed face of the existing pile cap to monitor the cap's rotation. Tiltmeter plates were installed at the eastern and the western edges of the pile cap; the third was located midway between them.

7. **Vertical Slope Inclinometers (SI):** Two inclinometers (SD128 and SD129) were installed to measure ground movement near the soil nail wall face. To ensure conformity with the surrounding ground movements the SI tubes were installed in 4 inch (100 mm) diameter holes that were later backfilled with loose native sand and pea gravel from the bottom of the hole to 3 feet (0.9 m) below the surface. A bentonite seal was placed in the top three feet of each 4-inch (100 mm) hole. The casing embedment tip was approximately 29.8 feet (9 m) and 32.7 feet (10 m), as measured from the top of casing at SD128 and SD129, respectively. The casings were extended a minimum of 10 feet (3 m) below the base of the excavation to provide a stable reference section.

The inclinometers were located 10 feet (3 m) (SD128) and 38 feet (12 m) (SD129) west of the bridge. Both inclinometers were located ±3.5 feet (1 m) behind the back face of wall. During the installation of the soil nails for lift No. 1, the casing for SD128 was severed. The damaged inclinometer was replaced by SD130 which was installed next to the abandoned SD128.

Two other vertical slope inclinometer tubings (SD131 and SD132) were attached to the initial shotcrete facing to monitor post-construction horizontal movement under the bridge portion of the wall. The tubings were located at previously established optical survey points (as described in 8 below) and cast into the final shotcrete lift full wall height. Neither SD131 nor SD132 tubings extended below the base of the wall.

8. **Optical Survey Monitoring Points:** These were at two wall locations under the bridge to monitor horizontal wall deflections during construction. The survey points were established following the placement of the shotcrete face at each lift.

Linear Variable Differential Transformers: Two linear variable differential transformers (LVDT-1 and LVDT-2) were installed to monitor vertical wall movement. (These were not part of the original instrumentation plan, but were added to the revision prior to construction completion.) The LVDT's were installed at the top-of-wall, within the bridge limits, after completion of soil nailing and prior to placing the final shotcrete lift.

5.0 FORCE-STRAIN RELATIONSHIP

5.1 BACKGROUND

Strain gauge readings measure the strain changes with time on the nail. The transformation of the strain readings into equivalent forces requires conversion from measured strains to computed stresses. For a composite section made of cement grout and reinforcement, the method of transformed areas can be used to calculate the stresses.

The method of transformed areas requires knowledge of the cross-sectional areas and the moduli of steel and cement grout compost section. This method has the following limitations:

1. The cement grout stress-strain relationship is nonlinear, especially at high stress levels. Therefore, the value for the cement grout modulus varies.

2. Cement grout in tension becomes ineffective as a load-carrying member due to cracking. When this happens, the axial stiffness of the cement grout-nail system is reduced substantially.

3. The method does not account for strains other than those caused by external stresses. Cement grout creep and shrinkage will cause additional strain deformation of the nail without an increase in nail load.

On the basis of the above, a special effort is required to properly interpret forces from the measured strain readings. Such effort should reasonably approximate the axial stiffness variations due to cement grout creep, shrinkage and cracking, and the nonlinearity of the cement grout stress-strain curve.

For the Swift-Delta project, a Dywidag bar was instrumented with a pair of vibrating wire strain gauges (Figure 29). A column of grout was cast around the bar and allowed to cure for a week (Figure 30). The specimen was load tested, in the lab, in direct tension (Figures 31 and 32). The resulting strain was measured for different, known, load levels. The force-strain relationship from this test nail was applied to strains measured at both instrumented sections convert the strain readings into equivalent forces.

One should recognize the limitations introduced by this procedure. The load-strain relationships were derived from data taken while applying the axial loads. Field measured strains are transferred through soil-grout interface shear to the bar. In addition, this procedure does not account for changes that occur at different curing ages.

5.2 TESTING PROCEDURE

Two vibrating wire strain gauges (G0 and G1) were mounted on a No. 9 Dywidag bar (uncoated Grade 60 steel). The gauges were spot welded opposite each other. The installation was performed by the subcontractor according to the specification requirements for field installation.

The test nail was load tested in the lab in direct tension prior to casting the column of grout. The Dywidag bar was incrementally tensioned to a tensile load of 40 kips (18 tonnes). The resulting strains were measured for each load increment (Table 2). Two load-strain relationships were obtained, one for each gauge (Figure 33). Strain gauge G0 had a change in strain with load equal to 40.25 microstrain/kip. Strain gauge G1 had a change in strain with load equal to 40.87 microstrain/kip. The average change in strain with load for the ungrouted bar was 40.56 microstrain/kip. The calculated modulus of elasticity was 24,845 ksi (1.7×10^5 MPa).

Following the calibration of the ungrouted bar, a column of grout was cast around the bar in a plastic cylinder. The column was 6 inches (150 mm) in diameter and 2-feet (0.6 m) long. (The 6 inch (150 mm) diameter was chosen for ease of casting and to better represent what the actual drill hole diameter would be following pressure grouting. Published results *(14, 15)* indicates that the drill hole diameter is typically enlarged by hydrofracturing of the cohesionless ground mass to give a grout diameter larger than the core diameter of the drill hole.) The grouting was performed in the field, during the construction of the wall. The fluid cement grout used to pressure grout the drill holes was also used to cast the test nail. The grouted test nail was left in the field for one week and covered with insulation blankets that were used to cure the shotcrete facing. Following the one-week curing period, the test nail was load tested in the lab, in direct tension. The nail was incrementally tensioned to a load of 40 kips (18 tonnes), and the resulting strains were measured for all load increments.

5.3 GROUTED NAIL LOAD-STRAIN RELATIONSHIP

Both strain gauges exhibited nearly identical load-strain relationships (Figure 34). One exception was noted between the 20 to 35 kips (9 to 16 tonnes) load range. This difference will be discussed later. Figure 35 shows the average force-strain relationship curve of the grouted test nail. The bar behaved elastically during the testing. Four different load strain zones were identified.

1. The first zone was in the zero to 6 kips (0 to 3 tonnes) load range. Strain gauges G0 and G1 registered a change in strain with load of 21.8 and 23.0 microstrain/kip respectively. The average change in strain with load was 22.3 microstrain/kip. The cement grout was intact.

2. The second zone was in the 6 kips to 20 kips (3 to 9 tonnes) load range. Strain gauges G0 and G1 registered a change in strain with load equal to 32.6 and 32.9 microstrain/kip respectively. The average change in strain with load was 32.8 microstrain/kip.

The average change in strain with load in this zone reflected a loss in the axial stiffness of the cement grout-Dywidag bar system. The increased strain in the bar was most likely due to the initiation of microcracks in the cement grout. This in turn caused the cement grout to shed part of its tensile load to the bar. The cement grout was partially ineffective in carrying the tensile load due to microcracking.

3. The third zone was in the 20 kips to 35 kips (9 to 16 tonnes) load range. Strain gauges G0 and G1 registered a change in strain with load equal to 54.0 and 45.8 microstrain/kip respectively. The average change was 49.9 microstrain/kip. An audible crack was heard between the 25 kip (11 tonnes) and the 28 kip (13 tonnes) load increment.

   The measured strains reflected a rapid loss in the axial stiffness of the cement grout-Dywidag bar system. The average change in strain of 49.9 microstrain/kip was larger than the measured average change of 40.4 microstrain/kip for the ungrouted bar. The larger increase in strain in the grouted bar system indicated that the cement grout was rapidly becoming ineffective as a load-carrying member due to widening of the cracks, and the rapid transfer of load from the cracked grout to the bar.

   The change in strain measured with gauge G0 was larger than the change measured with gauge G1. This difference could be attributed to the formation of the first grout crack adjacent to gauge G0.

4. The fourth zone was in the 35 kips to 40 kips (16 to 18 tonnes) (maximum test load) load range. Strain gauges G0 and G1 registered a change in strain with load equal to 41.0 and 38.0 microstrain/kip respectively. The average change was 39.4 microstrain/kip.

   The average change in strain of 39.4 microstrain/kip was nearly equal to the measured average change of 40.4 microstrain/kip for the ungrouted Dywidag bar. This indicates the Dywidag bar was acting alone in carrying the tensile load while the cement grout was totally ineffective.

The force-strain relationships obtained from these test were used to convert strain readings taken in the field.

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6.0 SHORT-TERM MONITORING

The short-term performance of the wall as measured by instrumentation is analyzed. The following dates were of interest during data collection:

a.	Completion of excavation and soil nailing	January 24, 1991
b.	Completion of wall construction (following placement of second shotcrete application)	February 14, 1991
c.	End of short-term performance evaluation	March 16, 1991

6.1 NAIL LOADS - GENERAL

The analysis of the short-term performance of the nails at both instrumented sections of the Swift-Delta project is separated into measured and computed performance during construction, and immediately following completion of construction. The measured performance consisted of load-time behavior as measured by the load cells installed at the nail anchorage in Rows 1, 3 and 5. The computed performance consisted of using the strain gauge data to compute the distribution of axial strain and the corresponding tensile loads along each instrumented nail. It should be noted that the tensile loads were calculated <u>without</u> correcting for creep in the cement grout.

6.1.1 Nail Strain Gauges

The strain gauge data at both sections provided considerable insight into the effect of excavating successive lifts, and the development of loads in the nails during and following construction. It also provided a preliminary, albeit limited, understanding of the pile-soil nail interaction which is discussed in detail in Chapter 8.

It was initially hoped that an understanding of the pile-soil nail interaction would be developed by direct comparison of load cell and strain data. Unfortunately, strain gauge data from Section 2, at Row 1 and Row 2 exhibited an unusual behavior that warrants further investigation. It is possible that such results may have been caused by one or more of the following:

a. Slope stability analysis indicates, in general, that the critical failure surfaces in cohesionless soils are close to the open face. The exposed face at Swift-Delta consisted of cohesionless sandy soil that experienced local sloughing on a regular basis and required stabilization with a flashcoat of shotcrete. It is possible that soil wedges immediately behind the face, slipping downward due to the low level of soil cohesion, may have caused compression in the nails. This will be discussed later in this section.

b. The nails were anchored through concrete rubble. Pressuremeter testing in the vicinity of Section 2 encountered concrete approximately 5 feet (1.5 m) below the surface. In addition, the drilling log for inclinometer SI 130 indicated that concrete was encountered at 8 to 9 feet (2.4 to 2.7 m) below ground surface and a concrete slab approximately 15 feet (5 m) in length and 4 to 5 inches (100 to 130 mm) thick was encountered between the instrumented nails corresponding to Rows 2 and 3 during the excavation of the open face. This debris could effect the behavior of the nails.

To that effect, no direct comparison was made between the strain gauge data from the two sections for the Rows 1 and 2 instrumented nails.

Figures 36 through 50 and Figures 51 through 65 show strain measurements plotted as a function of time for the top and bottom strain gauges, and the computed average axial strain at both Section 1 and 2. Using pairs of gauges attached opposite each other, and orienting them vertically at each location along the nail length allowed direct computation of the effect of bending on the total measured strains.

Tables 3 and 4 show the axial strain distribution along each nail at Section 1 and 2, following completion of lifts excavation (January 24, 1991), completion of construction (February 14, 1991) and at the end of the short-term performance evaluation (March 16, 1991). It can be seen that by the time short-term monitoring was completed, the axial tensile strains had generally increased over their values immediately following completion of the excavation.

Figure 36 shows the distribution of strain measurements at Section 1, Row 1, top strain gauges, plotted as a function of time during construction and one month following construction. It is representative of the general trend observed at most instrumented nails. The excavation of successive lifts after installation of the Row 1 nails is generally accompanied by an initial rapid increase in tensile strain. This is followed by a slow increase in strain with time as the lower nails and the shotcrete facing are installed. The influence of the staged excavation was most significant when the bottom of the excavation was within 12 feet (3.6 m) of the nail. The effect of subsequent excavation was less noticeable. Following the completion of excavation of all lifts (January 24, 1991), the measured tensile axial strains continued to increase, but at a slower rate. This behavior is consistent with other instrumented soil nail wall studies *(16, 17, 18)*.

The bottommost row of nails (Row 5) at both sections were least subjected to axial tensile strains following the completion of excavation and nail installation. This is consistent with other cases *(16, 18, 19)* where the higher tensile strains in the upper nails appear to be related to the arching effect developed in the reinforced soil mass following the excavation of successive lifts, while at the base of the wall, the strains are transferred mainly to the foundation material and felt to a lesser extent by the bottommost nails.

6.1.2 Computed Nail Loads

Figures 66 through 70 and Figures 71 through 75 show the change in tensile forces with time along the length of the instrumented nails in Section 1 and 2. They represent the equivalent nail forces calculated using the force-strain relationships described in Chapter 5. Tables 5 and 6 show the computed tensile loads distributed along the length of the nails following completion of all excavation and nail installation, completion of construction and at the end of the short-term performance evaluation.

Figures 76 through 79 summarize the distribution of the tensile forces at all nails at both instrumented sections immediately following the completion of excavation, on January 28, 1991, and at the end of the short-term performance evaluation on March 16, 1991.

Figures 68 and 73 show the nail force distribution and the change in nail forces with time for Row 3 at Sections 1 and 2. Both figures are representative of the general trend discussed earlier. The effect of excavating the final three lifts following the installation of this row of nails is clearly noticeable, in particular at Section 1. This is shown by the readings taken after January 5, 1991. Figure 68 shows a significant increase in load at Section 1, on January 16, 1991, approximately 3 feet from the shotcrete face. This increase is most likely due to bending strains associated with the excavation of Lift 5 on January 16, 1991. The general load distribution observed was as expected: nail tension varying over the whole length of the nail, with an increase in tension away from the shotcrete face to a maximum and then a decrease toward the tip of the nail. Most importantly, it can be seen that the tensile forces in the nails are maximum inside the soil nailed earth mass away from the facing. This behavior is consistent with other instrumented studies *(16, 17, 18, 19)*.

While the measured increase in nail forces at the end of the short-term performance evaluation is not very significant in general, it remains of interest to note how the distribution of tensile forces along the nails changed with time, particularly at Section 1. Tables 5 and 6 show the total percent of change in the axial tensile load distribution along the nails after January 28, 1991.

Table 5 shows that, at Section 1, the strain gauges located nearest the shotcrete facing indicated a general reduction in the tensile loads, while the strain gauges located further away from the shotcrete facing showed an increase in the tensile loads. The maximum short-term total percentage increases after January 28, 1991 were 38, 29, 57 and 28 percent at Rows 1 through 4 respectively, while it more than doubled at Row 5. This increase was measured at all gauges other than those located nearest the shotcrete facing.

Creep and microcracking normally renders the cement grout partially ineffective as a tensile load-carrying member. This in turn reduces the axial stiffness of the grout-nail system and leads to additional wall deflections. Soil creep also results in additional wall deflections leading to an increase in actual nail forces. Post-construction readings of the slope inclinometers verified the additional wall deflections. Therefore, this post-construction increase in the nail forces is most likely due to a combination of soil creep, creep and cracking of the cement grout, and the subsequent transfer of load from the grout onto the nails. It is also interesting to note that the nails at Section 1, Rows 1, 3, 4 and 5 have a nearly uniform force distribution (Figure 77). The nail at Section 1, Row 2 appears to have a distinct maxima.

6.1.3 Load Cells

Rows 1, 3 and 5 were equipped with load cells (Figures 13 and 14). Each load cell was placed at the nail head between two bearing plates following the nail installation and the grouting of the drill hole. No prestressing of the nails was specified. The bearing plates were fastened to the nail with a nut and secured with a minimum 100 foot-pound of torque to ensure the shotcrete facing was in good contact with the excavated face.

The load cell data may not reflect the actual load at the nail head. It is suspected the data was influenced by the shotcrete face bonding to the nail head. Row 1 nails were properly debonded from the shotcrete face with a sleeve, however, there is no documentation that Row 3 and Row 5 nails were also debonded. After the long-term monitoring period, the load cells at Row 3 and Row 5 were removed. The first layer of shotcrete, applied during the stage excavation, appeared to have bonded to the nails at Row 5 and to the nail at Section 2, Row 3. This would result in load transfer of an unknown magnitude from the nail to the facing and may explain why the load cell data does not show a post-construction increase.

Figures 80 and 81 show the load cell measurements as a function of time at Section 1 and Section 2, respectively. Tables 7 and 8 show selected load cell readings during the short-term performance evaluation including lock-off loads at Rows 1, 3 and 5. The lock-off loads varied between 2.2 kips and 3.7 kips (1.0 and 1.7 tonnes) at Section 1 and between 2.5 kips and 3.4 kips (1.1 and 1.5 tonnes) at Section 2. The load cells indicate a general loss of lock-off loads over time. Table 7 indicates an approximate loss of 27, 55 and 69 percent in lock-off loads at Section 1, Rows 1, 3 and 5, respectively, at the end of the short-term monitoring period (March 16, 1991). Likewise, Table 8 indicates an approximate loss of 67, 21 and 36 percent in lock-off loads at Section 2, Rows 1, 3 and 5, respectively.

Of interest is how the nail load at the face evolved at Section 1, Row 1 (Figure 80). The lock-off load was 3.7 kips (1.7 tonnes) on December 7, 1990. Approximately 8 percent load loss occurred by December 18, 1990. This loss was recovered shortly thereafter and the load increased 30 percent, to 4.8 kips (2.2 tonnes), by January 16, 1991. This increase is most likely associated with the bending strains due to the dead weight of the shotcrete face being partially carried by the nail. It should be noted that a similar increase was recorded at the face at Section 1, Row 3 on that date following an initial decrease in the lock-off load. The lock-off loads decreased significantly, particularly at Rows 3 and 5, following the completion of construction, and subsequently leveled off at all three instrumented rows through the latter part of the short-term performance evaluation.

A similar behavior was measured at Section 2 on February 16, 1991 where lock-off loads decreased, particularly at Row 1, and subsequently leveled off at all three rows.

The decrease in lock-off loads as measured by all nails could be attributed to the general outward movement in the wall and the nails. The movement of the nails is most likely related to the overall movement of the reinforced soil mass rather than displacement of the grouted nail relative to the surrounding soil. It could also be attributed to a possible redistribution of loads toward the bottom of the wall, particularly, into Row 5 nails (Figures 76 through 79).

6.1.4 Summary

This section summarizes some of the observed behavior through a review of the measured strain gauge readings.

1. Tables 7 and 8 show the ratio T_o / T_{max} (tensile force at the facing/maximum tensile force in the nail). The tensile forces at the facing were measured with load cells installed at Rows 1, 3, and 5. The facing forces at Rows 2 and 4 were linearly interpolated from measured tensile forces at Rows 1, 3, and 5. The ratio T_o / T_{max} decreased in general as construction progressed and following completion of construction. Immediately following nail lock-off, the ratio was generally larger than 1 and at the end of the short-term performance evaluation it varied between 0.13 and 0.4 at Section 1, and between 0.2 and 0.5 at Section 2. Such results are consistent with other instrumented studies with an average value of around 0.4 to 0.5 *(16, 18, 19)*. This is related to the lateral decompression of the soil during the staged excavation and prior to the installation of the corresponding soil nails. It is also due to the mobilization of the load-carrying capacity of the nails, with the maximum tension load in the nails occurring at some distance behind the face.

2. The performance of Rows 1, 2 and 3 at Section 1 on January 16, 1991 is of interest. Figures 36 and 37 show no change in the tensile strain in the top gauge A1 and a significant decrease in the tensile strain in the bottom gauge A2. A significant increase in tensile strain was also measured in the top gauges B1 and C1 at Row 2 and Row 3 (Figures 39 and 42). This was accompanied with a significant decrease in the tensile strain in the bottom gauges B2 and C2 (Figures 40 and 43), with gauge B2 measuring negative strain (compression). This behavior was only measured by the pair of strain gauges closest to the nail head (within 2 to 3 feet of the facing) at all three rows. This indicates the presence of bending strain due to the dead weight of the shotcrete face being partially carried by the nail head.

   The bending strain continued to slowly increase following additional excavation and subsequent to the completion of construction. The bottom gauge C2 at Row 3 measured negative strain (compression) after January 26, 1991. At the end of the short-term performance period, bending strain had increased to 84, 202 and 286 microstrains at Rows 1, 2 and 3, respectively.

3. The performance of Row 1 at Section 2 on January 10, 1991 is also of interest. Figure 51 shows a decrease in tensile strain in all top gauges along the nail, with gauges A1 and A3 reading negative strain (compression). The bottom strain gauges measured a similar behavior (Figure 52) with gauges A2, A4 and A6 reading negative strain (compression). This is most likely due to the collapse of a section of

the open face *(13)* following the excavation for Lift 4 on January 8, 1991. The measured performance on January 10, 1991 indicated that the upper portion of Row 1 is in total compression as measured by the pair of gauges A1/A2 and A3/A4 (Figure 53). It could be suggested that local instability, leading to downward slip movement within the retained earth mass may partially explain the unusual behavior measured along Rows 1 and 2 at Section 2.

4. Figures 54 and 55 show a significant increase in the tensile strain along Row 2 at Section 2 as measured by gauges A9 and A10, located near the shotcrete facing, on December 21, 1990. The excavation for Lift 3 took place on December 17, 1990 and lowered the ground an additional 2 feet (0.6 m) to a total of 11 feet (3.4 m) (Figure 16). The base reading for Row 2 took place on December 18, 1990, following the excavation for Lift 3. It is possible that the immediate effect of excavating Lift 3 on Row 2 was not captured in the readings taken after the excavation. The arching of the soil mass, after the excavation for a given lift, typically introduces higher forces in the upper nails and lower forces in the lower nails due to the proximity of the base of the excavation and the transfer of some of the load into the foundation material. Since small loads were introduced into Row 1 as of December 18, 1990, it can, therefore, be assumed that little to no load was introduced into Row 2 immediately following the excavation for Lift 3, and that the use of the December 18, 1990 reading as an initial base reading for Row 2 was valid. The same applies for Row 2 at Section 1.

5. Figure 67 shows the change in tensile forces with time at Row 2, Section 1 through the end of the short-term performance evaluation (March 16, 1991). Row 2 contains the only strain gauge that failed during the installation of the nails. This gauge, B7, is located along the lower portion of the nail, approximately 7 feet (2 m) from the tip (Figure 13). The tensile axial load variation appears to be uniformly distributed along the nail during the early stages of loading. After January 5, 1991, a distinct maxima was observed at the B7/B8 guage location. Since the measured performance at that location is based on a single gauge (B8), it could be suspected that the readings taken after January 5, 1991 may include bending strains that cannot be isolated with one gauge.

   A review of the measured strains in gauges B5/B6 and B9/B10, located adjacent to B7/B8, indicates that small tensile bending strains were introduced at B6, located along the bottom surface of the nail, and at B9, located along the top surface. While the bending strains are not significant, they do suggest that a point of contraflexure (point of zero bending) may exist between gauges B5/B6 and B9/B10 in the vicinity of B7/B8. Therefore, it could be assumed that measuring the axial strain with a single gauge at the B7/B8 location introduces negligible error.

6. Figures 45 and 46, and Figures 60 and 61 show significant axial strains on January 16, 1991 in Row 4 following the excavation for Lift 5. This measured performance was observed at all strain gauges along Row 4 (Figures 69 and 74). A review of

Table 1 shows that Row 4 nails were first-stage grouted on January 14, 1991. Second stage grouting was performed on January 15, 1991. The lowering of the ground and the low strength of the cement grout within two days of grout placement is most likely the reason for the high measured strain.

7. A review of the strain measurements, subsequent to Lift 3 excavation, indicates that both sections were behaving similarly, in general. The tensile force distribution along Row 3, prior to the excavation for Lift 4, is shown in Figures 82 and 83. The force distribution along Row 3 was near uniform and, generally, comparable in magnitude. This further indicates that the stiffness contribution of the existing pipe piles had become negligible. (A detailed discussion on pile-soil nail interaction is presented in Chapter 8, Section 8.1.)

8. The measured performance after the placement of the final lift of shotcrete on February 14, 1991, is of interest. Figures 36 and 37 show an increase in axial strains at Row 1, Section 1 measured on February 16, 1991. The increase was most pronounced at gauge locations near the shotcrete facing. A similar performance was measured at Section 2 (Figures 51 and 52).

   Figures 41 and 56 show little to no change in strains at Row 2 on February 16, 1991. This was followed by a decrease in strain on February 23, 1991 at gauges nearest the shotcrete facing. Rows 3 and 4 behaved similarly (Figures 44, 47, 59 and 62).

   Figures 50 and 65 show the tensile axial strain distribution at Row 5 is similar, in general, at both sections after February 16, 1991. However, while gauges D3/D4, near the facing at Section 2, measured a decrease in strain followed by a gradual increase through the short-term monitoring period, their counterparts, E1/E2, at Section 1 did not perform similarly. As shown in Figures 48 and 49, the top gauge, E1, measured a decrease in strain while the bottom gauge, E2, measured an increase. This reflects the presence of bending strain of approximately 38 microstrains. The measured performance, however, does not indicate that the shotcrete face is carried by the nail. Bending strains due to shotcrete dead weight would typically increase the strain in the top gauge, E1, and decrease the strain in the bottom gauge, E2. The bending strains continued to increase after construction with E1, measuring increased negative strain (compression) through the end of the short-term monitoring period (March 16, 1991). The bending strains increased to approximately 92 microstrains. This may indicate that, due to the increased inclination (25°) of Row 5, these elements are reacting partially in compression, similar to micropiles. This has been previously observed in steeply inclined soil nails *(21)*.

6.2 WALL FACING LOADS

6.2.1 Earth Pressure Cells

The earth pressure behind the shotcrete facing measured at 0.1 psi (700 Pa)during wall construction. The first post-construction reading on February 16, 1991 reflected an increase: the measured pressure was 0.4 and 0.5 psi (2,800 and 3,400 Pa) at Cell 1 and Cell 2, respectively. Additional post-construction readings showed a constant earth pressure of 0.5 psi (3,400 Pa) at both locations through the short-term performance evaluation. The cells are accurate in the 0.3 to 1.5 psi range (2,000 - 10,300 Pa). The maximum measured pressure of 0.5 psi (3,400 Pa) on the shotcrete facing appears to be insignificant and within the accuracy range of the cells. It is possible that earth pressure under-registration may have occurred due to improper installation of the cells; the significant soil arching over the cells; or the relatively flexible facing which allows increased deformation of the wall while reducing the lateral earth pressure applied to the shotcrete facing.

6.3 PILE LOADS

6.3.1 Measured Pile Strain and Computed Stress

The strain gauges attached on the excavation side of the piles were intended to evaluate stress induced in the piles as a result of the excavation and soil nail wall construction. Figure 84 shows the response of the strain gauges over the short-term monitoring period. The readings are generally small, in the range of $\pm$ 100 microstrain. Based on an elastic modulus for steel of 29,000 ksi (2.0×10^5 MPa), this corresponds to a nominal pile stress of not more than 3 ksi (21 MPa) (Figure 85).

The general trends of strain may be of more interest over this time period Following the excavation of Lift 1 all readings are initially negative, indicating compression. The strains gradually trended positive through the successive excavation steps. This may indicate an initial leaning (or free head rotation) of the pile cap, followed by a translation, without additional rotation. When compared to the short-term extensometer measurements (Figure 86), this appears consistent with the approximate doubling of pile cap movement following excavation of Lift 2.

6.4 DEFLECTIONS

6.4.1 Pile Cap Extensometer

Figure 86 shows the horizontal deflection of the pile cap with time through March 16, 1991.

The effect of the earth berm removal is clearly noticeable. Increases in the pile cap deflection were measured following the excavation for successive lifts. Of interest is the maximum deflection of 0.087 inch (2.2 mm) measured on December 15, 1990, following the excavation for Lift 2. This is less than the minimum relative movement of 1/8 inch (3.2 mm) between the soil and the nail that is required to mobilize the tensile capacity of the nails as reported in the literature. This agrees with the behavior of Row 1 at Section 1 following the excavation for Lift 2. As shown in Figure 38, little to no load was introduced in Row 1 at that time period.

Of great interest are the increases in pile cap deflection of 0.12 inch (3.0 mm) and 0.23 inch (5.8 mm) measured respectively on December 19 and 22, 1990 following the excavation for Lift 3. These could signal a mobilization of the tensile capacity of the soil nails as evidenced in the significant axial strains and equivalent force increases at Row 1 measured on December 18 and 21, 1990 (Figures 38 and 66).

The pile cap deflection measured 0.32 inch (8.1 mm) on January 30, 1991 following the excavation for the final lift. Post-construction readings through the short-term performance period indicate horizontal movement of the pile cap to be leveling-off at a deflection of 0.33 inch (8.4 mm) (Figure 86).

This maximum horizontal movement is well within the tolerances of the bridge deck expansion joint located two pier bents away at Pier-8 (Figure 2). However, it could become a concern if the pile cap and the superstructure movements were relative to each other rather than acting together as a unit. This could overstress the hinge dowels which are spaced at 12-inch (0.30 m) centers that connect the superstructure prestressed beams to the pile cap.

The superstructure connection to the pile cap was visually inspected for sign of distress subsequent to completion of construction. While the hinge dowels could not be inspected, as they were encased in both the pile cap and the diaphragm beam (Figure 6), the following observation was made: Concrete laitance from the time of casting the diaphragm beam gave the pile cap and the diaphragm beam a monolithic appearance. The fact that the concrete laitance was intact indicates the pile cap and the superstructure prestressed beams moved as a unit rather than relative to each other.

As noted earlier in this report, (Section 3.4) the piles were modeled by assuming the pile head was fixed against translation and that a horizontal strut force, introduced by the superstructure, was acting at the top of the pile model. This condition was assumed to exist at the onset of the excavation. The slight movement (0.087 inch (2.2 mm)) measured on December 15, 1990 following the excavation for Lift 2, validates this assumption. However, the significant increase in pile cap movement following the excavation of Lift 3 and subsequent lifts, indicates a softening in the horizontal strut force as the superstructure moved longitudinally under the effect of the lateral earth pressure of the embankment fill at the bridge abutment.

The maximum short-term horizontal movement the pile cap of 0.33 inches (8.4 mm), is less than the maximum short-term top-of-wall movement of 0.48 inch (12 mm) and 0.73 inch (18.5 mm) as measured by the slope inclinometers. This again suggests that the superstructure is acting as a strut, therefore limiting the horizontal movement of the pile cap.

6.4.2 Slope Inclinometers

Figures 87 and 88 show the vector magnitude deflection of inclinometers SD129 and SD130, over the short-term performance period. SD130 did not capture wall movements from the beginning of construction, as the original inclinometer casing at this location (SD128) was severed. The installation of SD130 and the first base reading took place following the excavation for Lift 2.

The base of the wall is about 19 feet (5.8 m) below the top of the casing at SD129, and about 22 feet (6.7 m) below the top of the casing at SD130. The readings shown begin (vertically) at the point coincident with the top-of-wall at each inclinometer location. Maximum top-of-wall movements were 0.44 inch (11 mm) (SD129) and 0.55 inch (14 mm) (SD130) on February 2, 1991 and January 26, 1991, respectively. Maximum top-of-wall movements increased to 0.48 inch (12 mm) (SD129) and 0.73 inch (18.5 mm) (SD130) at the end of the short-term performance evaluation.

This means that top-of-wall movements at the end of construction, 0.2 percent to 0.3 percent of the height of the wall at that location. These values are within the range published in the literature for acceptable soil nail wall deflections for granular soils.

The bulge shown in Figure 88 at 14 feet (4.3 m) below the top-of-wall is not easily explained. It may be due to variable ground conditions, or a variation in load carried by the nails above, at, and below this location.

The bulging may also be the result of soil arching stress redistribution due to the stress relief at the wall face after a lift has been cut and left unsupported. Soil arching may have redistributed stresses to the nails above the cut and to the base of the excavation. The stress redistribution in turn may have induced soil straining above and below the excavation lift and therefore soil deformation. Deflection plots for SD130 (Figure 88), show the bulge stopping above the base of the excavation as if the stress was being redistributed onto the nails above. Deflection plots for SD129 (Figure 87), show a less pronounced bulge below the base of the excavation as if stress redistribution took place below the excavated ground as well.

Both SD129 and SD130 show less than 0.06 inch (1.5 mm) lateral deformation below the base of the excavation.

6.4.3 Tiltmeters

A ceramic tiltplate was attached to the exposed face of the pile cap to monitor the rotation of the bridge pile cap as a result of excavation and removal of the existing approach fill end slope.

Figure 89 shows the pile cap rotation measured by all three tiltmeters during construction and through the short-term performance evaluation. Tiltmeters SDT2 and SDT3 indicate insignificant magnitudes of rotation that are within the precision error of $\pm$ 50 arc-second ($\pm$ 0.014 degrees) for the tiltmeter model used. For all practical purposes they can be assumed to equal zero.

Tiltmeter SDT1 shows a less than 0.11 degree (2.0 mrad) rotation during excavation and the lowering of the ground in front of the pile-supported bridge pile cap. Unfortunately, data from SDT1 was interrupted two months into construction when one seating peg on the ceramic tiltplate broke.

Based on the measured data and visual observation, pile cap rotation was insignificant.

6.4.4 Optical Survey Points

Survey points were established at two wall locations under the bridge to measure horizontal wall deflections during construction. The survey points were established at each nail location and midway between nails at each wall location.

The data generated was judged to be unreliable due to human error and changes in the optical survey equipment used to survey the wall. As a result the data was discarded.

6.4.5 Linear Variable Differential Transformers (LVDT's)

Data generated from the LVDT's was judged to be unreliable and subsequently discarded. The hardware was deemed unreliable and later abandoned.

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7.0 LONG-TERM MONITORING

The long-term performance evaluation was initially envisioned to continue for two years following completion of construction. It was later decided to extend the study for a total of five years in recognition of:

(a) wall movement and tensile load redistribution along the nails, in particular, which had not completely stabilized during the second year of monitoring,

(b) the excellent performance of the vibrating wire strain gauges and the reliability of the data,

(c) the need to develop a long-term database to further our understanding of soil nail wall behavior in similar applications, and

(d) the academic interest in using the study data to validate working stress (finite element) analyses, and to evaluate existing design methods and computer-aided soil nailing design programs.

Instruments were read on the average of once a week, from March 16, 1991 through July 13, 1991. Thereafter, they continued monthly through February 1992. They were then decreased to four-month intervals through March 1993, and to six-month intervals thereafter. The readings will continue at six-month intervals until the end of the five-year monitoring program (April 1996).

The long-term performance of the wall as measured by the strain gauges, the load cells, the inclinometers, and the extensometer are analyzed in this section. Data from the earth pressure cells and the LVDT's was judged to be unreliable and data from the tiltmeters of no further interest and, therefore, no discussion is provided on their long-term performance. The following dates are of interest:

a.	End of short-term performance evaluation	March 16, 1991
b.	Post-Scotts Mills earthquake evaluation	March 31, 1993
c.	End of present long-term performance evaluation	October 23, 1993

The reading taken on March 31, 1993 is of importance because it was taken following the March 23, 1993 Scott Mills earthquake that was centered approximately 40 miles (64 km) south of the Interstate-5 Swift-Delta soil nail wall. The magnitude 5.6 earthquake and was probably the largest earthquake in the historical record of northwest Oregon *(20)*. Strong motion instruments at the US Army Corps of Engineers Detroit Dam, approximately 22 miles (35 km) southeast of the epicenter recorded a peak acceleration of 0.06 g at the downstream toe of the dam, while an instrument located in a gallery within the dam recorded a peak of 0.18 g. A digital instrument in

Portland recorded a peak acceleration of 0.03 g, while older analog instruments in Portland and Vancouver, Washington, both recorded peak accelerations of 0.02 g. The stations in Portland are approximately 35 miles (56 km) north of the epicenter, and the Vancouver location is approximately 44 miles (62 km) north.

The earthquake caused significant structural damage to a number of unreinforced masonry buildings in and around the epicentral area of Molalla, Oregon. Minor damage, such as cracked plaster and foundations, were reported in the Portland metropolitan area. The total damage estimate was approximately $29 million.

7.1 NAIL LOADS

The long-term performance evaluation consisted of monitoring the change in the strain gauge data to compute the distribution of axial strain and the corresponding long-term tensile loads along each instrumented nail. The strain gauge readings taken immediately after the installation and the grouting of each nail served again as the base zero reading to which all long-term readings were compared.

7.1.1 Nail Strain Gauges

Long-term strain gauge readings presented in this report are through October 23, 1993. For the purpose of this report, this date is considered the end of the present long-term performance evaluation (approximately 2 ½ years after construction).

Figures 90 through 104 and Figures 105 through 119 show long-term strain measurements plotted with time for the top and bottom gauges, and the computed average axial strain at Rows 1 through 5.

Tables 9 and 10 show the average axial strain distribution along Rows 1 through 5 for selected dates through the long-term performance evaluation period.

A review of Table 9 shows that at the time monitoring was completed (October 23, 1993), strain measurements at Section 1 had generally increased by a factor of two over their values at the end of the short-term performance evaluation (March 16, 1991). Strains more than doubled at Row 5. Similar behavior was observed along Rows 3, 4 and 5 at Section 2 shown in (Table 10). This behavior is consistent with other instrumented studies *(10, 17)*. A comparison of the measured strain at Section 1 to the strains at Section 2 indicates that higher tensile strains were introduced at Section 1. This was expected since Section 1, under the bridge, has a 10-foot surcharge fill above the top of the wall.

The long-term increase in tensile strains could be attributed to

a) continued straining of the soil over time in response to the wall excavation, and

b) to additional creep and microcracking of the cement grout under tension which causes the grout to shed some of its load onto the steel nail. It is important to note that until the grout cracks, it does have some tensile strength and can carry a portion of the total load transferred by the surrounding ground to the combined bar/grout system.

The strain gauges, mounted on the bar, only measure the portion of the load transferred to the bar. The strain gauges will not measure the total nail load until the grout cracks in the immediate vicinity of the strain gauges allowing all the load to be transferred into the bar.

Microcracking of the cement grout renders it partially ineffective as a tensile load-carrying member. This in turn reduces the axial stiffness of the bar/grout system leading to additional wall deflections with time. Long-term readings of the slope inclinometers SD131 (Figure 149) and SD 132 (Figure 150), which were cast into the final wall facing under the bridge, also show additional long-term movements. It is notable that long-term strain readings correspond to the second zone on the load-strain relationship for the lab tested grouted nail (Figure 34). This zone reflected a softening in the axial stiffness of the grouted bar due to cracking of the cement grout. Therefore, this increase in nail strain is most likely due to a combination of creep and microcracking of the cement grout; redistribution of the total nail load from the cement grout to the steel; and the continued straining of the soil over time.

The strain gauges nearest the shotcrete facing generally showed a small increase in axial strain measurements through the long-term performance evaluation Tables 9 and 10. The strain gauges farther away from the shotcrete facing indicate a more significant increase in tensile strains.

While the magnitude of bending strain observed in gauges near the wall face remains relatively constant over the long-term monitoring period, there is a general trend for the bottom strain gauge reading to go from compressive to tensile (Figures 93, 94, 96, and 97).

It was stated earlier that the increase in long-term strain measurements by a factor of two over their values at the end of the short-term performance evaluation could partially be attributed to soil creep. Experience has shown the rate of soil creep in response to the wall excavation may decrease, remain constant or accelerate with time. It is important to determine whether the creep rate is constant or accelerating. Figures 120 through 129 represent a semi-log plot of measured average axial tensile strains versus time. They represent long-term creep curves at both sections. An upward concave creep curve typically indicates excessive creep; a constant slope creep curve (a straight line), indicates that critical creep tension has not been reached; a downward, concave creep curve indicates a decrease in creep with time. Figures 120 through 129 show that the rate of creep is generally constant at all rows.

7.1.2 Computed Nail Loads

Figures 130 through 139 show the change in computed tensile loads with time along the nails for Sections 1 and 2. Tables 11 and 12 show the computed tensile load distribution along the nails for selected readings during the long-term performance evaluation, including the percent change in computed loads for each set of selected consecutive readings.

Figures 140 through 141 summarize and compare the distribution of the computed tensile forces at all rows for Sections 1 and 2 at the end of the short-term (March 16, 1991) and the long-term (October 23, 1993) performance evaluations.

The general trend observed during the short-term performance evaluation continued during the long-term evaluation, mainly:

(a) additional increase in tension in the nails away from the shotcrete facing to a maximum and then a decrease toward the tip of the nail, and

(b) tensile forces in the nails continued to be maximum inside the reinforced earth mass away from the shotcrete facing.

Rows 1, 3, 4 and 5 at Section 1 continued to reflect a rather uniform force distribution (Figure 140) while Row 2 continued to have a distinct maxima. Figure 141 indicates that the force distribution is nearly uniform along Rows 3, 4 and 5 at Section 2. At the end of the long-term performance evaluation, the gauges nearest the shotcrete facing and corresponding to Rows 1 and 2 at Section 2 measured negative strain (compressive). This is reflected as zero tensile axial nail load in Figure 141.

7.1.3 Load Cells

Figures 142 and 143 show the load cell measurements with time during the long-term performance period at Sections 1 and 2, respectively. As mentioned earlier in Chapter 6, Section 6.1.3, this data is suspected to be influenced by the shotcrete facing.

The load cell readings at Row 1, Section 1, reflect a general increase in the nail load at the face during the first year of long-term monitoring. It subsequently leveled off at 3.7 kips (1.7 tonnes) which is equivalent to the initial lock-off load measured on December 9, 1990. The load cell readings at Rows 3 and 5, Section 1, show a small initial increase. The loads quickly leveled off at 1.0 kips (0.45 tonnes) and 1.3 kips (0.59 tonnes), respectively. This is well below the lock-off loads of 2.2 (1 tonnes) and 2.3 kips (1.04 tonnes) respectively (Table 7).

The performance at Section 2 was generally similar to the behavior measured at Section 1. The measured performance at Row 1 (Figure 143) reflects some fluctuation in the load intensity at the facing with the loads leveling off at 1.3 kips (0.59 tonnes). This is significantly lower than the initial lock-off load of 3.3 kips (1.5 tonnes). The measured performance at Row 3, reflects a small increase with time: from 2.7 kips (1.2 tonnes) at the end of the short-term monitoring, to

3.3 kips (1.5 tonnes) as of October 23, 1993. This is nearly equal to the initial lock-off load of 3.4 kips. The readings at Row 5 continued to reflect a leveling off at a service load of 1.7 which is less than the initial lock-off load of 2.5 kips (1.1 tonnes).

The load cells in general do not show an increase in face loading over time. The loads at the face as measured by the load cells range from 10 to 26 percent of the maximum nail load at Section 1. They range from 17 to 32 percent of the maximum nail load at Section 2. This is consistent with the typical range of 30 to 40 percent measured by others *(16)*.

7.1.4 Summary

This section summarizes general observations of relevance to the long-term performance evaluation as measured by the strain gauges and the load cells.

1. It is of interest to note how the long-term strains along the nails varied with time at Section 1. A review of Figures 92, 95, 98, 101 and 104 shows that the long-term increase in axial tensile strain (as measured by gauges A1/A2, B1/B2, C1/C2, D1/D2, and E1/E2, located nearest the shotcrete facing and corresponding to Rows 1 through 5, respectively) appears to have leveled off at the end of the present long-term performance evaluation. The strains measured by the gauges located one and two gauges removed from the shotcrete facing appear to be rapidly leveling off at Rows 1 through 4. All other gauges located farther away from the facing, including most of the gauges at Row 5, do not appear to be leveling off at the end of the present long-term performance evaluation. This indicates load transfer down the nail length with time, from the upper rows to the lower rows.

   Similar behavior was measured at Section 2 where tensile strain measurements appear to have generally leveled off more rapidly than at Section 1. Measurements taken from gauges located farthest from the shotcrete facing, Rows 2 through 4, continued to increase (Figures 110, 113, and 116). In addition, none of the strain gauges in Row 5, with the exception of gauges D3/D4 located nearest the shotcrete facing, leveled off at the end of the long-term evaluation (Figure 119). Most importantly, the behavior of Row 5 is consistent, at both Sections.

2. Long-term strain gauge measurements appear to have captured the effect of environmental loads, such as temperature fluctuations, on nail loads. In particular, this is evidenced at Section 2, which is located outside the bridge limits and subject to longer periods of sun exposure than Section 1.

   Axial strain readings at Section 2 increased during the first three summers and decreased during the following three winters Figures 107, 110, 113, 116, and 119,. The change in measured strain was most pronounced at the gauges located nearest the shotcrete face.

   Similar behavior, although generally less pronounced, was observed at Section 1 (Figures 92, 95, 98, and 101).

3. The Swift-Delta soil nail wall was inspected shortly after the Scotts Mills earthquake of March 25, 1993. The inspection revealed no cracking of the wall face, no indications of lateral movement or tension cracks in the fill slope behind the wall.

Tables 9 and 10 show the axial strain readings along each nail on March 31, 1993, six days after the earthquake. Tables 11 and 12 show the computed tensile forces corresponding to the March 31, 1993 strain readings and the incremental percent change in computed loads. The previous reading was on October 28, 1992. Tables 11 and 12 show a general significant increase in nail loads, in particular, among the upper rows as measured by the strain gauges located close to the shotcrete facing, and smaller increases toward the tip of the nails. However, it cannot be concluded that the increases are solely due to the earthquake.

7.2 PILE LOADS

7.2.1 Pile Strain Gauges

The results of long-term pile strain gauge monitoring are shown on Figure 144. This data is less consistent than the short-term readings. It would be expected that the gauges at the same distances below the pile cap would indicate similar strains. However, only the gauge 5 feet below the cap on the right pile indicates an appreciable pile strain over the 2 ½ years of monitoring . This gauge has shown an increase in strain of up to about 220 microstrain (tension) where the reading has apparently leveled off. The remaining three gauges have remained relatively constant at nominal values of less than about 50 microstrain, following initial variations during and shortly after construction.

7.2.2 Computed Pile Stress

Since it was not feasible to install strain gauges on both sides of the piles (facing the wall and away from the wall), average axial strain and true bending strain could not be measured. Figure 145 shows the stress in the pile calculated from the measured strain on the front of the steel pipe pile. It does not account for the possibility of composite section response due to the unreinforced concrete filling the pipe piles. The calculated increase in pile stress on the pile front face is about 6.4 ksi (44 MPa) (tension). The piles were designed to a maximum allowable compressive stress of 12 ksi (83 MPa). Assuming the compressive stress increase on the pile back side is equal to the tension stress increase on the front side, the maximum compressive stress currently existing in the piles is 18.4 ksi (127 MPa). This is well below the pile steel yield stress of 36 ksi (248 MPa).

7.3 DEFLECTIONS

7.3.1 Pile Cap Extensometer

Figure 146 shows the horizontal deflection at the bridge cap during the long-term performance evaluation period.

Long-term increases in the pile cap horizontal deflection were insignificant. The maximum measured deflection increased from 0.33 inch (8.4 mm) at the end of the short-term to 0.39 inch (9.9 mm) on January 20, 1992. Subsequent readings indicate that the pile cap horizontal movement stabilized at a maximum deflection of 0.39 inch (9.9 mm) (Figure 146).

With insignificant post-construction movement in the bridge pile cap (as indicated by the single point extensometer), and 0.36 inch (9.1 mm) of maximum post-construction wall movement (as measured by slope inclinometer SD132 on June 6, 1992), it appears that the reinforced soil movement due to soil straining is uncoupled from the bridge pile cap movement.

7.3.2 Slope Inclinometers

Long-term slope inclinometer data are presented in Figures 147 through 150 for the two inclinometers located away from the bridge, as well as the inclinometer tubes cast into the final wall facing (second shotcrete layer) under the bridge after completion of lift excavations and soil nailing.

Away from the bridge, maximum top-of-wall movements measured 0.85 inches (22 mm)(SD130) and 0.95 inches (24 mm) (SD129). At the higher wall section (SD130), these deflections are about 0.35 percent of the wall height. This is still a reasonable amount of deflection for acceptable wall performance. At the shorter wall section (SD129), the top-of-wall movements are in the range of 0.6 percent of the wall height. This may seem to be of concern, however, 0.4 inches (10 mm) of the total deflection occurs within 1 foot (305 mm) of the top of the wall. This, along with some observed erosion of the near surface materials in the 2:1 fill slope above the wall indicates that solifluction, or near surface soil movement due to gravity, may be occurring. Additionally, foot traffic in the area may strongly influence the near-surface readings of both SD130 and SD129. The total vector magnitude displacements reported above and shown on the plots includes a component that is parallel to the wall face. In the upper 5 feet (1.5 m) of inclinometer casing, this parallel component is large enough to support the conclusion that some near surface solifluction is occurring and some topsoil is displacing to the west, parallel to the wall face.

Under the bridge, inclinometers SD131 and SD132 have indicated continued wall movement, in a leaning fashion, well into the second year of performance evaluation. Total top-of-wall deflections measured 0.35-inch (8.9 mm) by both inclinometers. The top-of-wall deflections away from the bridge nearly doubled during the long-term performance evaluation period. Furthermore, the extensometer did not indicate significant pile cap movement after completion of construction, while the inclinometers in the wall facing have shown a continued straining of the wall facing elements.

When the total pile cap deflection measured by the extensometer is added to post-construction deflection measured by SD132, the total wall deflection is about 0.75 inches (20 mm) under the bridge. This is less than the overall deflection of 0.85 inches (22 mm) measured by SD130 at a similar wall height away from the bridge. Furthermore, it could be assumed the difference in overall wall deflection would have been even higher had inclinometer SD130 captured the wall movement from the beginning of construction, and that associated with the excavation of the top two lifts. Smaller overall horizontal movements were measured even though 10 feet (3 m) of bridge abutment surcharge was acting on top of the wall section under the bridge. This appears to indicate that existing pipe piles are acting as stiffening members, limiting wall movements under the bridge.

Although the pile cap seems to have stopped translation essentially with the last step of excavation, the soil and soil nail wall elements continued to strain over a period of more than one year following construction.

Inclinometer readings at SD129, SD131 and SD132 taken on March 31, 1993, six days after the Scotts Mills earthquake show wall movements into the fill backslope (Figures 147, 149, and 150). A subsequent reading of these instruments as well as SD130 on October 26, 1993, shows further movement into the fill backslope. It was later concluded that the inward measured movement was due to a faulty inclinometer probe rather than the earthquake. Further, it may be inferred since the magnitude of both movements are small, these observations may be more indicative of the precision of the instruments and human error rather than actual wall movement.

8.0 SUMMARY AND CONCLUSIONS

8.1 PILE-SOIL NAIL INTERACTION

The following is a an interpretation of the interaction between the soil nails and the piles. It deals only with instrumentation data from beneath the bridge, or Section 1 data, except as noted for comparison with Section 2.

A preliminary understanding of the pile-soil nail interaction was developed through a review of the computed tensile loads at Row 1, the measured horizontal deflection of the pile cap as represented by the extensometer readings, and the computed tensile loads at Row 3 for Sections 1 and 2.

Figure 82 shows the nail force distribution and the change in nail forces with time for Rows 1 through 3 following the excavation for Lifts 2 and 3. The excavation for Lift 2 was performed on December 10, 1990 and lowered the ground 5 feet (1.5 m), to a total of 6.5 feet (2 m) below Row 1. The excavation for Lift 2 introduced little to no load into Row 1. The maximum computed load, as represented by the reading on December 13, 1990, was 1.3 kips (0.6 tonnes) and occurred at the strain gauges located 2 feet (0.6 m) from the face. No measurable load was introduced into the remaining strain gauges. The 1.3 kips (0.6 tonnes) load may have been introduced when the nail was partially pretensioned by the tightening of the nut against the bearing plate prior to the excavation for Lift 2 to ensure that the shotcrete facing was in good contact with the ground. The load cell reading at Row 1 indicated a preload of 3.75 kips (1.7 tonnes).

The excavation for Lift 3 was performed on December 17, 1990 and the ground was lowered an additional 2 feet (0.6 m). Owing to construction delays due to cold weather conditions, no additional excavation was performed until January 8, 1991. Several readings were taken during this time interval.

The first of those readings were taken on December 18, 1990. The maximum computed tensile load in Row 1 was 2 kips (0.9 tonnes) and occurred 10 feet (3 m) from the face. The computed tensile load in Row 1 at 2 feet (0.6 m)from the facing was 1.7 kips (0.8 tonnes). Figure 82 indicates a significant increase in the nail forces at Row 1 (December 21, 1990) followed by a slow increase (January 5, 1991). The maximum computed nail force in Row 1 was 6.8 kips (3 tonnes) on January 5, 1991 and occurred 2 feet (0.6 m) from the face.

The observed behavior at Section 1, Row 1, following the excavation for Lifts 2 and 3, does not agree with published results of several full-scale Instrumented soil nail walls *(16, 17, 18, and 19)*. Such results indicate that the increase in nail tension is due primarily to excavation lifts subsequent to the installation of the nail. Published results also suggest a rapid increase in nail forces immediately after the excavation for a lift, followed by a slow increase in nail forces with time as the next lift nails and the corresponding shotcrete facing are installed.

The observed behavior at the Interstate-5 Swift-Delta soil nail wall, as represented by Section 1, suggests that the arching of the soil mass following the excavation for Lift 2, resulted in a transfer of stress into the existing pipe piles as evidenced by the introduction of little to no load into Row 1. Immediately following the excavation for Lift 3, the arching of the soil mass may have introduced additional load into the pipe piles. Most importantly, it also resulted in a stress transfer into Row 1 as evidenced by the reading on December 18, 1990. Of great interest is the significant increase in axial strains and their equivalent nail forces that were measured several days later, on December 21, 1990, as Row 3 and the shotcrete facing corresponding to Lift 3 were being installed. The nail forces in Row 1 more than doubled in magnitude (Figure 82). This significant increase is most likely due to a decrease in the lateral carrying capacity (i.e. bending stiffness) of the existing steel pipe piles subsequent to excavating Lift 3. This in turn appears to have caused additional wall movement as evidenced in the extensometer measured reading of 0.23 inches (5.8 mm) taken on December 22, 1990 (Figure 86). This appears to indicate that a limiting lateral load-carrying capacity had been reached in the piles, and that excess load was now being carried by Rows 1 and 2 (Figure 82).

The decrease in the lateral carrying capacity of the existing steel pipe piles appears to have caused a decrease in the overall soil/structure (wall) stiffness at the bridge portion of the soil nail wall. Most importantly, any additional contribution of the steel pipe piles to the overall wall stiffness seems to have stopped subsequent to Lift 3. Figures 82 and 83 show the nail force distribution along Row 3 for the first two readings associated with Lift 3 (December 27, 1990 and January 5, 1991). The December 27th reading was taken a few days after the first stage grouting of the drill hole (Table 1) and prior to the placement of the shotcrete face corresponding to Lift 3. The force distribution along Row 3 at Sections 1 and 2 was nearly uniform; the maximum computed nail force was 1.1 kips (0.5 tonnes) and 1.5 kips (0.7 tonnes) at Sections 1 and 2, respectively. The January 5th reading was taken the day after second stage grouting (Table 1), and placement of the shotcrete face at Row 3. Figures 82 and 83 show a small increase in Row 3 tensile forces at Sections 1 and 2 on that date. Most importantly, the nail forces represented by these two readings were introduced into Row 3 prior to the excavation for Lift 4. The tensile force distribution along these nails (Figures 82 and 83) appear to indicate that both wall sections were performing similarly. This is further evidence that additional stiffness contribution of the steel pipe piles seems to have ceased once the excavation proceeded below Lift 3.

It must be emphasized that the observed behavior at the Interstate-5 soil nail wall is for the defined conditions of a single row of piles spaced at 4.5 feet (1.4 m), located immediately behind the wall, and founded at depth in a dense stratum. A different pile geometry, number of rows, pile spacing, and subsurface condition would most likely lead to a different pile-soil nail interaction.

8.2 LOAD TRANSFER TO PILES

The effects of load transfer to the piles has been one of the more difficult interpretations of the Swift-Delta data. Conceptually, three modes of pile/pile cap behavior are possible: 1) true fixed-head conditions where the piles and pile cap are restrained from both lateral translation and rotation due to strutting action of the bridge superstructure, 2) true free-head conditions where the pile cap is free to both translate laterally and rotate under a lateral load, and 3) some intermediate condition between these two boundary conditions. Unfortunately, measured data from the instrumentation is not conclusive as to which condition may have existed.

The pile strain gauge located 5 feet (1.5 m) below the cap on the instrumented westerly pile indicates that tensile stress is present on the front (wall) side of the pile. This would indicate that rotation of the pile cap is prevented by strutting action of the bridge superstructure. The short-term zero readings of two of the three tiltmeters also seem to support the conclusion that pile cap rotation is being restrained. However, the other three strain gauges attached to the piles do not show appreciable strain or associated stress (either positive or negative). This would seem to indicate that bending stress in the piles is being accommodated at a depth lower than the gauges which would account for the observed pile cap horizontal deflection as measured by the extensometer. The third tiltmeter SDT-1, also indicates that slight rotation of the pile cap has occurred.

The following interesting correlation was noted: the amount of expected pile cap rotation for the lateral translation measured by the extensometer, corresponds well with the last reading from tiltmeter SDT-1. That is, two months into the monitoring program, 0.3-inch(7.6 mm) of horizontal top-of-wall movement was measured by the extensometer. Over a 19-foot (5.8 mm) wall height, the magnitude of rotation would be expected to be about 0.08 degrees (1.4 mrad) for this horizontal deflection. The two month reading of tiltmeter SDT-1 was about 0.08 degrees (1.4 mrad). Unfortunately, the extensometer and tiltmeter SDT-1 are located approximately 80 feet (24 m) apart. The tiltmeter located near the extensometer (SDT-2) did not indicate similar magnitudes of pile cap rotation.

Theoretically, there is no reason to believe that the pile cap would be restrained from either translation or rotation, since the superstructure prestressed beams/pile cap joint is constructed as a true hinge (Figure 6). The bridge deck expansion joint located two pier bents away still remains free to accommodate additional movement. Judgment suggests that the true behavior is somewhere between the extremes of fixed- and free-head conditions. The behavior is also not likely to be uniform along the length of the cap, due to varied wall height, nail locations (vertically, with respect to the pile cap) and the possibility of variable soil strengths and stiffness.

In the absence of more definitive data, a more conclusive determination of the behavior of piles and bridge pile cap does not seem warranted.

8.3 COMPARISON OF MEASURED AND PREDICTED NAIL LOADS

Figures 151 and 152 show the maximum calculated nail loads within the reinforced earth mass for each nail at Sections 1 and 2 as measured by the nail strain gauges and the theoretical (design) nail loads derived from the Davis method. The maximum nail loads were plotted versus the depth of excavation. Figures 151 and 152 also show the maximum measured nail loads at the facing as measured by the load cells, and those predicted by use of Terzaghi and Peck's empirical earth pressure diagram for a braced excavation as was suggested by Juran and Elias *(10)* to provide reasonable approximation of maximum service nail loads distribution.

The Davis method clearly overestimates the nail loads in the lower nails in comparison to the nail loads calculated from the field-measured axial strains and those obtained using the Terzaghi and Peck empirical diagram. In addition, the Davis method is likely to underestimate the nail loads in the upper nails. The overestimation of the nail loads is not surprising once the limitations of the method are understood. The Davis method assumes the frictional resistance at each nail is calculated along the effective nail length behind the critical slope stability failure surface. Therefore, as the excavated ground is lowered, the critical failure surface extends deeper within the reinforced soil mass and as a consequence the effective length of the upper nails decreases while it increases for the lower nails. This results in a triangular distribution of predicted nail loads that increases with increasing depth of excavation. This is not consistent with the measured-load distribution on this project nor with other cases *(16, 18, 19)* where the higher tensile loads in the upper nails are related to the excavation of successive lifts, while at the base of the wall, the strains are transferred mainly to the foundation base and felt to a lesser extent by the lowest nail. Another factor contributing to the overestimation of nail loads by the Davis method is the use of a safety factor on the soil properties to compute the predicted maximum loads. Also, the Davis method overestimation of the nail loads in the upper nails at Section 1 may also in part be the result of modeling the 10-foot (3m) of bridge abutment surcharge as an assumed additional wall height with fictitious nails having zero frictional resistance within this height.

Figures 151 and 152 show that, in general, the Terzaghi and Peck empirical earth pressure diagram underestimated the service nail loads derived from field-measured axial strains. This is particularly true for the long-term monitored maximum nail loads. An initial review of Figure 151 appears to indicate that the measured variation of maximum tensile loads with depth are similar to the empirical diagram. It also indicates that in-service working loads could be assumed to be represented by the maximum nail loads calculated from measured strains at the end of the short-term monitoring period, soon after all external loads have been applied to the nails and before grout creep had much influence on the nail loads. The larger nail loads calculated from measured strains during the long-term monitoring period may then be considered as upper bound nail loads. Unfortunately, inclinometers SD131 and SD132 show long-term movements which indicate continued straining of the soil over time. Since soil creep increases the actual nail forces, it is suspected this empirical earth pressure approach may underestimate in-service working nail loads when soil creep is a factor. From a practical standpoint, the predicted services loads, from Terzaghi and Peck empirical earth pressure diagram, could be viewed to be in reasonable agreement with measured loads to date.

It is also interesting to note the distribution of the maximum measured tensile loads with depth at Section 1 is nearly rectangular in shape (Figure 151). This type of load distribution may be due to the presence of a relatively homogeneous soil over the whole excavation depth as discussed in Chapter 3, Section 3.1. A similar observation could be made at Section 2, with the exception that the uppermost nail measured a low tensile force. This could be attributed to the non-uniform condition and presence of concrete rubble and metal debris in the upper soil layers as indicated in Chapter 3, Section 3.1.

Another interesting observation is that the measured maximum nail forces at Section 2 are nearly comparable in magnitude to those at Section 1, even though Section 1 has 10 feet (3 m) of abutment vertical surcharge fill while Section 2 has a 2:1 slope surcharge. Such resemblance may be due to a combination of the low load carried by the uppermost nail at Section 2, different ground conditions, and the presence of the pipe piles at Section 1.

The load cell data indicates the loads transferred to the facing are substantially lower than the maximum tensile forces measured in each row of nails. Many current design methods assume substantially higher loads will be transferred to the shotcrete facing than those measured at Swift Delta. An assumed load transfer equivalent to Terzaghi and Peck's Empirical rectangular earth pressure diagram, as was assumed for design of the wall facing on this project, is a very conservative design for the shotcrete facing (Figures 151 and 152).

8.4 COMPARISON OF MEASURED MAXIMUM NAIL FORCES AND THEORETICAL LOG-SPIRAL FAILURE SURFACE

Figures 153 and 154 show the nail force distribution along the nails at Sections 1 and 2, respectively, for strain measurements immediately following the completion of the excavation and soil nailing. (January 28,1991), at the end of the short-term performance evaluation (March 16, 1991), and at the end of the present long-term performance evaluation (October 23, 1993). The nail forces were calculated using field-measured strains and the force-strain relationships described in Chapter 5. This method of converting from strain to force does not affect the location of the maximum nail force. Also shown in Figures 153 and 154 are the log-spiral critical (minimum F.S.) slope stability failure surfaces predicted by the Davis method at both sections. The calculated peak forces along the nails locate the locus of maximum strain in the reinforced soil mass.

8.5 CONCLUSIONS

- The measured strains, corresponding nail forces, and wall movements as measured by slope inclinometers and single point extensometer, indicate the soil nail wall is performing well within structural safety limits for both the wall and the bridge abutment.

- The measured performance suggests the maximum nail forces as a function of depth within the reinforced mass to be relatively constant with depth.

- Soil nailing, a passive retaining system, requires relative horizontal soil movement in order for loads to be mobilized in the nails. A relative movement in the range of 1/8 to 1/4 inch (3.2 to 6.3 mm), as measured by the pile cap mounted extensometer, was the minimum necessary movement to begin mobilizing the tensile capacity of the soil nails under the bridge abutment. This compares well to the minimum relative movement of 1/8 inch (3.2 mm) reported in the literature for soil nailing in general cut slope applications.

- Nail tension varies over the whole length of the nail and can be described as an increase in tension away from the shotcrete face to a maximum, and a decrease toward the tip of the nail. Most importantly, the tensile forces are maximum inside the soil nailed earth mass at some distance from the facing. Implied high forces near the shotcrete facing are primarily due to measured bending in some of the nails.

- The measured strain history plots at both instrumented sections reflect continued soil movement (creep) subsequent to completion of wall construction. This was also evident in the long-term inclinometer readings, in particular, those from SD131 and SD132.

- The Davis method clearly overestimates the nail forces in the lower nails and underestimates the nail forces in the upper nails. The predicted near-triangular distribution of nail forces that increase with depth of excavation is not consistent with the generally uniform force distribution measured on this and other projects.

- Terzaghi and Peck's braced cut empirical earth pressure diagram appears, in general, to be in reasonable agreement with measured loads to date. Since soil creep increases the nail forces, it is suspected this empirical earth pressure approach may underestimate the in-service working nail forces measured on this project, in particular when soil creep is a factor.

- The maximum measured top-of-wall movements of 0.004 H under the bridge, and 0.005 H away from the bridge are slightly higher than the typically observed 0.002 H and 0.003 H range.

- The vertical piles may be strengthening the reinforced soil mass by providing additional ground support. Consequently, the overall wall stiffness at the bridge portion of the wall was enhanced by the close spacing of the piles, the soil nails and the shotcrete facing.

- The existing pipe piles appear to be limiting wall deflections under the bridge. Maximum top-of-wall movements of 0.75 inch (20 mm) under the bridge was less than the measured top-of-wall movement of 0.85 inch (22 mm) at a similar wall height away from the bridge; smaller movement was measured even though 10 feet (3 m) of bridge abutment vertical surcharge fill was acting on top of the wall section under the bridge.

- The maximum pile cap horizontal movement as measured by the extensometer was 0.32 inch (8.1 mm) at the end of lift excavation and soil nailing (January 28, 1991). It measured 0.33 inch (8.4 mm) (at the end of the short-term performance evaluation (March 16, 1991). Long-term increases in the pile cap horizontal deflection were insignificant with long-term readings indicating the pile cap movement to have stabilized at a maximum deflection of 0.39 inch (9.9 mm). In comparison, slope inclinometer SD132 (embedded in the shotcrete face under the bridge abutment subsequent to completion of excavation and soil nailing) measured an additional 0.36 inch (9.1 mm) of horizontal movement over the long term. This indicates the soil nailed soil movement is uncoupled from the bridge pile cap movement during the post-construction monitoring.

- Soil arching following initial excavation to a certain depth appears to transmit the loads to the vertical piles with little to no load introduced onto the soil nails. Additional ground excavation causes a decrease in the lateral carrying capacity of the piles leading to a reduction in the overall wall stiffness which in turn results in additional wall movement. This increase in wall movement mobilizes the nails and causes a redistribution of forces from the pile elements to the soil nail wall elements.

- The inferred stresses introduced in an existing pipe pile and measured by the pile mounted strain gauges indicate the piles are performing well within the working stress for the piles. It should be noted, however, the limited and somewhat inconsistent pile strain data measurements made on this project do not lead to a final conclusion.

8.6 SUGGESTIONS FOR FURTHER RESEARCH

- Additional research is needed to develop a more rational approach to determine the total required nail force and corresponding distribution among the individual nails. Limit equilibrium methods are currently used to design soil nail walls. They have provided satisfactory designs for soil nailing in-situ soils. Unfortunately, existing limit equilibrium methods, including the Davis method, do not provide good estimates of the magnitude and distribution of maximum nail forces and are not able to distribute the total force explicitly. They also do not directly address the issue of wall deformation, a primary consideration for soil nail walls.

- Better methods are needed to predict how the concrete grout contributes to the axial stiffness of the soil nails. Measured strain readings were converted to equivalent nail forces using force-strain relationships developed from laboratory tests on grouted nails. This procedure has limitations; the load-strain relationships were derived by applying the axial loads directly to the nail reinforcement, while field-measured strains are transferred through soil-grout interface shear to the bar reinforcement, on which the strain gauges are attached. In addition, this procedure does not account for creep and stress relaxation of the cement grout, nor for loading the soil nails in stages at different curing ages.

- Two new methods were recently proposed to better predict the total equivalent nail force. The first method *(22)* accounts for both curing and creep of concrete to estimate an effective modulus for the grout and to determine the tensile loads in the grout. This method interprets the concrete grout response from the measured strains at the grout-rebar interface and estimates the actual force developed in the composite soil nail. The second method requires the installation of a grout breaker, a teflon disk or equivalent, at each nail-mounted strain gauge location to ensure that the measured strain in the steel rebar is the total nail load at that point. This method is being developed as part of the Federal Highway Administrations (FHWA) Demonstration Project (DP) 103 on soil nail walls and should help eliminate data interpretation problems associated with the grout steel rebar interaction.

- Additional full-scale instrumentation of soil nail walls used to retain bridge fill embankments are needed. This will further our understanding of soil-structural interaction in soil nail retaining wall and pile foundation systems. Of particular interest are bridge applications with multiple rows of piles, larger pile spacing, piles located farther away from the soil nail wall, or a combination thereof. Expanding our database should also help validate future computer modeling techniques of soil nail structural interaction problems.

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APPENDIX A: TABLES AND FIGURES

TABLE 1
SCHEDULE OF INSTRUMENTED NAIL INSTALLATION

Row	Section - 1	Section - 2
1	12-3-90 (a) 12-6-90 (b)	12-4-90 (a) 12-5-90 (b)
2	12-13-90 (a) 12-15-90 (b)	12-13-90 (a) 12-15-90 (b)
3	12-21-91 (a) 1-4-91 (c)	12-21-90 (a) 1-4-91 (c)
4	1-14-91 (a) 1-15-91 (d)	1-14-91 (a) 1-15-91 (d)
5	1-18-91 (a) 1-21-91 (e)	1-18-91 (a) 1-21-91 (e)

a) Low pressure grouting of bore hole following nail installation

b) Topping bore hole with tremie grout, dry packing block-out around nail head and placement of bearing plate.

c) Topping bore hole with shotcrete mix and placement of bearing plate.

d) Topping bore hole with tremie grout, shotcreting blockout around nail head and placement of bearing plate grout.

e) Dry packing around nail head and placement of bearing plate.

TABLE 2
CALIBRATION TEST NAIL LOAD-STRAIN RELATIONSHIP

UNGROUTED DYWIDAG BAR			GROUTED NAIL		
LOAD (Kips)	G0 (Microstrain)	G1 (Microstrain)	LOAD (Kips)	G0 (Microstain)	G1 (Microstain)
1	35	45	.1	2	2
2	72	93	.2	3	4
3	104	134	.3	5	5
4	138	178	.4	6	7
5	181	223	.5	8	9
6	219	264	.6	10	11
7	258	305	.7	11	13
8	295	344	.8	13	15
9	339	385	.9	15	16
10	381	425	1	17	18
11	423	462	1.1	18	20
12	462	508	1.2	20	22
13	504	546	1.3	22	23
14	545	581	1.4	24	25
15	588	622	1.6	27	19
16	625	661	1.8	31	33
17	****	****	2	35	37
18	707	741	2.2	39	41
19	748	783	2.4	43	45
20	790	819	2.6	47	54
21	828	864	2.8	51	58
22	877	907	3	56	64
23	914	942	3.2	60	68
24	955	982	3.4	69	73
25	992	1029	3.6	74	78
26	1034	1063	3.8	78	83
27	1074	1108	4	83	88
28	1117	1147	4.2	87	94
29	1155	1193	4.4	92	99
30	1198	1229	4.6	97	105
31	1240	1270	4.8	102	110
32	1281	1310	5	107	116
33	1322	1352	5.2	113	122
34	1363	1392	5.4	119	129
35	1401	1433	5.6	125	135
36	1444	1473	6	132	141
37	1485	1514	6.2	138	148
38	1527	1554	6.4	144	154
39	1567	1592	6.6	152	161
40	1610	1635	7	165	174
			7.4	181	189
			7.8	194	203
			8.2	210	219
			8.6	222	230
			9	235	244
			9.5	251	261
			10	268	278
			10.5	285	295
			11	300	311
			12	333	344
			13	365	380
			14	398	409
			15	429	442
			20	588	602
			25	810	831
			28	990	981
			30	1072	1056
			35	1398	1290
			40	1603	1480

TABLE 3
SHORT TERM PERFORMANCE
AXIAL STRAIN DISTRIBUTION ALONG NAILS AT SECTION 1

Row No.	Date	**Average Axial Strain (Microstrain)**				
		A1 & A2	A3 & A4	A5 & A6	A7 & A8	A9 & A10
1	1-28-91 (a)	108	124	151	128	76
	2-16-91 (b)	132	143	174	143	90
	3-16-91 (c)	129	150	187	158	105
		B1 & B2	B3 & B4	B5 & B6	B7 & B8	B9 & B10
2	1-28-91 (a)	91	116	102	157	108
	2-16-91 (b)	103	124	115	183	126
	3-16-91 (c)	89	121	118	195	141
		C1 & C2	C3 & C4	C5 & C6	C7 & C8	C9 & C10
3	1-28-91 (a)	193	119	142	132	67
	2-16-91 (b)	212	141	163	154	87
	3-16-91 (c)	192	145	172	170	105
		D1 & D2	D3 & D4	D5 & D6	D7 & D8	D9 & D10
4	1-28-91 (a)	162	177	175	169	127
	2-16-91 (b)	169	183	194	192	145
	3-16-91 (c)	150	187	216	213	176
		E1 & E2	E3 & E4	E5 & E6	E7 & E8	E9 & E10
5	1-28-91 (a)	87	7	46	50	17
	2-16-91 (b)	58	2	53	67	29
	3-16-91 (c)	66	20	77	96	56

(a) Following Completion of Excavation and nail installation (1-24-91)

(b) Following Completion of Construction (2-14-91)

(c) End of Short Term Performance Evaluation

TABLE 4
SHORT TERM PERFORMANCE
AXIAL STRAIN DISTRIBUTION ALONG NAILS AT SECTION 2

Row No.	Date	**Average Axial Strain (Microstrain)**			
		A1 & A2	A3 & A4	A5 & A6	A7 & A8
1	1-28-91 (a)	-73	- 6	42	64
	2-16-91 (b)	-43	24	69	83
	3-16-91 (c)	-71	5	67	88
		A9 & A10	B1 & B2	B3 & B4	B5 & B6
2	1-28-91 (a)	86	220	-34	35
	2-16-91 (b)	80	260	-11	57
	3-16-91 (c)	59	241	- 9	70
		B7 & B8	B9 & B10	C1 & C2	C3 & C4
3	1-28-91 (a)	124	40	26	64
	2-16-91 (b)	151	57	44	93
	3-16-91 (c)	154	70	59	112
		C5 & C6	C7 & C8	C9 & C10	D1 & D2
4	1-28-91 (a)	216	142	100	68
	2-16-91 (b)	273	189	141	102
	3-16-91 (c)	271	218	176	138
		D3 & D4	D5 & D6	D7 & D8	D9 & D10
5	1-28-91 (a)	41	- 6	11	- 4
	2-16-91 (b)	50	27	33	7
	3-16-91 (a)	73	66	65	31

(a) Following Completion of Excavation and nail installation (1-24-91)
(b) Following Completion of Construction (2-14-91)
(c) End of Short Term Performance Evaluation

TABLE 5
SHORT TERM PERFORMANCE
AXIAL TENSILE LOAD DISTRIBUTION ALONG NAILS AT SECTION 1

Row No.	Date	**Axial Tensile Load (Kips)** (Including Total percent change subsequent to 1-28-91)									
		A1 & A2		A3 & A4		A5 & A6		A7 & A8		A9 & A10	
1	1-28-91 (a)		4.8		5.6		6.5		5.7		3.4
	2-16-91 (b)		5.9		6.3		7.2		6.3		4.1
	3-16-91 (c)	21%	5.8	16%	6.5	17%	7.6	18%	6.7	38%	4.7
		B1 & B2		B3 & B4		B5 & B6		B7 & B8		B9 & B10	
2	1-28-91 (a)		4.1		5.2		4.6		6.7		4.8
	2-16-91 (b)		4.6		5.6		5.2		7.5		5.7
	3-16-91 (c)	-2%	4.0	4%	5.4	15%	5.3	18%	7.9	29%	6.2
		C1 & C2		C3 & C4		C5 & C6		C7 & C8		C9 & C10	
3	1-28-91 (a)		7.8		5.4		6.2		5.9		3.0
	2-16-91 (b)		8.4		6.2		6.9		6.6		3.9
	3-16-91 (c)	0%	7.8	17%	6.3	16%	7.2	20%	7.1	57%	4.7
		D1 & D2		D3 & D4		D5 & D6		D7 & D8		D9 & D10	
4	1-28-91 (a)		6.9		7.3		7.3		7.1		5.7
	2-16-91 (b)		7.1		7.5		7.8		7.8		6.3
	3-16-91 (c)	-6%	6.5	4%	7.6	16%	8.5	18%	8.4	28%	7.3
		E1 & E2		E3 & E4		E5 & E6		E7 & E8		E9 & E10	
5	1-28-91 (a)		3.9		0.3		2.1		2.2		0.8
	2-16-91 (b)		2.6		0.1		2.4		3.0		1.3
	3-16-91 (c)	-23%	3.0	200%	0.9	67%	3.5	95%	4.3	212%	2.5

(a) Following Completion of Excavation and nail installation (1-24-91)
(b) Following Completion of Construction (2-14-91)
(c) End of Short Term Performance Evaluation

TABLE 6
SHORT TERM PERFORMANCE
AXIAL TENSILE LOAD DISTRIBUTION ALONG NAILS AT SECTION 2

Row No.	Date	**Axial Tensile Load (Kips)** (Including <u>Total</u> percent change subsequent to 1-28-91)			
		<u>A1 & A2</u>	<u>A3 & A4</u>	<u>A5 & A6</u>	<u>A7 & A8</u>
1	1-28-91 (a)	0.0	0.0	1.9	2.9
	2-16-91 (b)	0.0	1.1	3.1	3.7
	3-16-91 (c)	0.0	0.2	58% 3.0	38% 4.0
		<u>A9 & A10</u>	<u>B1 & B2</u>	<u>B3 & B4</u>	<u>B5 & B6</u>
2	1-28-91 (a)	3.9	8.6	0.0	1.6
	2-16-91 (b)	3.6	9.9	0.0	2.6
	3-16-91 (c)	-31% 2.7	8% 9.3	0.0	94% 3.1
		<u>B7 & B8</u>	<u>B9 & B10</u>	<u>C1 & C2</u>	<u>C3 & C4</u>
3	1-28-91 (a)	5.6	1.8	1.2	2.9
	2-16-91 (b)	6.5	2.6	2.0	4.2
	3-16-91 (c)	18% 6.6	77% 3.2	117% 2.6	72% 5.0
		<u>C5 & C6</u>	<u>C7 & C8</u>	<u>C9 & C10</u>	<u>D1 & D2</u>
4	1-28-91 (a)	8.5	6.3	4.5	3.0
	2-16-91 (b)	10.2	7.7	6.2	4.6
	3-16-91 (c)	20% 10.2	36% 8.6	62% 7.3	103% 6.1
		<u>D3 & D4</u>	<u>D5 & D6</u>	<u>D7 & D8</u>	<u>D9 & D10</u>
5	1-28-91 (a)	1.8	0.2	0.5	0.0
	2-16-91 (b)	2.3	1.2	1.5	0.3
	3-16-91 (c)	83% 3.3	3.0	480% 2.9	1.4

(a) Following Completion of Excavation and nail installation (1-24-91)
(b) Following Completion of Construction (2-14-91)
(c) End of Short Term Performance Evaluation

TABLE 7
SHORT TERM PERFORMANCE
CHANGE IN THE RATIO T_o / T_{max} WITH TIME AT SECTION 1

Row No.	Date	Load Cell Reading at the Facing	Maximum Load in Nail	Ratio T_o / T_{max}
1	(a) 2-07-90	3.7	0.0	---
	12-18-90	3.4	2.0	1.7
	12-21-90	3.8	6.4	0.6
	1-10-91	4.3	7.1	0.6
	(b) 1-28-91	4.4	6.5	0.7
	(c) 2-16-91	4.3	7.2	0.6
	(d) 3-16-91	2.7	7.6	0.4
(Ʌ) 2	(b) 1-28-91	2.9	6.7	0.4
	(c) 2-16-91	3.1	7.5	0.4
	(d) 3-16-91	1.85	7.9	0.2
3	(a) 1-05-91	2.2	2.0	1.1
	1-10-91	1.6	3.2	0.5
	1-16-91	1.9	6.7	0.3
	(b) 1-28-91	1.7	7.8	0.2
	(c) 2-16-91	1.9	8.4	0.2
	(d) 3-16-91	1.0	7.8	0.13
(ɅɅ) 4	(b) 1-28-91	2.3	7.3	0.3
	(c) 2-16-91	1.65	7.8	0.2
	(d) 3-16-91	0.95	8.5	0.1
5	(a)(b) 1-28-91	2.9	3.9	0.7
	(c) 2-16-91	1.4	3.0	0.5
	(d) 3-16-91	0.9	4.3	0.2

T_o = Tensile force at the facing
T_{max}= Maximum Tensile force in the nail

(a) Lock-off Load Cell Reading
(b) Following Completion of Excavation and Nail Installation (1-24-91)
(c) Following Completion of Construction (2-14-91)
(d) End of Short Term Performance Evaluation

Ʌ Load Cell Readings at Row 2 interpolated from Row 1 and Row 3
ɅɅ Load Cell Readings at Row 4 interpolated from Row 3 and Row 5

TABLE 8
SHORT TERM PERFORMANCE
CHANGE IN THE RATIO T_o / T_{max} WITH TIME AT SECTION 2

Row No.	Date	Load Cell Reading at the Facing	Maximum Load in Nail	Ratio T_o / T_{max}
1	(a) 12-07-90	3.3	0.0	---
	12-18-90	2.8	1.2	2.3
	1-16-91	2.0	2.8	0.7
	(b) 1-28-91	1.2	2.9	0.4
	(c) 2-16-91	2.3	3.7	0.6
	(d) 3-16-91	1.1	4.0	0.3
(Δ) 2	(b) 1-28-91	2.0	8.6	0.2
	(c) 2-16-91	2.6	9.9	0.3
	(d) 3-16-91	1.9	9.3	0.2
3	(a) 1-05-91	3.4	2.0	1.7
	1-16-91	3.0	5.2	0.6
	(b) 1-28-91	2.8	5.6	0.5
	(c) 2-16-91	2.9	6.5	0.4
	(d) 3-16-91	2.7	6.6	0.4
(ΔΔ) 4	(b) 1-28-91	2.65	8.5	0.3
	(c) 2-16-91	2.6	10.2	0.25
	(d) 3-16-91	2.15	10.2	0:2
5	(a) (b) 1-31-91	2.5	2.0	1.2
	(c) 2-16-91	2.3	2.3	1.0
	(d) 3-16-91	1.6	3.3	0.5

T_o = Tensile force at the facing
T_{max} = Maximum Tensile force in the nail

(a) Lock-off Load Cell Reading
(b) Following Completion of Excavation and Nail Installation (1-24-91)
(c) Following Completion of Construction (2-14-91)
(d) End of Short Term Performance Evaluation

(Δ) Load Cell Readings at Row 2 interpolated from Row 1 and Row 3
(ΔΔ) Load Cell Readings at Row 4 interpolated from Row 3 and Row 5

TABLE 9
LONG TERM PERFORMANCE
AXIAL STRAIN DISTRIBUTION ALONG NAILS AT SECTION 1

Row No.	Date	Average Axial Strain (Microstrain)				
		A1 & A2	A3 & A4	A5 & A6	A7 & A8	A9 & A10
1	(a) 3-16-91	129	150	187	158	105
	2-22-92	207	244	307	250	194
	6-06-92	227	285	354	282	217
	10-28-92	210	265	348	273	222
	(b) 3-31-93	260	314	388	319	252
	(c)10-23-93	232	323	404	316	259
		B1 & B2	B3 & B4	B5 & B6	B7 & B8	B9 & B10
2	(a) 3-16-91	89	121	118	195	141
	2-22-92	165	196	204	324	194
	6-06-92	178	209	226	354	209
	10-28-92	137	233	218	369	218
	(b) 3-31-93	192	280	260	393	234
	(c)10-23-93	153	254	257	404	255
		C1 & C2	C3 & C4	C5 & C6	C7 & C8	C9 & C10
3	(a) 3-16-91	192	145	172	170	105
	2-22-92	262	255	280	272	187
	6-06-92	271	275	301	291	204
	10-28-92	258	273	307	301	222
	(b) 3-31-93	297	311	338	327	238
	(c)10-23-93	277	304	343	336	265
		D1 & D2	D3 & D4	D5 & D6	D7 & D8	D9 & D10
4	(a) 3-16-91	150	187	216	213	176
	2-22-92	267	301	334	321	282
	6-06-92	258	312	354	340	301
	10-28-92	259	315	368	356	318
	(b) 3-31-93	296	350	398	379	337
	(c)10-23-93	271	331	408	387	350
		E1 & E2	E3 & E4	E5 & E6	E7 & E8	E9 & E10
5	(a) 3-16-91	66	20	77	96	56
	2-22-92	97	130	180	190	148
	6-06-92	91	158	204	213	164
	10-28-92	106	184	235	244	187
	(b) 3-31-93	116	200	249	256	196
	(c)10-23-93	117	230	281	285	221

(a) End of Short Term Performance Evaluation
(b) Following the Scotts Mills Earthquake of March 25, 1993
(c) End of the Long Term Performance Evaluation

TABLE 10
LONG TERM PERFORMANCE
AXIAL STRAIN DISTRIBUTION ALONG NAILS AT SECTION 2

Row No.	Date	Average Axial Strain (Microstrain)			
		A1 & A2	A3 & A4	A5 & A6	A7 & A8
1	(a) 3-16-91	-71	5	67	88
	2-22-92	-26	47	106	119
	6-06-92	-17	51	110	120
	10-28-92	-53	21	75	96
	(b) 3-31-93	-34	34	97	118
	(c)10-23-93	-21	43	93	109
		A9 & A10	B1 & B2	B3 & B4	B5 & B6
2	(a) 3-16-91	59	241	-9	70
	2-22-92	82	298	67	158
	6-06-92	45	271	68	181
	10-28-92	-27	250	43	183
	(b) 3-31-93	62	310	102	212
	(c)10-23-93	4	274	70	232
		B7 & B8	B9 & B10	C1 & C2	C3 & C4
3	(a) 3-16-91	154	70	59	112
	2-22-92	254	171	154	208
	6-06-92	251	195	185	234
	10-28-92	213	183	172	237
	(b) 3-31-93	264	237	208	256
	(c)10-23-93	231	235	214	281
		C5 & C6	C7 & C8	C9 & C10	D1 & D2
4	(a) 3-16-91	271	218	176	138
	2-22-92	358	339	305	258
	6-06-92	310	345	329	286
	10-28-92	305	342	322	293
	(b) 3-31-93	363	401	363	326
	(c)10-23-93	300	376	357	348
		D3 & D4	D5 & D6	D7 & D8	D9 & D10
5	(a) 3-16-91	73	66	65	31
	2-22-92	134	159	164	102
	6-06-92	122	189	196	125
	10-28-92	122	193	213	138
	(b) 3-31-93	154	219	229	150
	(c)10-23-93	122	231	258	173

(a) End of Short Term Performance Evaluation
(b) Following the Scotts Mills Earthquake of March 25, 1993
(c) End of the Long Term Performance Evaluation

TABLE 11
LONG TERM PERFORMANCE
AXIAL TENSILE LOAD DISTRIBUTION ALONG NAILS AT SECTION 1

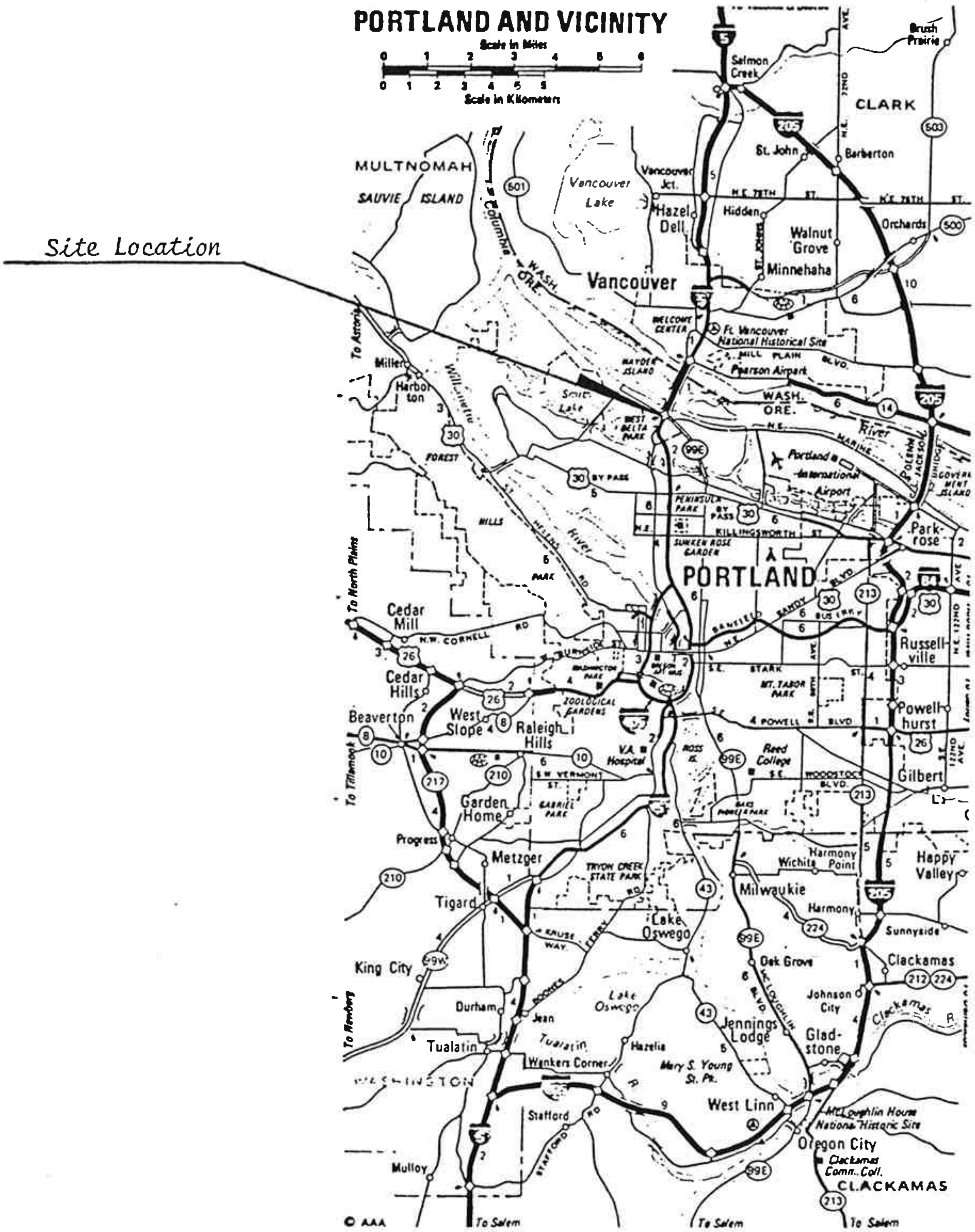
Row No.		Date	Axial Tensile Load (Kips) (Including incremental percent change in computed loads)									
			A1 & A2		A3 & A4		A5 & A6		A7 & A8		A9 & A10	
	(a)	3-16-91		5.8		6.5		7.6		6.7		4.7
		10-19-91	24%	7.2	28%	8.3	33%	10.1	25%	8.4	47%	6.9
		2-22-92	14%	8.2	13%	9.4	12%	11.3	14%	9.6	13%	7.8
1		6-06-92	7%	8.8	13%	10.6	12%	12.7	9%	10.5	9%	8.5
		10-28-92	-6%	8.3	-6%	10.0	-2%	12.5	-2%	10.3	2%	8.7
	(b)	3-31-93	18%	9.8	15%	11.5	10%	13.8	13%	11.6	10%	9.6
	(c)	10-23-93	-8%	9.0	3%	11.8	3%	14.2	-1%	11.5	2%	9.8
			B1 & B2		B3 & B4		B5 & B6		B7 & B8		B9 & B10	
	(a)	3-16-91		4.0		5.4		5.3		7.9		6.2
		10-19-91	25%	5.0	20%	6.5	32%	7.0	37%	10.8	19%	7.4
		2-22-92	40%	7.0	22%	7.9	16%	8.1	9%	11.8	5%	7.8
2		6-06-92	4%	7.3	5%	8.3	9%	8.8	8%	12.7	6%	8.3
		10-28-92	-16%	6.1	8%	9.0	-2%	8.6	4%	13.2	4%	8.6
	(b)	3-31-93	28%	7.8	17%	10.5	14%	9.8	5%	13.9	6%	9.1
	(c)	10-23-93	-15%	6.6	-8%	9.7	0%	9.8	2%	14.2	6%	9.7
			C1 & C2		C3 & C4		C5 & C6		C7 & C8		C9 & C10	
	(a)	3-16-91		7.8		6.3		7.2		7.1		4.7
		10-19-91	10%	8.6	35%	8.5	29%	9.3	32%	9.4	49%	7.0
		2-22-92	15%	9.9	14%	9.7	13%	10.5	9%	10.2	9%	7.6
3		6-06-92	3%	10.2	6%	10.3	6%	11.1	6%	10.8	6%	8.1
		10-28-92	-4%	9.8	-1%	10.2	2%	11.3	3%	11.1	7%	8.7
	(b)	3-31-93	12%	11.0	12%	11.4	8%	12.2	7%	11.9	6%	9.2
	(c)	10-23-93	-5%	10.4	-2%	11.2	2%	12.4	8%	12.2	7%	9.9
			D1 & D2		D3 & D4		D5 & D6		D7 & D8		D9 & D10	
	(a)	3-16-91		6.5		7.6		8.5		8.4		7.3
		10-19-91	34%	8.7	32%	10.0	31%	11.1	27%	10.7	34%	9.8
		2-22-92	15%	10.0	11%	11.1	9%	12.1	9%	11.7	7%	10.5
4		6-06-92	-2%	9.8	3%	11.4	3%	12.5	5%	12.3	6%	11.1
		10-28-92	0%	9.8	1%	11.5	5%	13.1	4%	12.8	5%	11.6
	(b)	3-31-93	11%	10.9	10%	12.6	8%	14.1	5%	13.5	5%	12.2
	(c)	10-23-93	-6%	10.2	-5%	12.0	2%	14.4	2%	13.7	3%	12.6
			E1 & E2		E3 & E4		E5 & E6		E7 & E8		E9 & E10	
	(a)	3-16-91		3.0		0.9		3.5		4.3		2.5
		10-19-91	17%	3.5	444%	4.9	100%	7.0	67%	7.2	136%	5.9
5		2-22-92	26%	4.4	18%	5.8	6%	7.4	7%	7.7	8%	6.4
		6-06-92	-7%	4.1	16%	6.7	9%	8.1	9%	8.4	8%	6.9
		10-28-92	17%	4.8	12%	7.5	12%	9.1	12%	9.4	10%	7.6
	(b)	3-31-93	8%	5.2	7%	8.0	4%	9.5	3%	9.7	4%	7.9
	(c)	10-23-93	2%	5.3	11%	8.9	11%	10.5	9%	10.6	10%	8.7

(a) End of Short Term Performance Evaluation
(b) Following the Scotts Mills Earthquake of March 25, 1993
(c) End of the Long Term Performance Evaluation

TABLE 12
LONG TERM PERFORMANCE
AXIAL TENSILE LOAD DISTRIBUTION ALONG NAILS AT SECTION 2

Row No.		Date	Axial Tensile Load (Kips) (Including incremental percent change in computed loads)							
			A1 & A2		A3 & A4		A5 & A6		A7 & A8	
	(a)	3-16-91		0.0		0.2		3.0		4.0
		10-19-91		0.0	600%	1.4	37%	4.1	20%	4.8
		2-22-92		0.0	50%	2.1	17%	4.8	10%	5.3
1		6-06-92		0.0	10%	2.3	2%	4.9	2%	5.4
		10-28-92		0.0	-61%	0.9	31%	3.4	-20%	4.3
	(b)	3-31-93		0.0	33%	1.2	26%	4.3	23%	5.3
	(c)	10-23-93		0.0	58%	1.9	-2%	4.2	-8%	4.9
			A9 & A10		B1 & B2		B3 & B4		B5 & B6	
	(a)	3-16-91		2.7		9.3		0.0		3.1
		10-19-91		0.0	-3%	9.0		0.7	84%	5.7
		2-22-92		3.7	22%	11.0	328%	3.0	18%	6.7
2		6-06-92		2.0	-7%	10.2	3%	3.1	10%	7.4
		10-28-92		0.0	-7%	9.5	-35%	2.0	1%	7.5
	(b)	3-31-93		2.8	20%	11.4	130%	4.6	12%	8.4
	(c)	10-23-93		0.2	-10%	10.3	-30%	3.2	7%	9.0
			B7 & B8		B9 & B10		C1 & C2		C3 & C4	
	(a)	3-16-91		6.6		3.2		2.6		5.0
		10-19-91	17%	7.7	66%	5.3	96%	5.1	48%	7.4
		2-22-92	26%	9.7	34%	7.1	29%	6.6	12%	8.3
3		6-06-92	-1%	9.6	11%	7.9	15%	7.6	8%	9.0
		10-28-92	-13%	8.4	-5%	7.5	-5%	7.2	1%	9.1
	(b)	3-31-93	19%	10.0	23%	9.2	15%	8.3	7%	9.7
	(c)	10-23-93	-10%	9.0	-1%	9.1	1%	8.4	8%	10.5
			C5 & C6		C7 & C8		C9 & C10		D1 & D2	
	(a)	3-16-91		10.2		8.6		7.3		6.1
		10-19-91	-2%	10.0	17%	10.2	33%	9.7	41%	8.6
		2-22-92	28%	12.8	21%	12.3	16%	11.2	14%	9.8
4		6-06-92	-11%	11.4	1%	12.4	7%	12.0	8%	10.6
		10-28-92	-2%	11.2	0%	12.4	-3%	11.7	2%	10.8
	(b)	3-31-93	16%	13.0	14%	14.1	11%	13.0	10%	11.9
	(c)	10-23-93	-15%	11.1	-5%	13.4	-2%	12.8	5%	12.5
			D3 & D4		D5 & D6		D7 & D8		D9 & D10	
	(a)	3-16-91		3.3		3.0		2.9		1.4
		10-19-91	24%	4.1	97%	5.9	114%	6.2	186%	4.0
		2-22-92	46%	6.0	15%	6.8	11%	6.9	15%	4.6
5		6-06-92	-8%	5.5	13%	7.7	15%	7.9	22%	5.6
		10-28-92	0%	5.5	1%	7.8	6%	8.4	9%	6.1
	(b)	3-31-93	20%	6.6	10%	8.6	6%	8.9	6%	6.5
	(c)	10-23-93	-17%	5.5	5%	9.0	10%	9.8	11%	7.2

(a) End of Short Term Performance Evaluation
(b) Following the Scotts Mills Earthquake of March 25, 1993
(c) End of Long Term Performance Evaluation



PORTLAND AND VICINITY
Scale in Miles
Scale in Kilometers
Site Location
MULTNOMAH
SAUVIE ISLAND
CLARK
Vancouver
Vancouver Lake
Salmon Creek
St. John
Barberton
Brush Prairie
Vancouver Jct.
Hazel Dell
Hidden
Walnut Grove
Minnehaha
Orchards
Ft. Vancouver National Historical Site
Pearson Airport
WASH.
ORE.
Columbia River
Portland International Airport
Miller
Harborton
Willamette River
Smith Lake
FOREST
HILLS
PARK
PORTLAND
Parkrose
Russellville
Powellhurst
Gilbert
Cedar Mill
Cedar Hills
Beaverton
West Slope
Raleigh Hills
Garden Home
Progress
Metzger
Tigard
King City
Durham
Tualatin
Jean
Lake Oswego
Hazelia
Wankers Corner
Stafford
Mulloy
Reed College
Milwaukie
Harmony
Wichita
Happy Valley
Sunnyside
Clackamas
Oak Grove
Johnson City
Jennings Lodge
Gladstone
West Linn
Oregon City
McLoughlin House National Historic Site
Clackamas Comm. Coll.
Mary S. Young St. Pk.
CLACKAMAS
WASHINGTON
To Astoria
To North Plains
To Tillamook
To Newberg
To Salem
© AAA

Figure 1

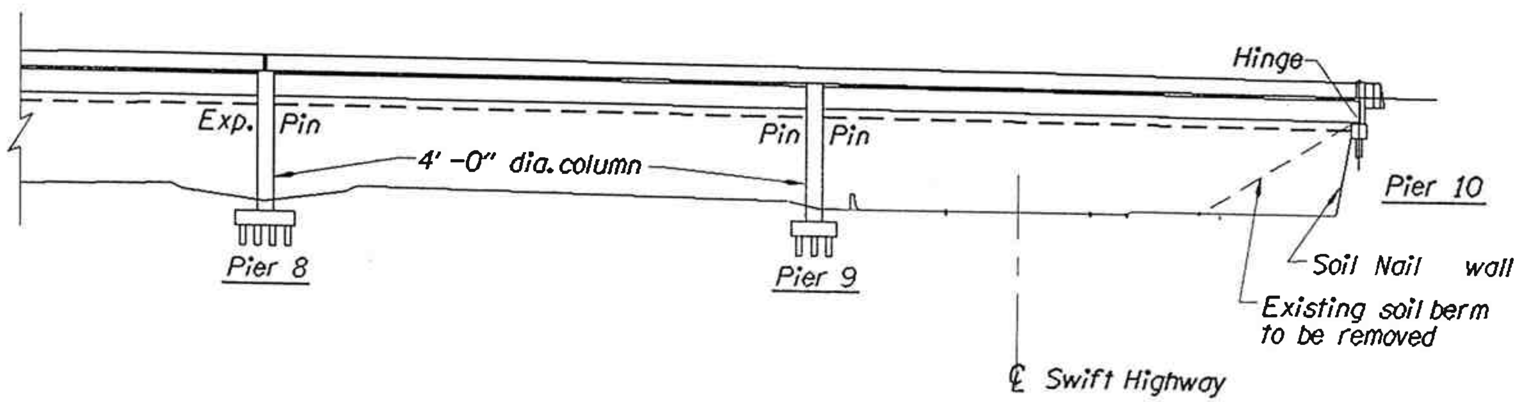


Figure 2 Oregon Slough Bridge

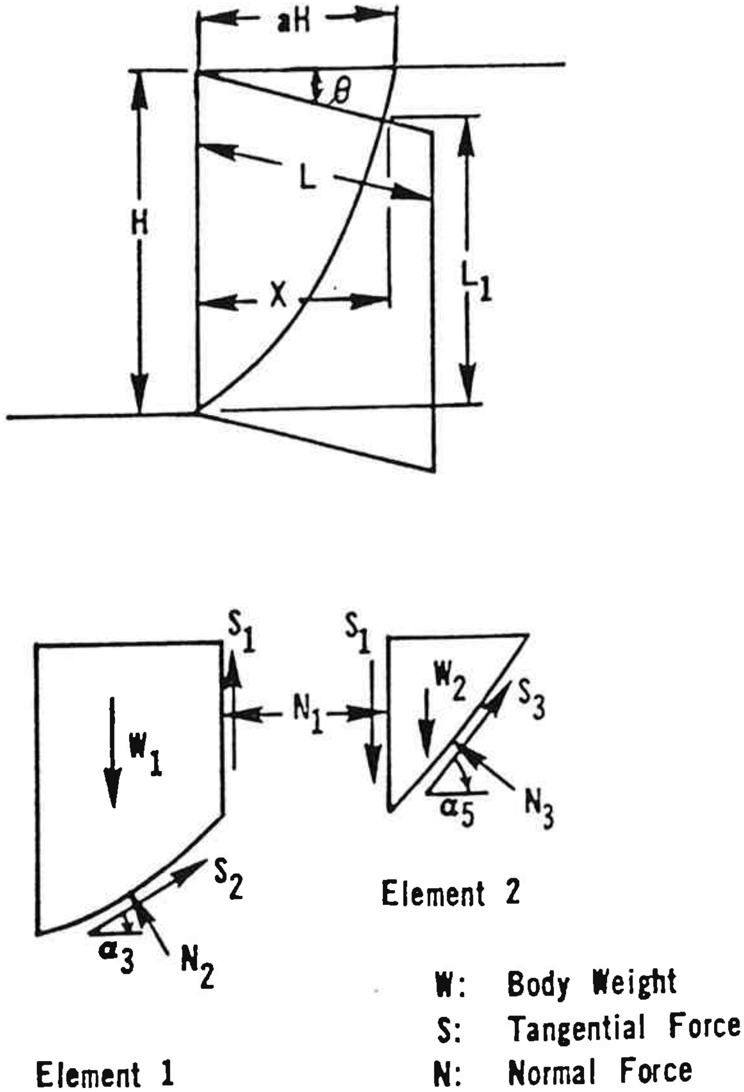


Figure 3 Free Body Diagram when $a < a_T$

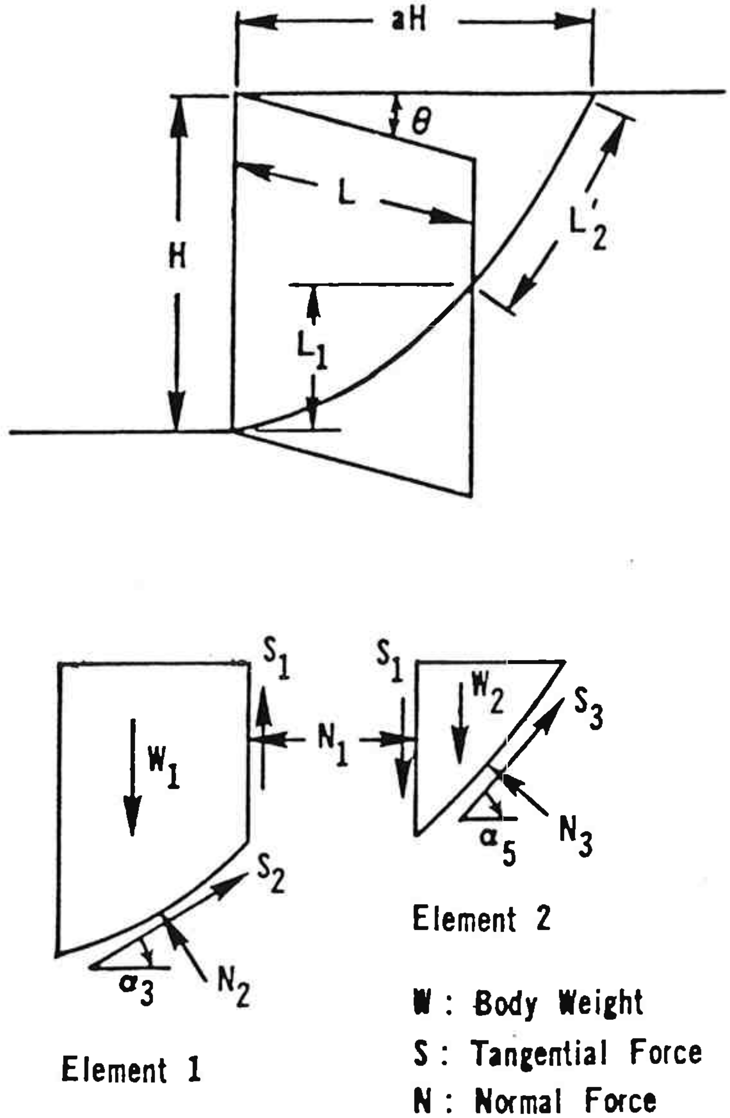


Figure 4 <u>Free Body Diagram when $a > a_T$</u>

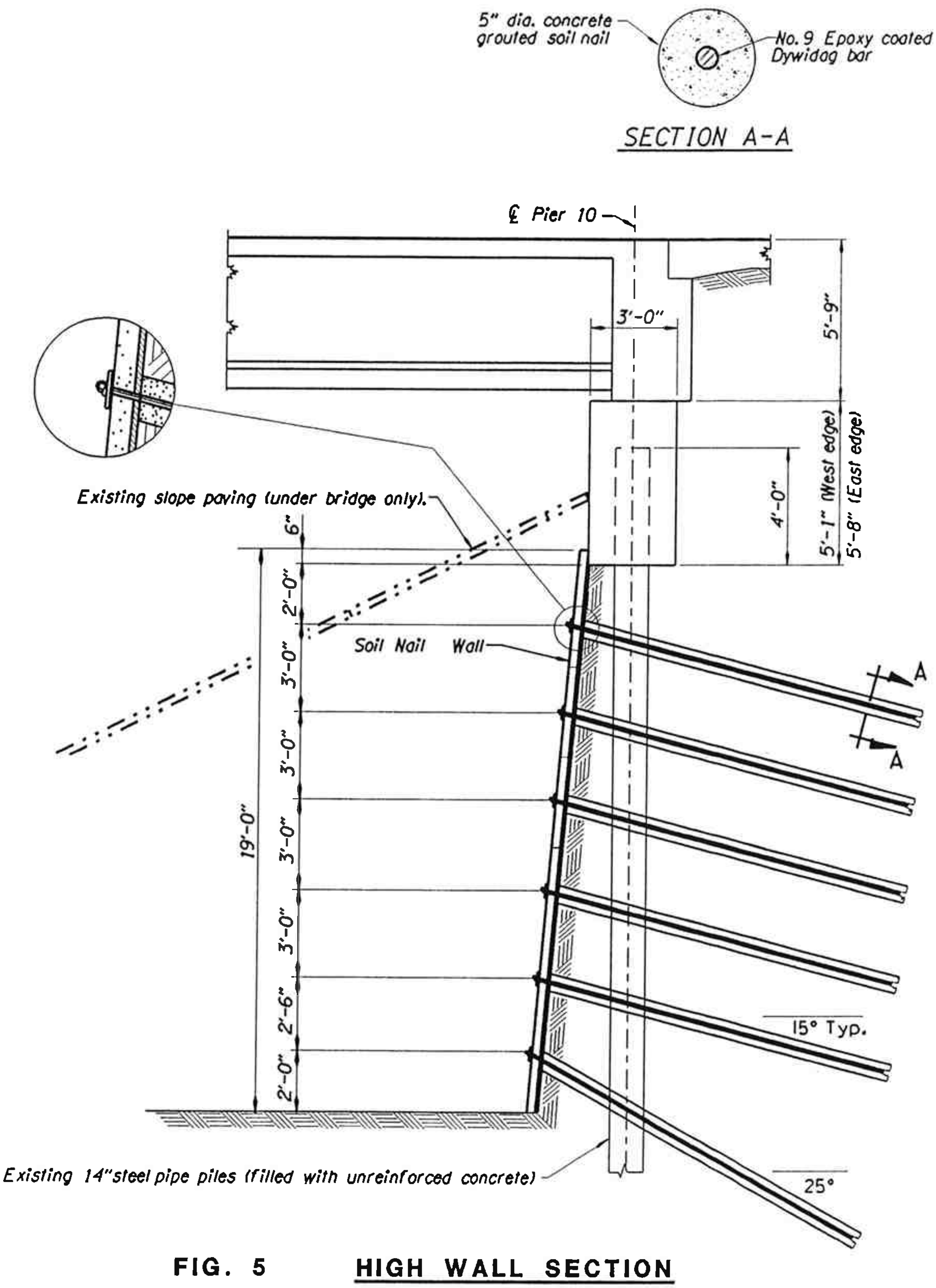
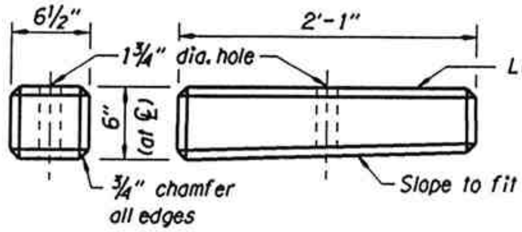
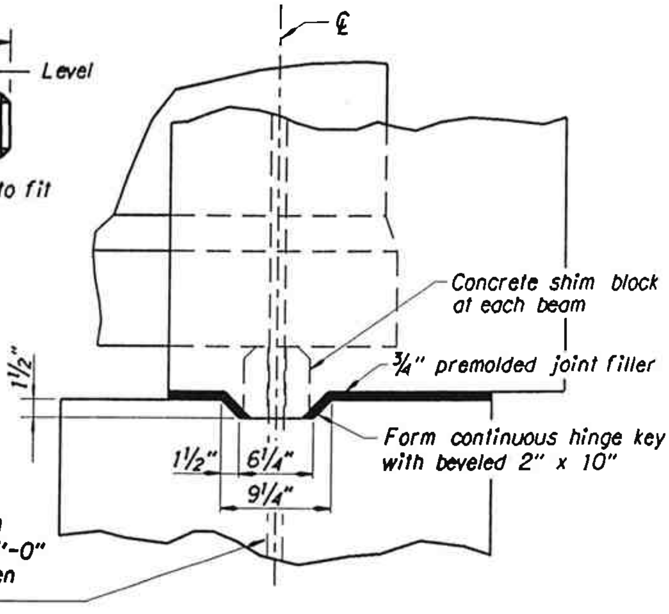


FIG. 5 **HIGH WALL SECTION AT BRIDGE ABUTMENT**



CONCRETE SHIM BLOCK



HINGE DETAIL

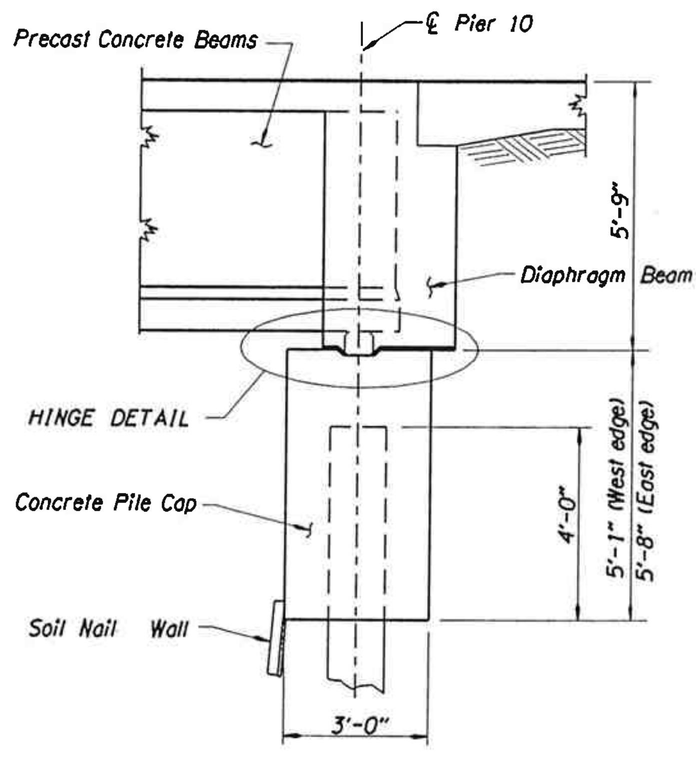


FIG. 6 TYPICAL SECTION AT PIER 10
OREGON SLOUGH BRIDGE

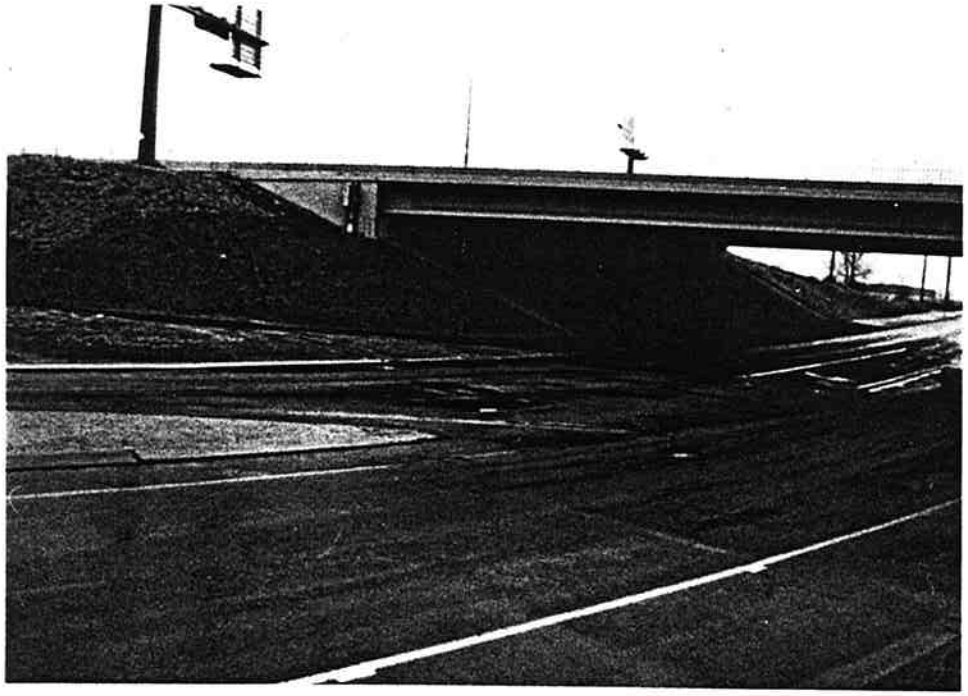


FIG. 7 - South abutment of the Oregon Slough Bridge prior to any soil nailing activities

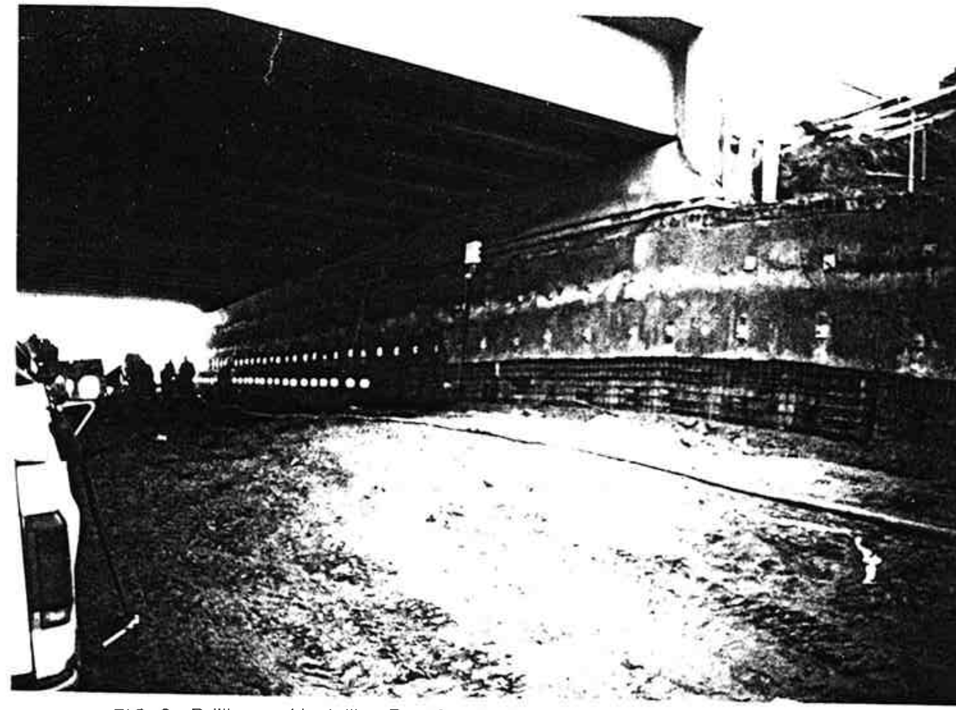


FIG. 8 - Drilling and installing Row 3 nails. The use of blockouts at nail locations (in the background) allowed the contractor to drill through the one-day old shotcrete facing

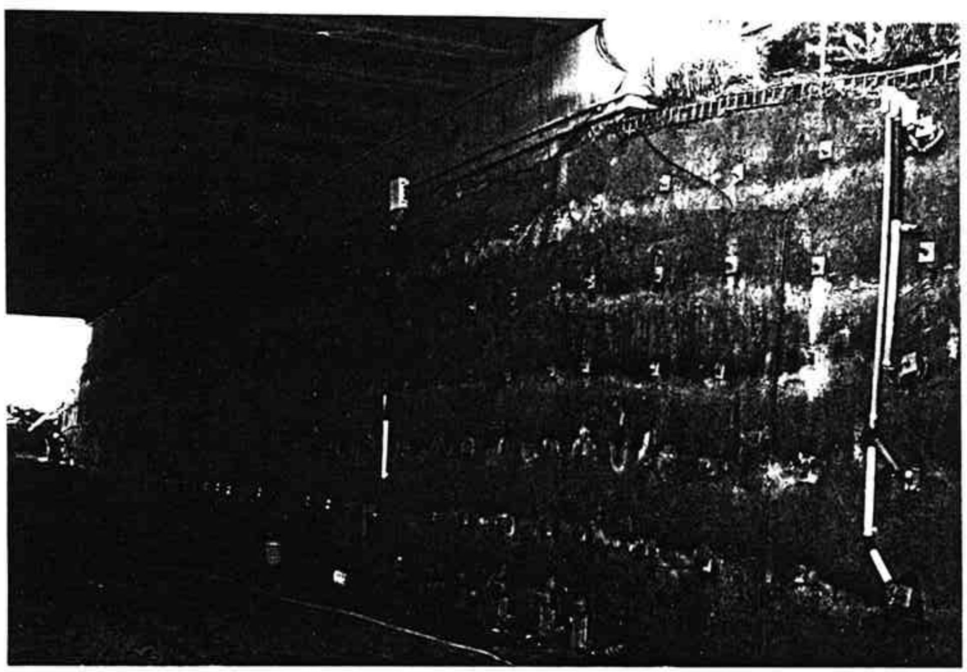
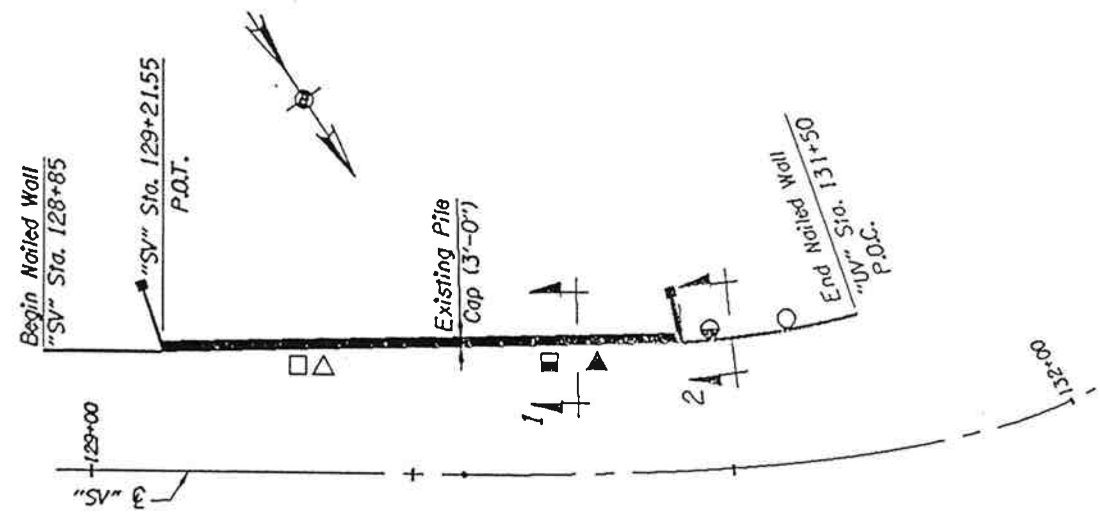


FIG. 9 - Wall after completion of all soil nailing activities. Notice instrumentation wire conduits at instrumented Sections 1 & 2

FIG. 10 - Wall after placement of the second shotcrete application and removal of architectural scoring strips. Notice blockouts at instrumented Section 1.



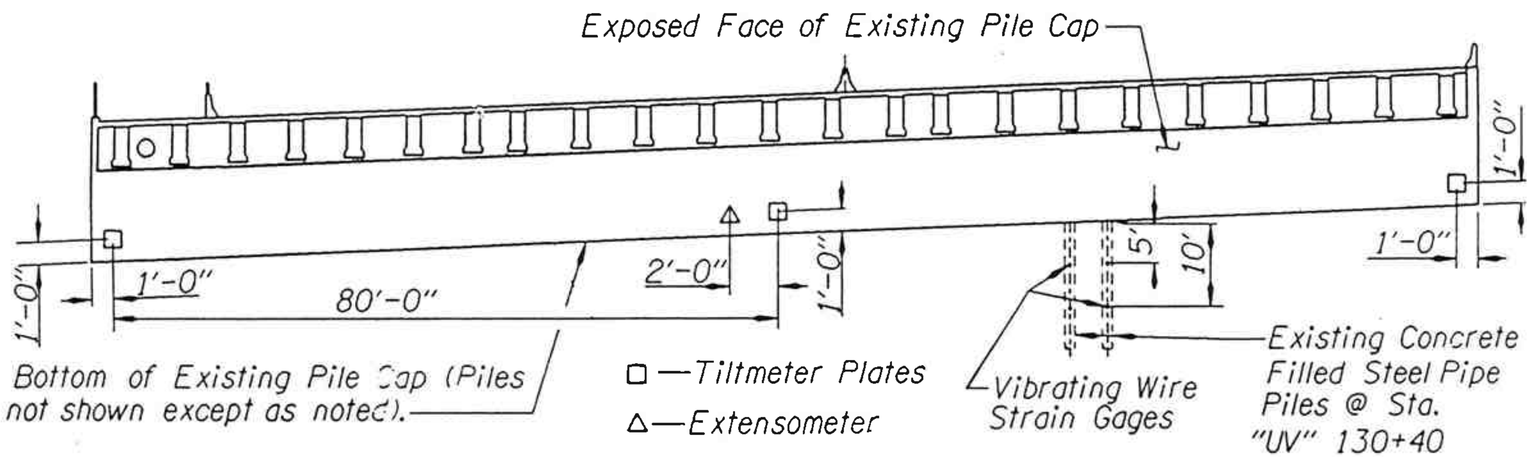
LEGEND:

- ○ SI 129 (1) "SV" 131 + 27.32
- ◒ SI 130 "SV" 130 + 96.74
- △ SI 131 (2) "SV" 129 + 64.52
- ▲ SI 132 "SV" 130 + 62.90
- □ LVDT-1 (3) "SV" 129 + 62.02
- ■ LVDT-2 "SV" 130 + 40.90
- Instrumental section (4) no. 1 "SV" 130 + 58.95
- Instrumental section no. 2 "SV" 131 + 04.81

NOTE:

1. Slope inclinometer 3.5' behind wall at SI 129 and SI 130.
2. Slope inclinometer embedded in the second shotcrete application: full wall height, at SI 131 and SI 132.
3. Linear variable differential transformers installed at top of wall prior to placing the second shotcrete application at LVDT-1 and LVDT-2.
4. Five rows of nails instrumented with strain gauges at instrumented section no. 1 and no. 2.

Figure 11 Instrumentation Plan



Front View of Pile Cap

No Scale

Figure 12 **Soil Nailing Instrumentation**

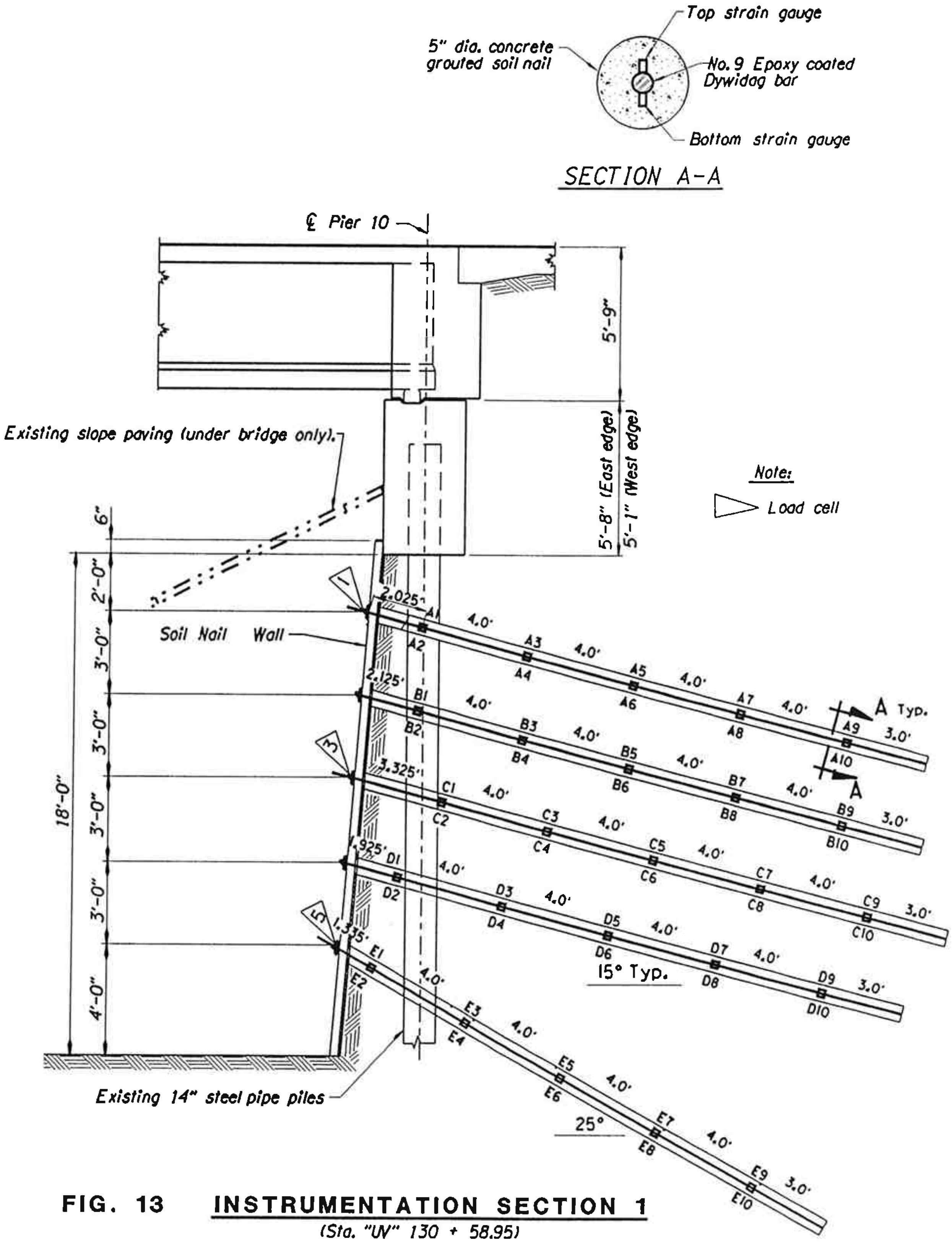
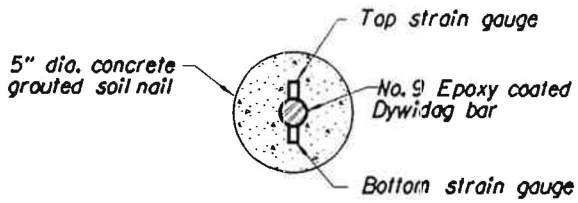


FIG. 13 INSTRUMENTATION SECTION 1

(Sta. "UV" 130 + 58.95)



SECTION A-A

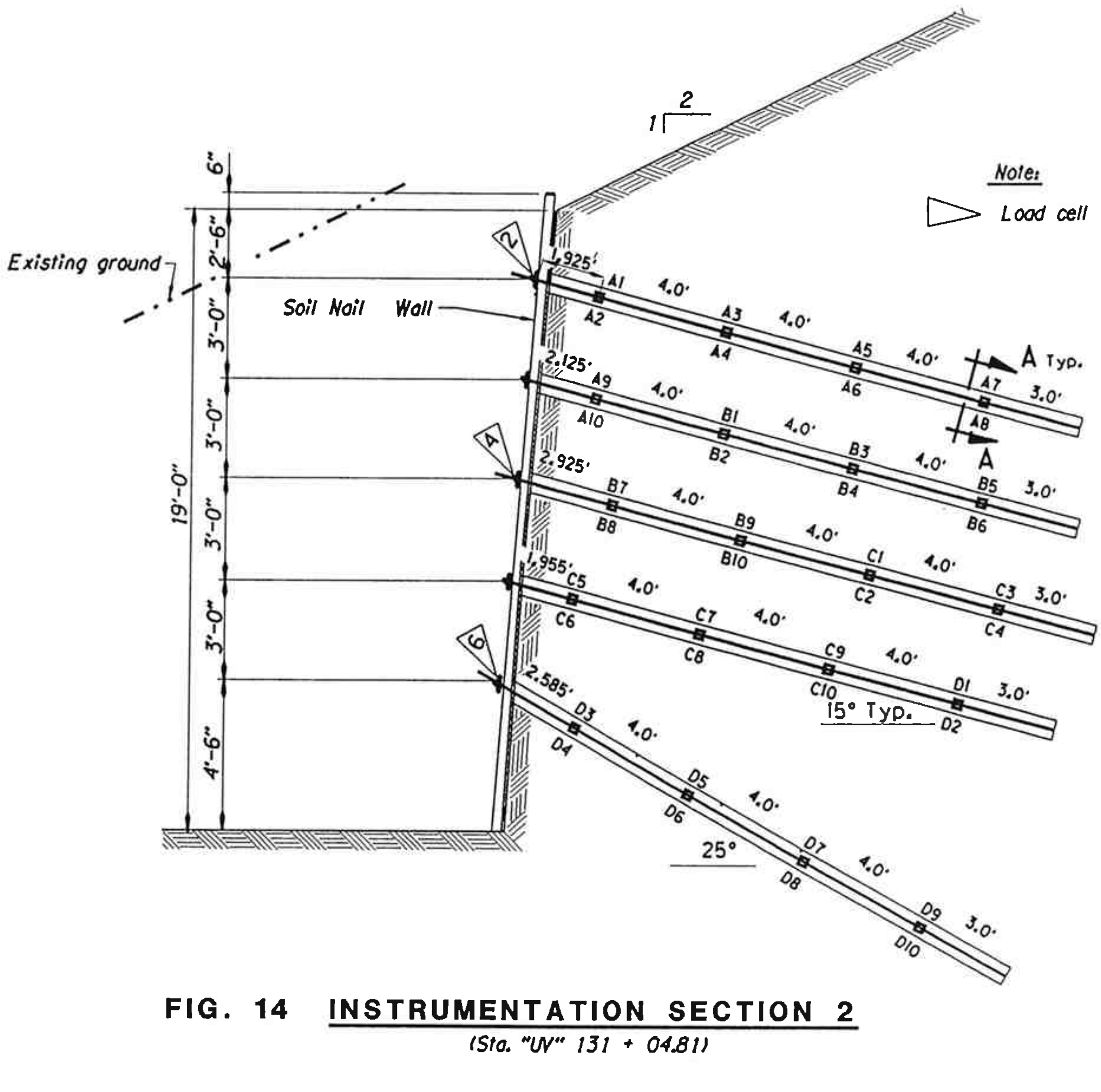


FIG. 14 INSTRUMENTATION SECTION 2

(Sta. "UV" 131 + 04.81)

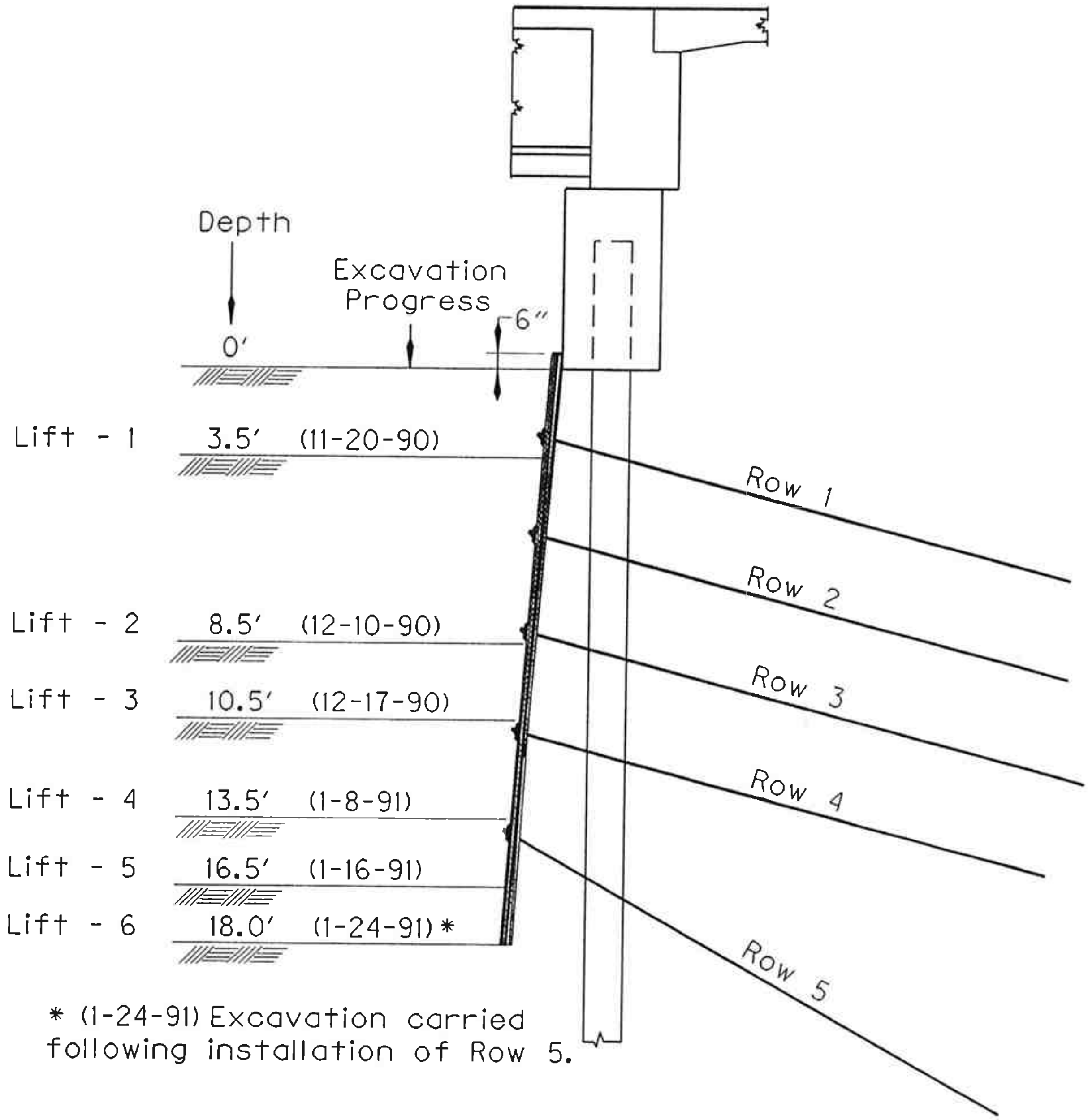


FIG. 15 EXCAVATION PROGRESS AT INSTRUMENTED SECTION 1

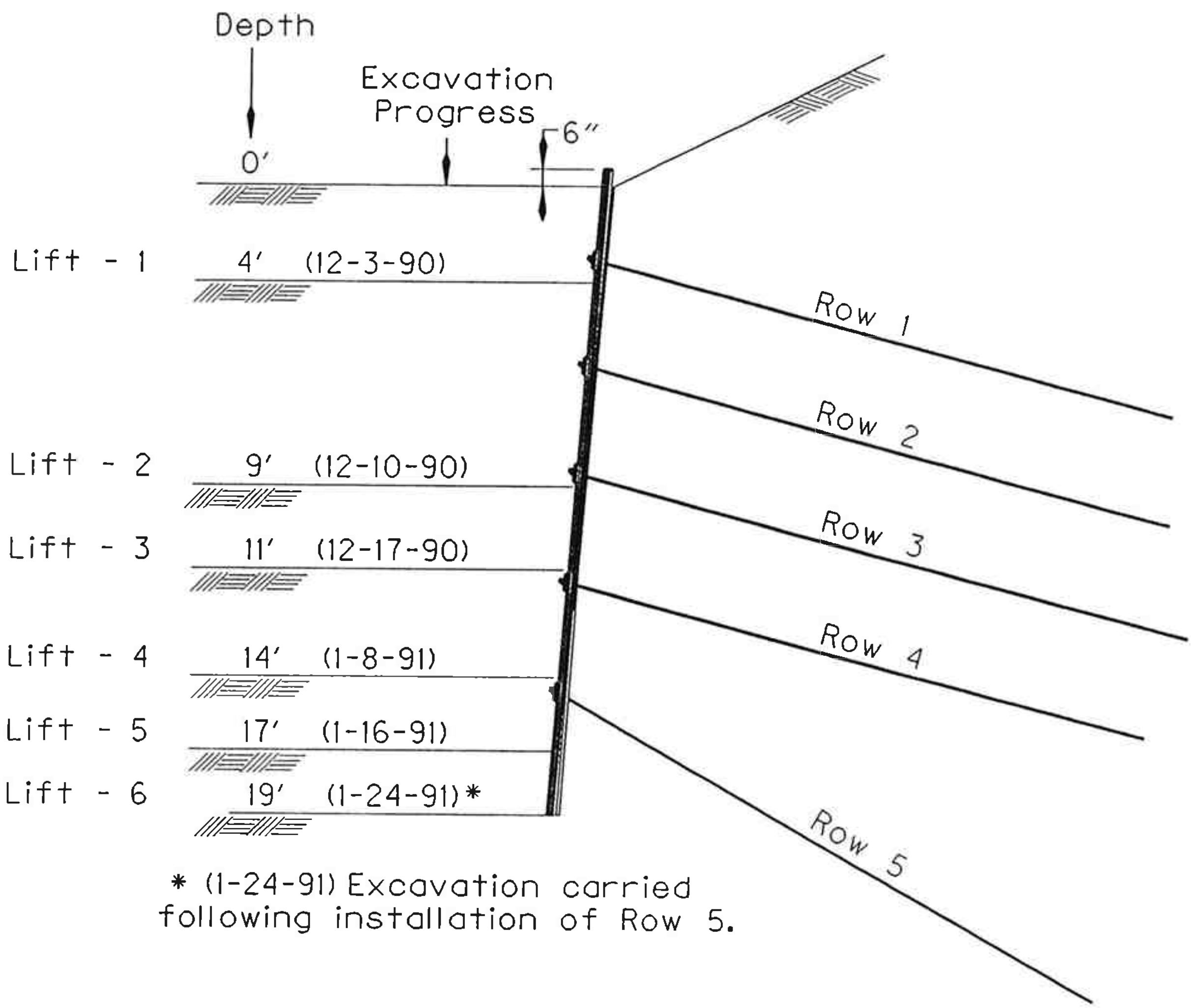


FIG. 16 EXCAVATION PROGRESS AT INSTRUMENTED SECTION 2

FIG. 17 - Instrumented nail with vibrating wire strain gauges

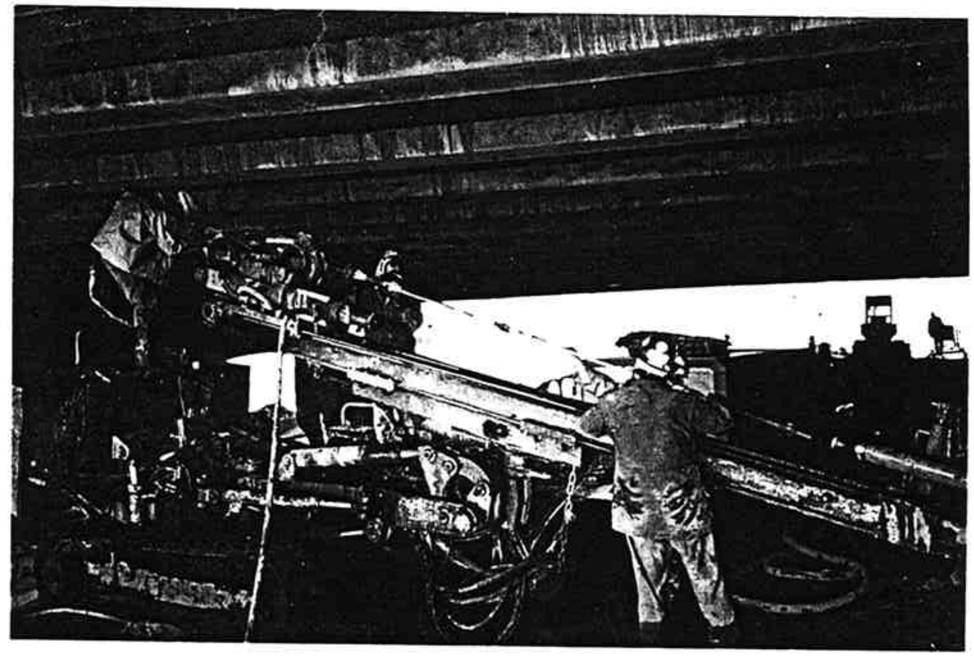


FIG. 18 - Placing an instrumented nail inside a cased drill hole at section 1

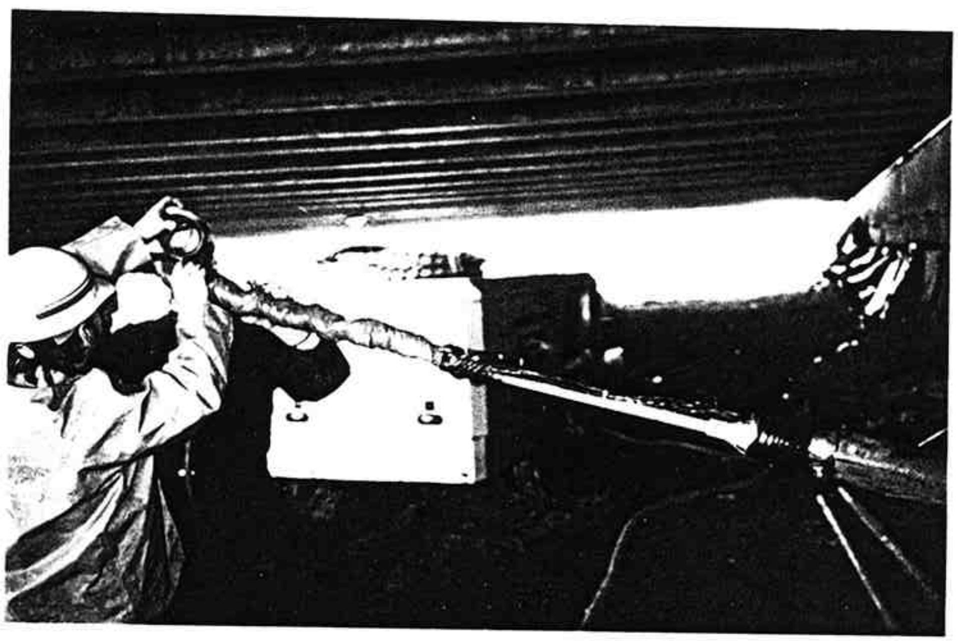
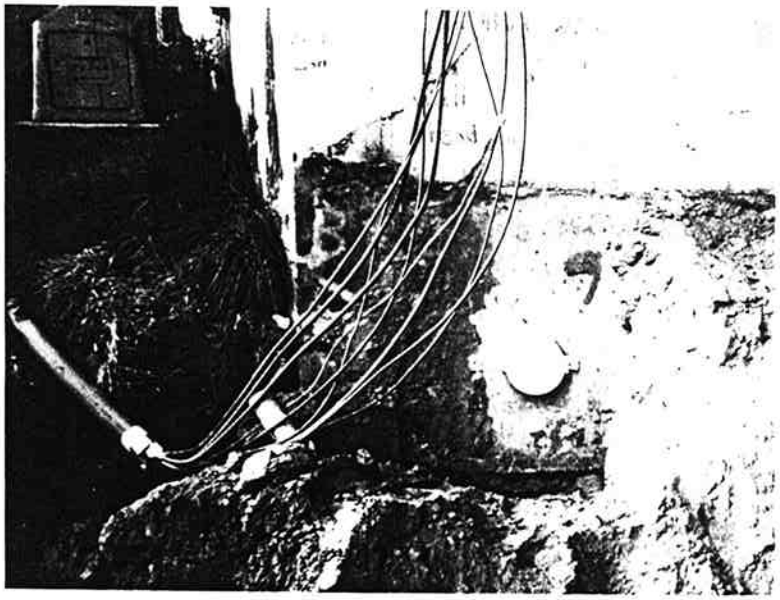


FIG. 19 - Protecting instrumentation wire leads at the head of the nail

FIG. 20 - Advancing the drilling head around the instrumented nail in preparation for grouting the cased drill hole



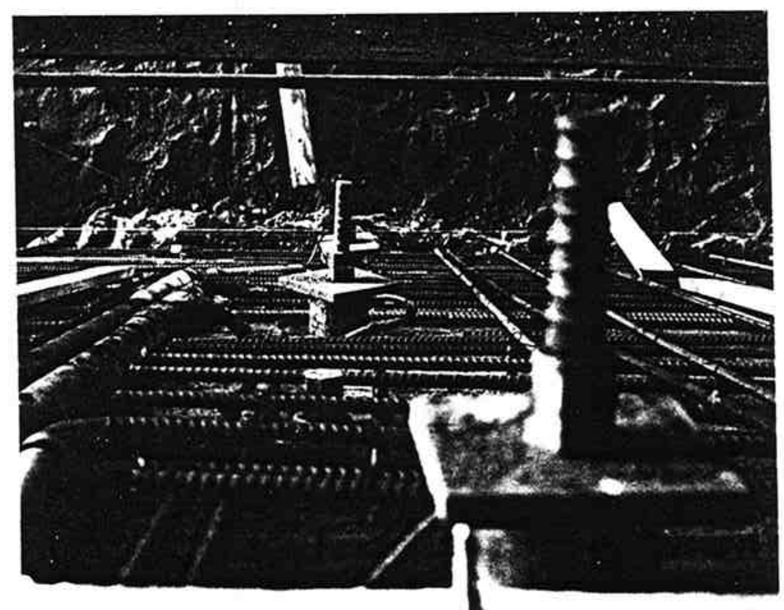
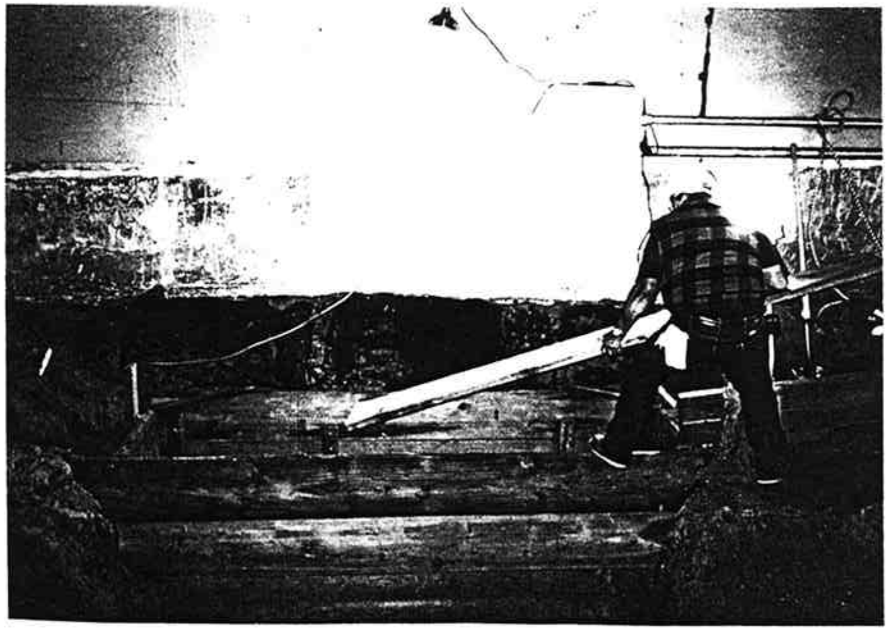


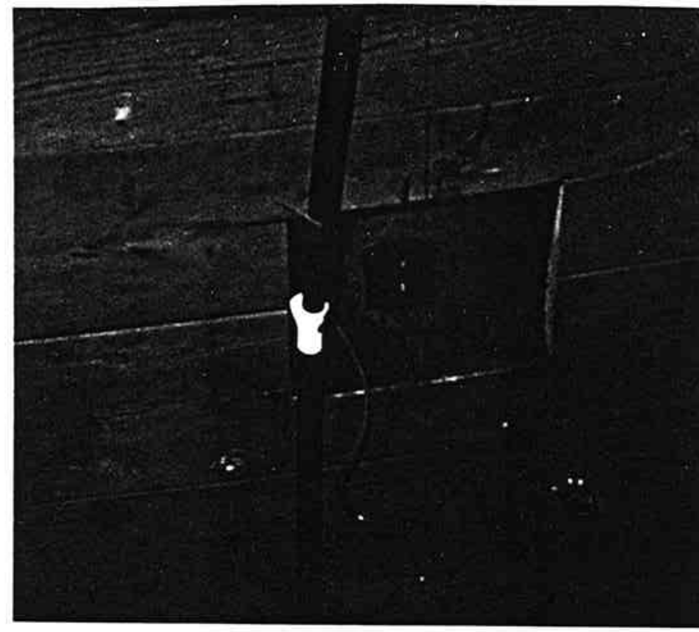
FIG. 21 - (Top left)
Ceramic tiltmeter plate mounted at the east end of the existing bridge pile cap prior to any excavation.

FIG. 22 - (Top right)
Reading the bridge pile cap rotation with a portable tiltmeter.

FIG. 23 - (Bottom left)
Base reading inclinometer SD132 prior to embedding into the final shotcrete lift.

FIG. 24 - (Bottom right)
Looking top-down at the electric load cells at instrumented Section 1.





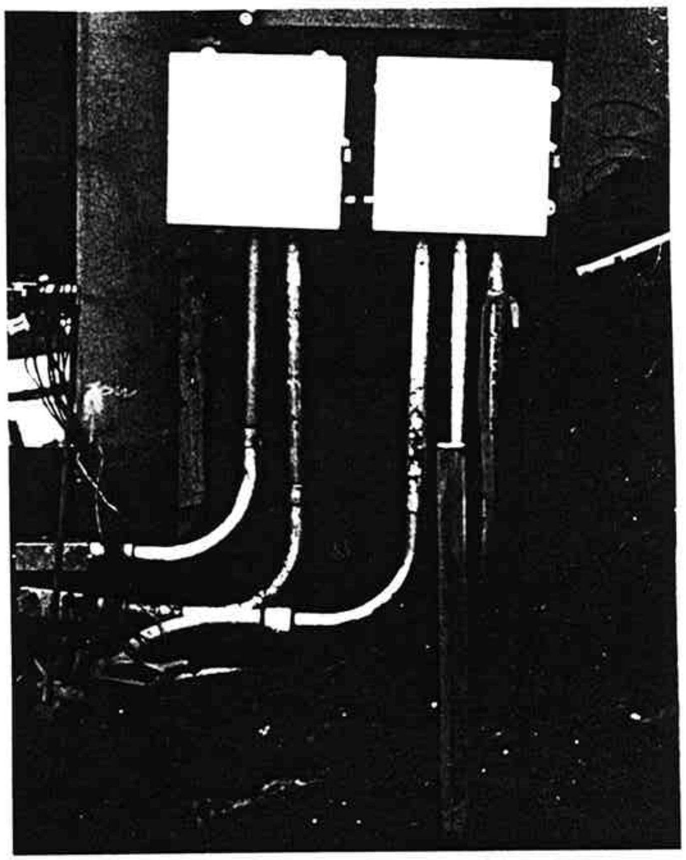


FIG. 25 - (Top left)
Hand excavating to instrument two existing piles prior to any soil nailing activity. Notice the exposed steel pipe piles in the background.

FIG. 26 - (Top right)
Instrumenting an existing steel pipe pile with vibrating wire strain gauges.

FIG. 27- (Bottom left)
Vertical slope inclinometer SD130. Notice the instrumentation control panels in the background.

FIG. 28 - (Bottom right)
Completed wall at instrumented Section 1. Notice blockouts at load cell locations.

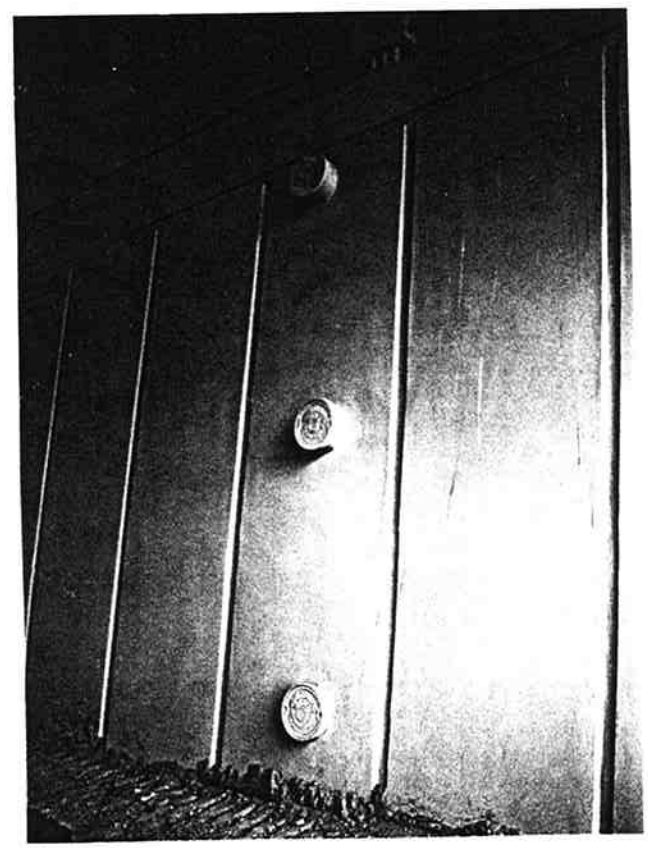


FIG. 29 - Ungrouted Dywidag test bar with vibrating wire strain gauges

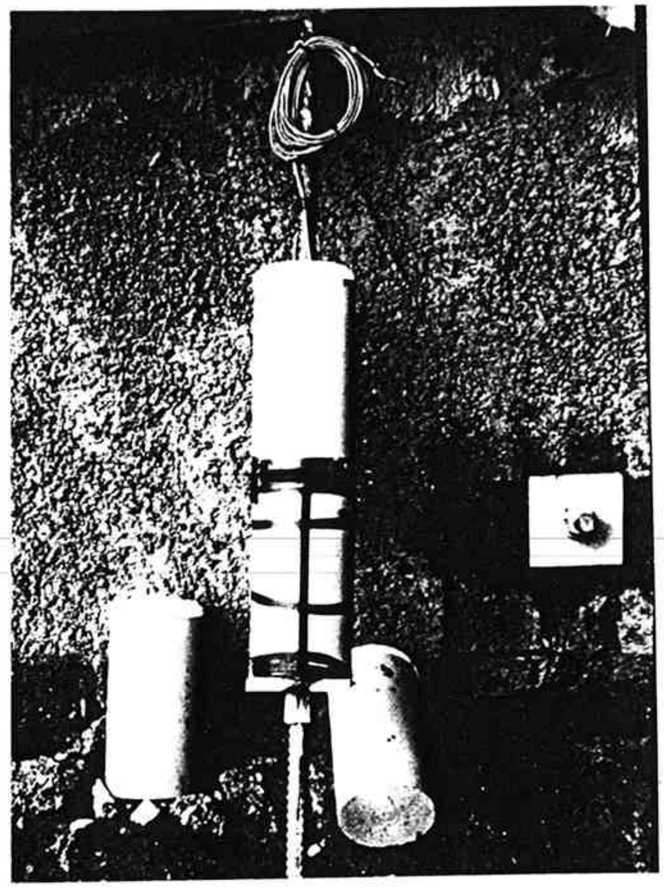
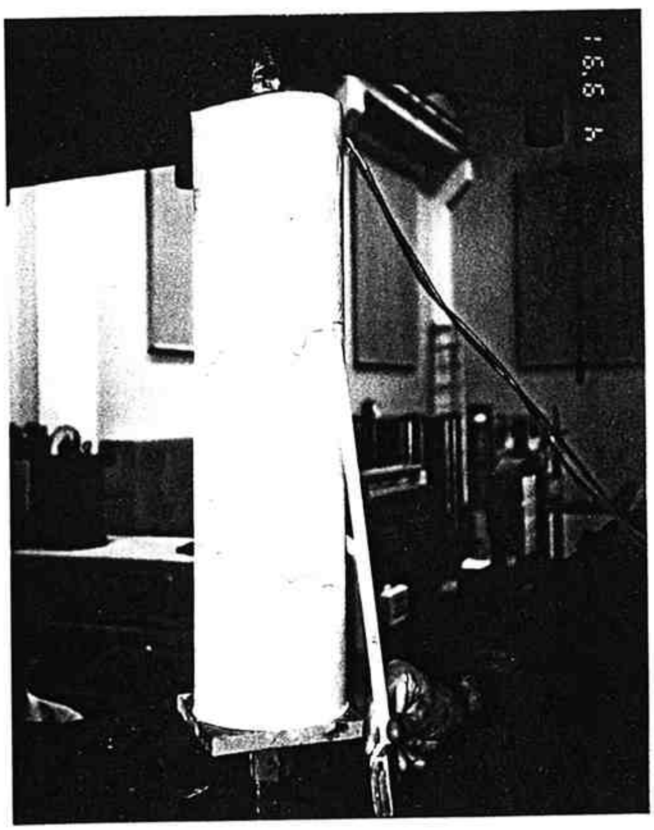


FIG. 30 - Grouted test nail



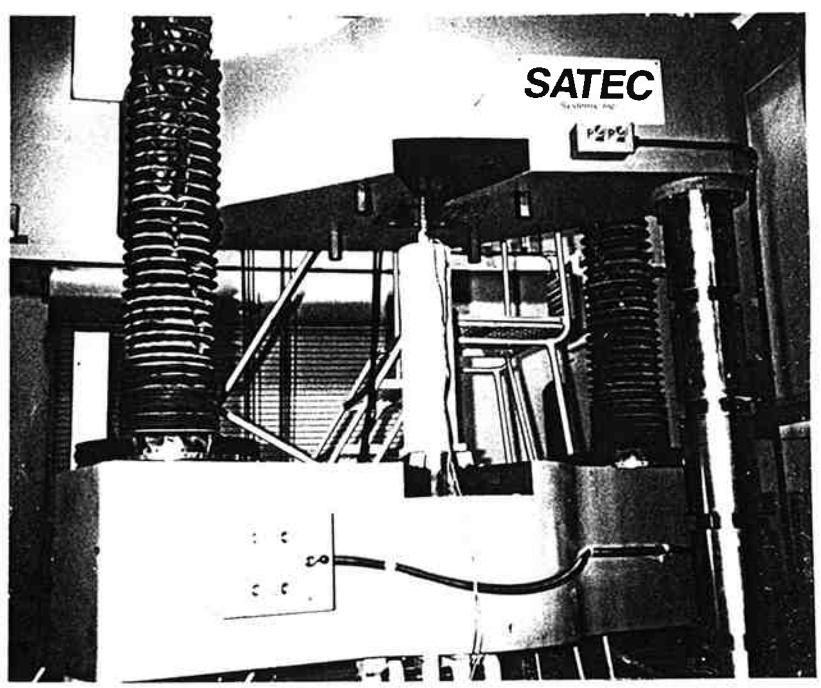


FIG. 31 and FIG. 32 - Lab testing of grouted test nail in direct tension

• Area of no. 9 bar = 1.0 in.^{2}

• Average change in strain with load = 40.56 microstrain/kip

• Modulus of elasticity of bar = $1.0^{K} / 40.56 \times 10^{-6}$ = 24,845,000 psi

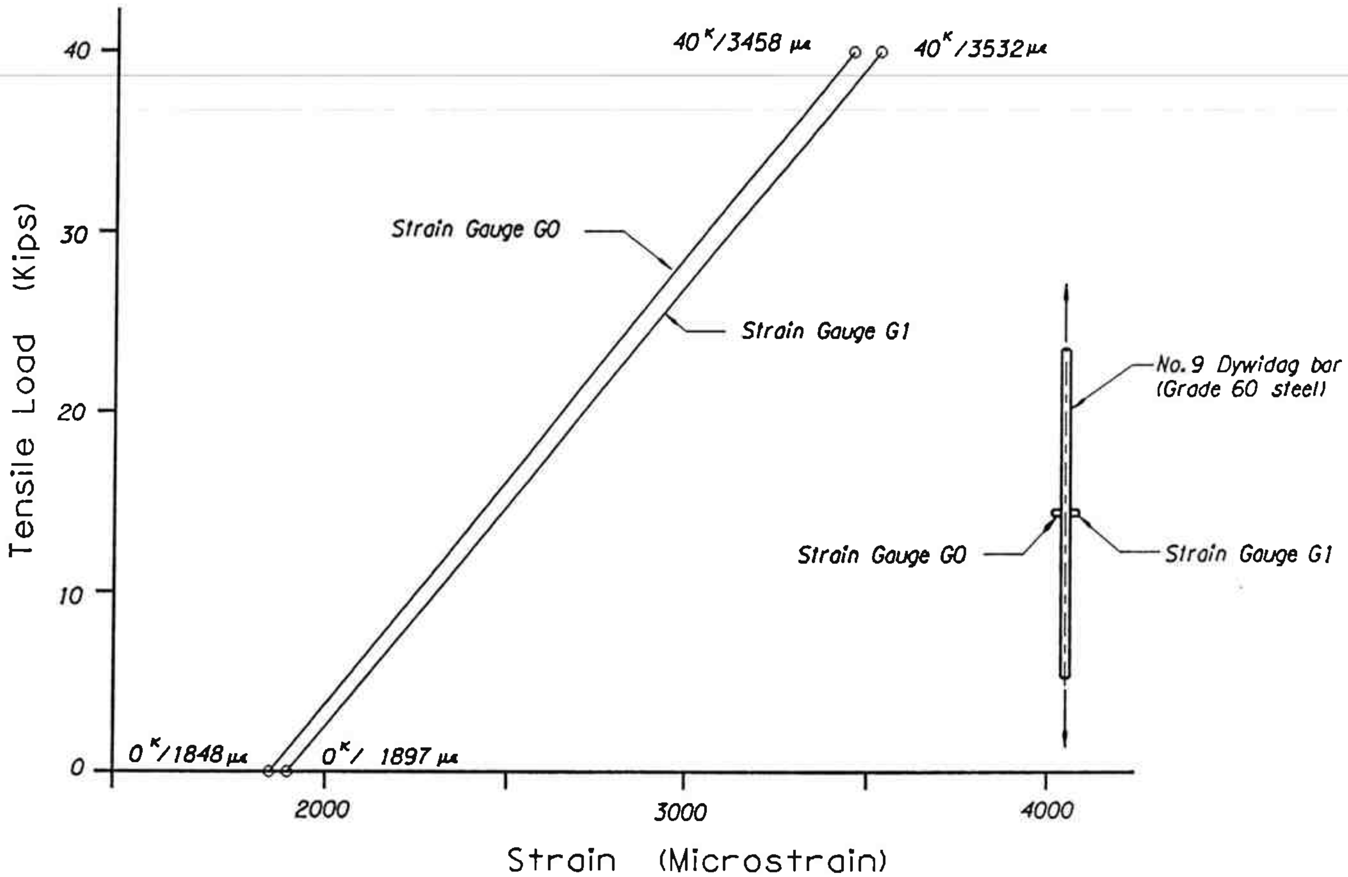


Figure 33 **FORCE - STRAIN RELATIONSHIP CURVES OF A NO. 9 DYWIDAG TEST BAR**

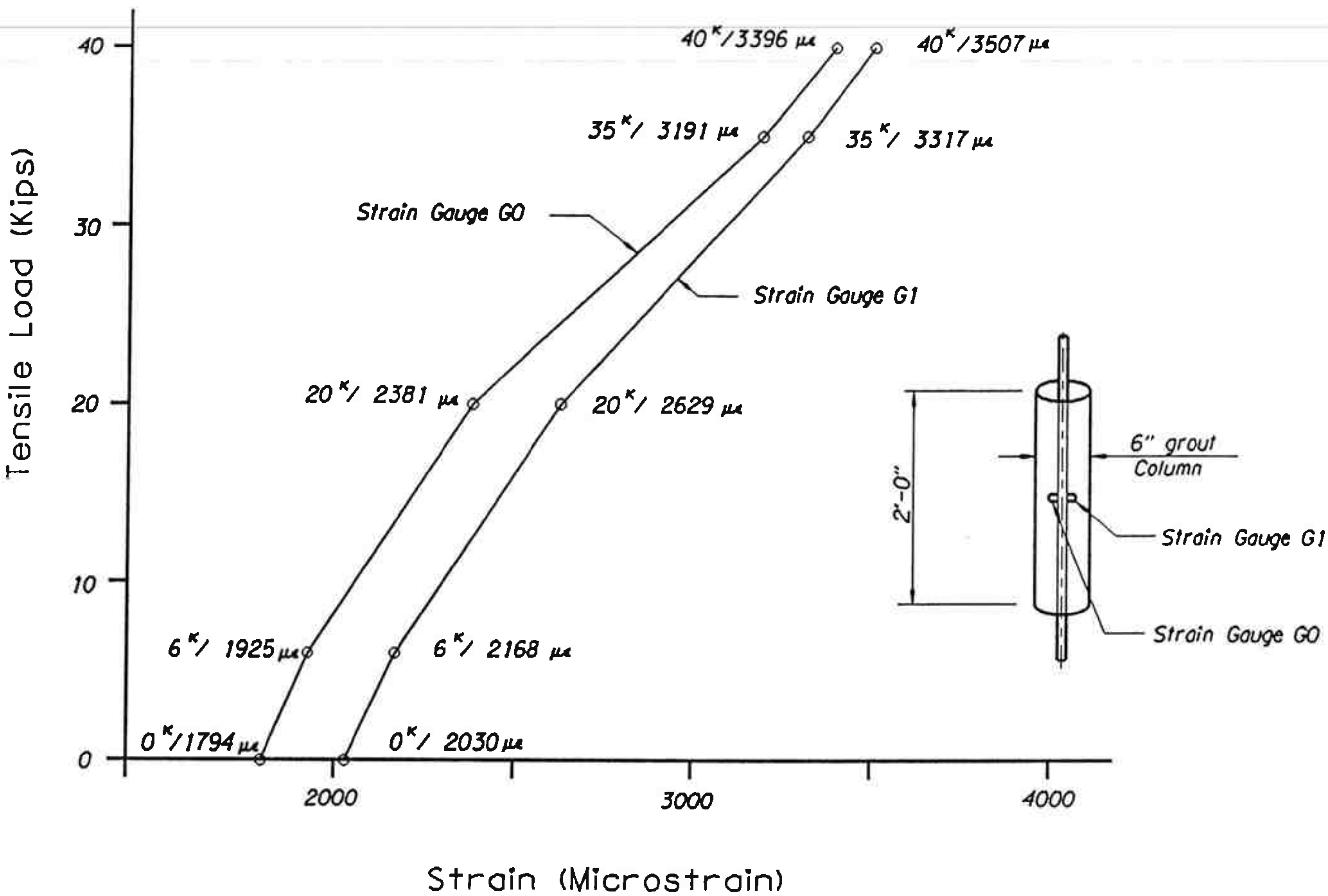


Figure 34 FORCE - STRAIN RELATIONSHIP CURVES OF A GROUTED TEST NAIL

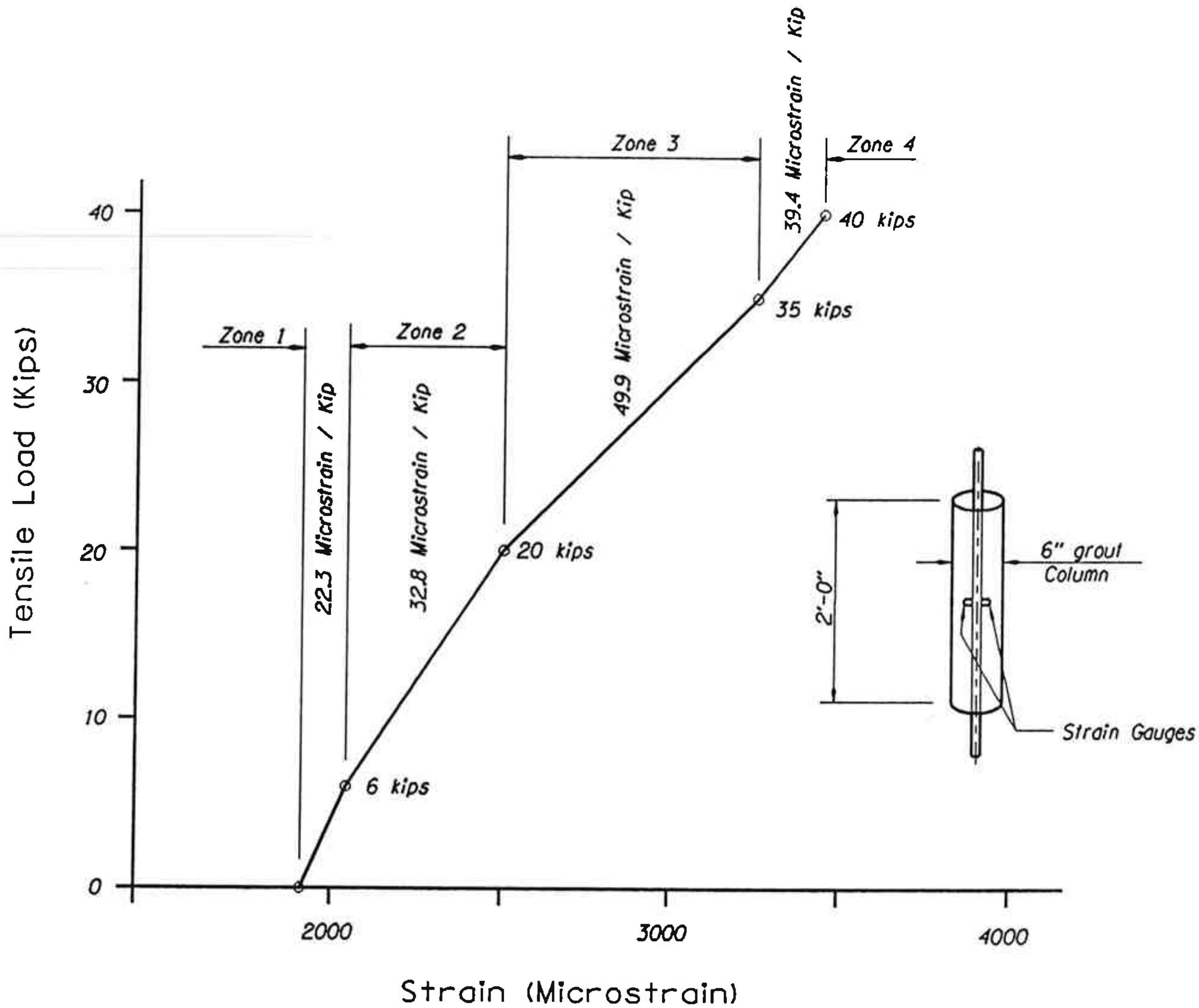


Figure 35 AVERAGE FORCE - STRAIN RELATIONSHIP CURVE OF A GROUTED TEST NAIL

Fig. 36 - Short Term Performance

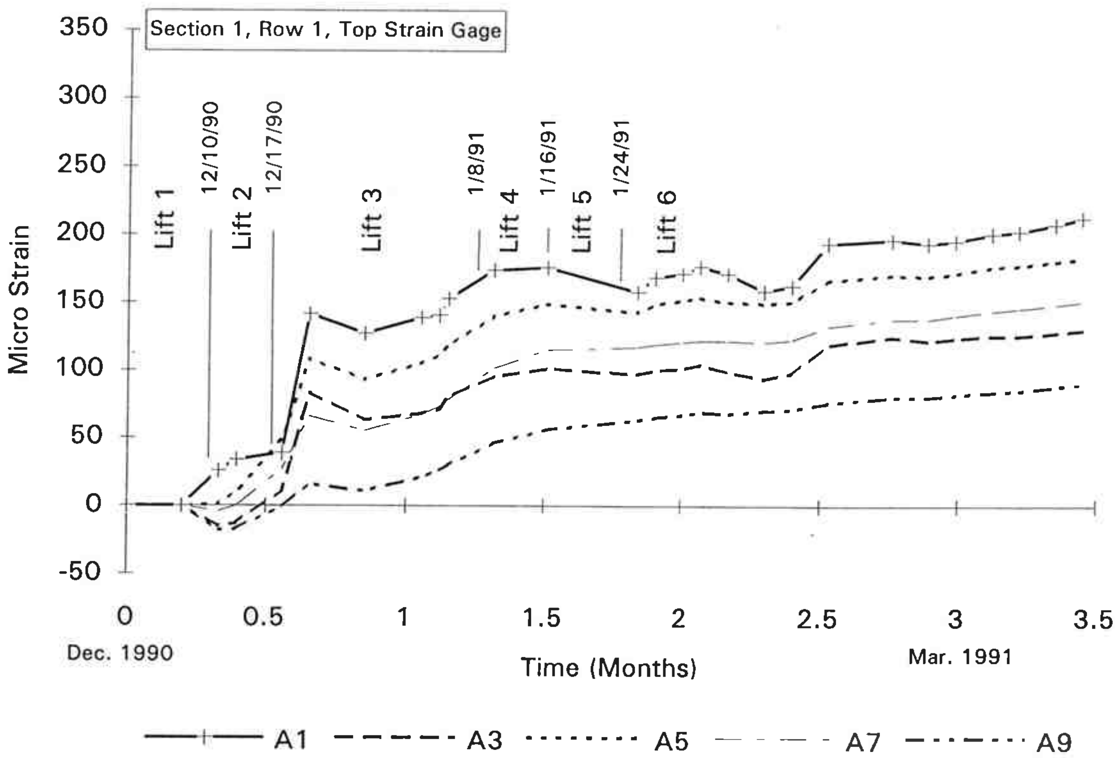


Fig. 37 - Short Term Performance

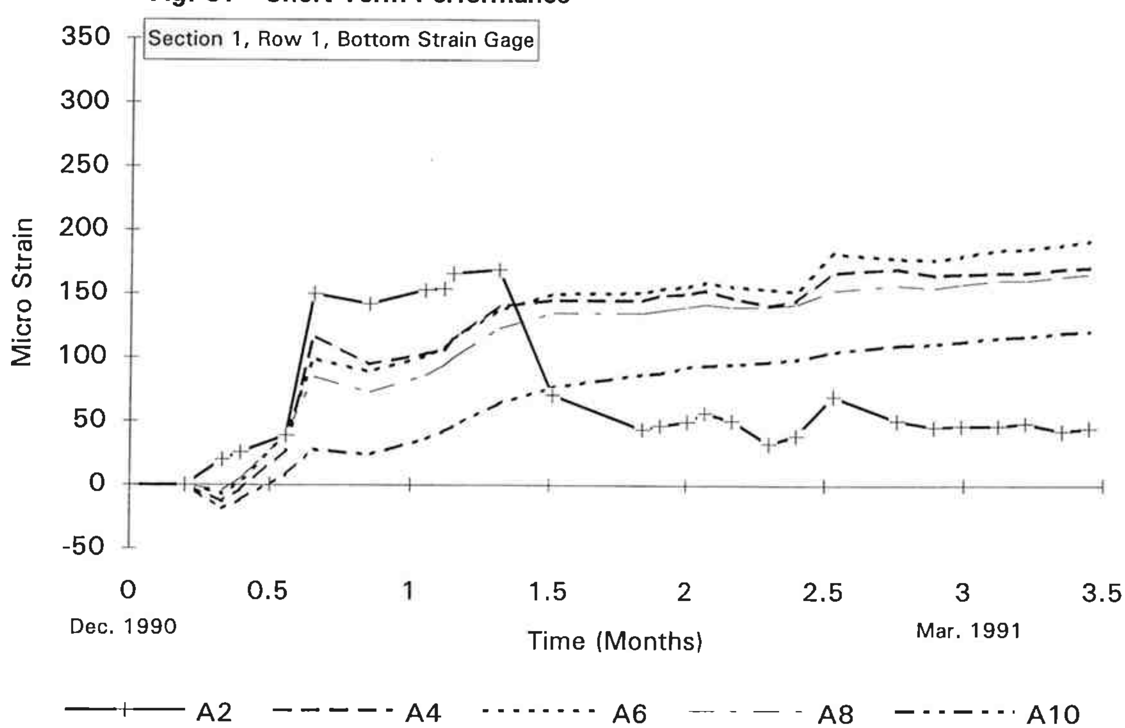


Fig. 38 - Short Term Performance

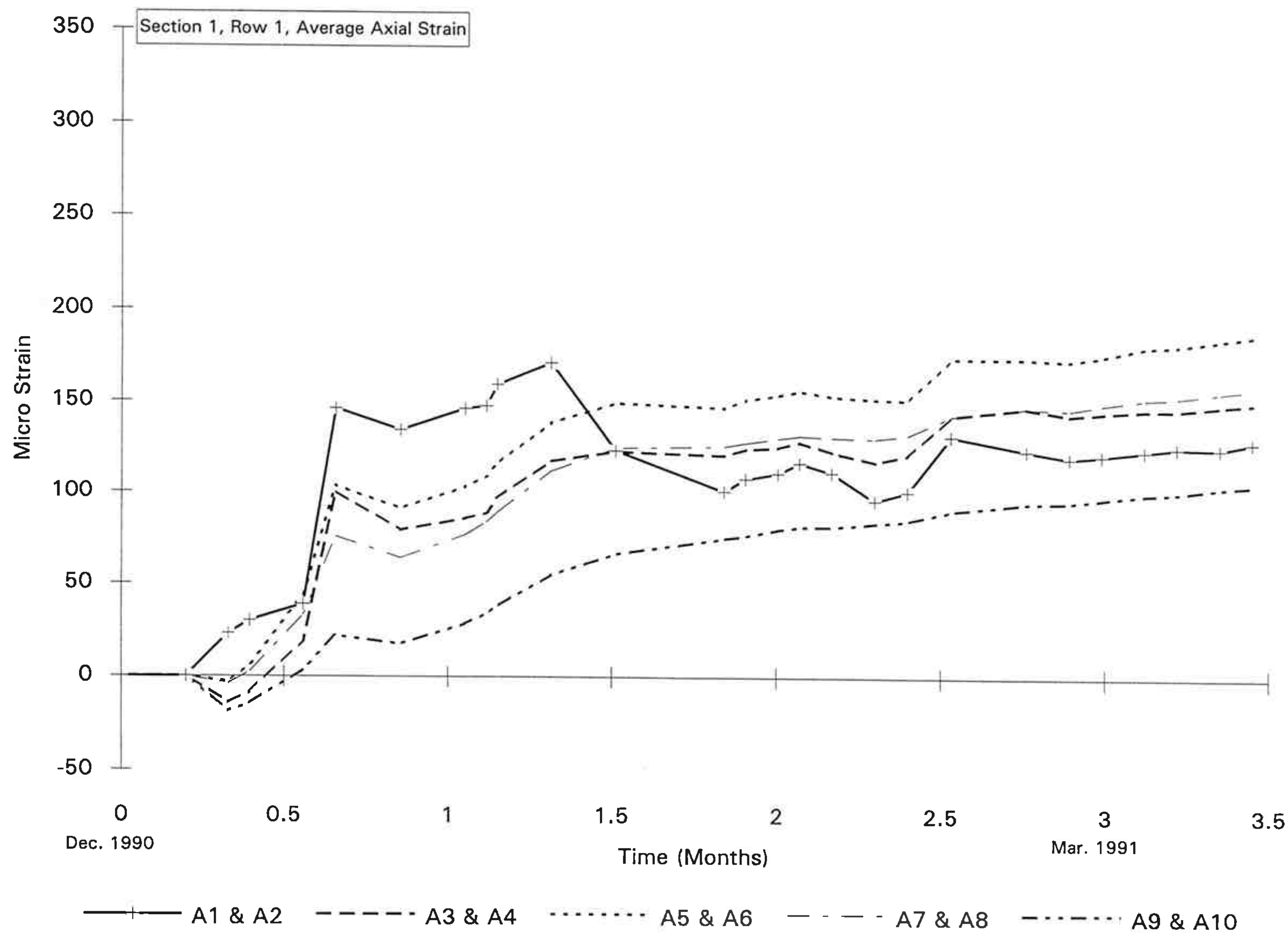


Fig. 39 - Short Term Performance

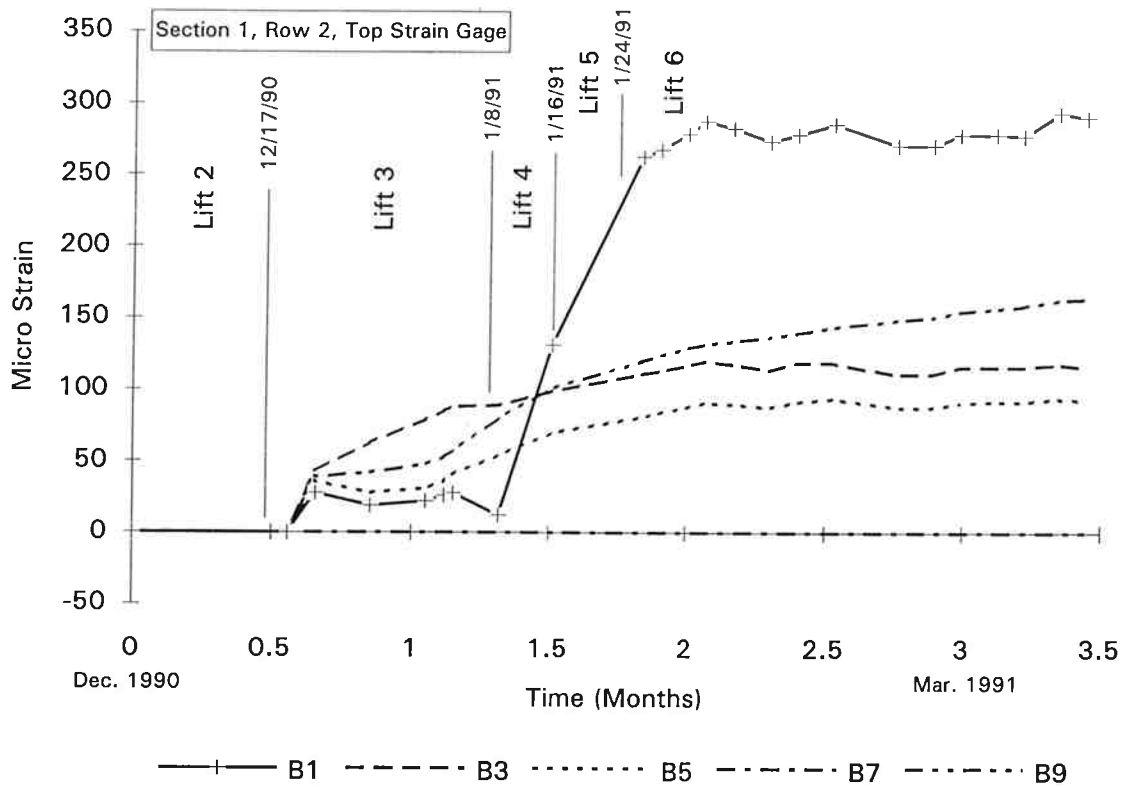


Fig. 40 - Short Term Performance

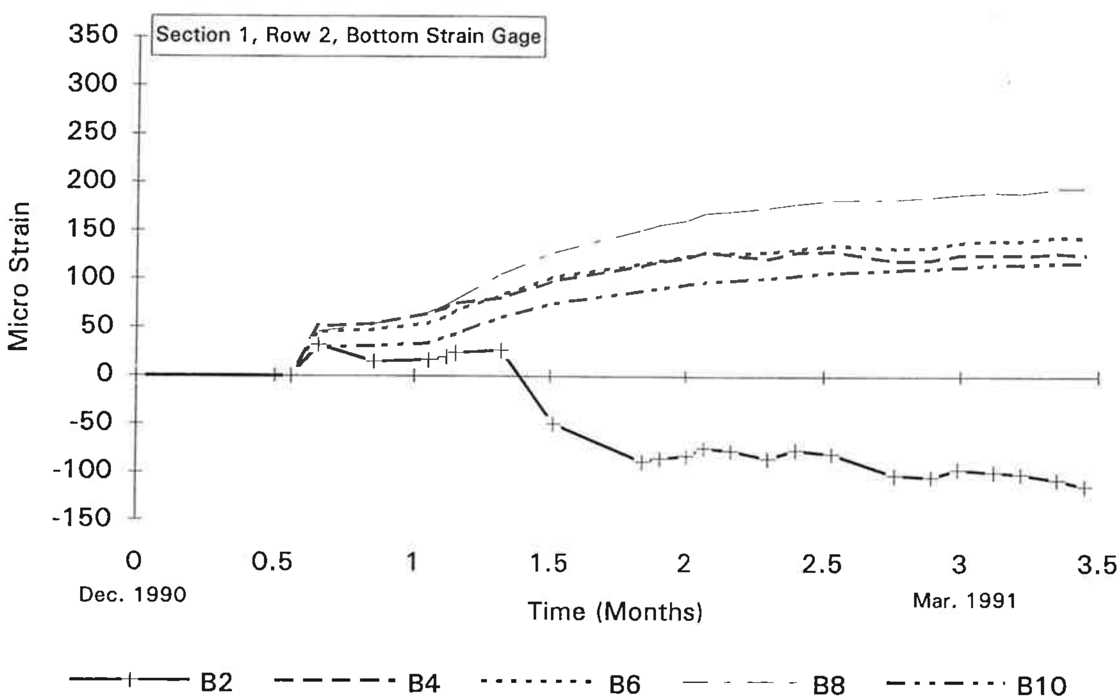


Fig. 41 - Short Term Performance

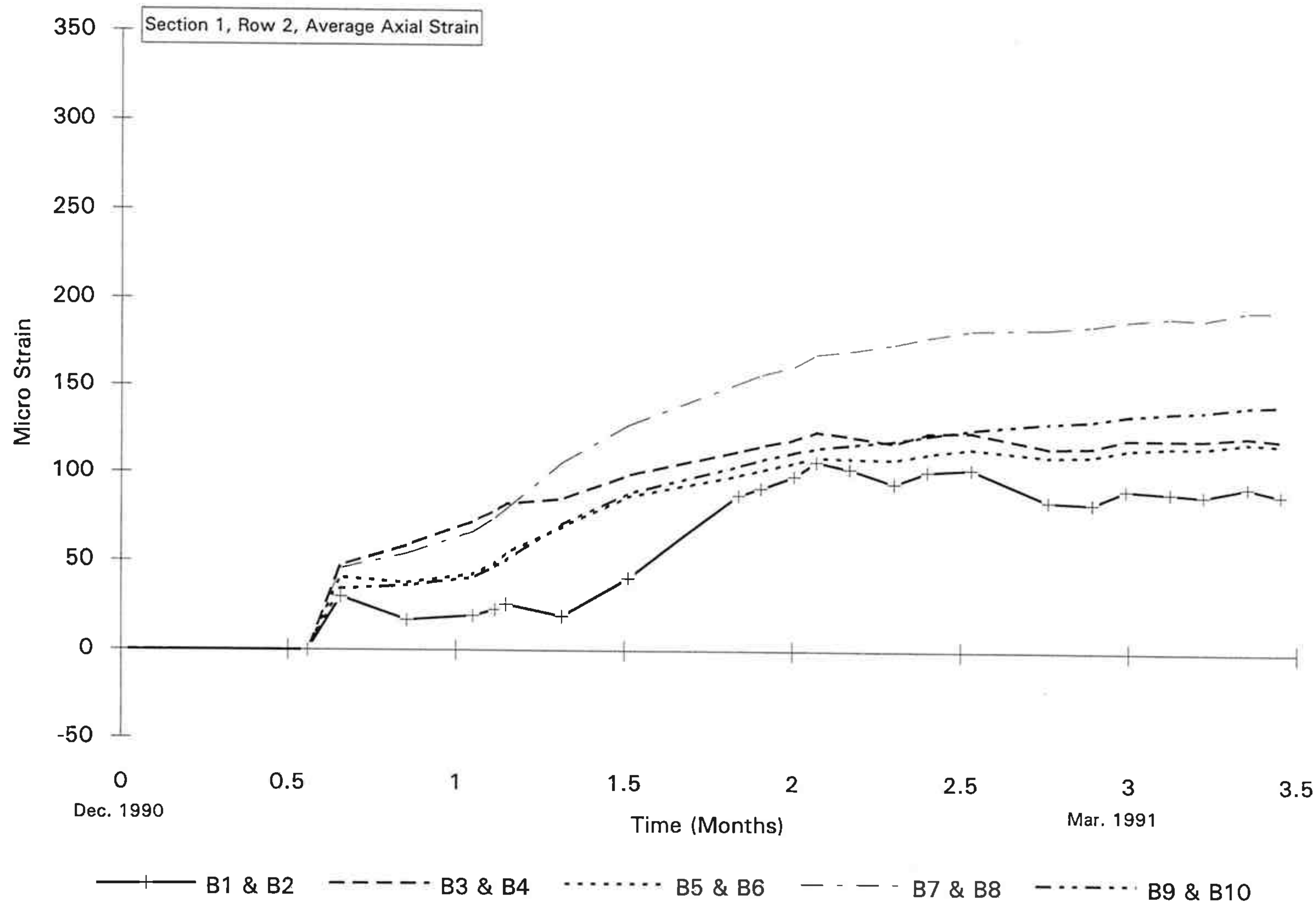


Fig. 42 - Short Term Performance

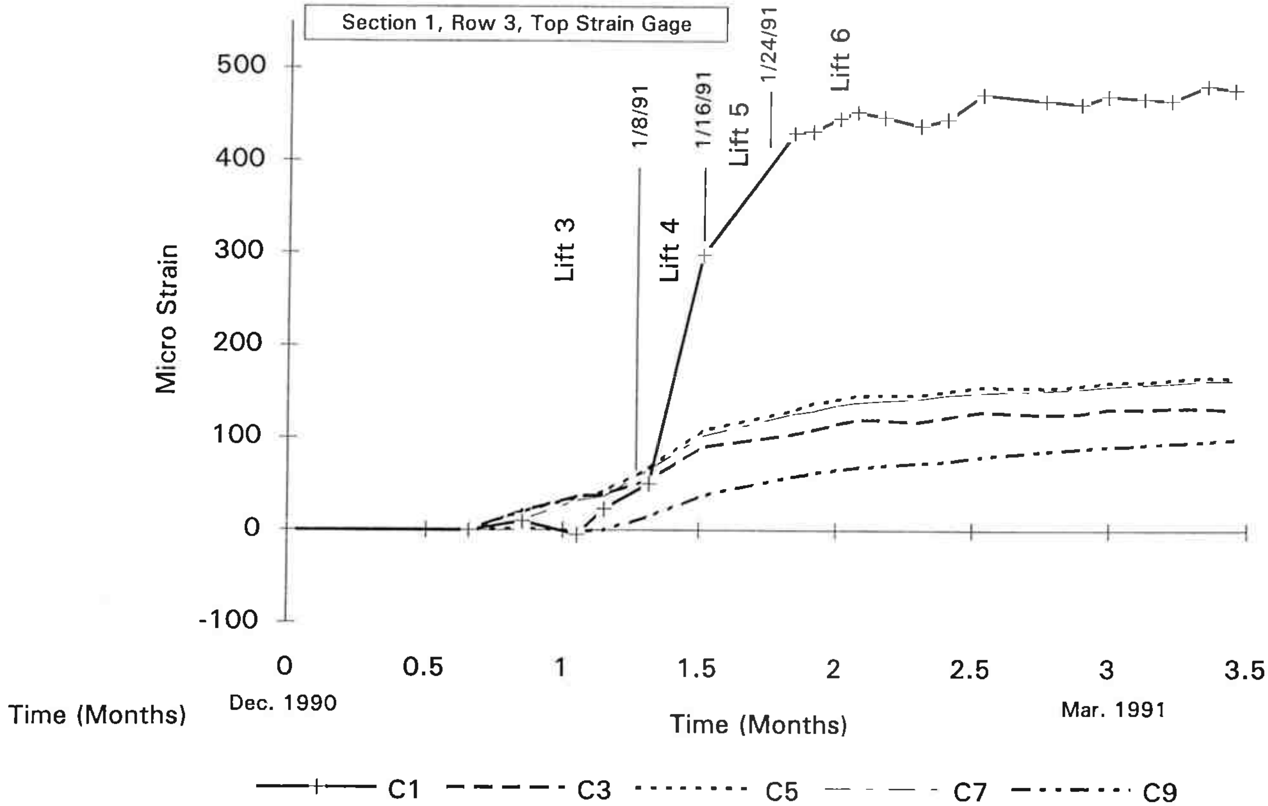


Fig. 43 - Short Term Performance

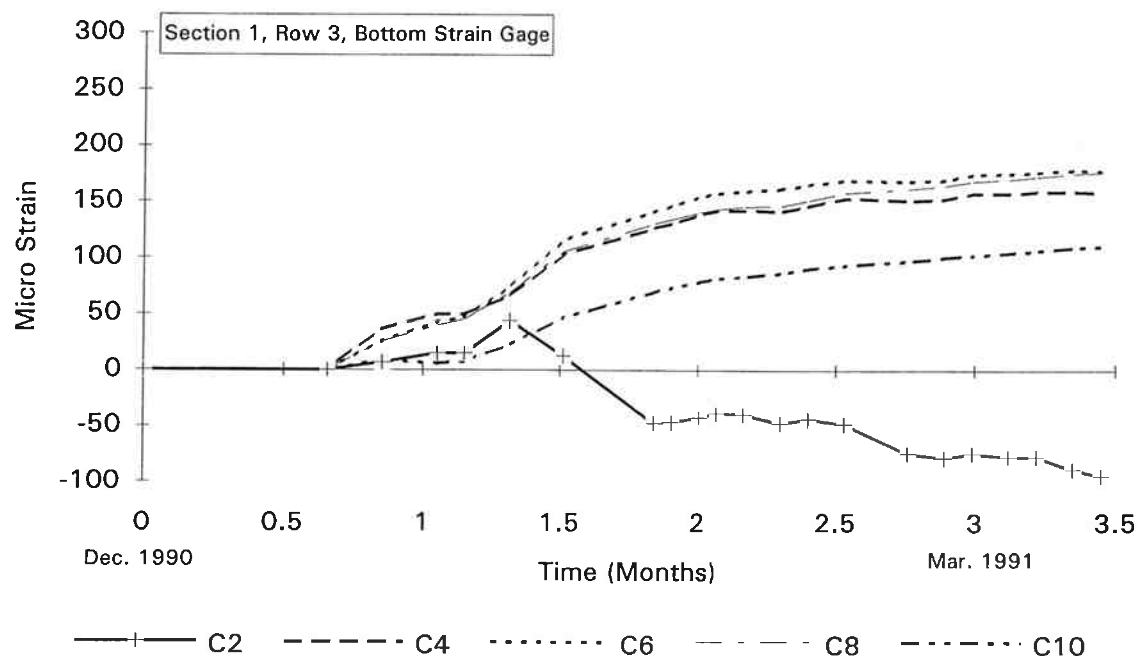


Fig. 44 - Short Term Performance

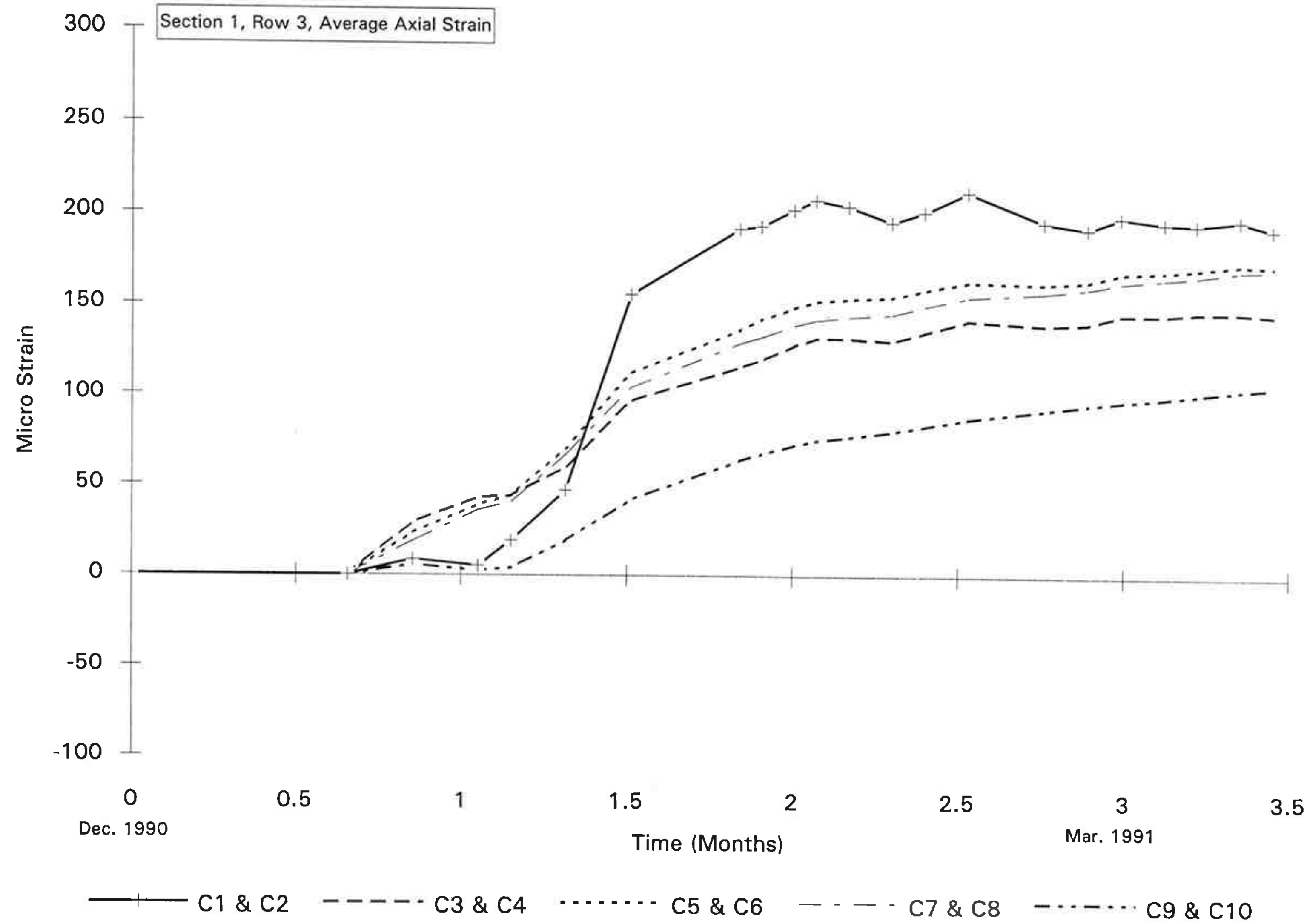


Fig. 45 - Short Term Performance

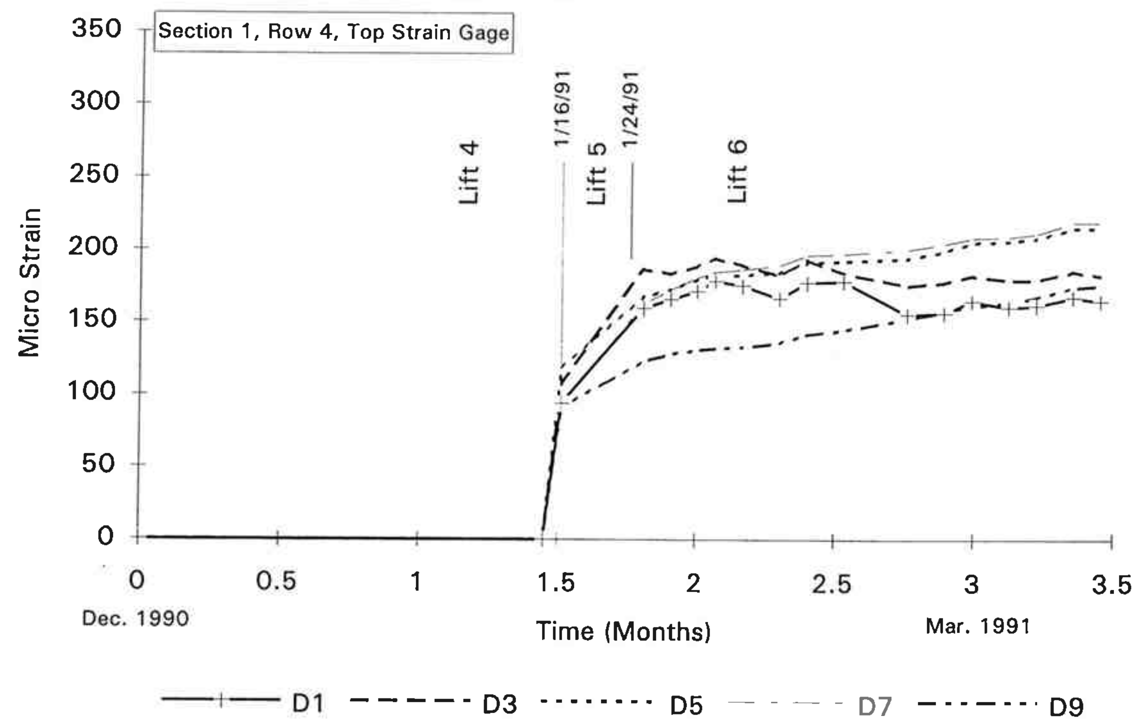


Fig. 46 - Short Term Performance

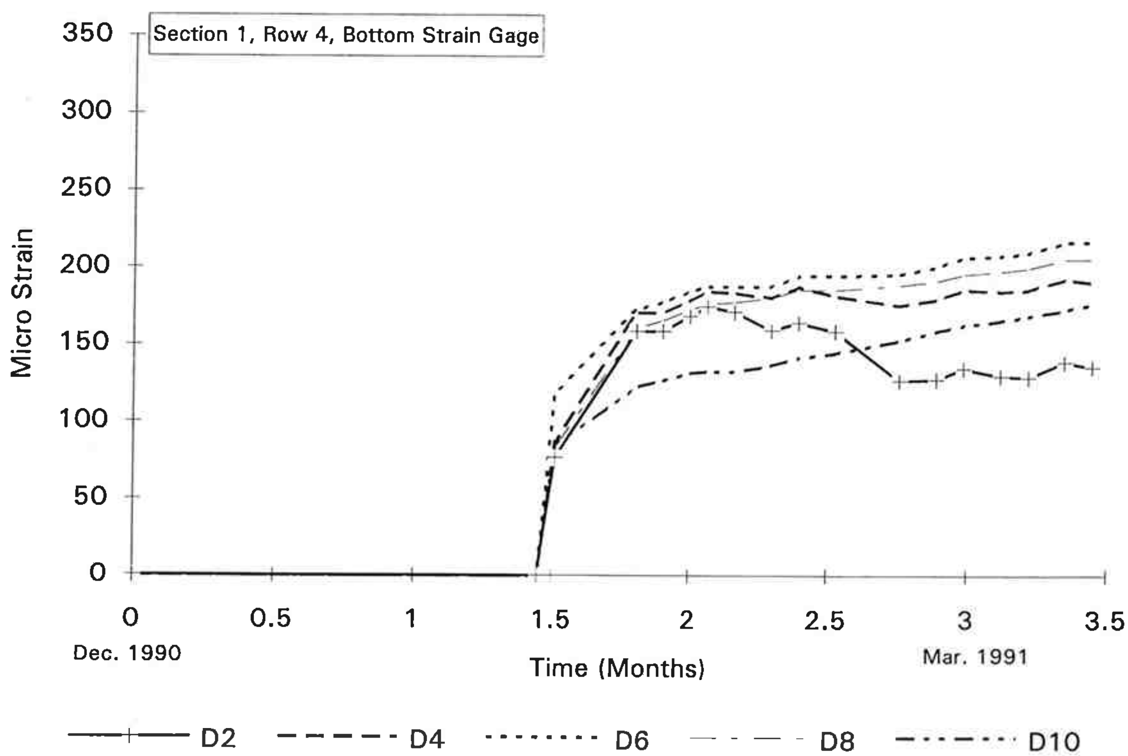


Fig. 47 - Short Term Performance

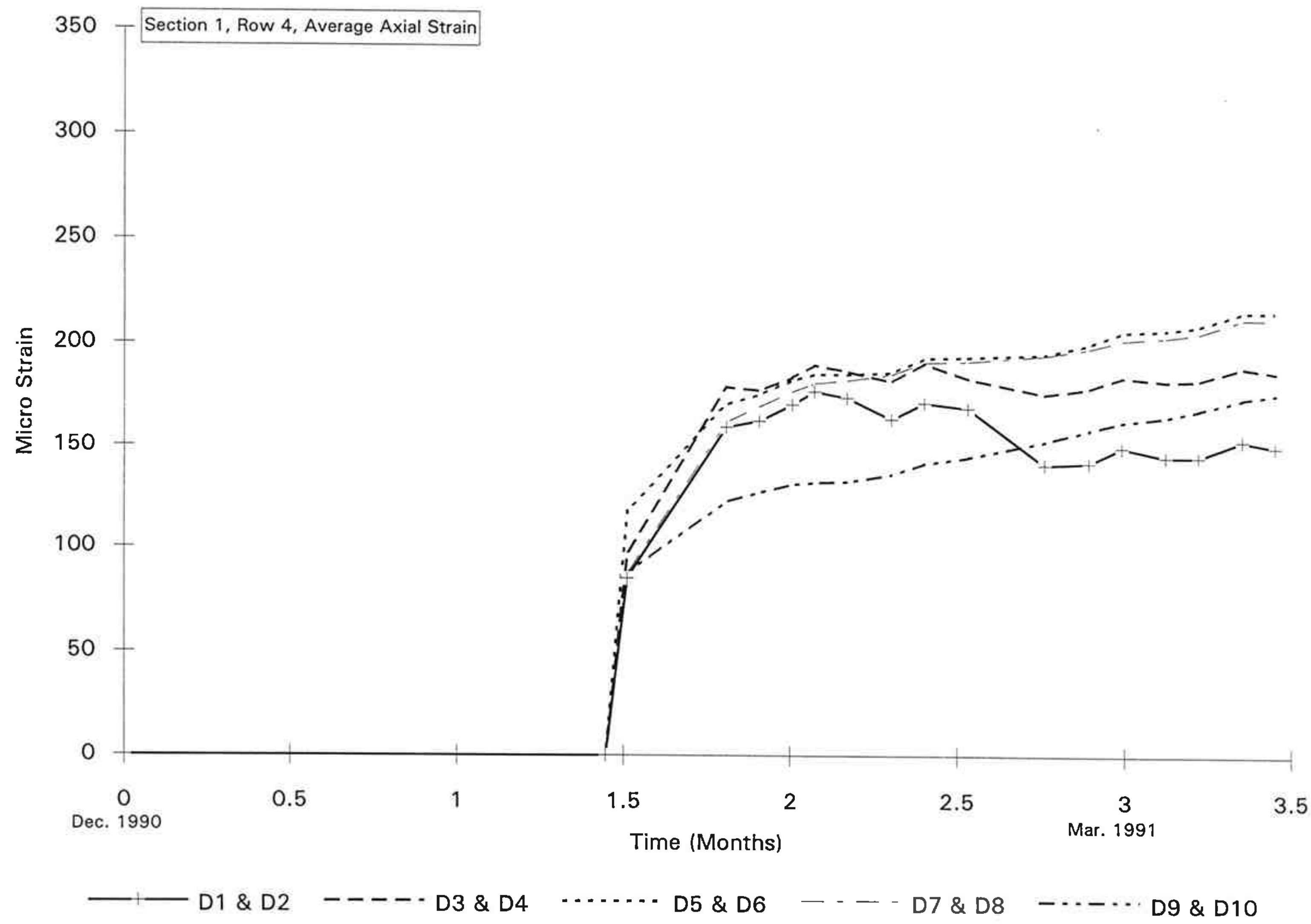


Fig. 48 - Short Term Performance

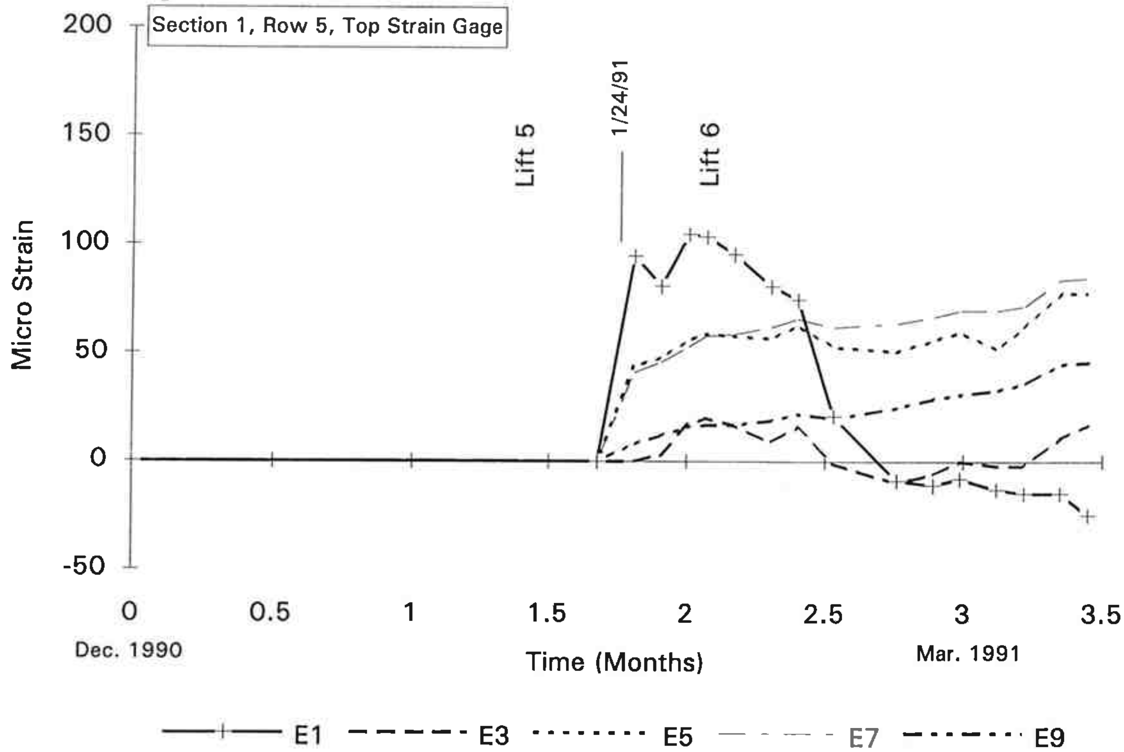


Fig. 49 - Short Term Performance

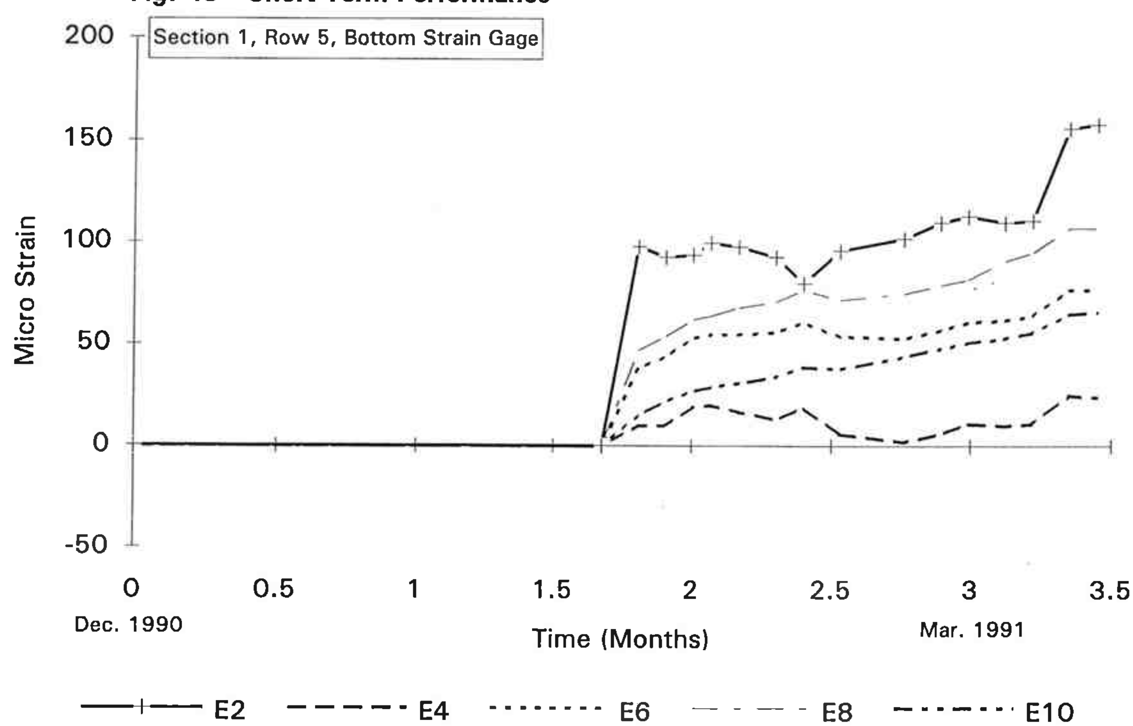


Fig. 50 - Short Term Performance

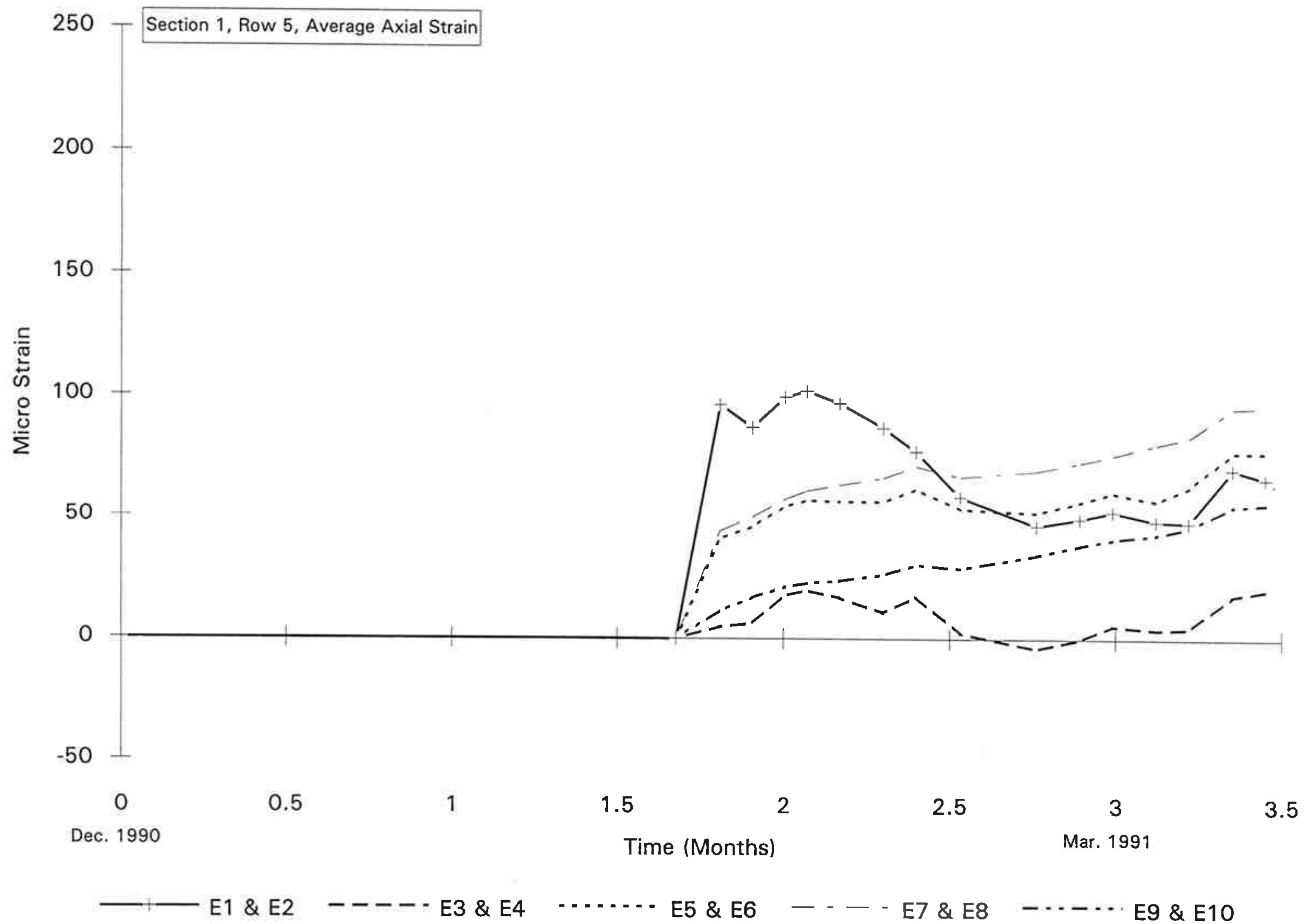


Fig. 51 - Short Term Performance

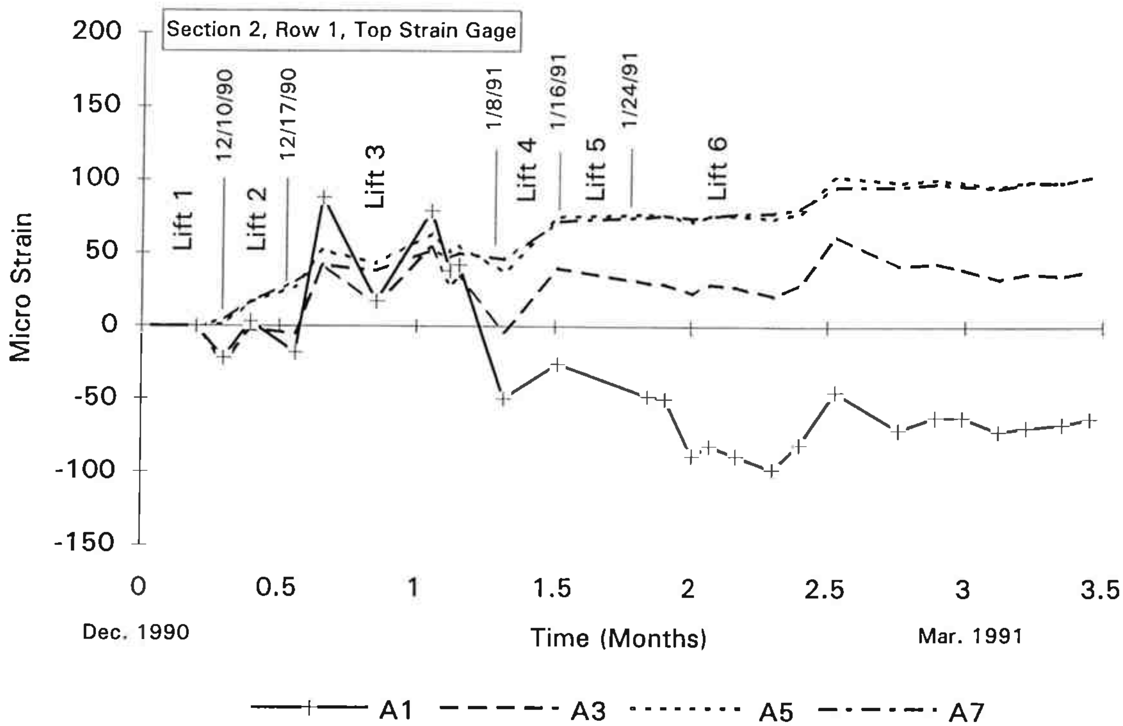


Fig. 52 - Short Term Performance

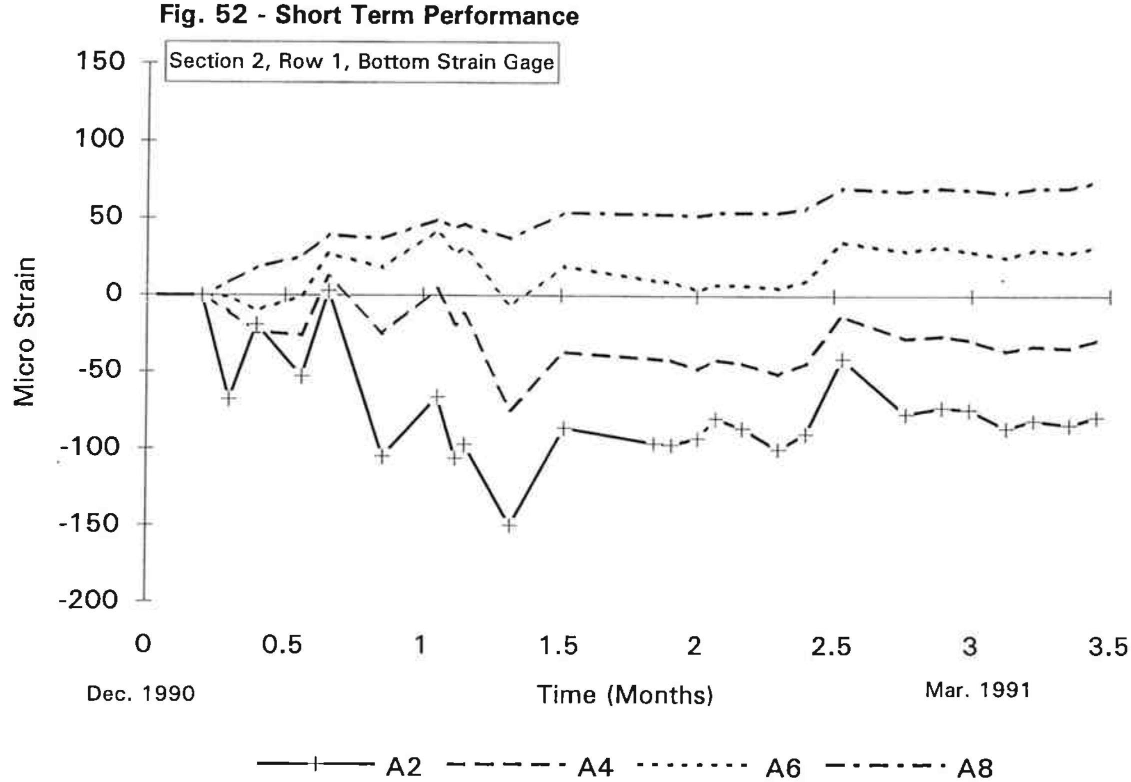


Fig. 53 - Short Term Performance

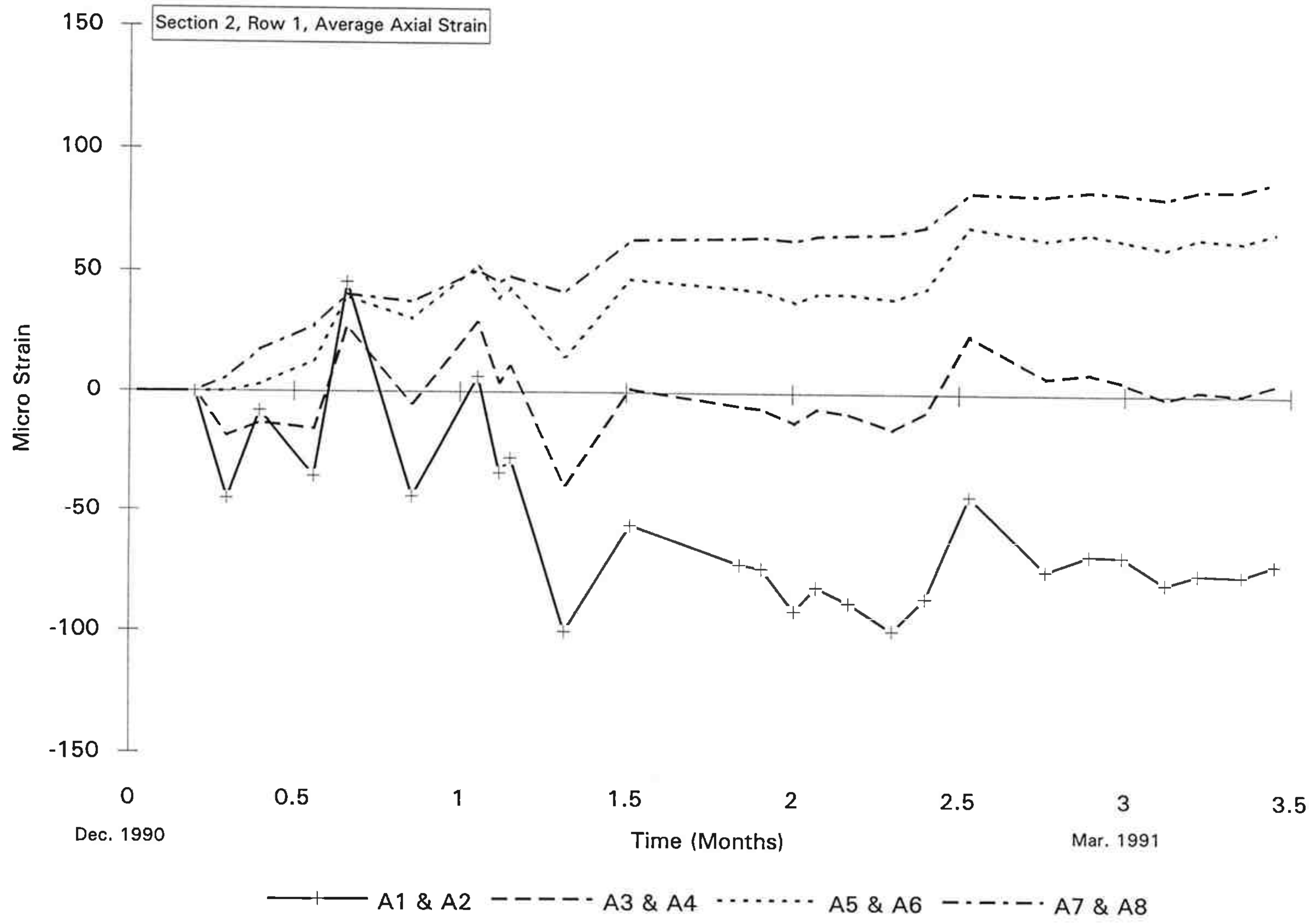


Fig. 54 - Short Term Performance

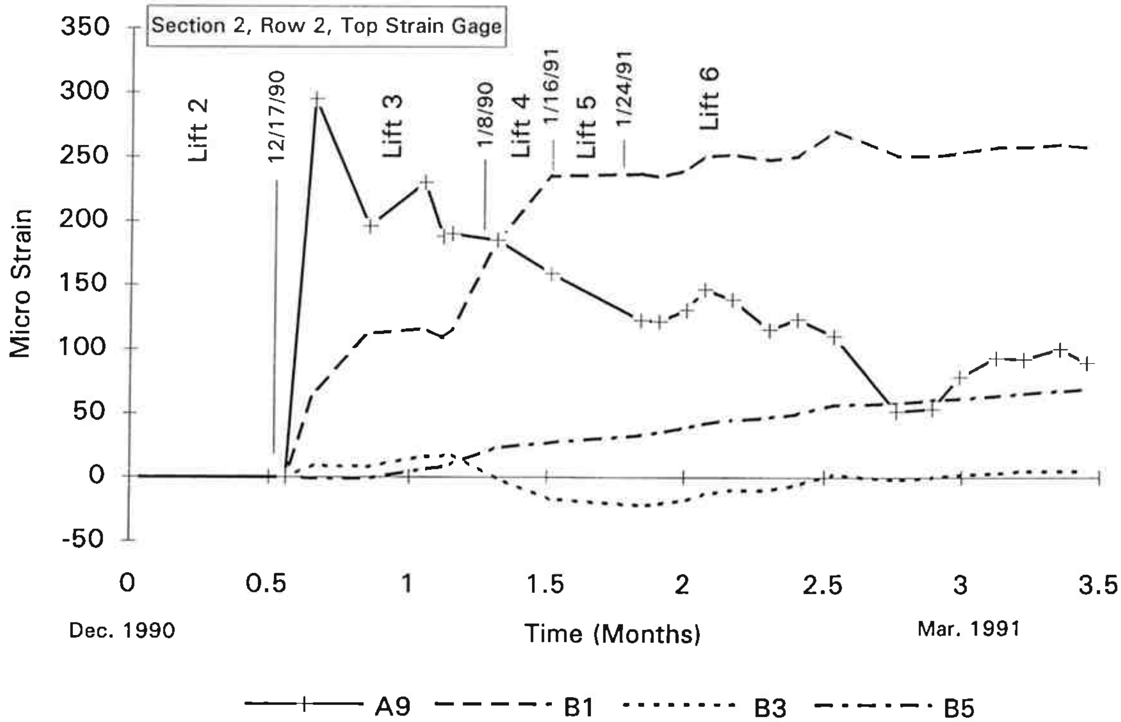


Fig. 55 - Short Term Performance

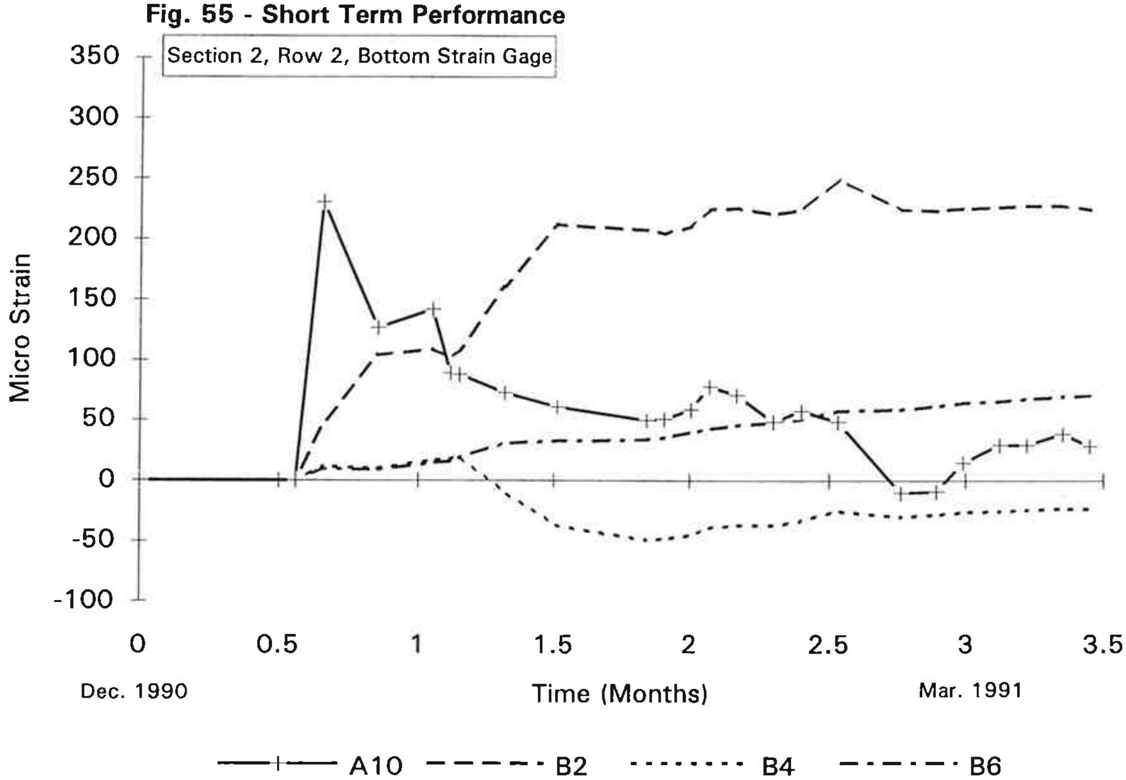


Fig. 56 - Short Term Performance

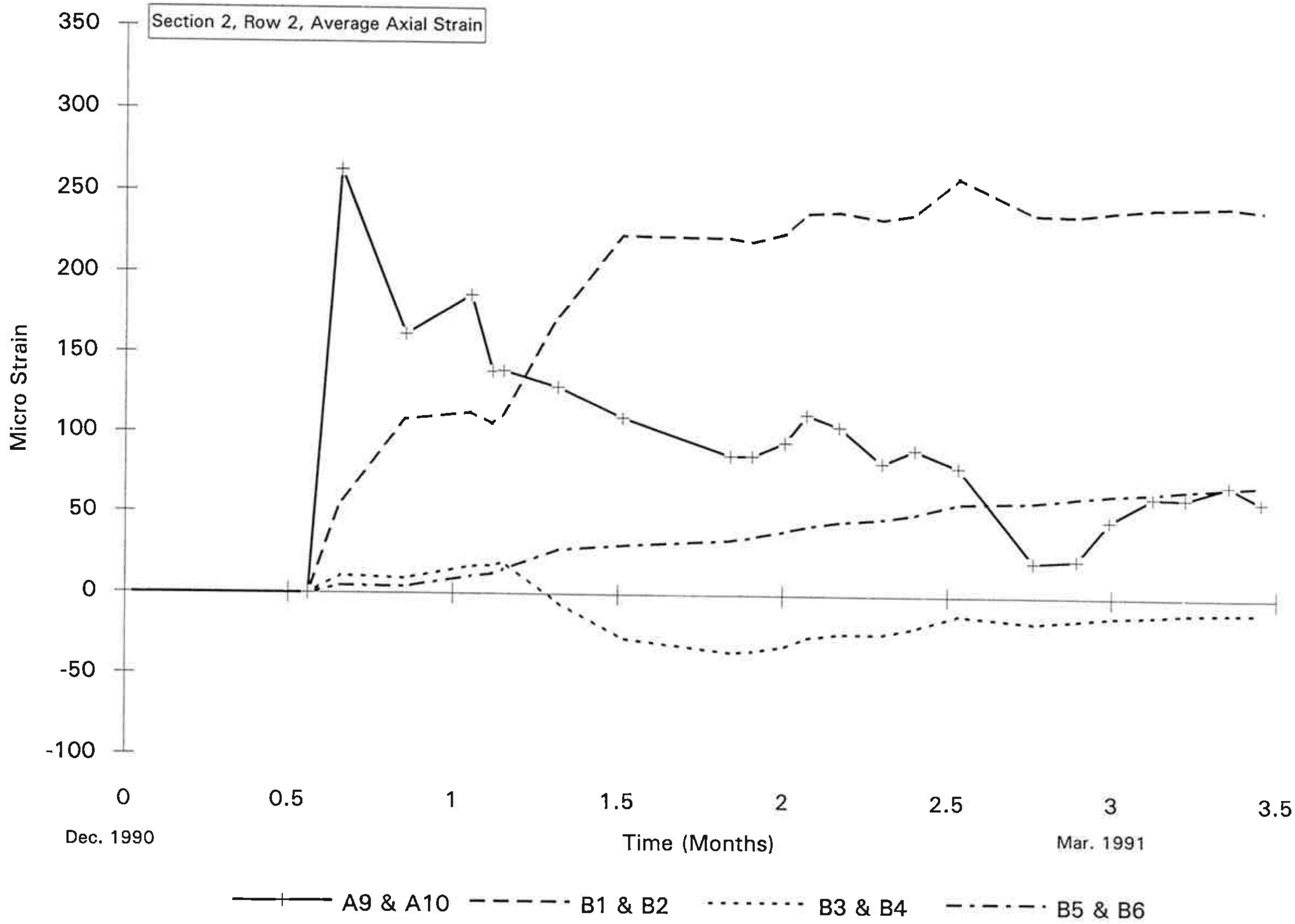


Fig. 57 - Short Term Performance

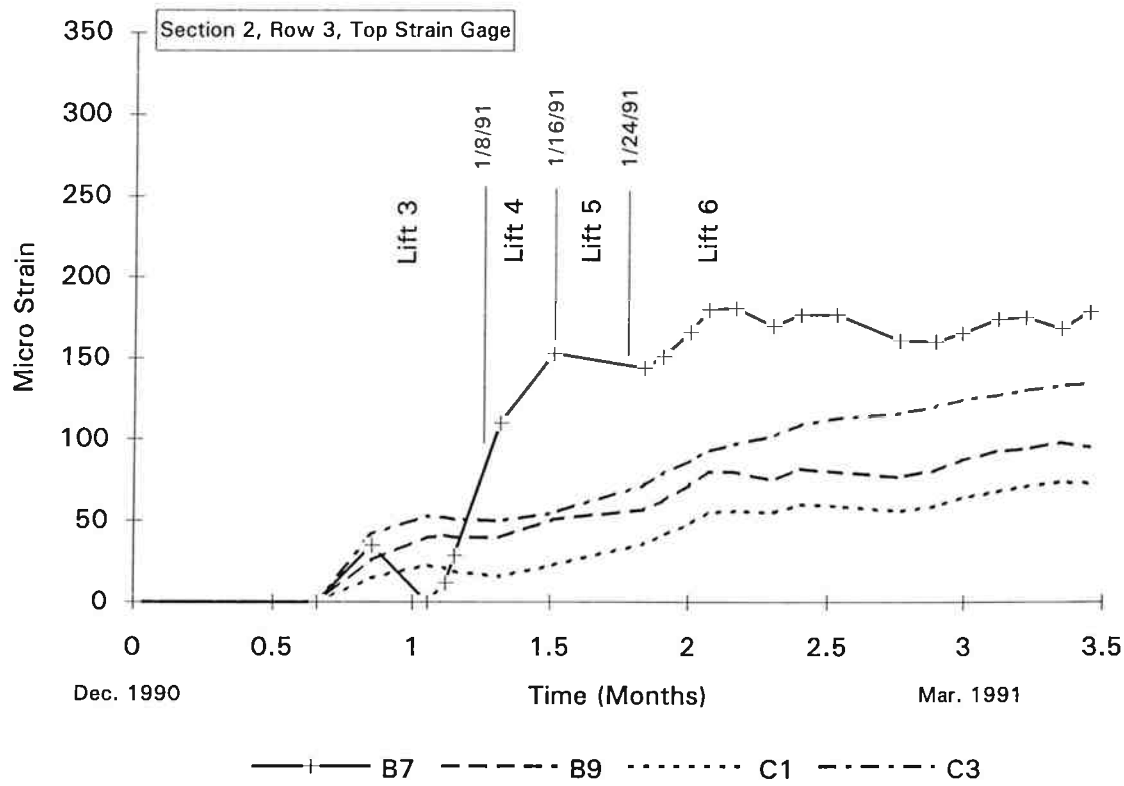


Fig. 58 - Short Term Performance

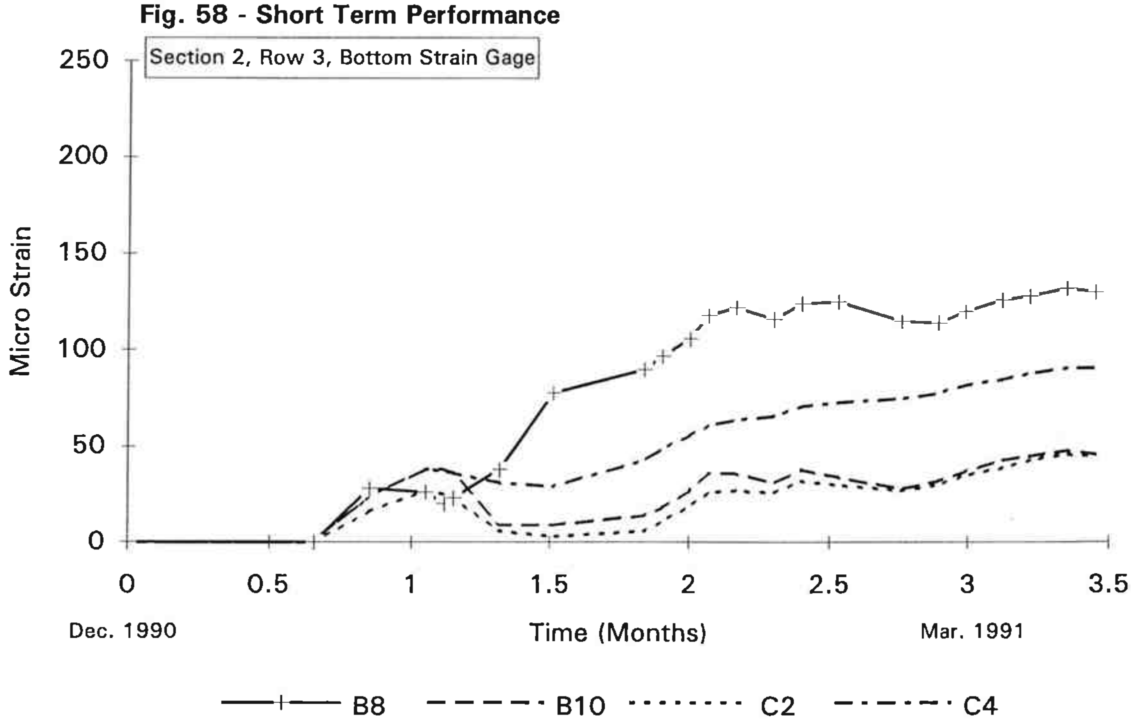


Fig. 59 - Short Term Performance

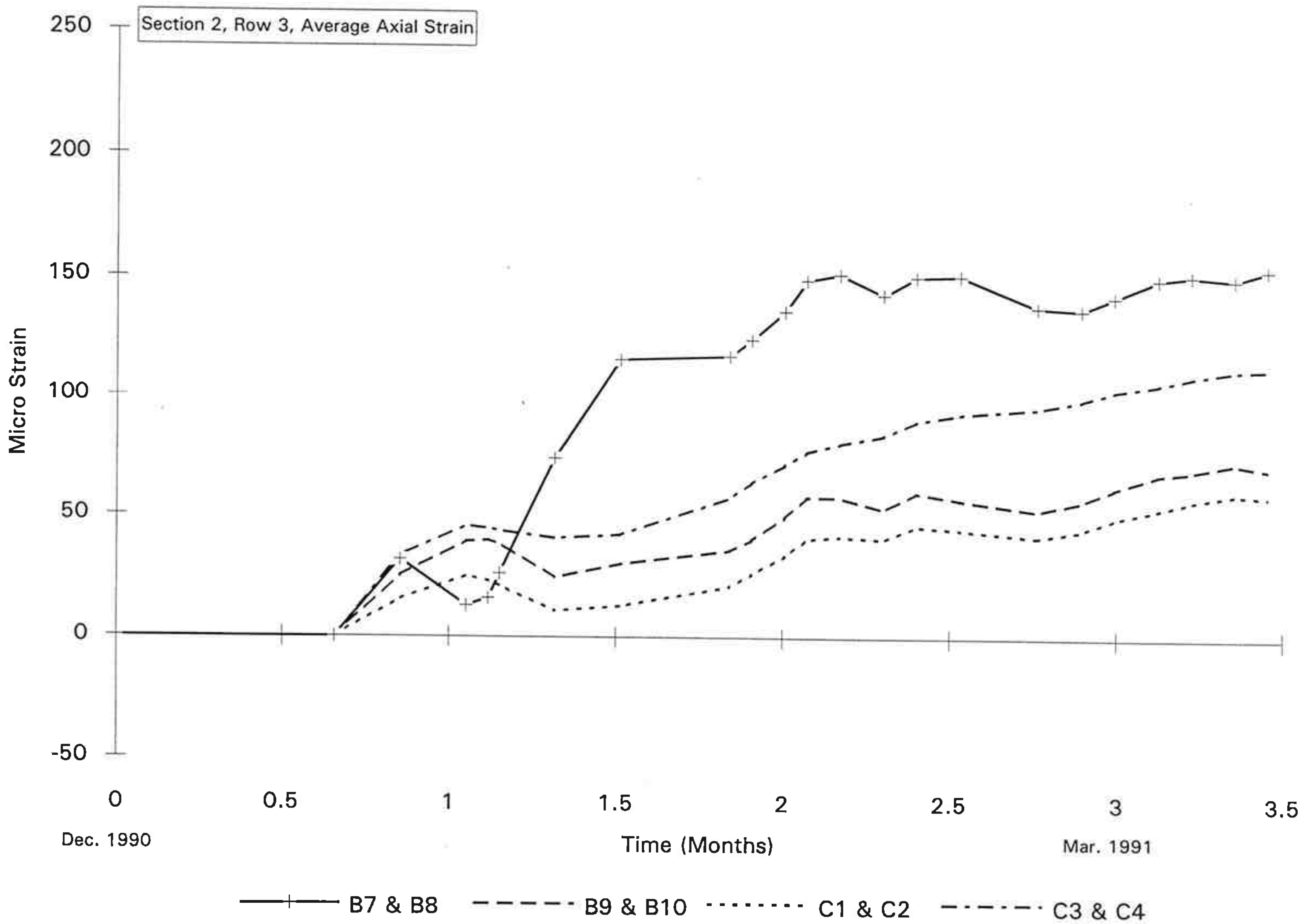


Fig. 60 - Short Term Performance

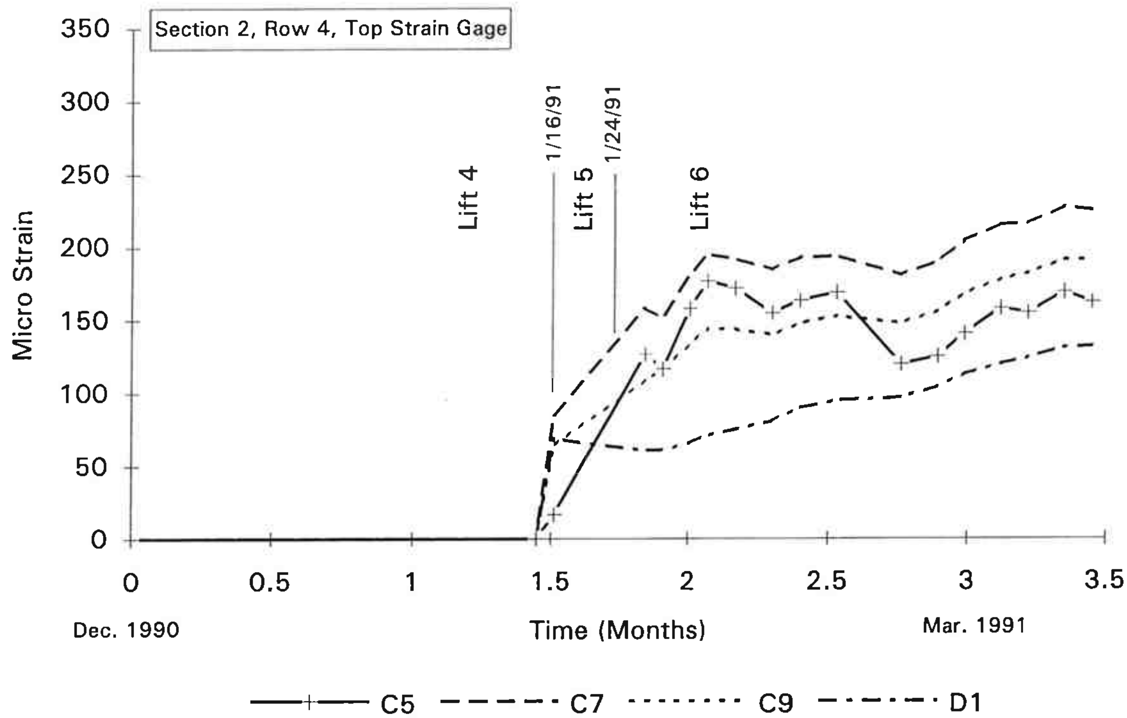


Fig. 61 - Short Term Performance

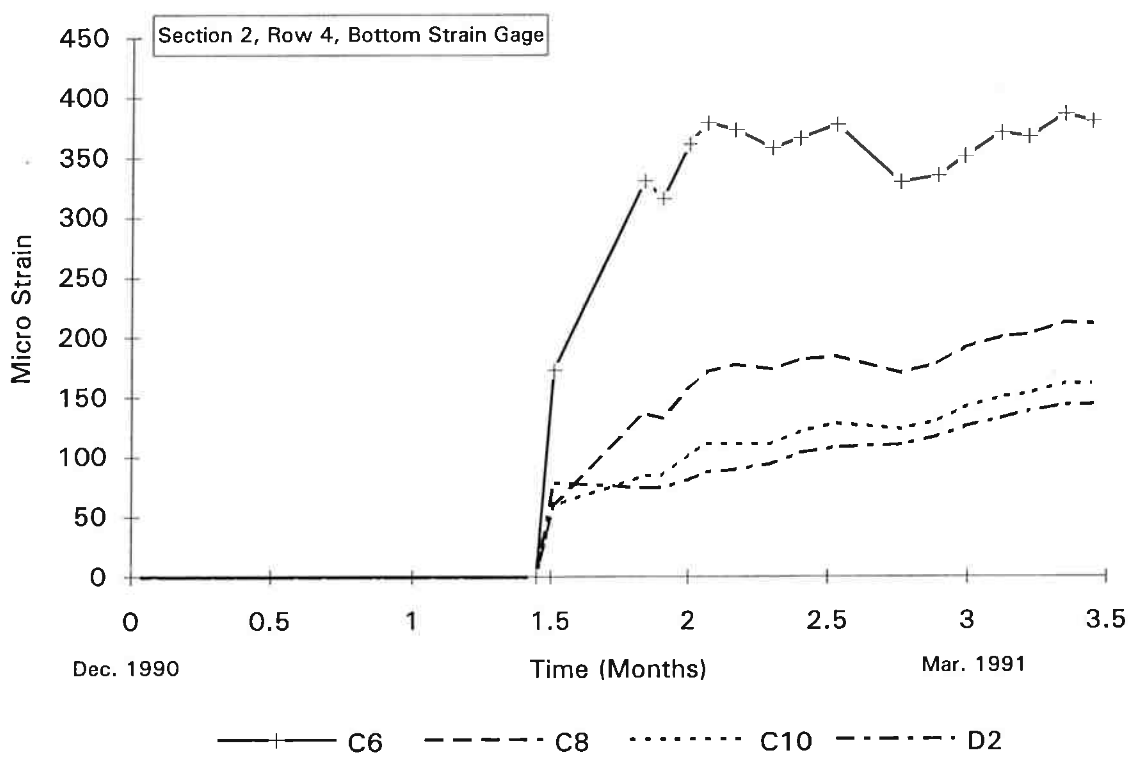


Fig. 63 - Short Term Performance

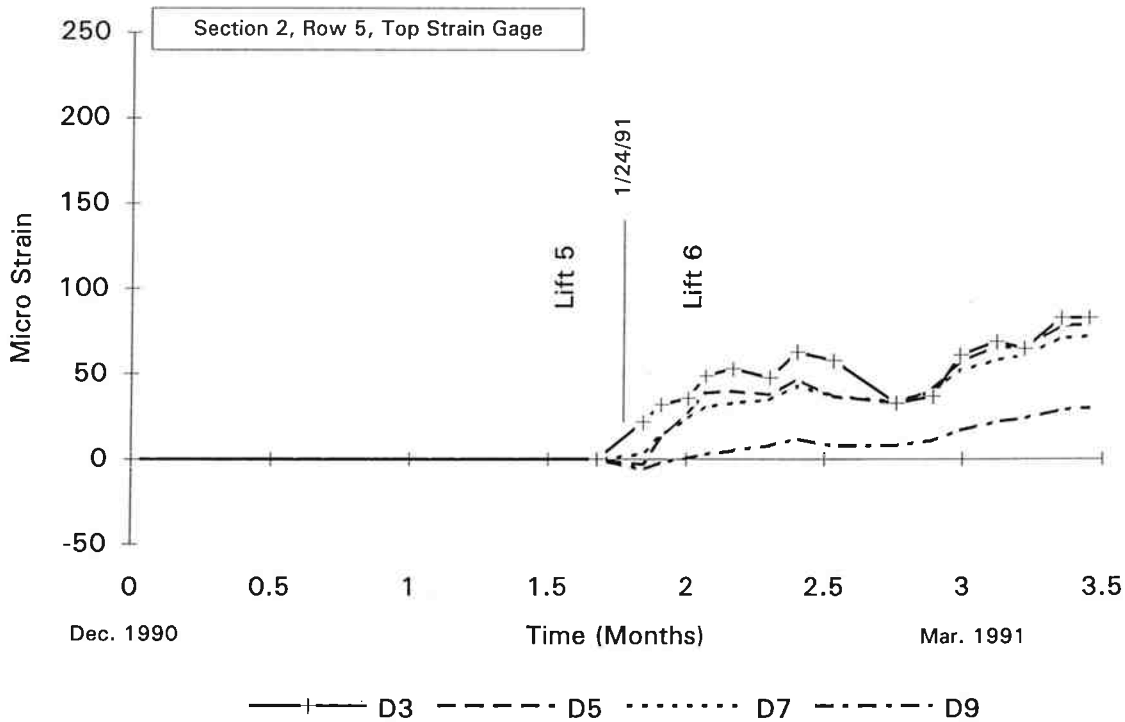


Fig. 64 - Short Term Performance

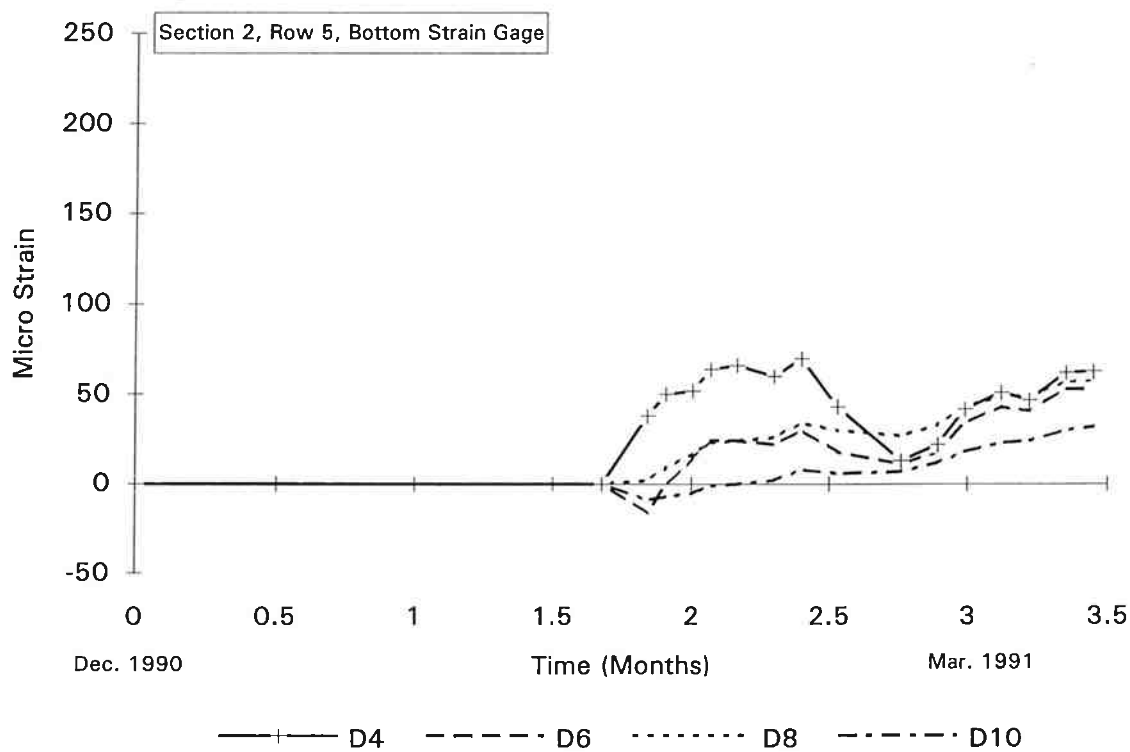


Fig. 65 - Short Term Performance

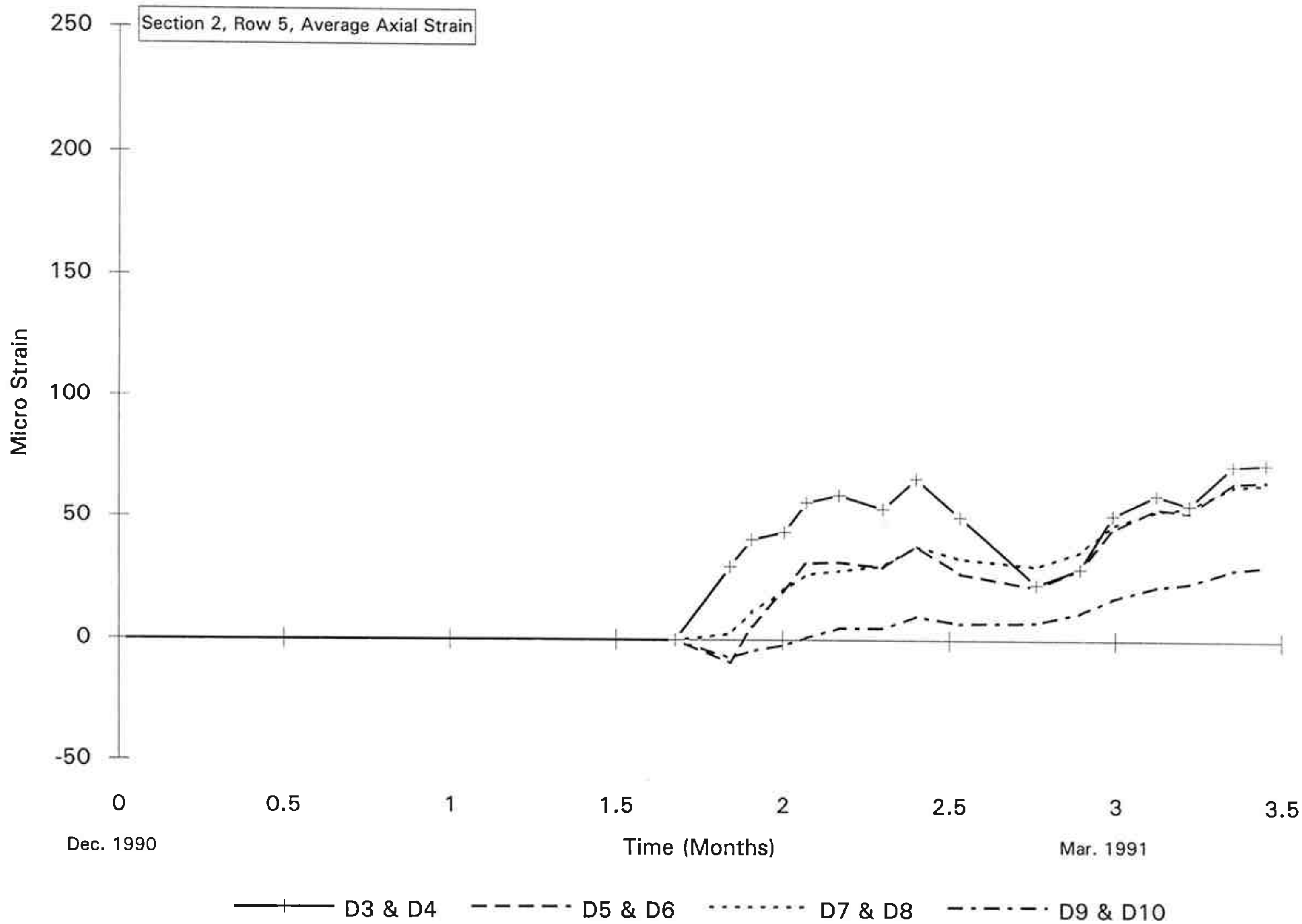


Fig. 66 - Short Term Tensile Nail Loads at Section 1, Row 1

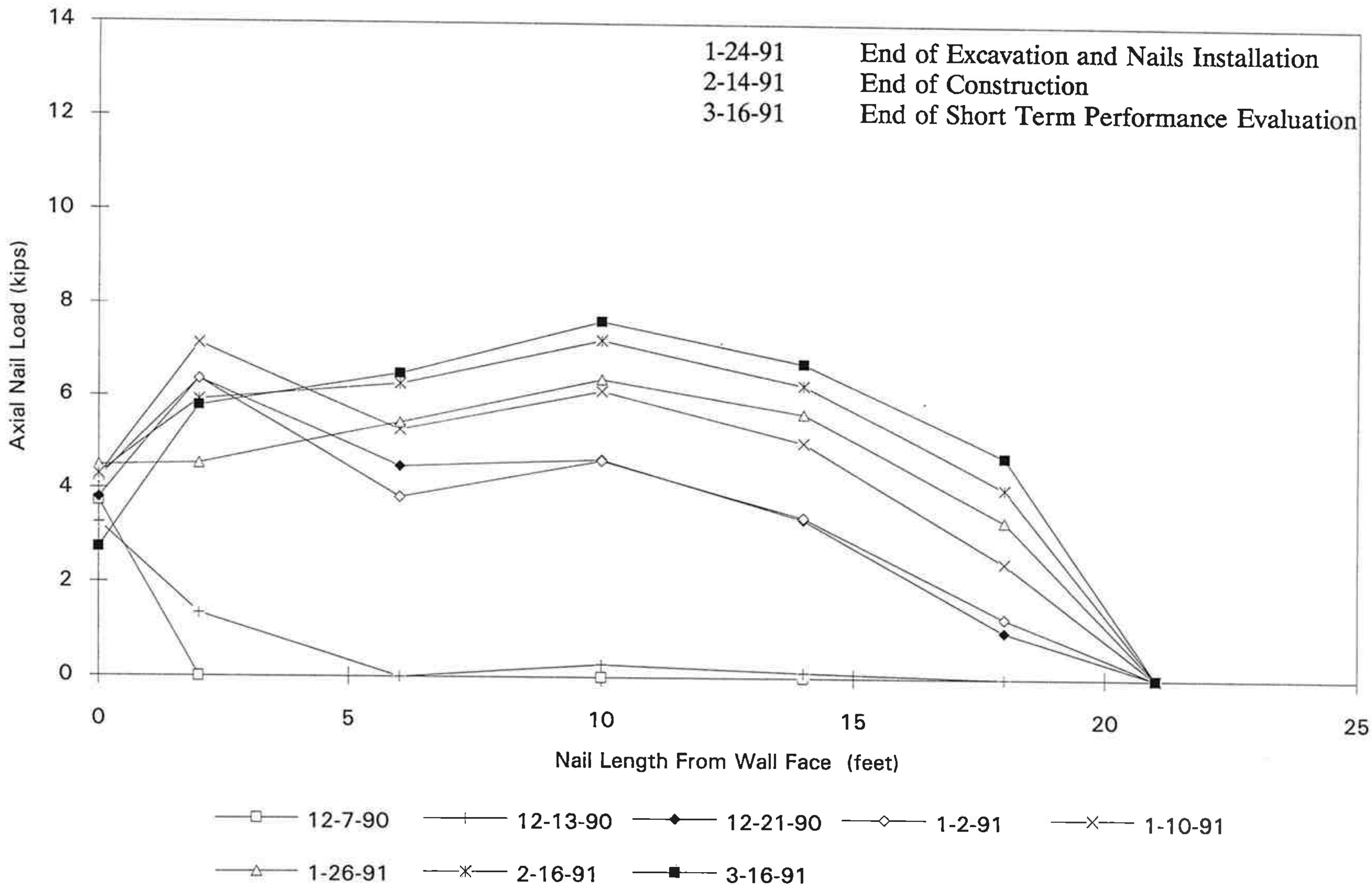


Fig. 67 - Short Term Tensile Nail Loads at Section 1, Row 2

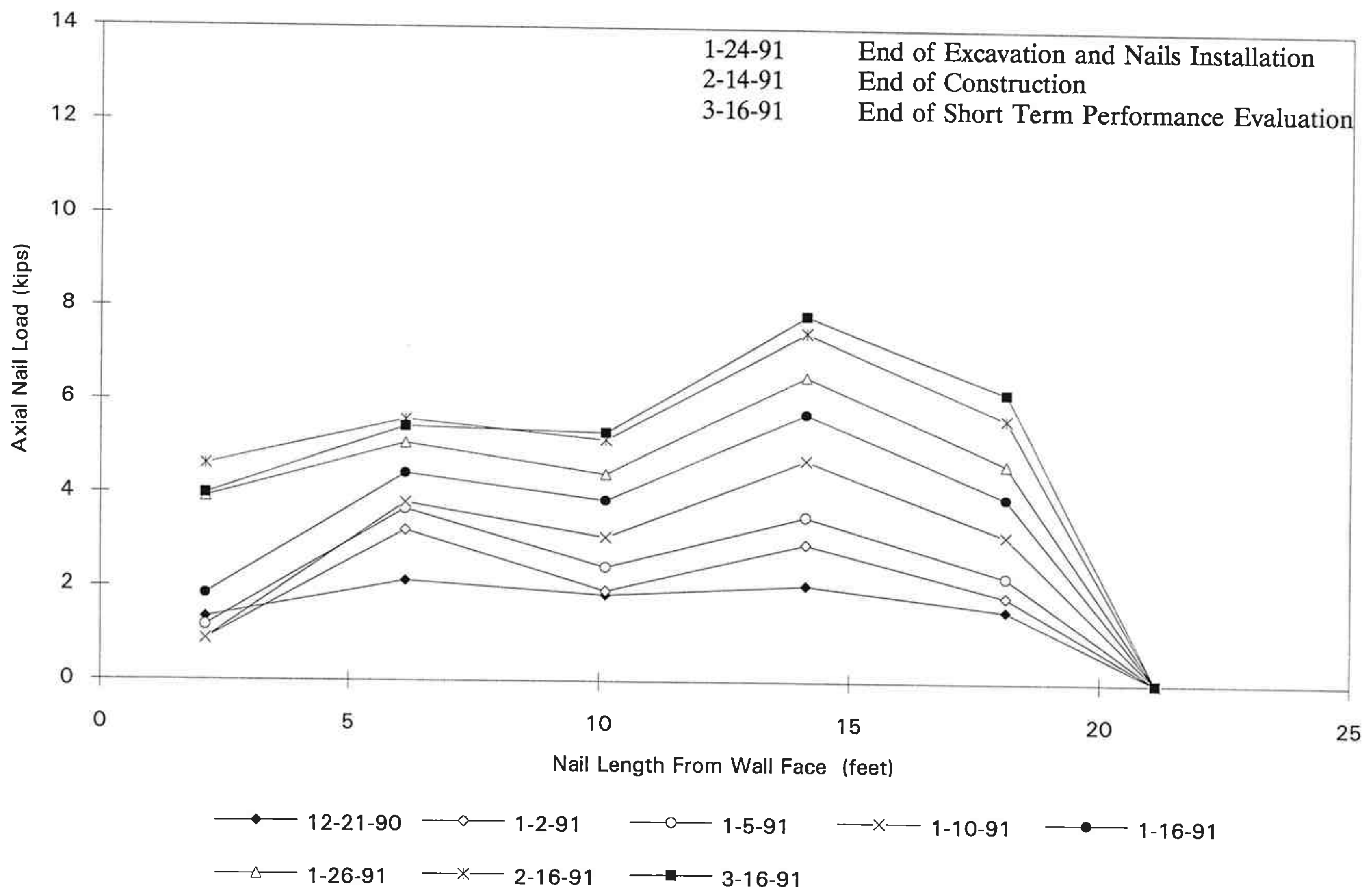


Fig. 68 - Short Term Tensile Nail Loads at Section 1, Row 3

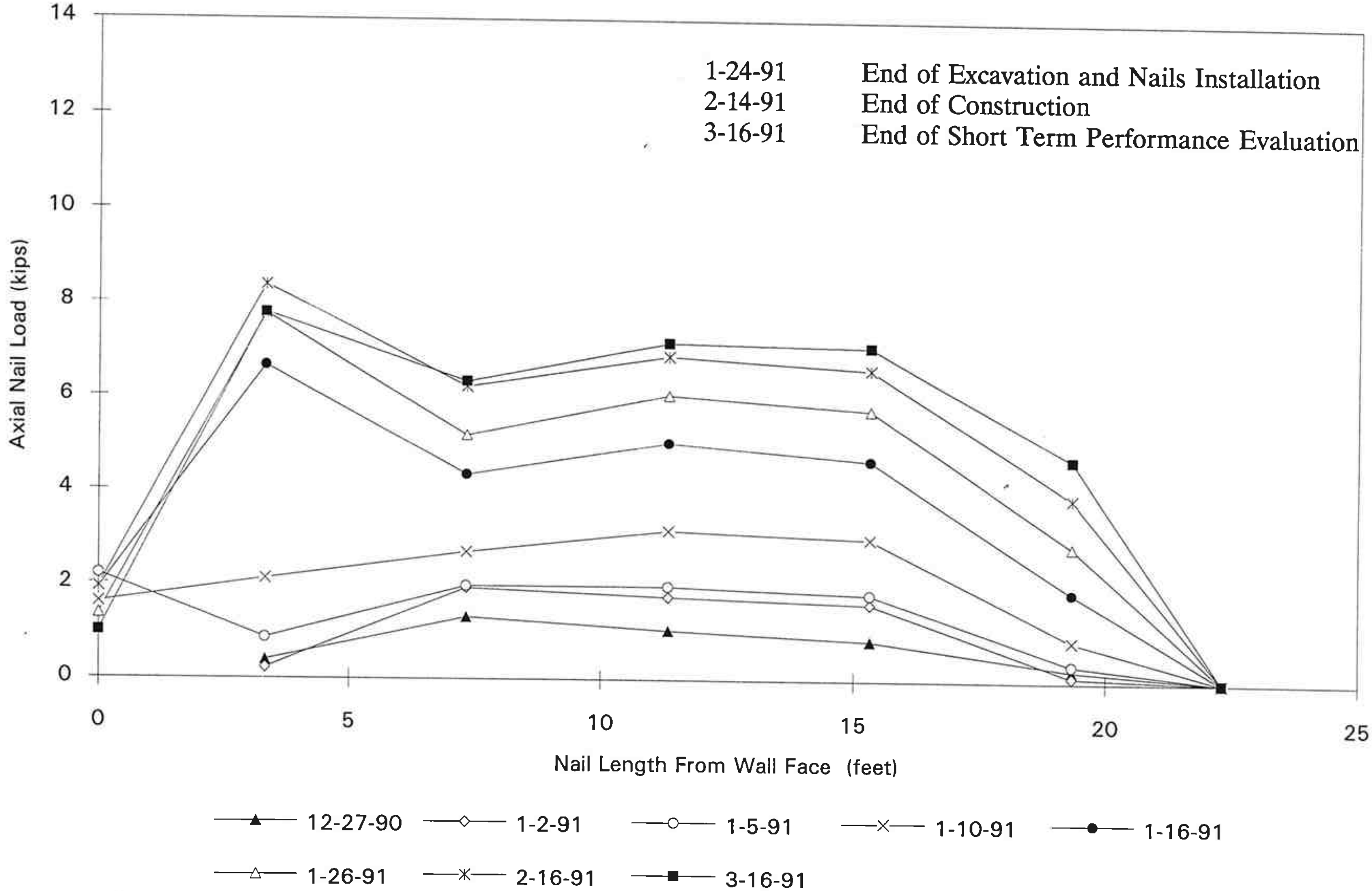


Fig. 69 - Short Term Tensile Nail Loads at Section 1, Row 4

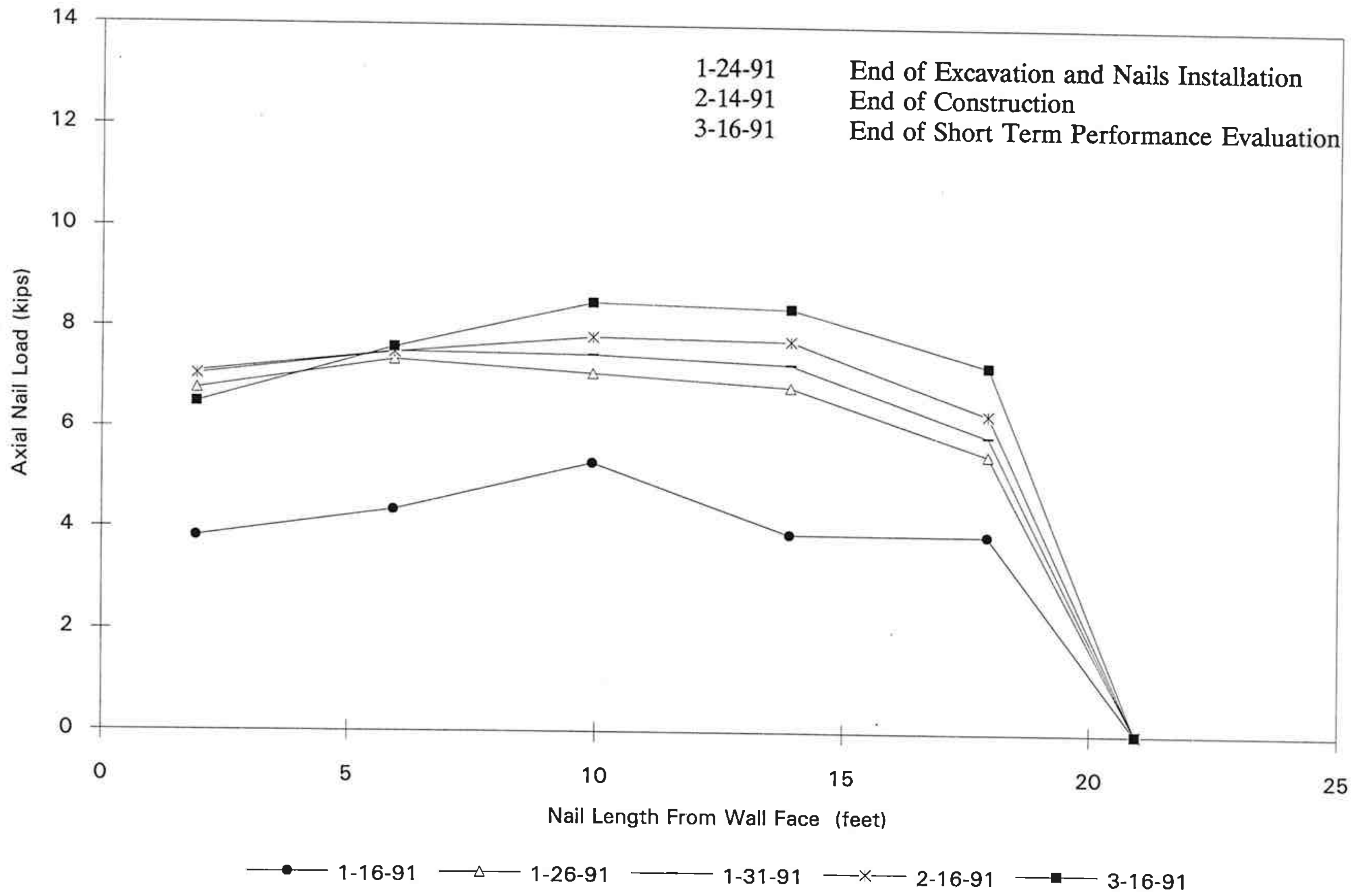


Fig. 70 - Short Term Tensile Nail Loads at Section 1, Row 5

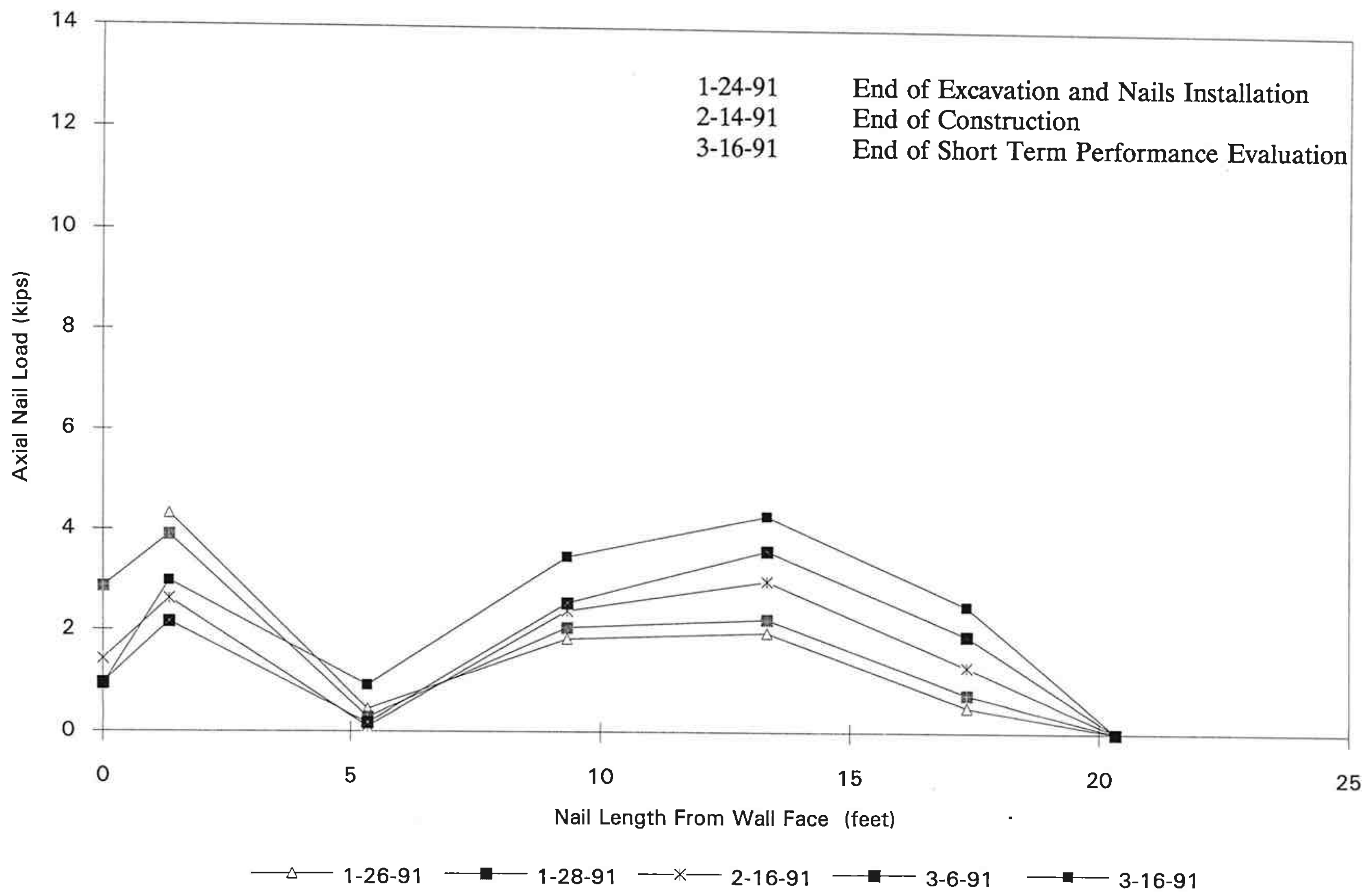


Fig. 71 - Short Term Tensile Nail Loads at Section 2, Row 1

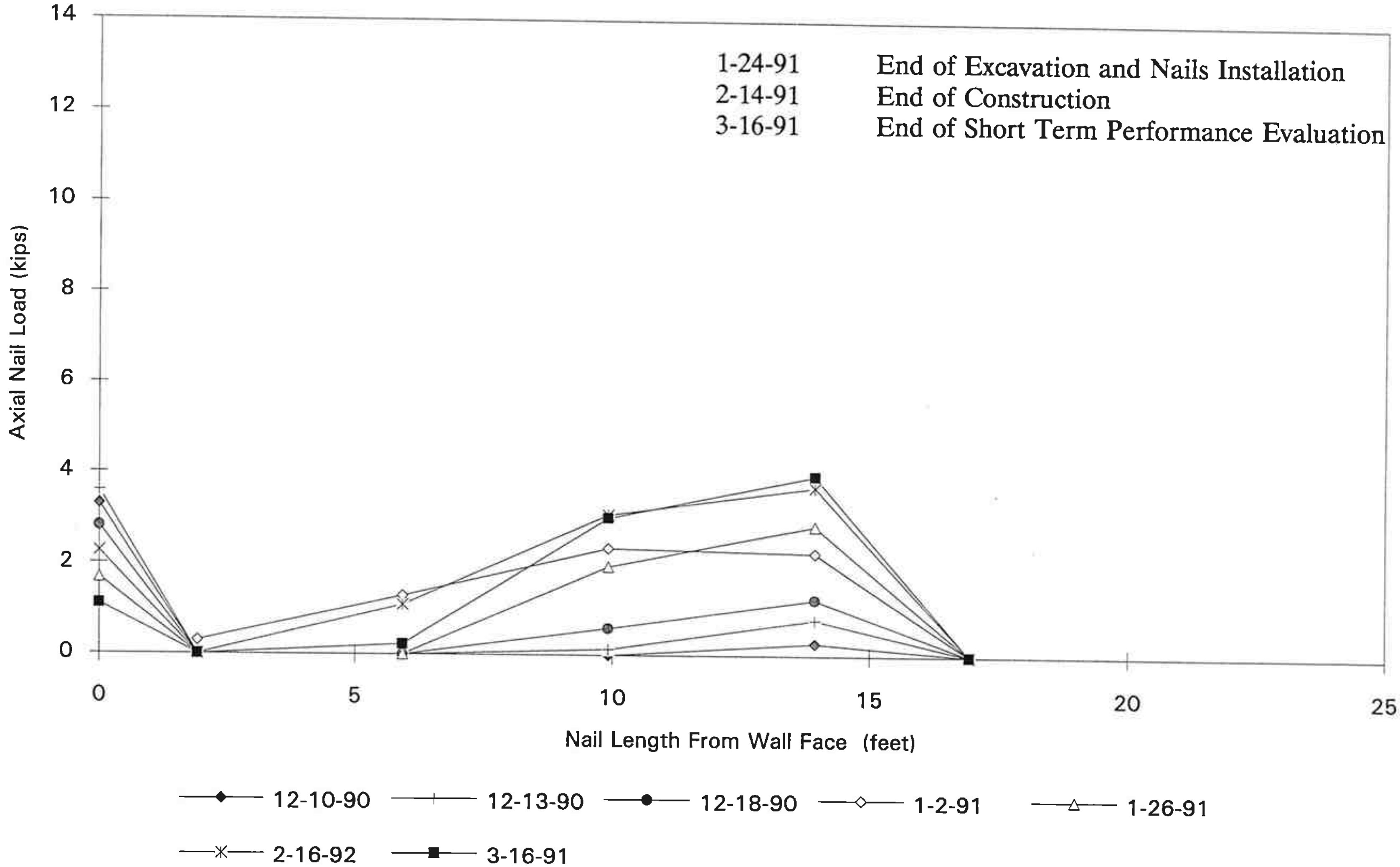


Fig. 72 - Short Term Tensile Nail Loads at Section 2, Row 2

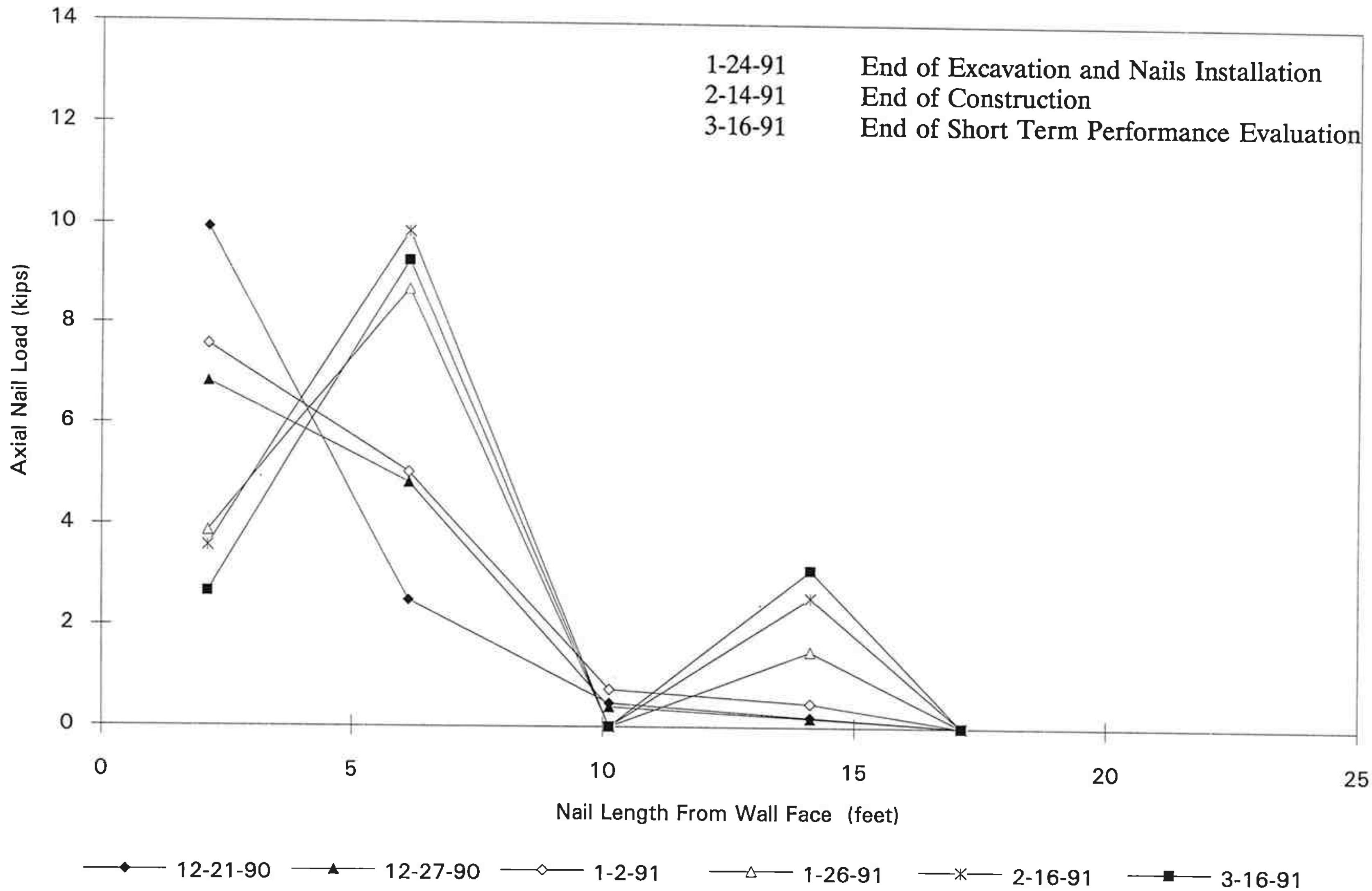


Fig. 73 - Short Term Tensile Nail Loads at Section 2, Row 3

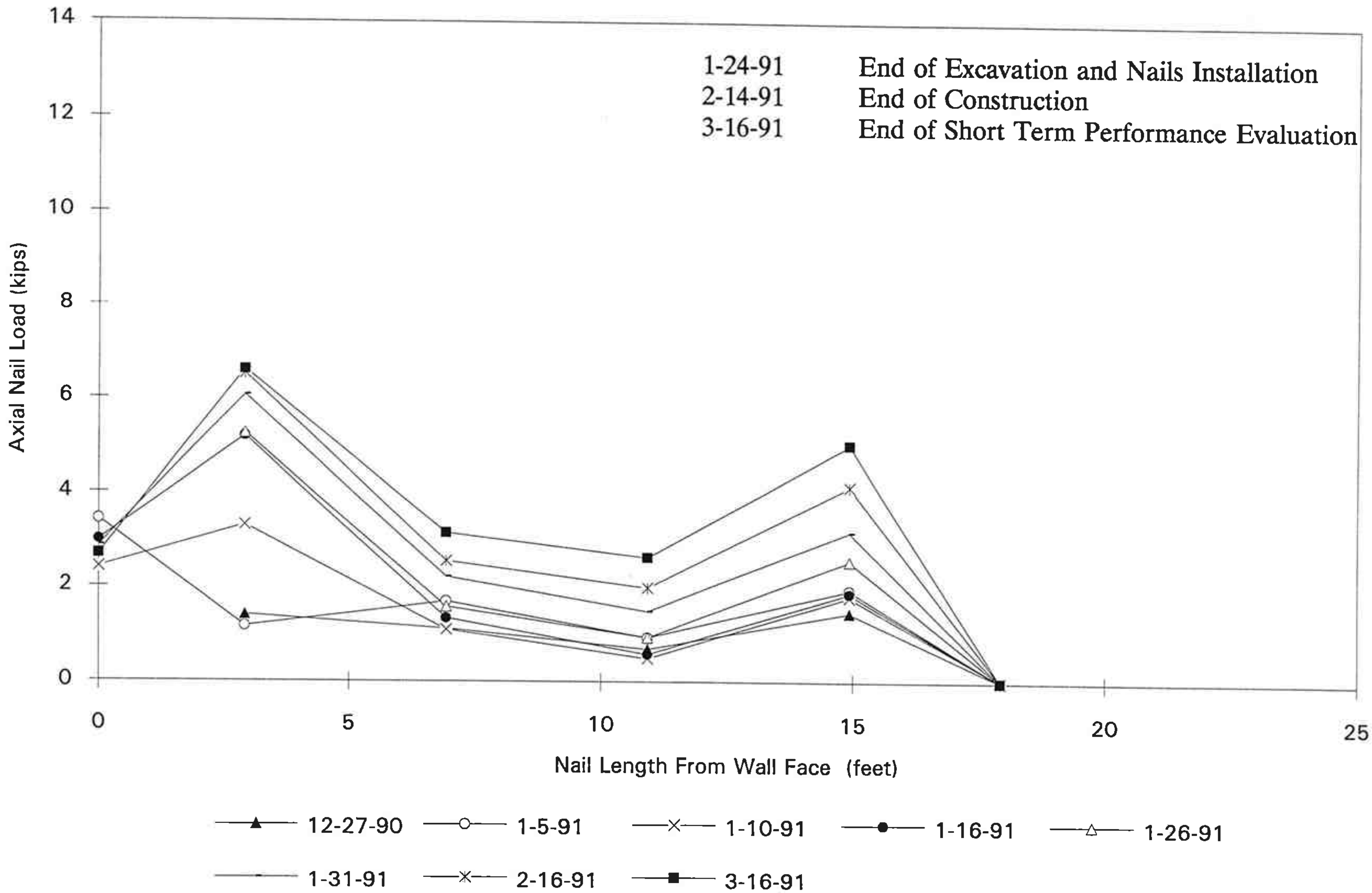


Fig. 74 - Short Term Tensile Nail Loads at Section 2, Row 4

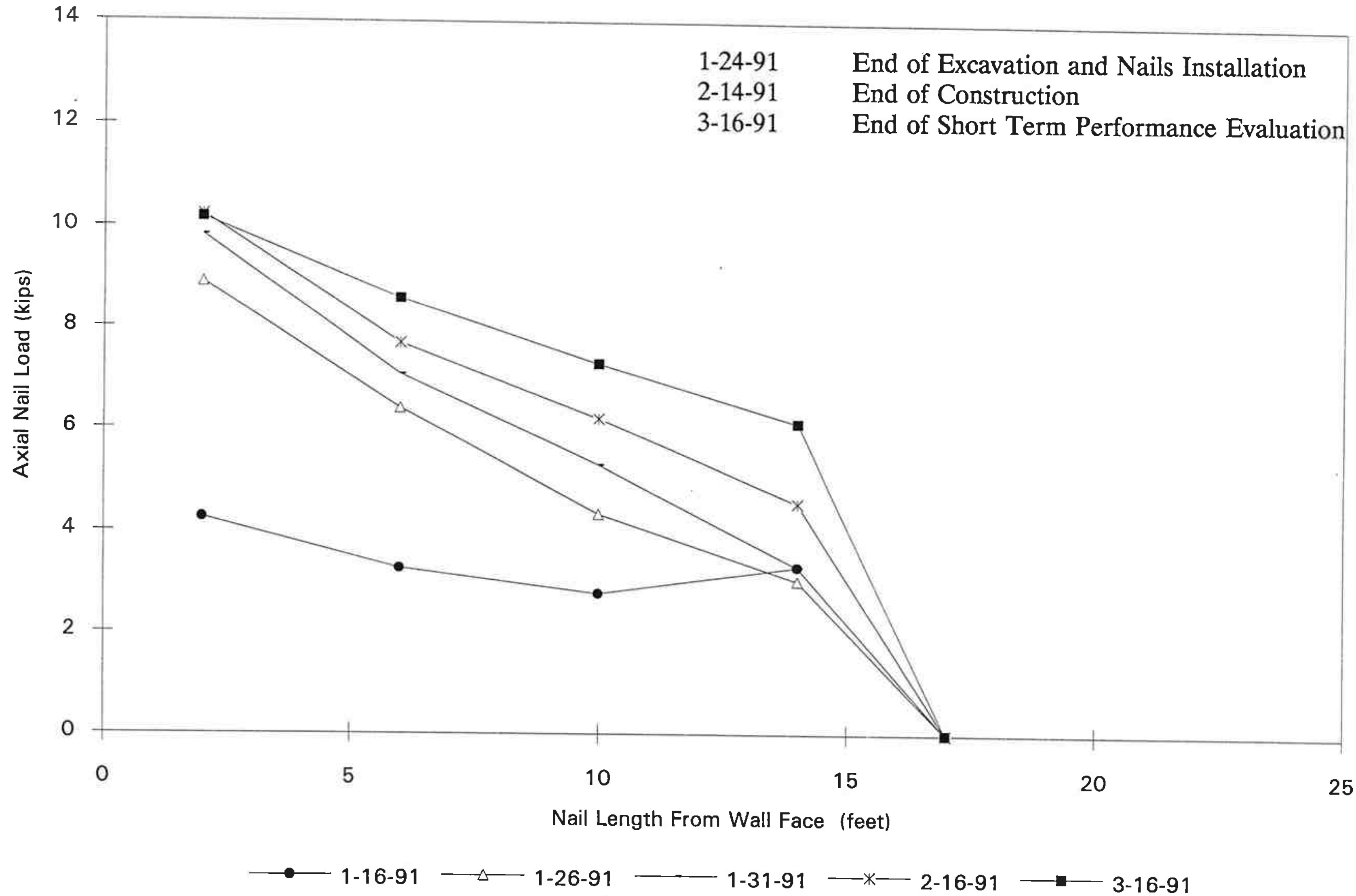


Fig. 75 - Short Term Tensile Nail Loads at Section 2, Row 5

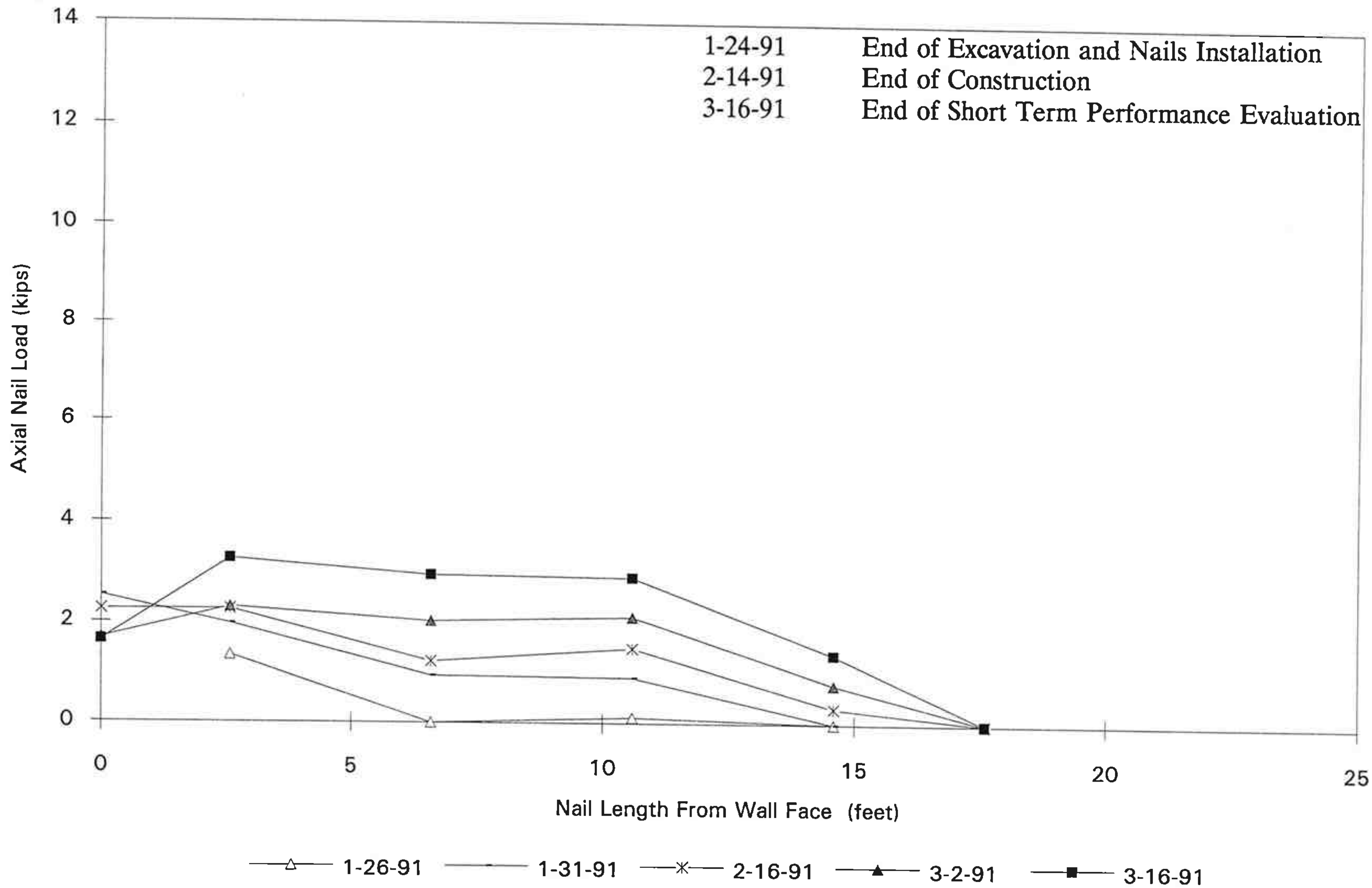


Fig. 76 - Distribution of Nail Forces at Section 1

At Completion of Nail Installation - January 28, 1991

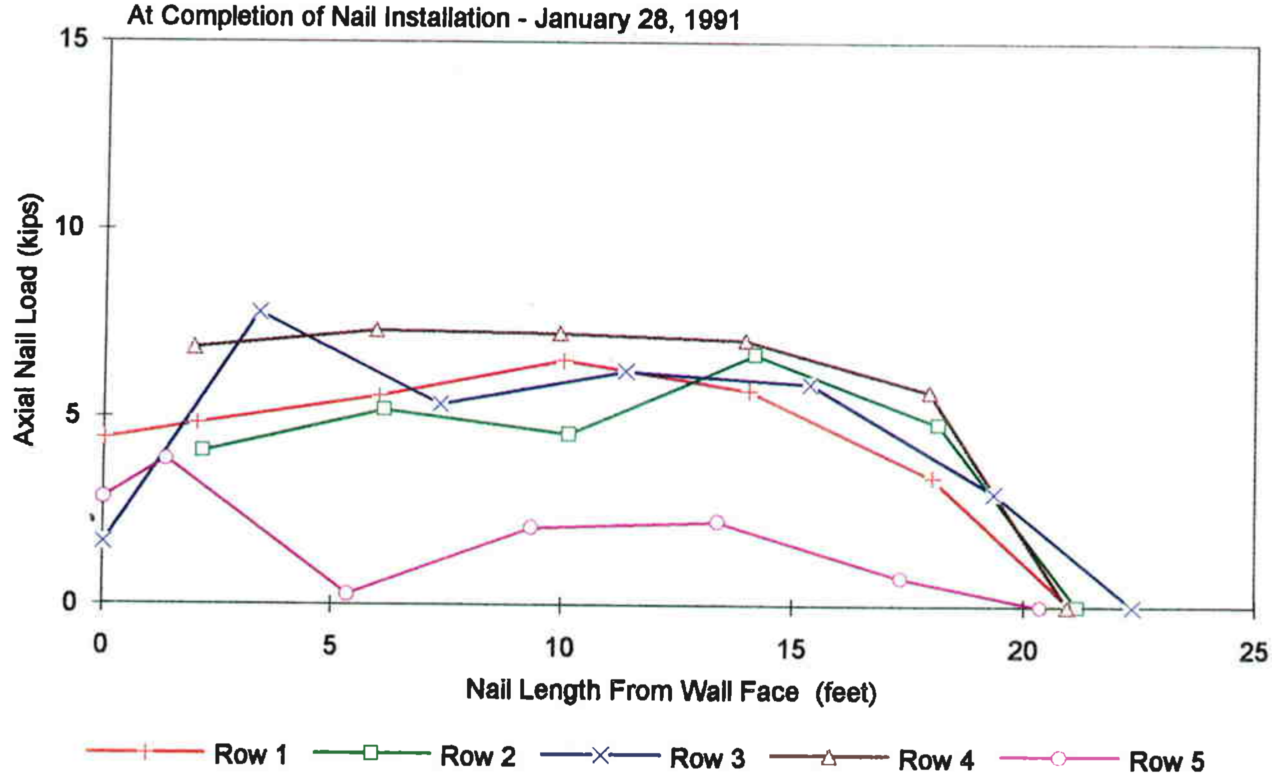


Fig. 77 - Distribution of Nail Forces at Section 1

At End of Short Term Performance Period - March 16, 1991

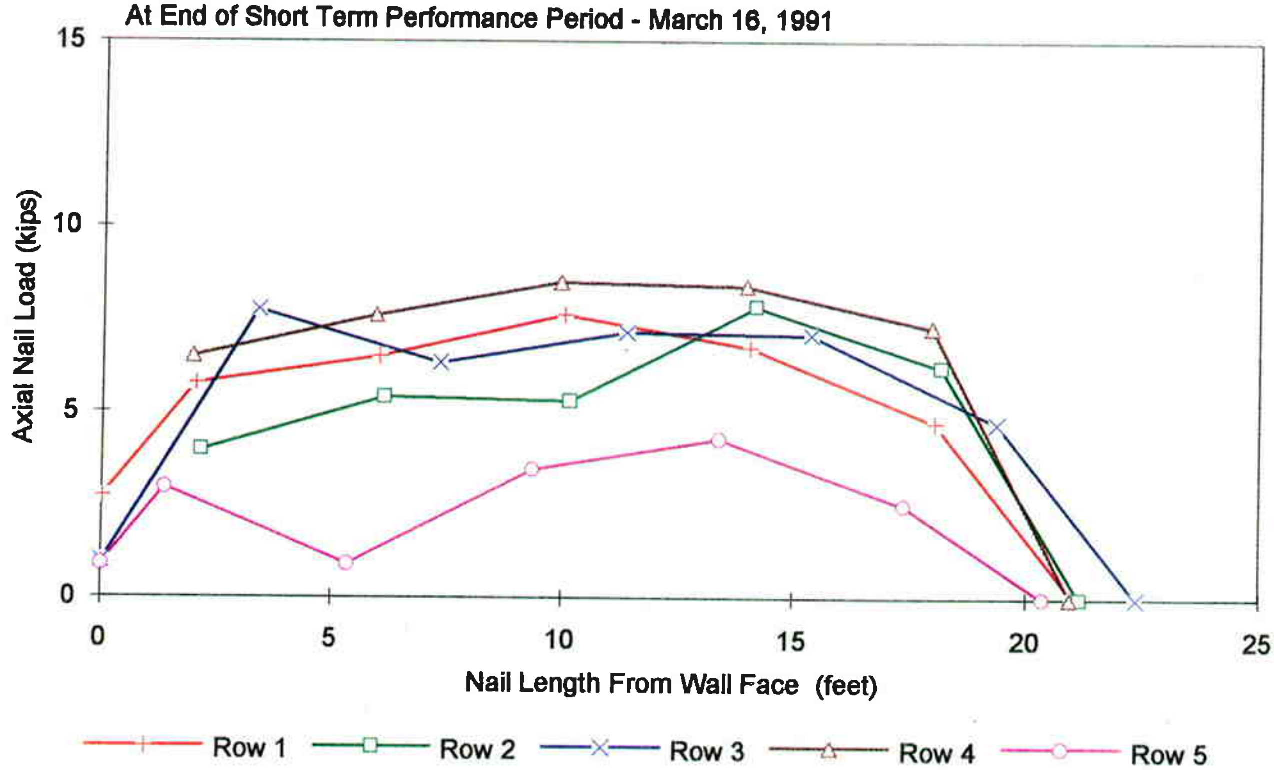


Fig. 78 - Distribution of Nail Forces at Section 2

At Completion of Nail Installation - January 28, 1991

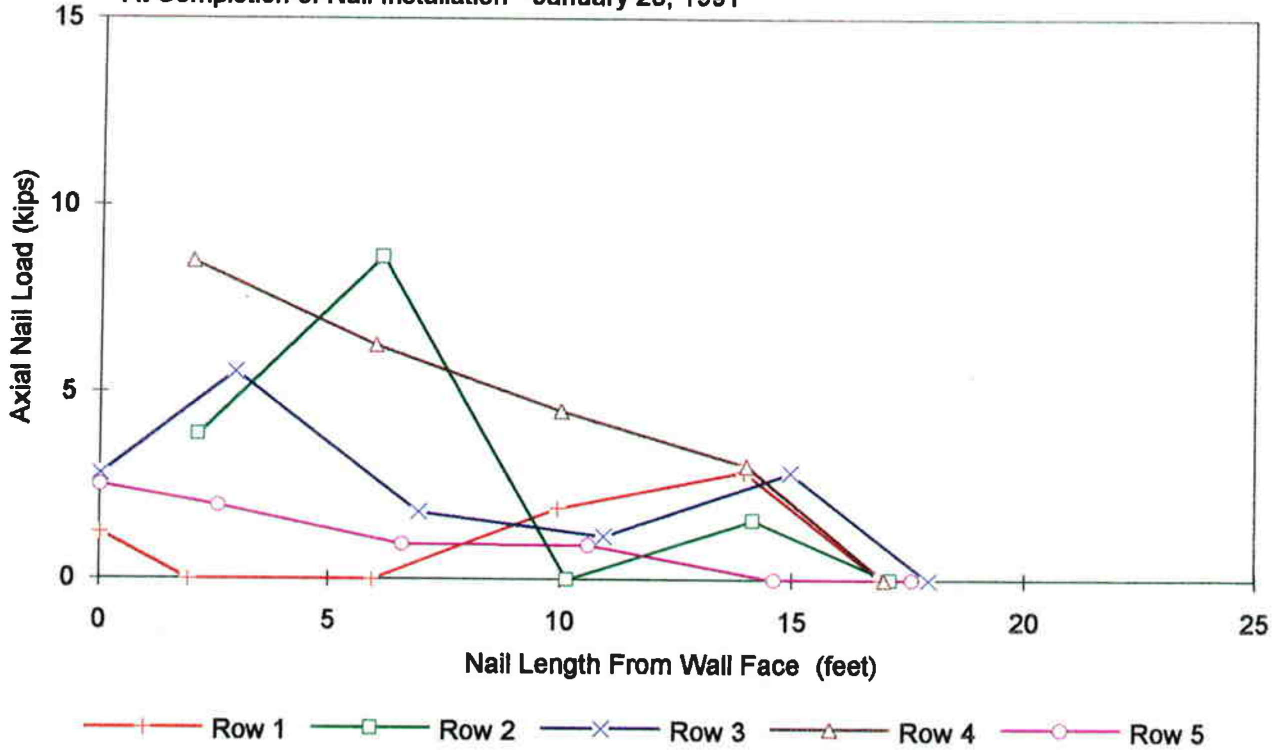


Fig. 79 - Distribution of Nail Forces at Section 2

At End of Short Term Performance Period - March 16, 1991

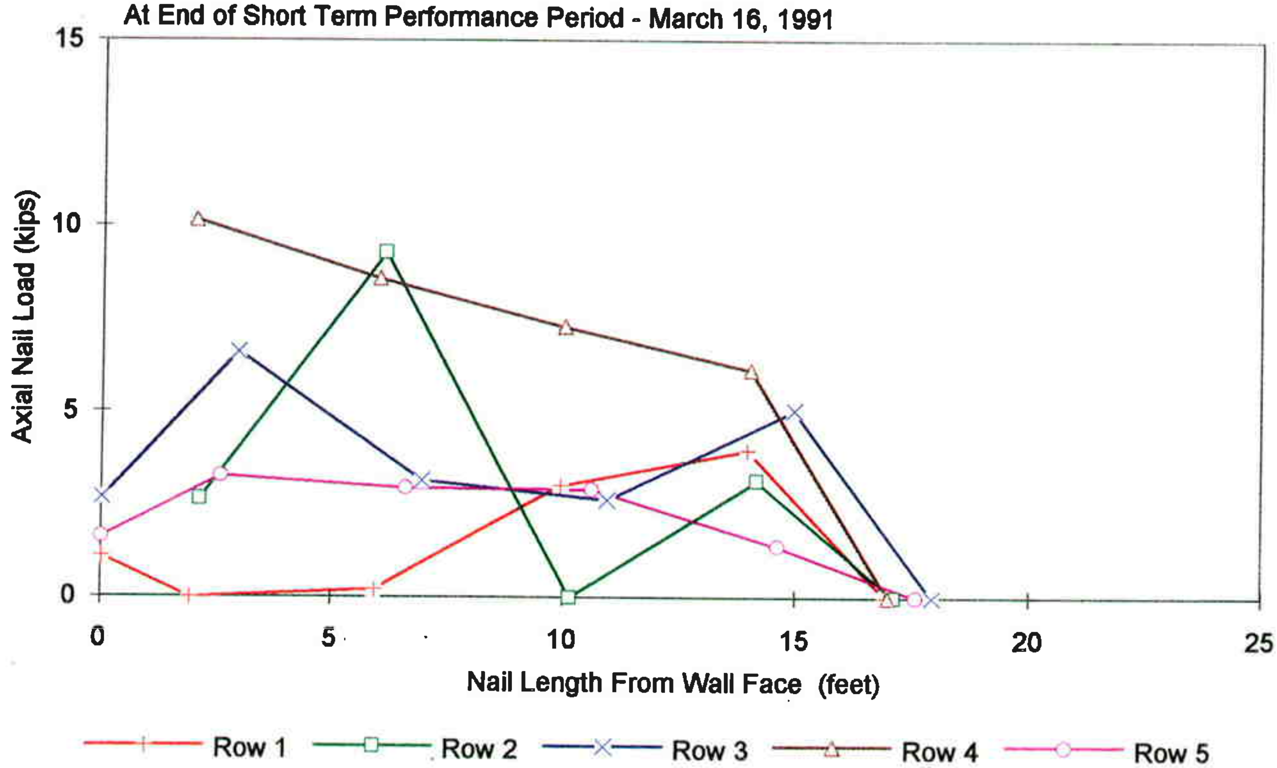


Fig. 80 - **Load Cell Readings**

Short Term Performance at Section 1

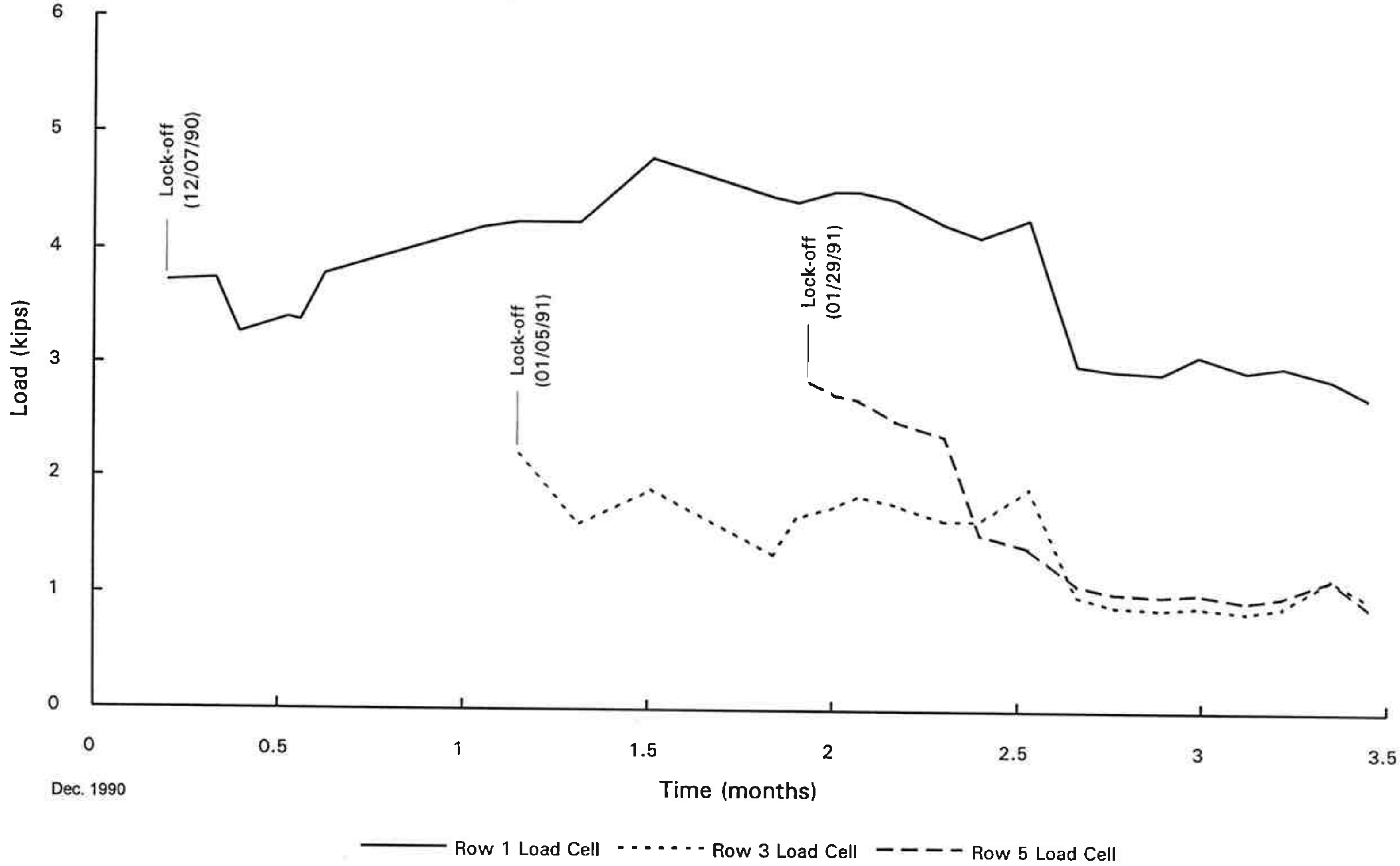
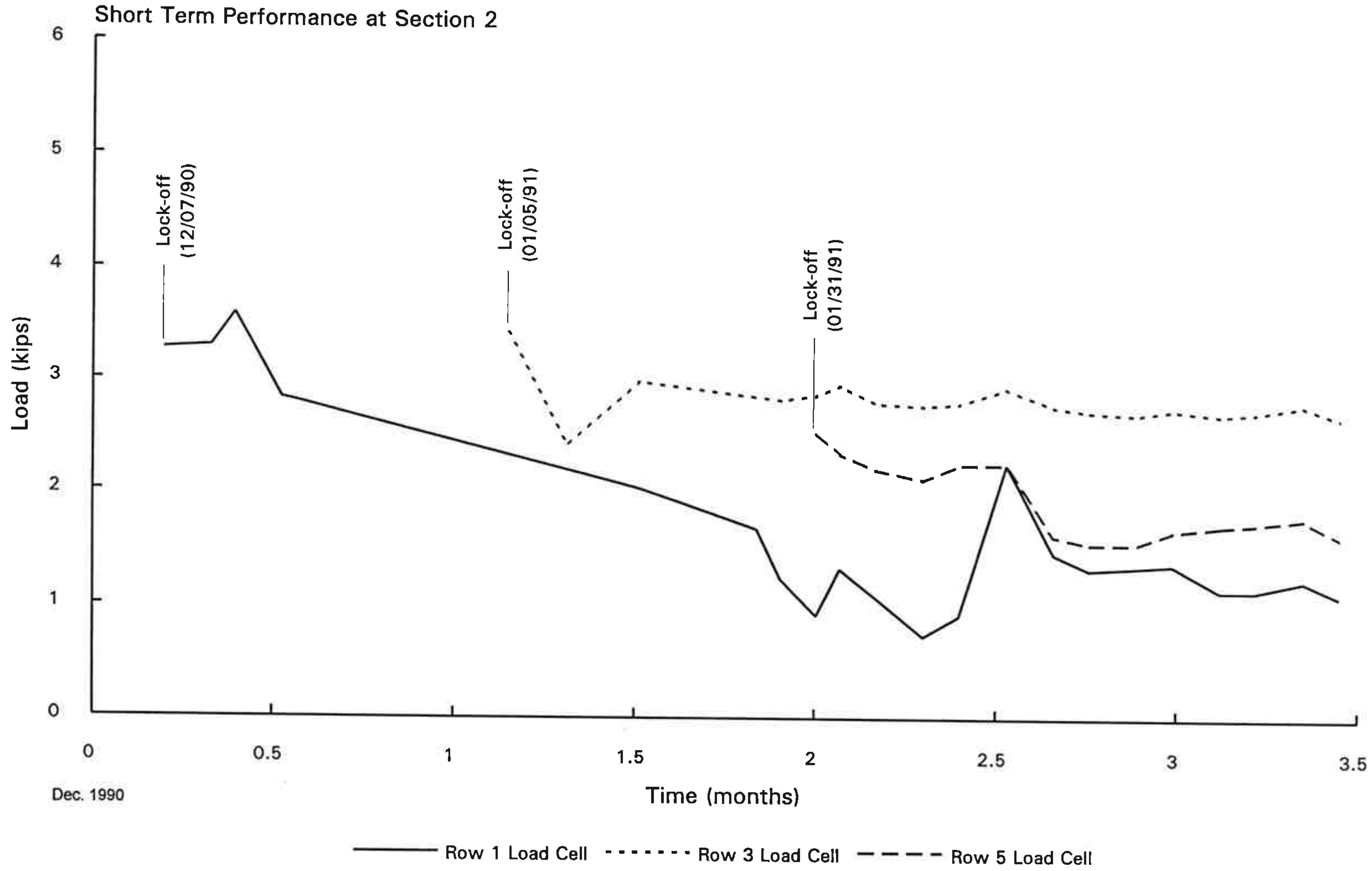


Fig. 81 - Load Cell Readings



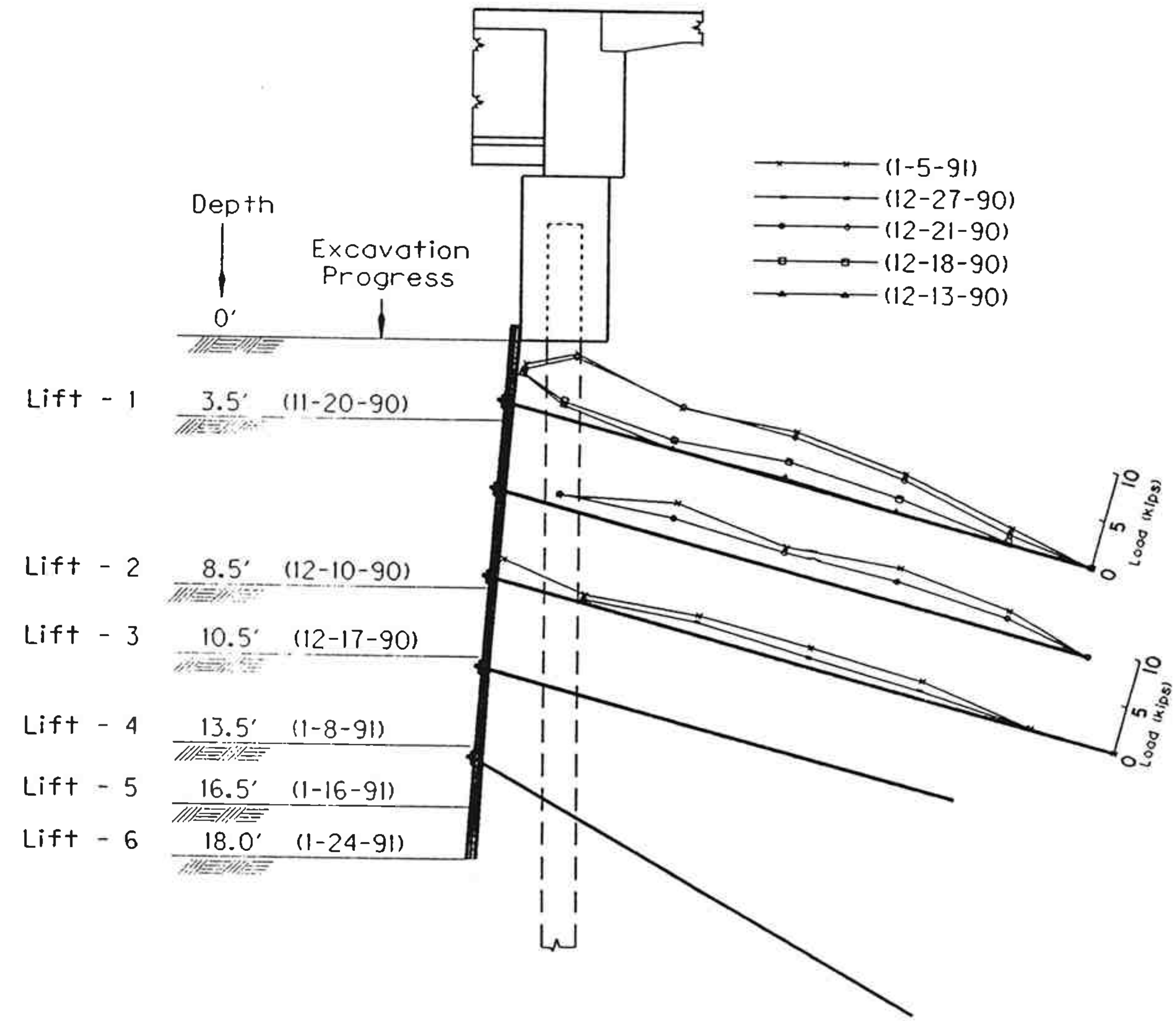


FIG. 82 DISTRIBUTION OF NAIL FORCES AT SECTION 1 FOLLOWING EXCAVATION OF TOP THREE LIFTS

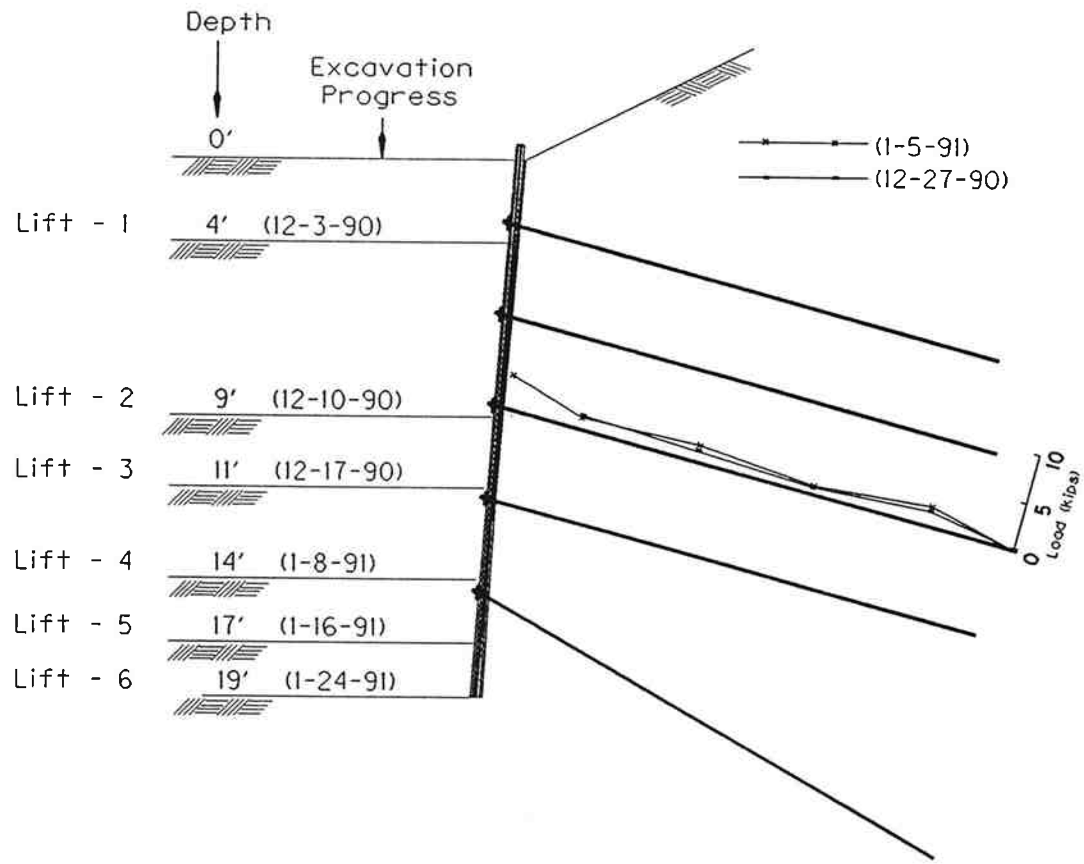


FIG. 83 **DISTRIBUTION OF NAIL FORCES AT SECTION 2, ROW 3 FOLLOWING EXCAVATION FOR LIFT 3**

Fig. 84 - Short Term Pile Strain Gauge Readings

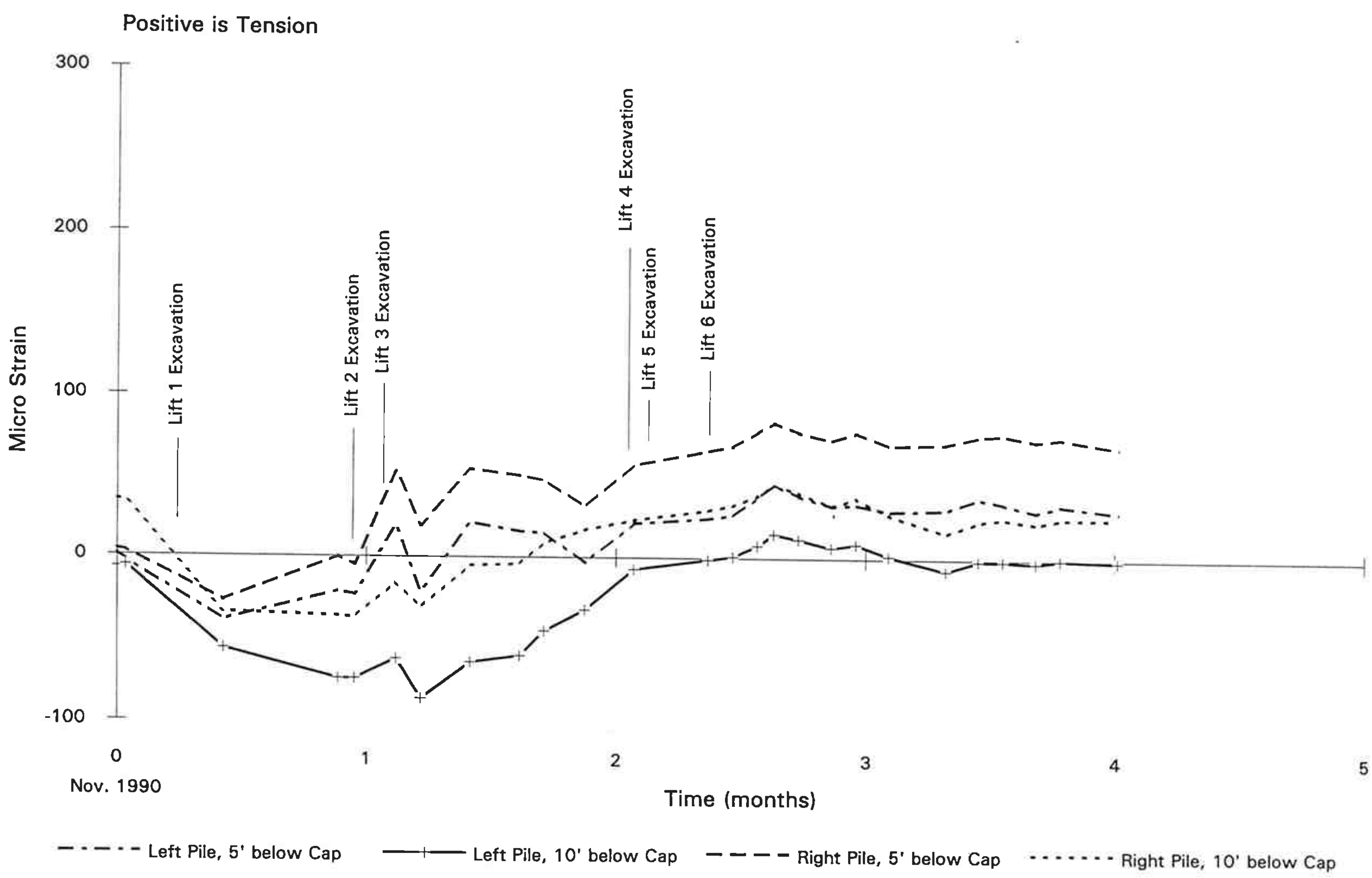


Fig. 85 - Short Term Pile Stress

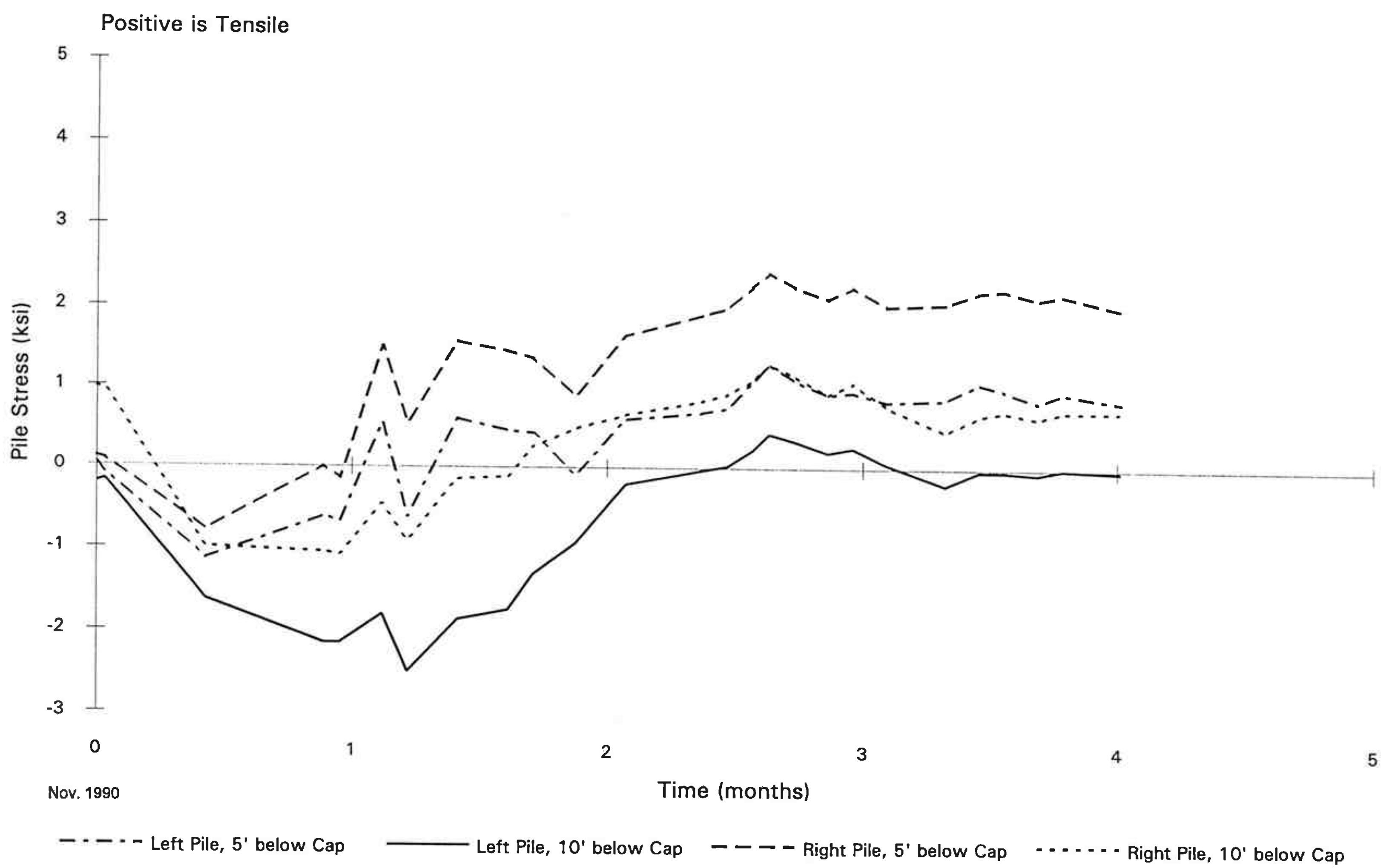


Fig. 86 - Short Term Pile Cap Deflection Measured by Single Point Extensometer

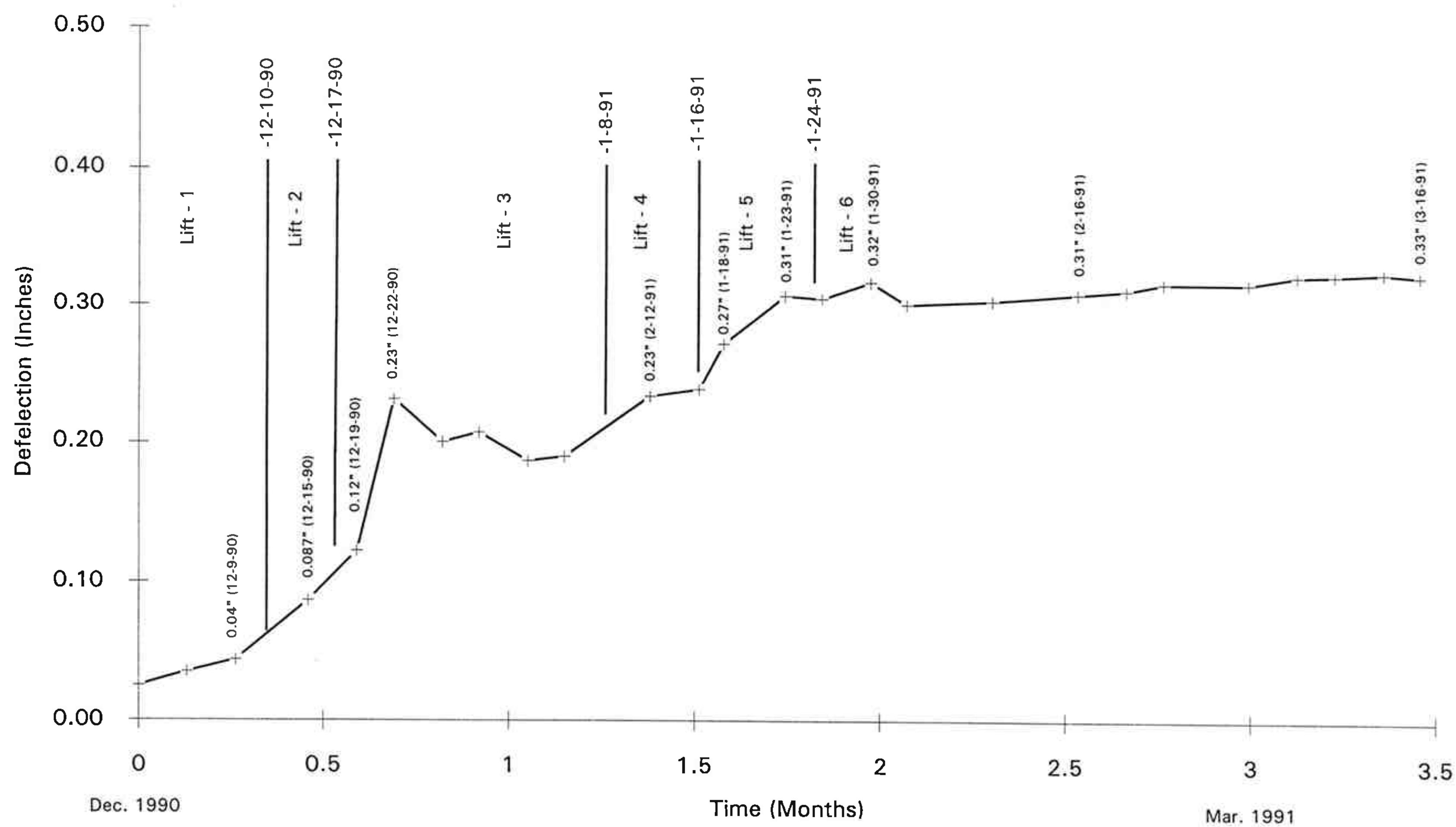


Fig. 87 - Short Term Deflections - Slope Inclinometer SD129

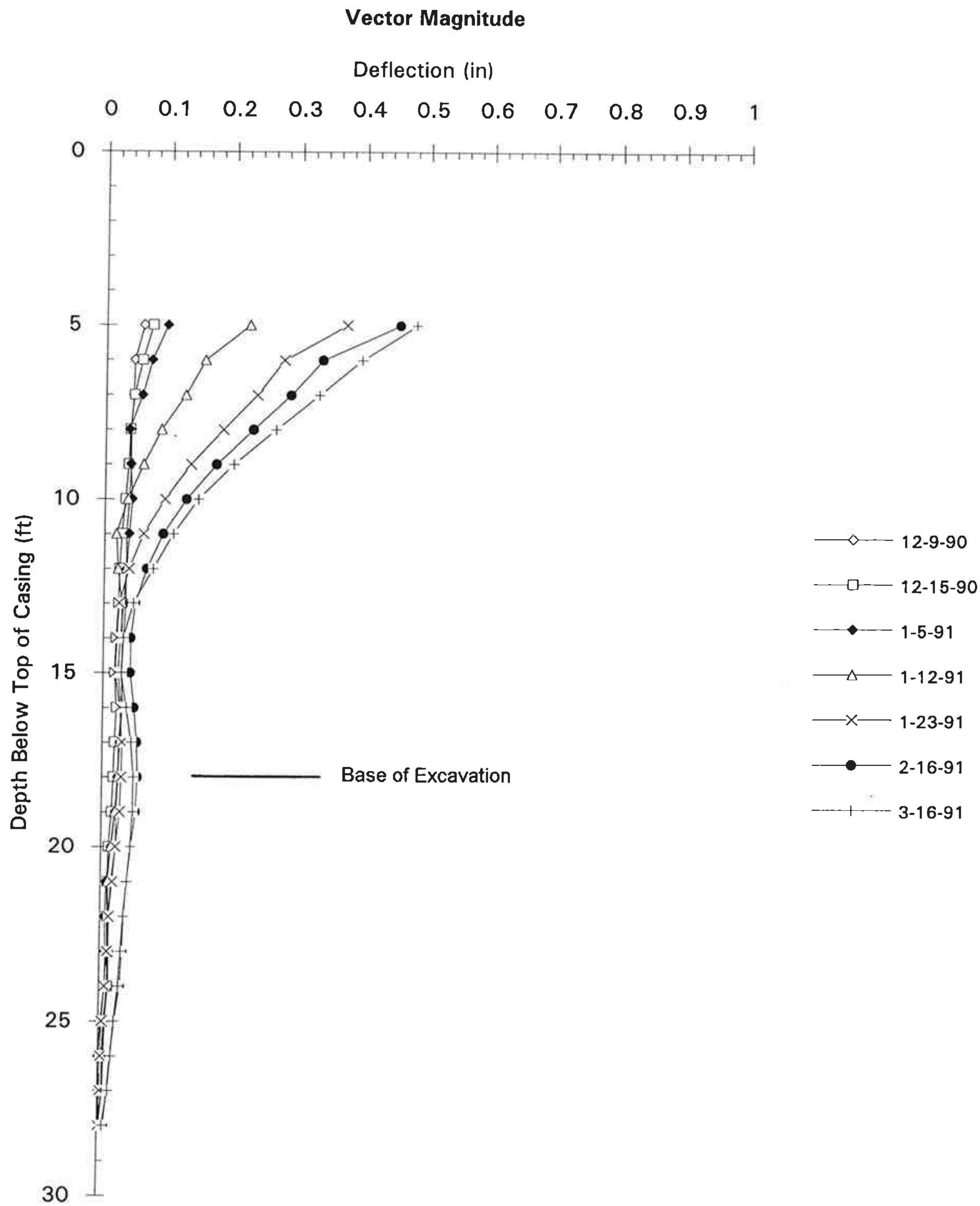


Fig. 88 - Short Term Deflections - Slope Inclinometer SD130

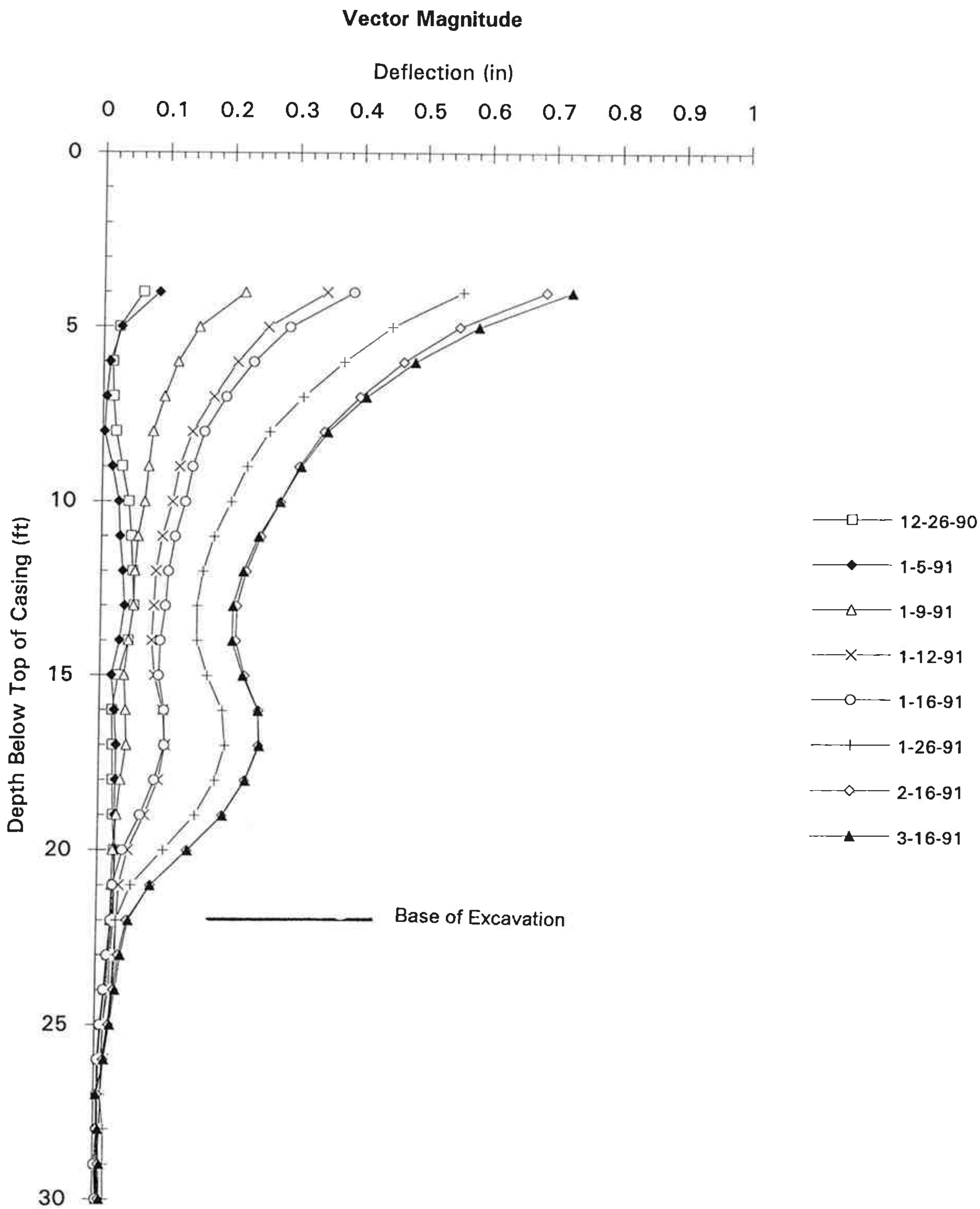


Fig. 89 - Short Term Pile Cap Rotation

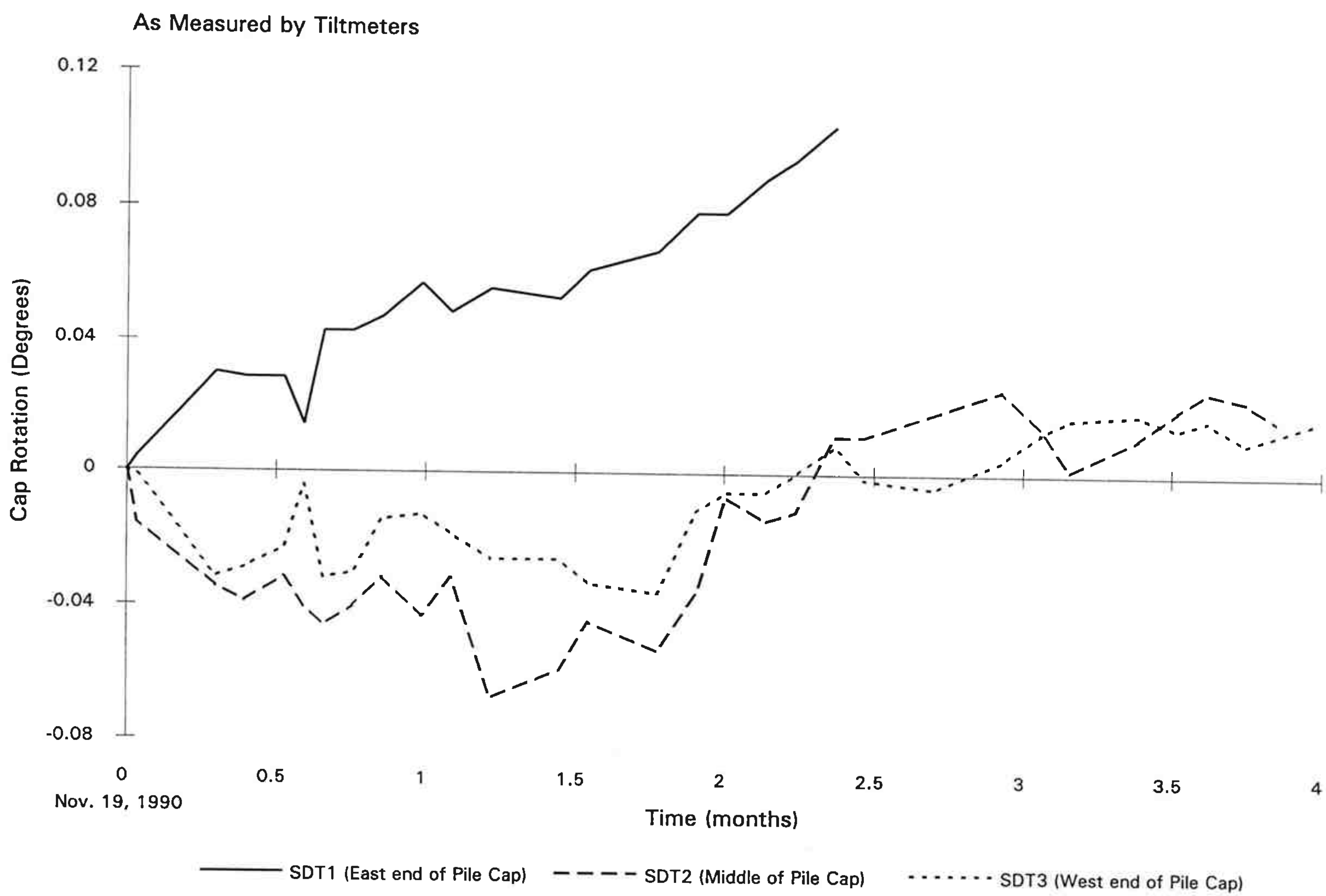


Fig. 90 - Long Term Performance

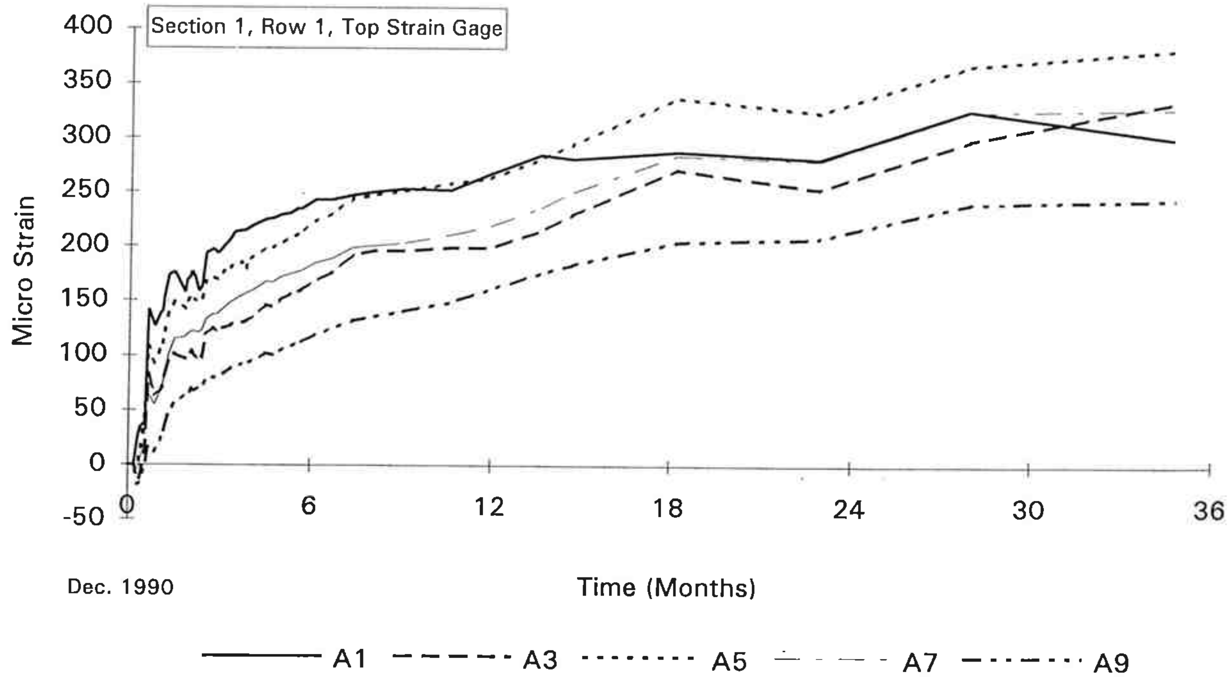


Fig. 91 - Long Term Performance

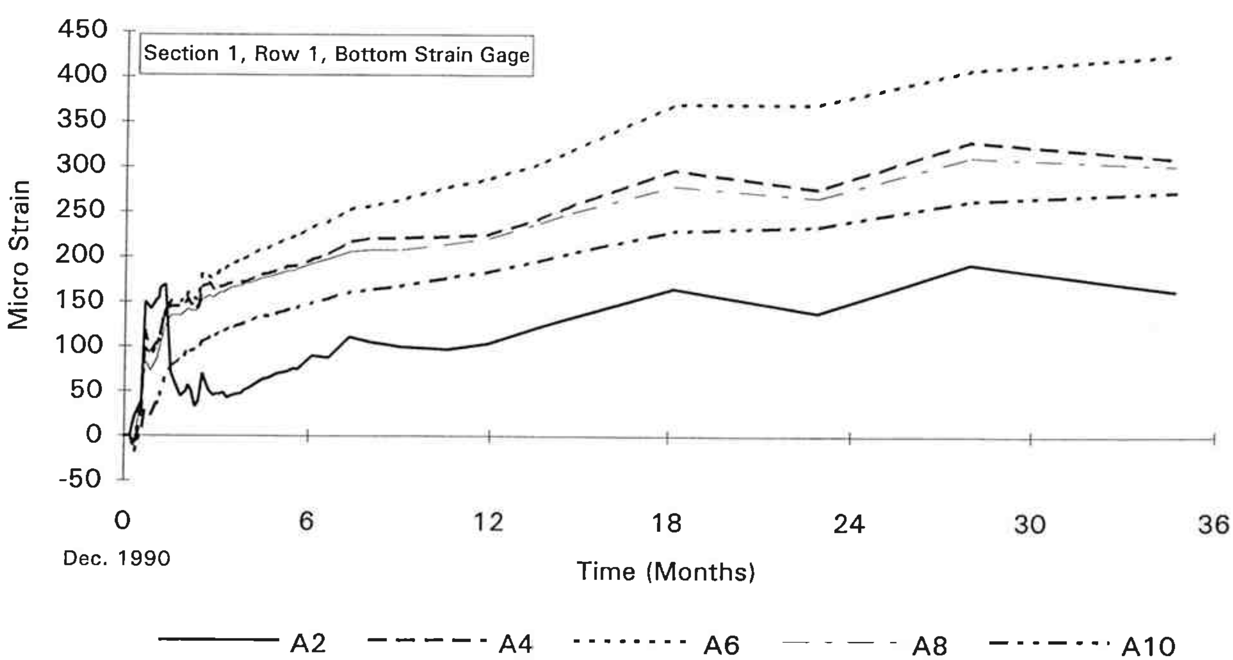


Fig. 92 - Long Term Performance

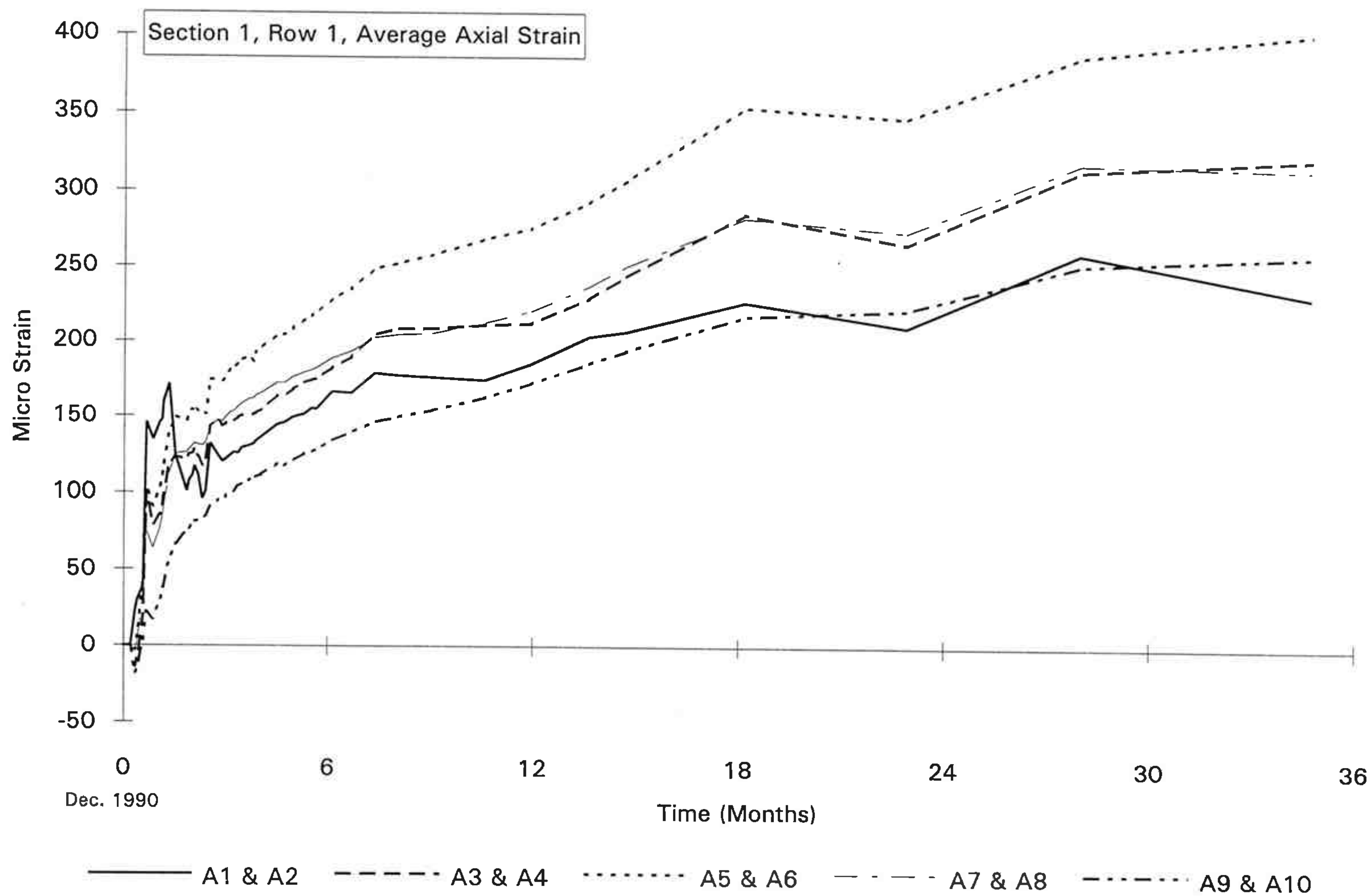


Fig. 93 - Long Term Performance

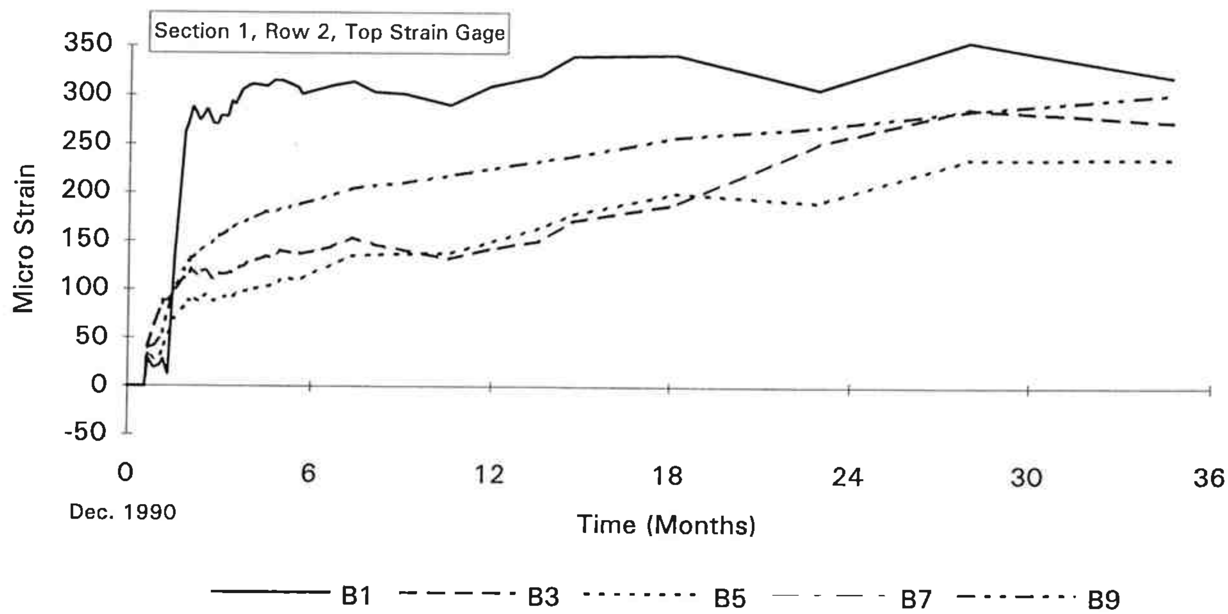


Fig. 94 - Long Term Performance

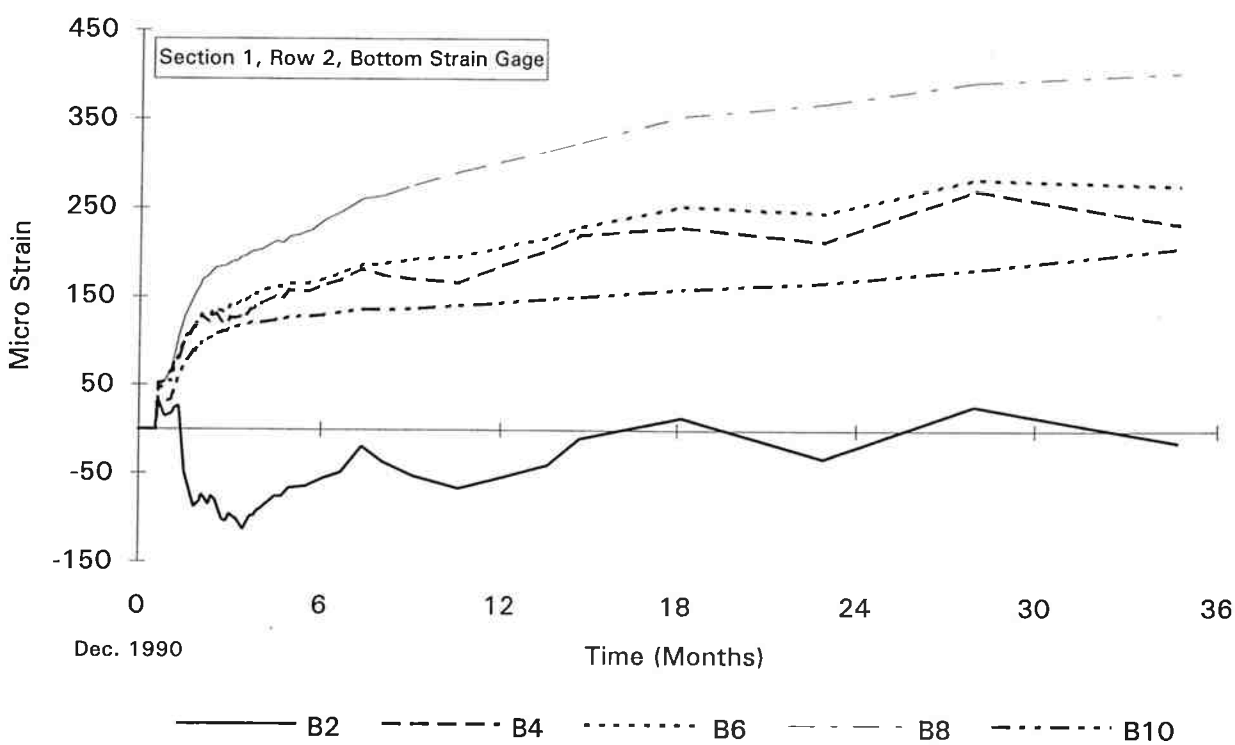


Fig. 95 - Long Term Performance

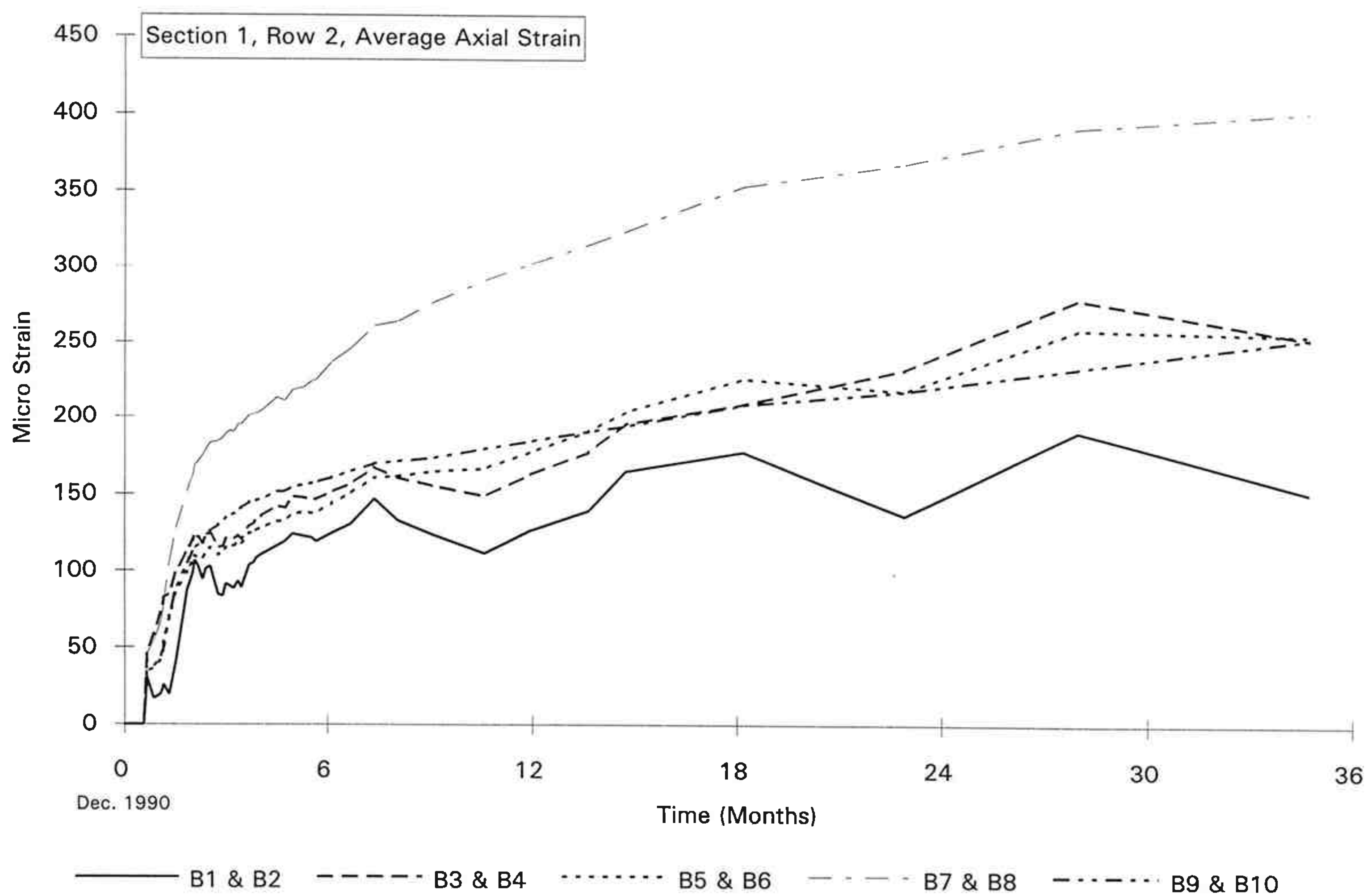


Fig. 96 - Long Term Performance

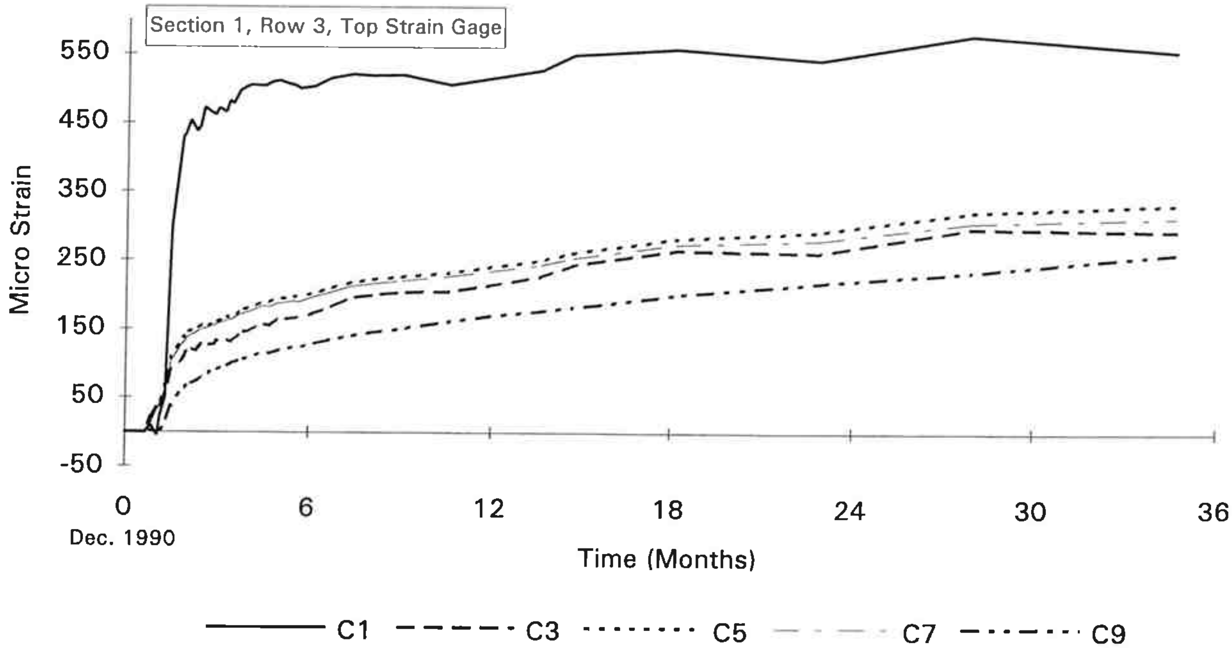


Fig. 97 - Long Term Performance

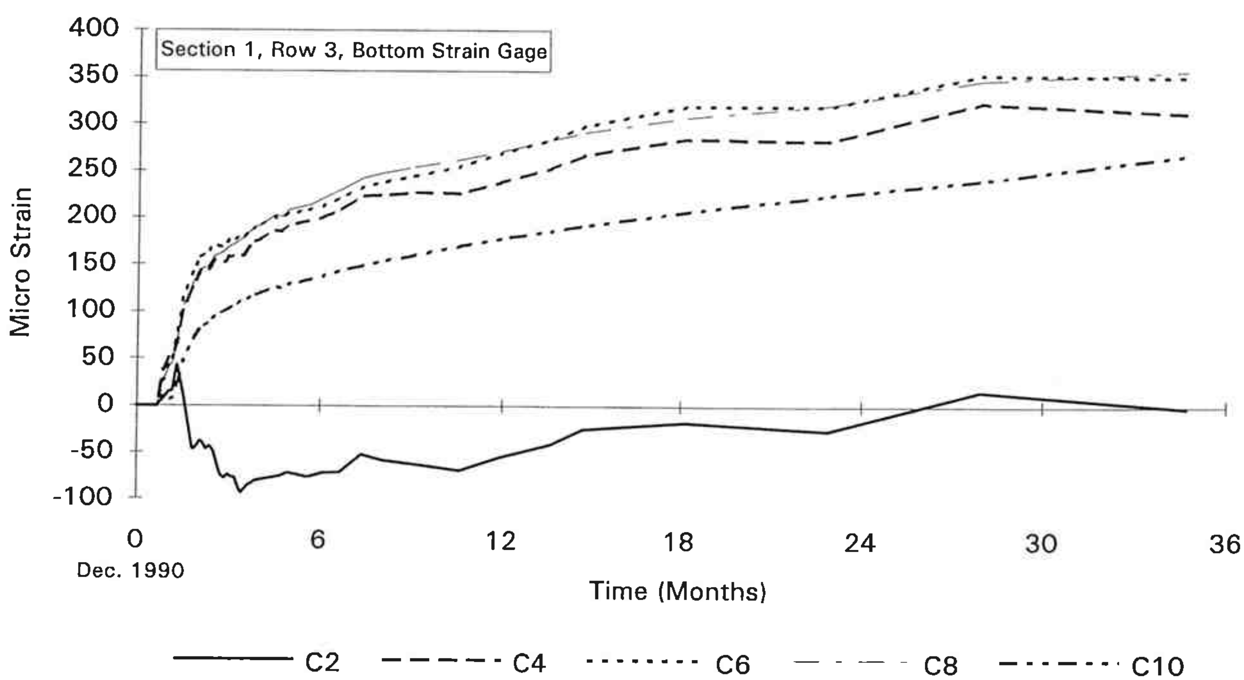


Fig. 98 - Long Term Performance

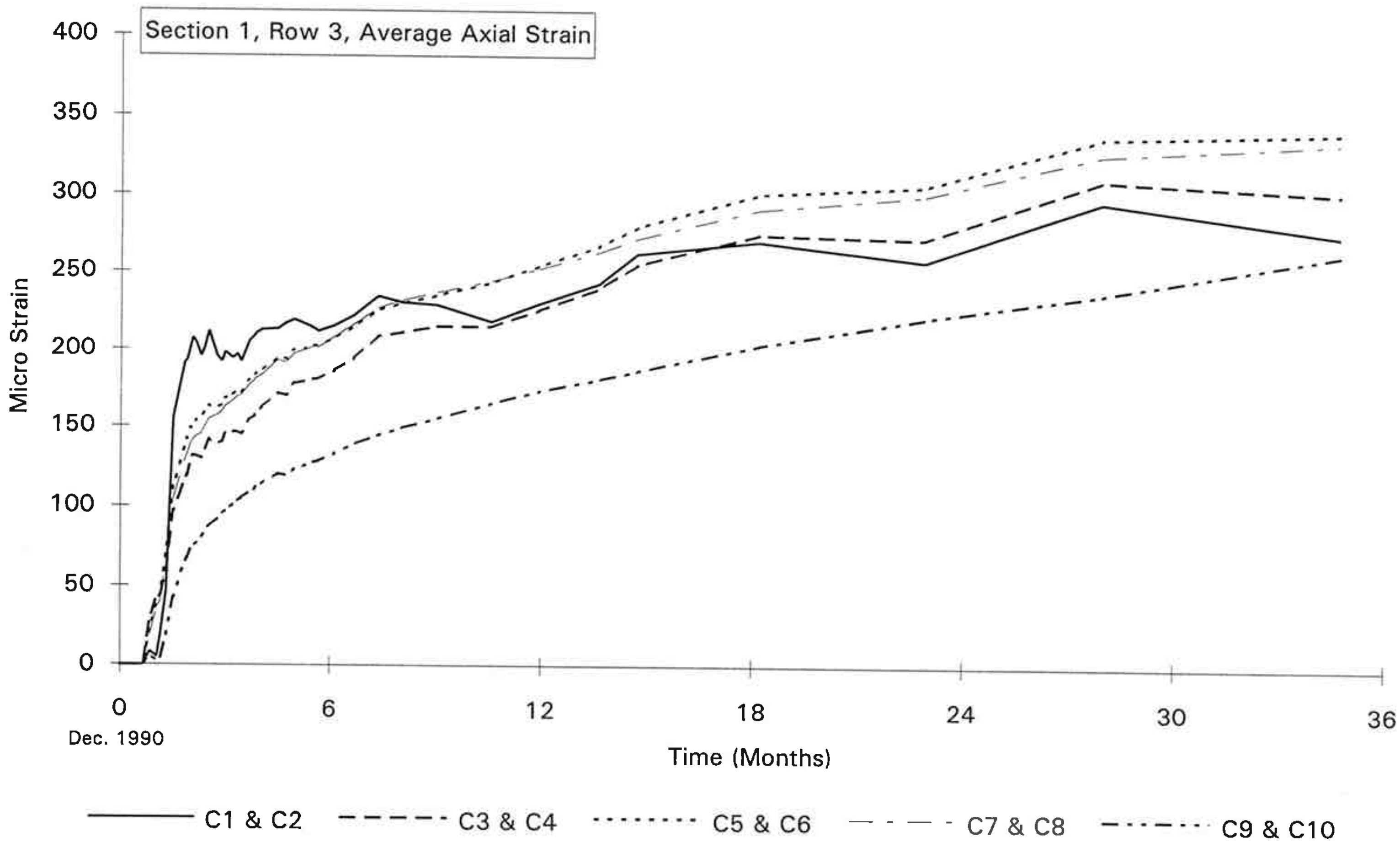


Fig. 99 - Long Term Performance

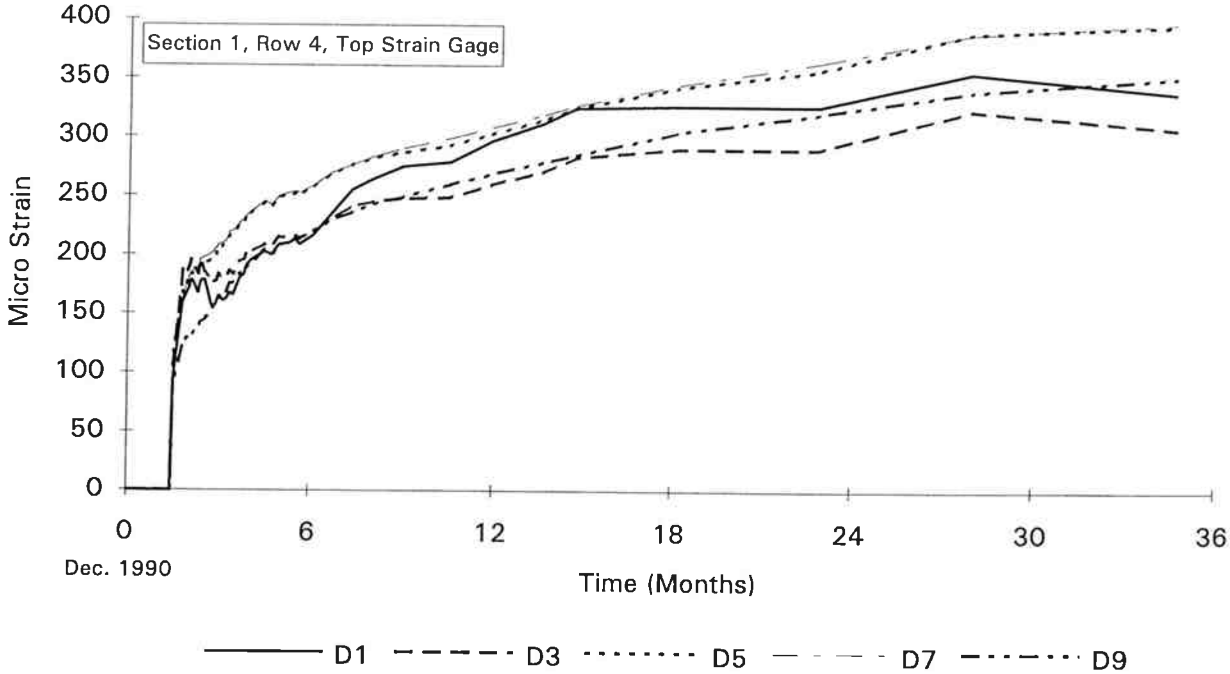


Fig. 100 - Long Term Performance

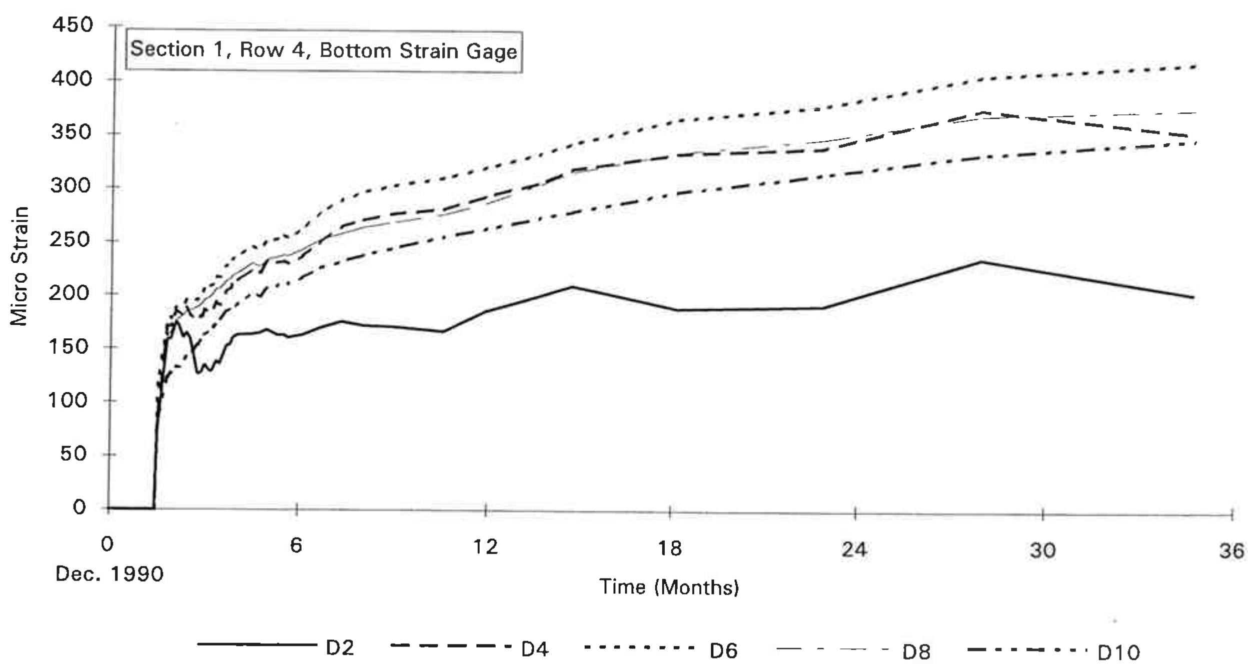


Fig. 101 - Long Term Performance

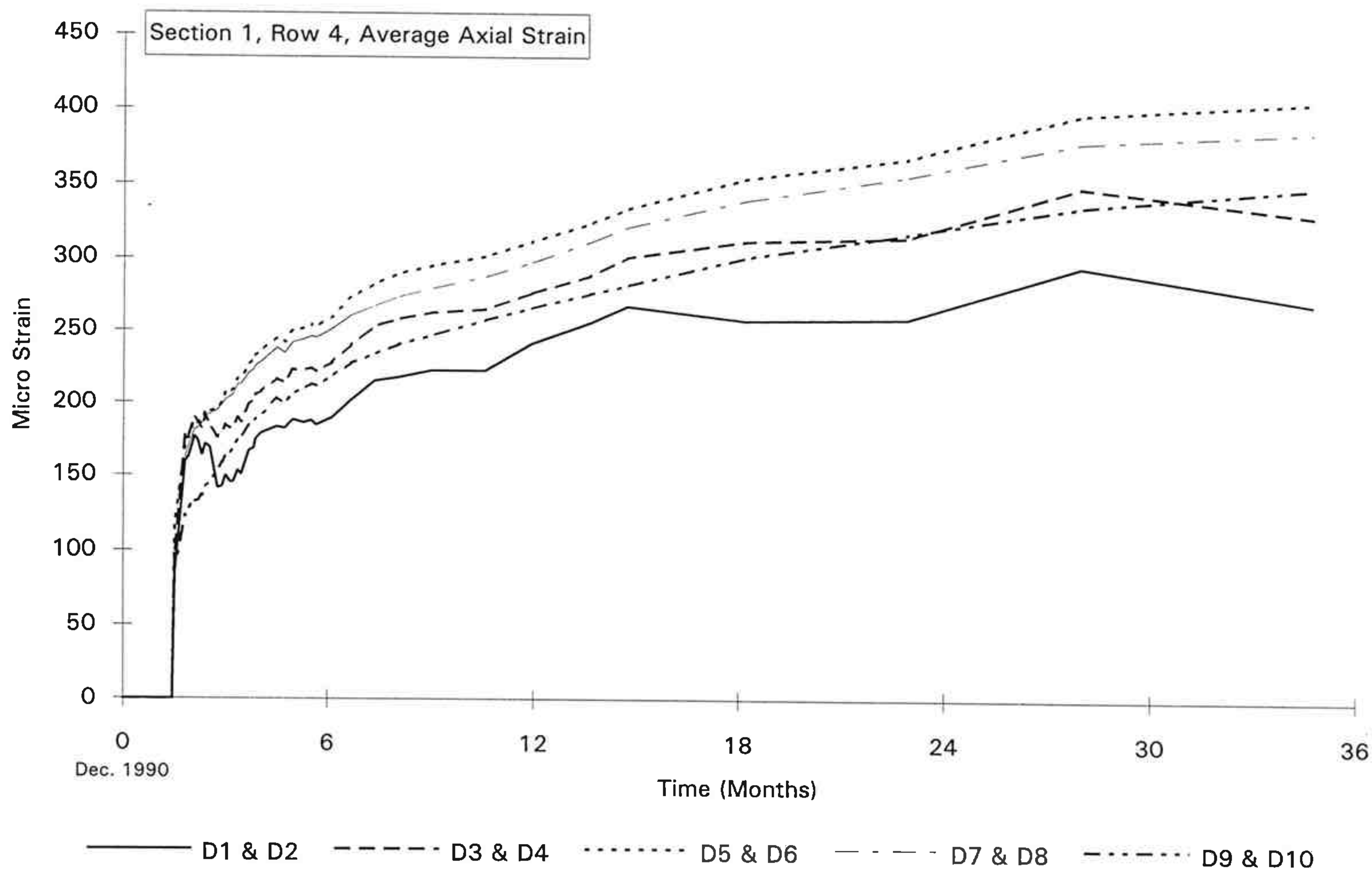


Fig. 102 - Long Term Performance

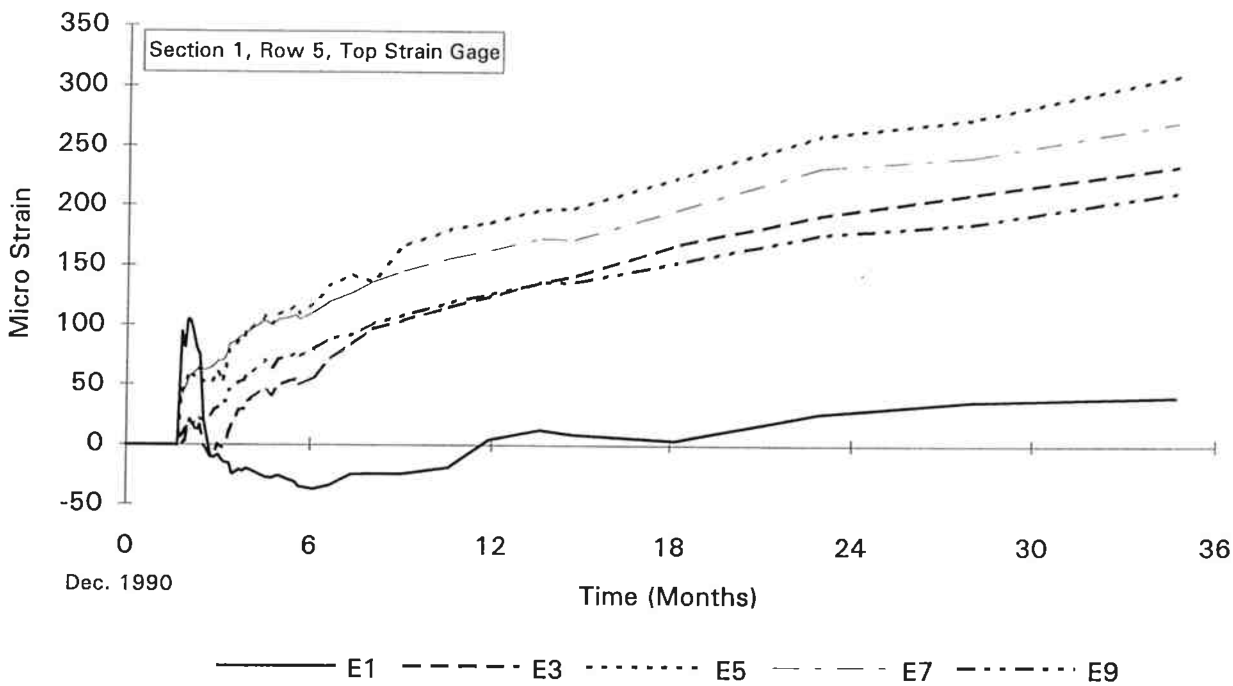


Fig. 103 - Long Term Performance

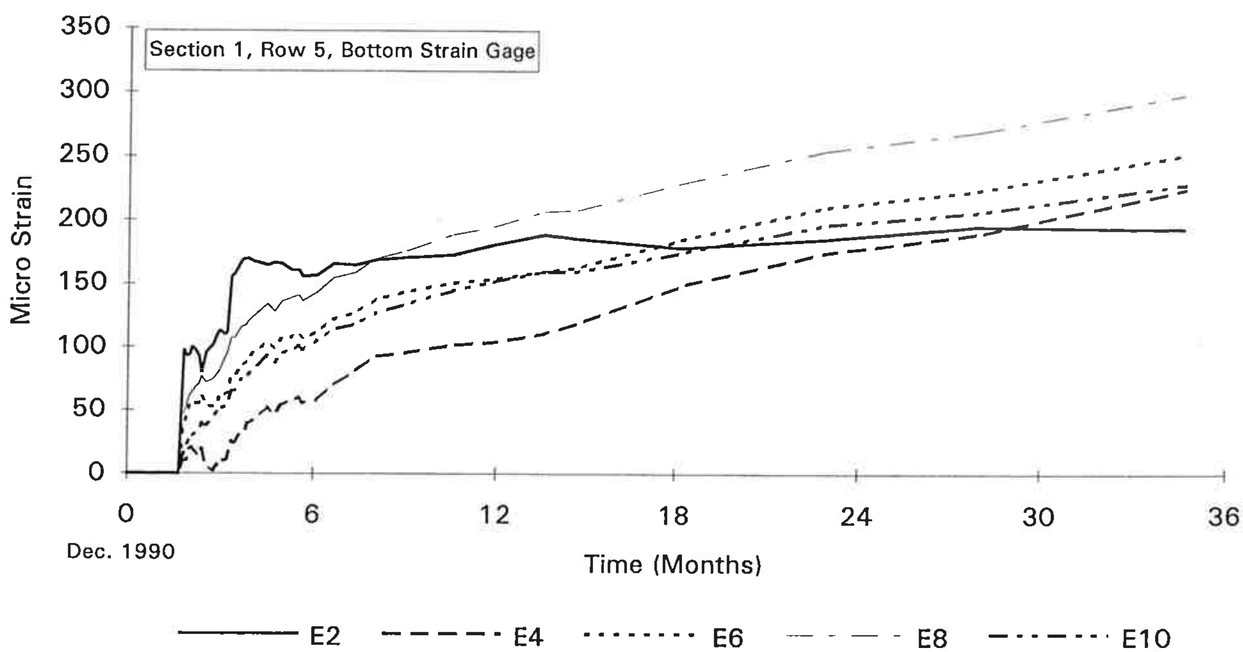


Fig. 104 - Long Term Performance

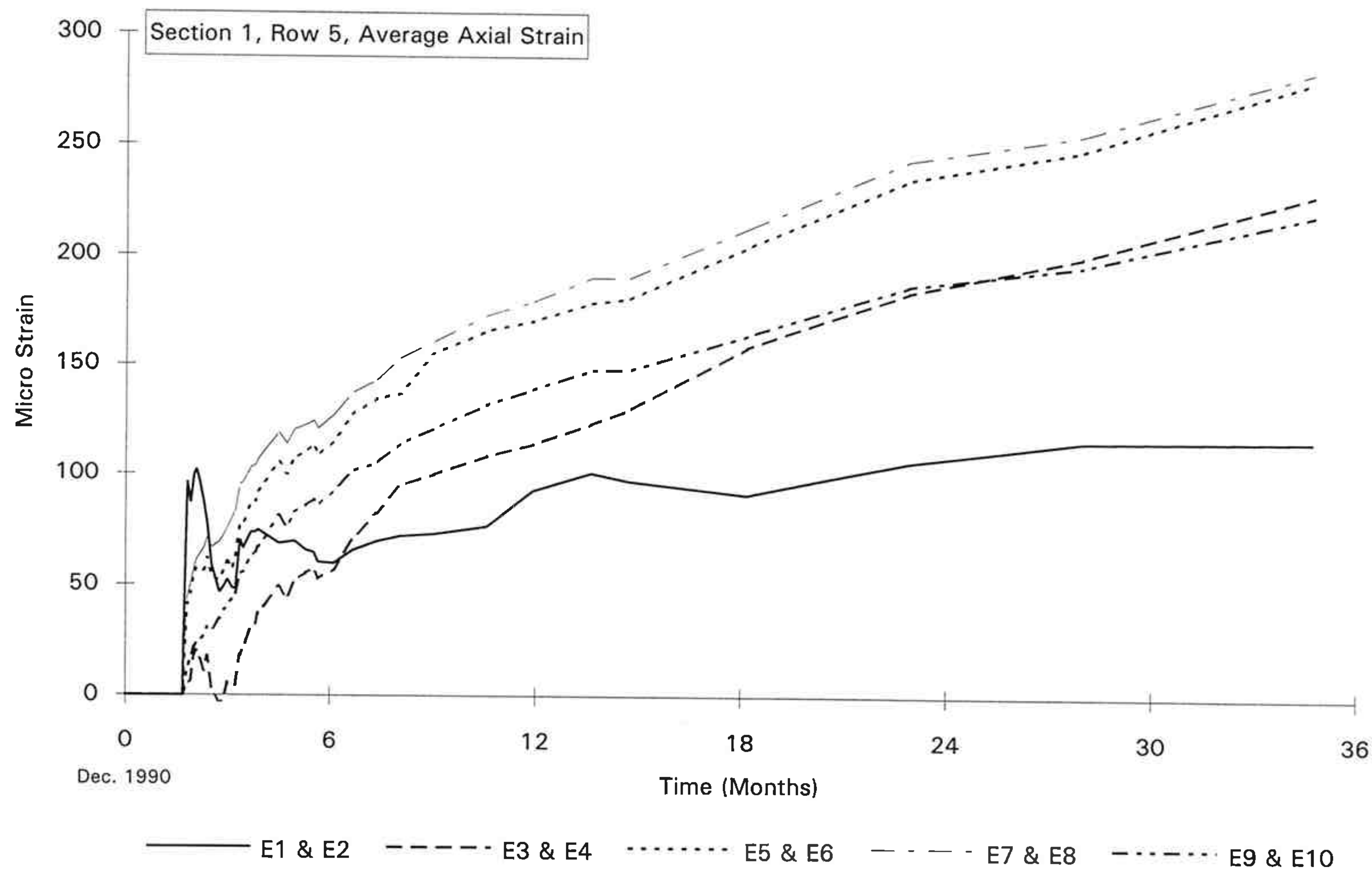


Fig. 105 - Long Term Performance

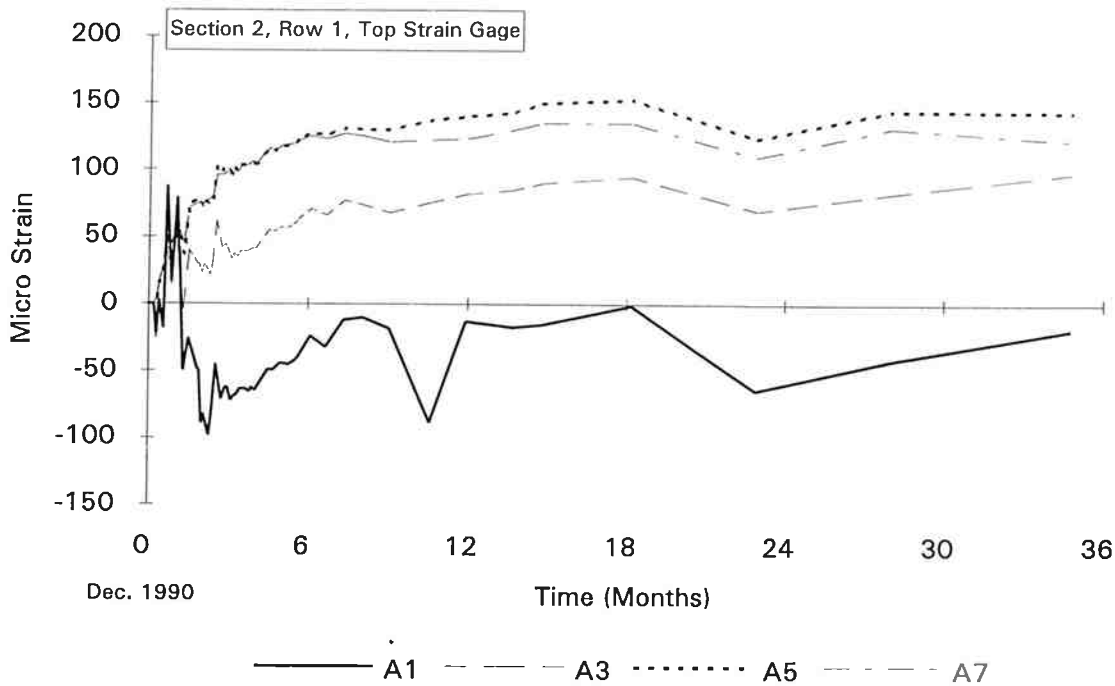


Fig. 106 - Long Term Performance

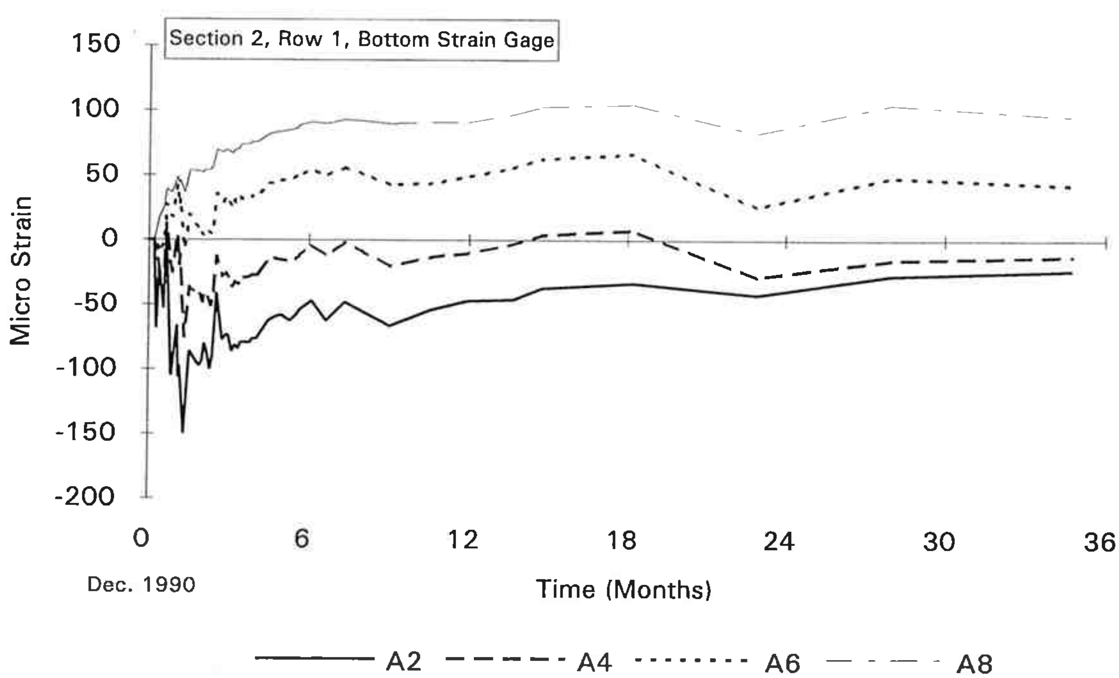


Fig. 107 - Long Term Performance

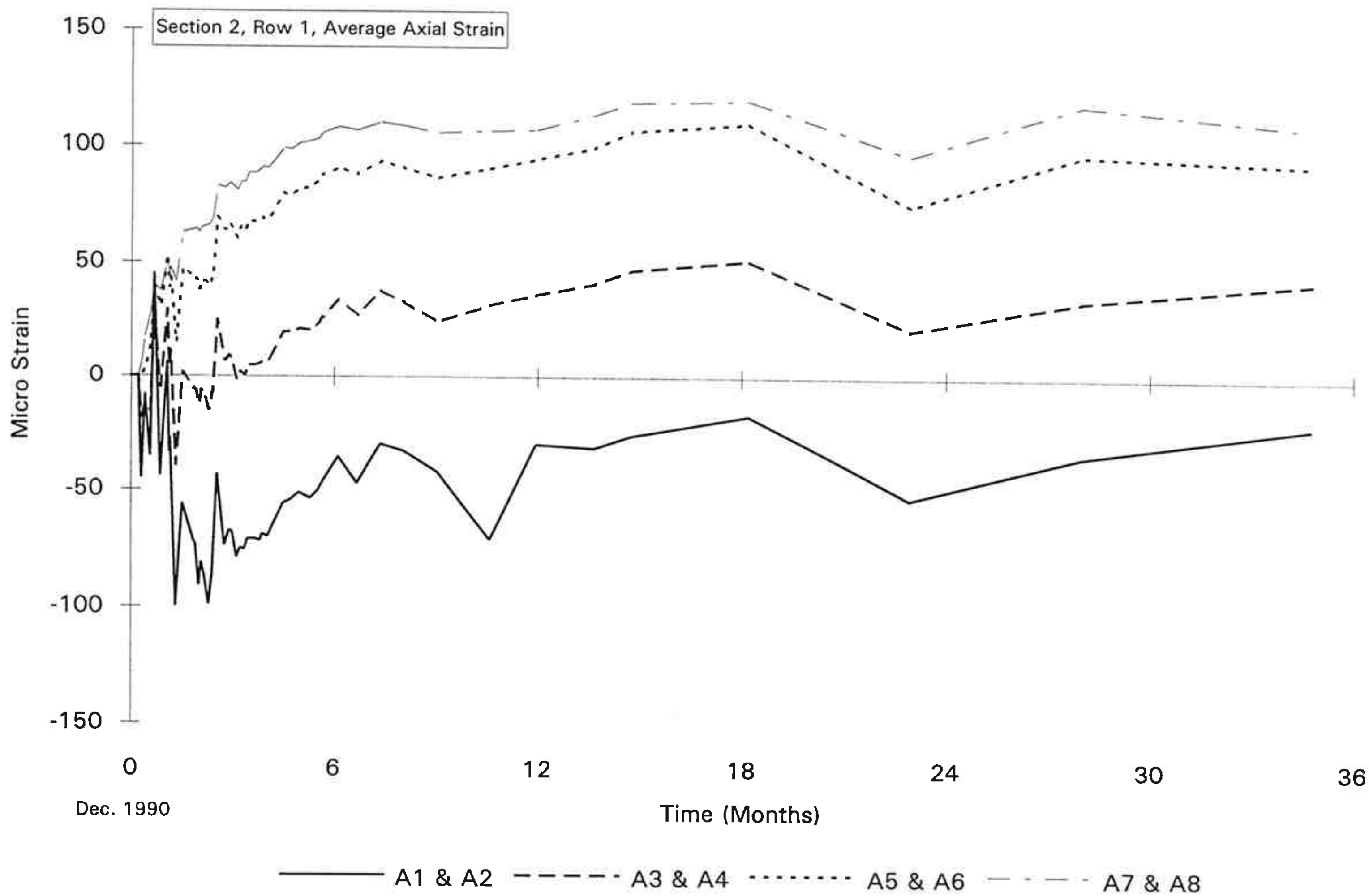


Fig. 108 - Long Term Performance

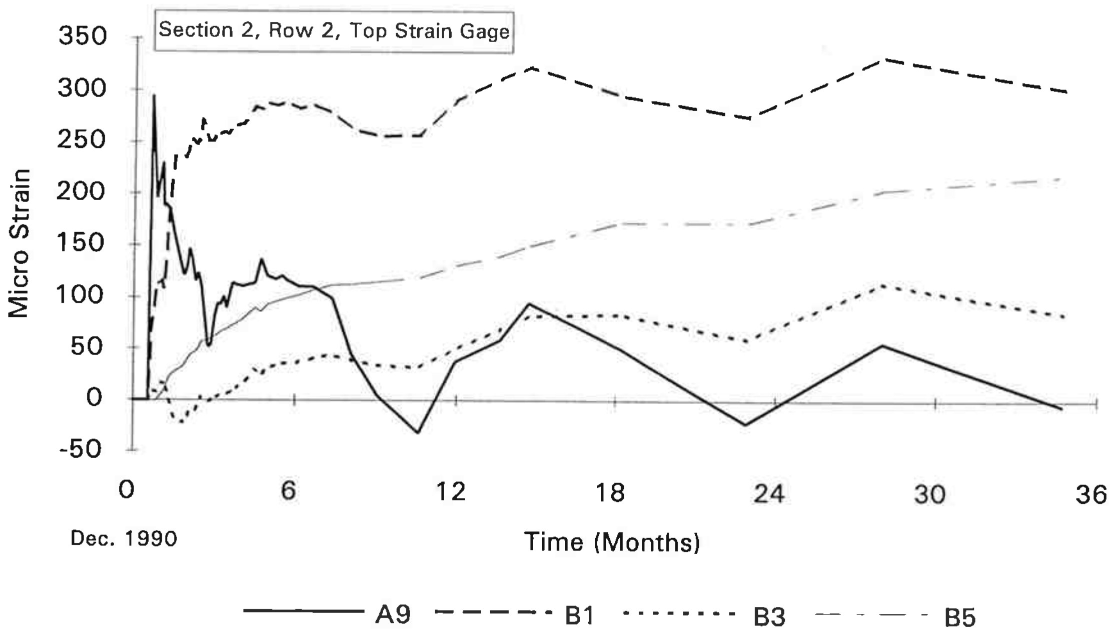


Fig. 109 - Long Term Performance

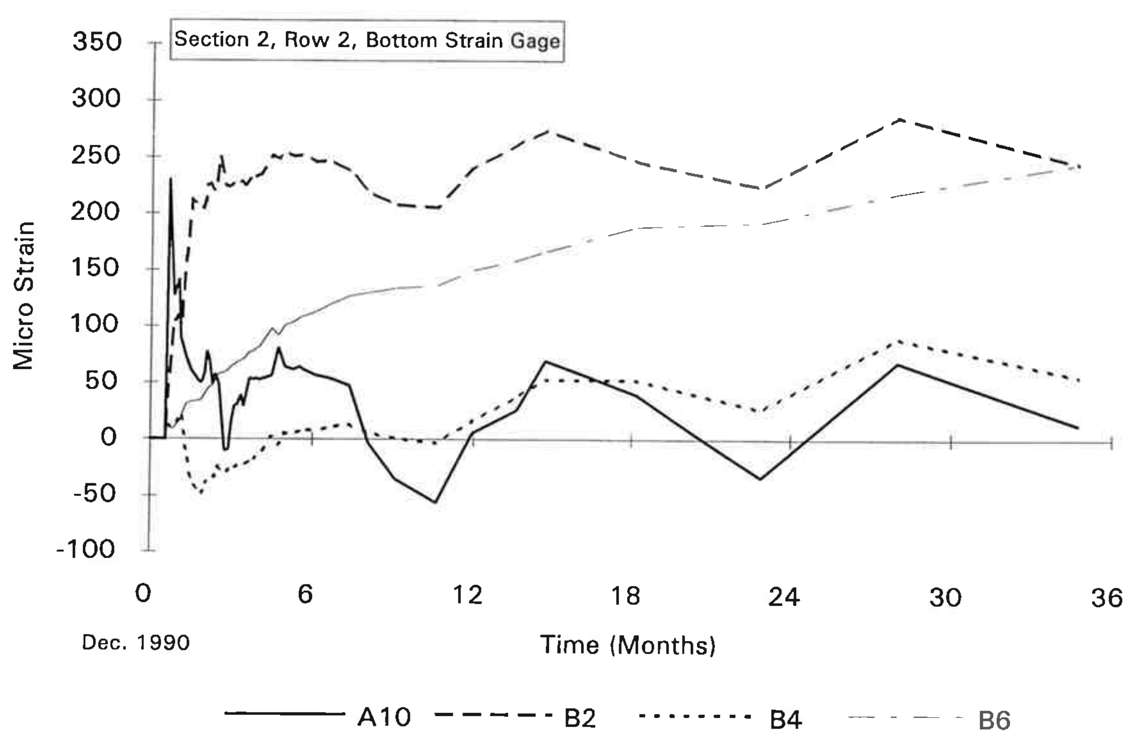


Fig. 110 - Long Term Performance

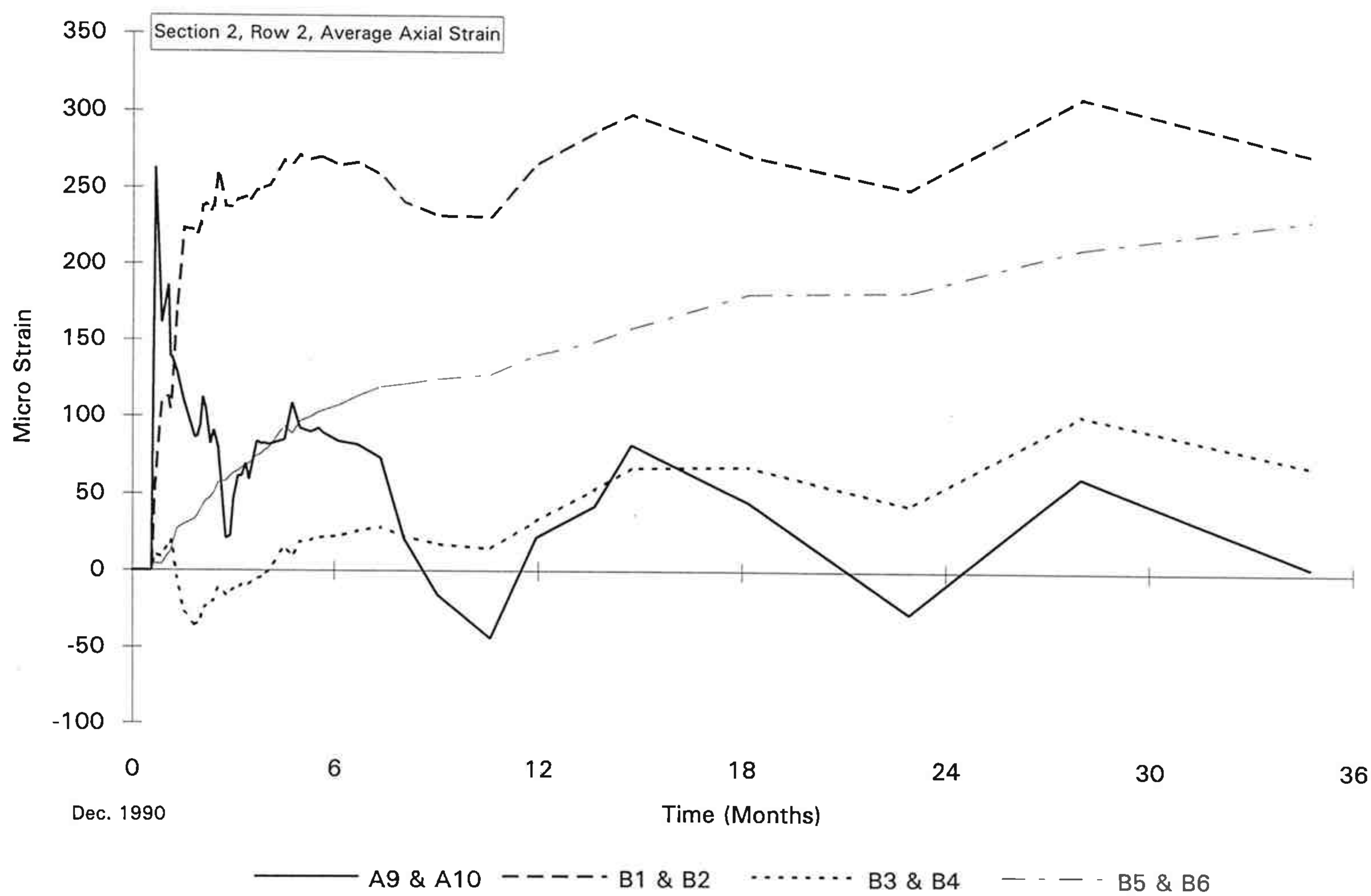


Fig. 111 - Long Term Performance

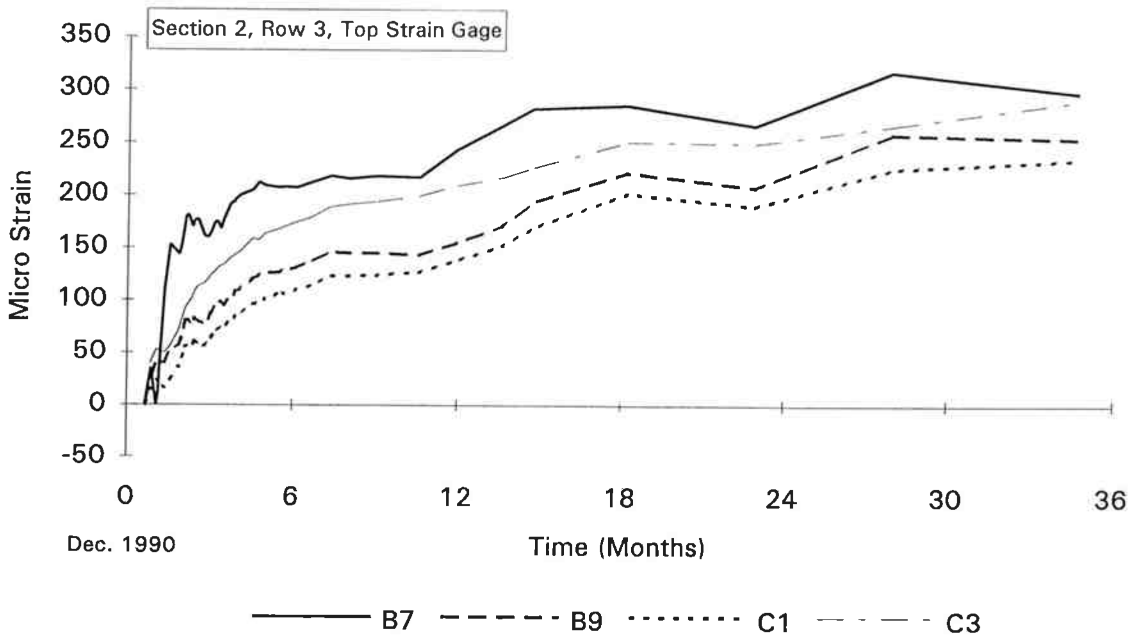


Fig. 112 - Long Term Performance

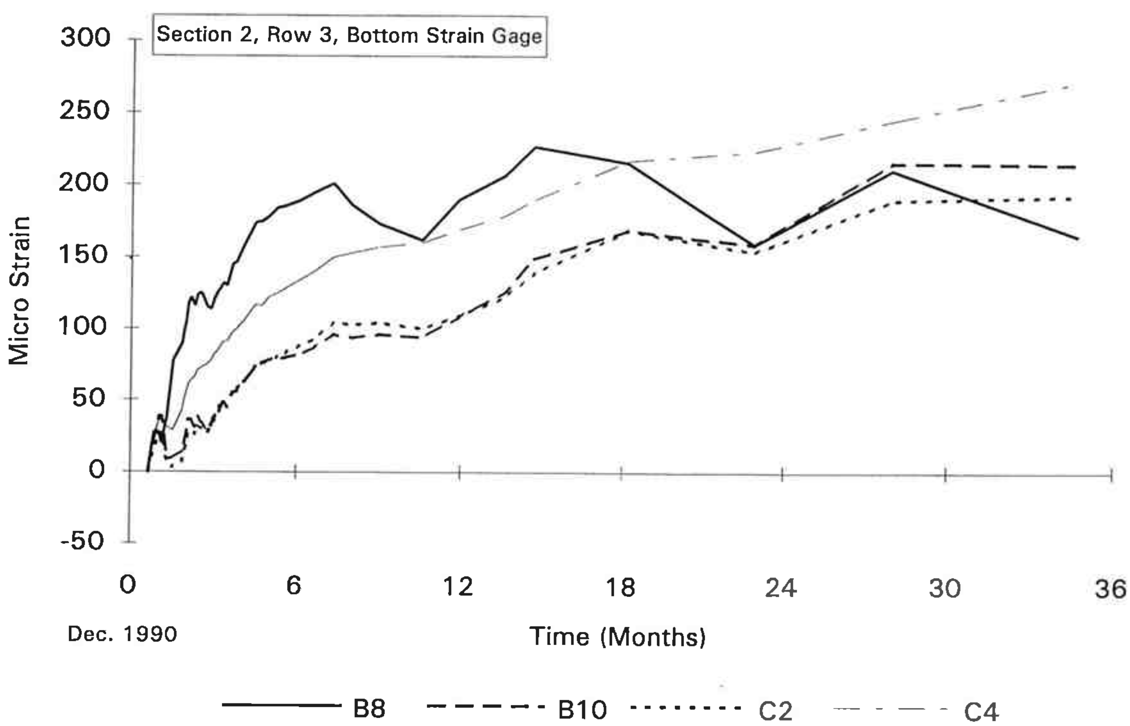


Fig. 113 - Long Term Performance

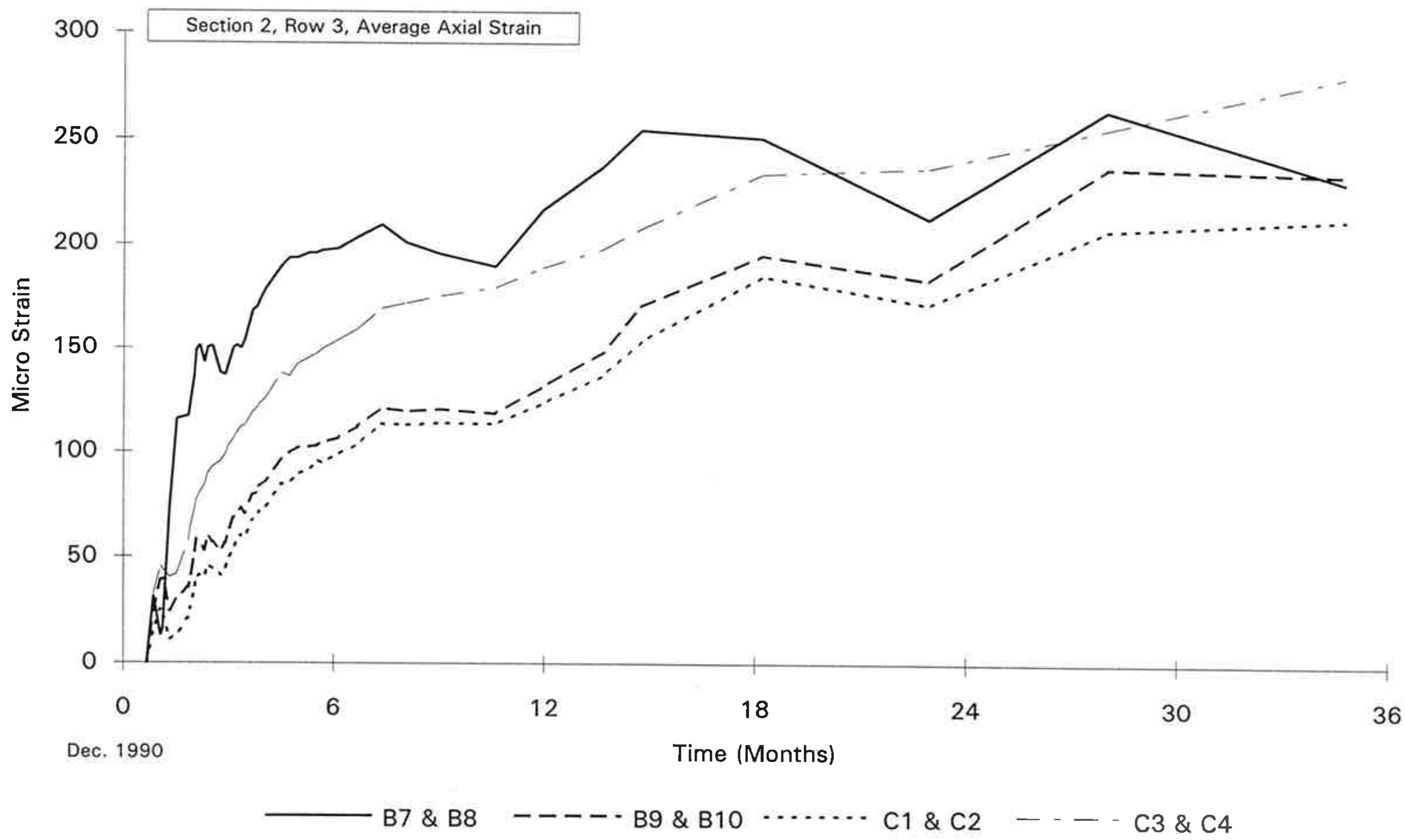


Fig. 114 - Long Term Performance

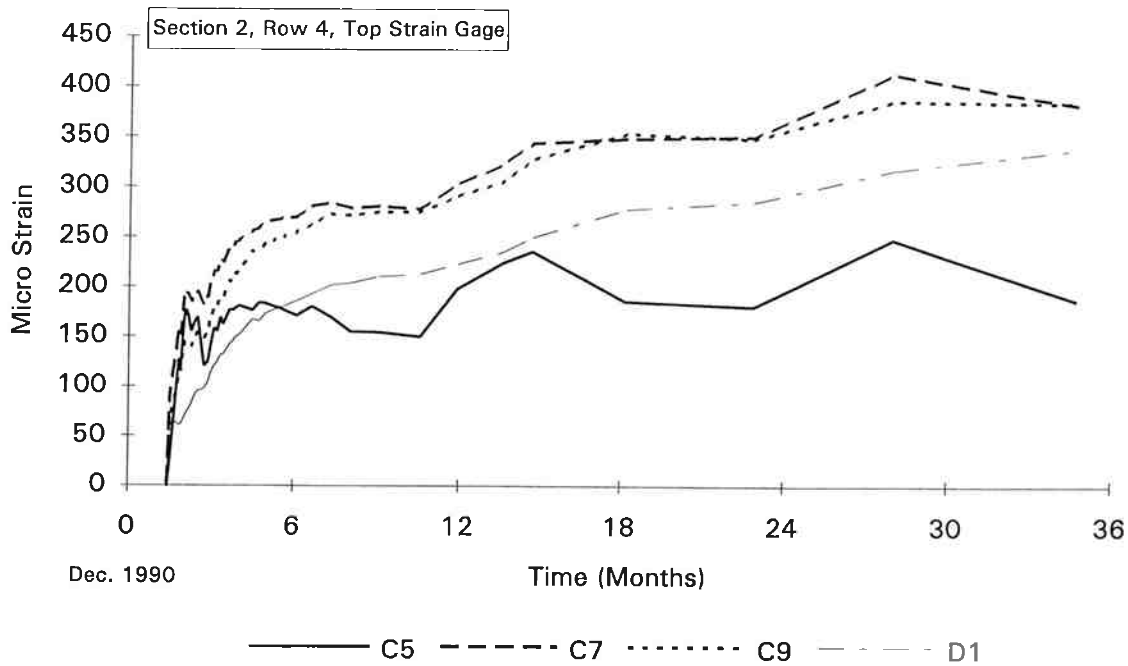


Fig. 115 - Long Term Performance

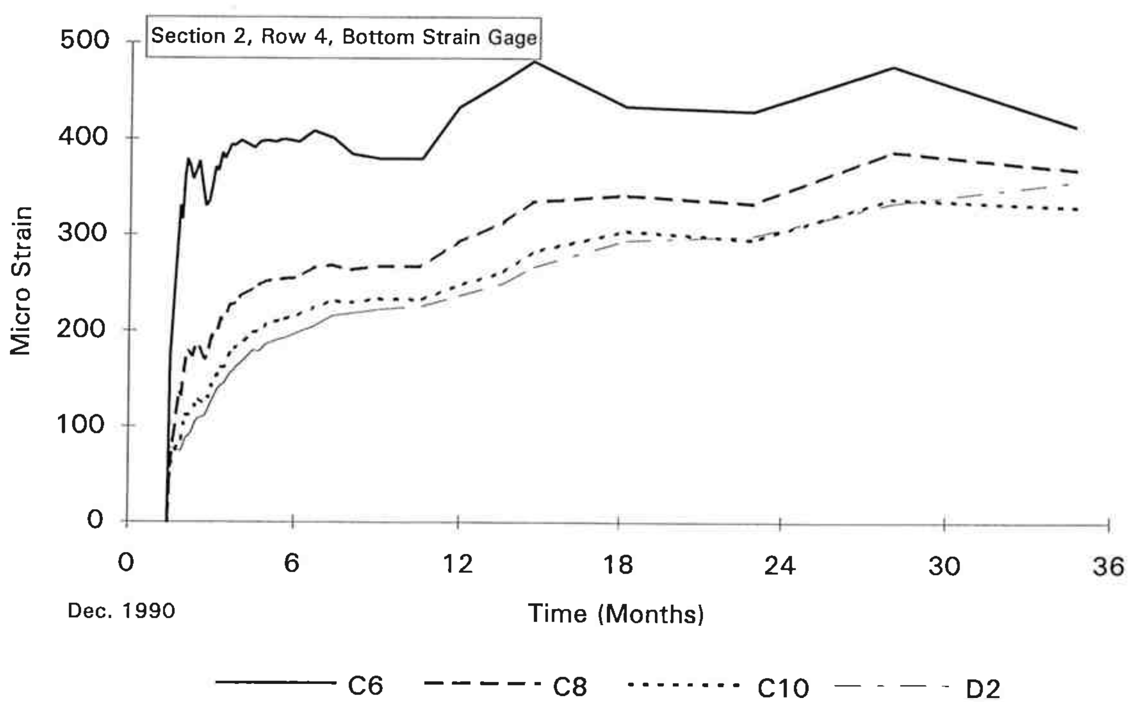


Fig. 116 - Long Term Performance

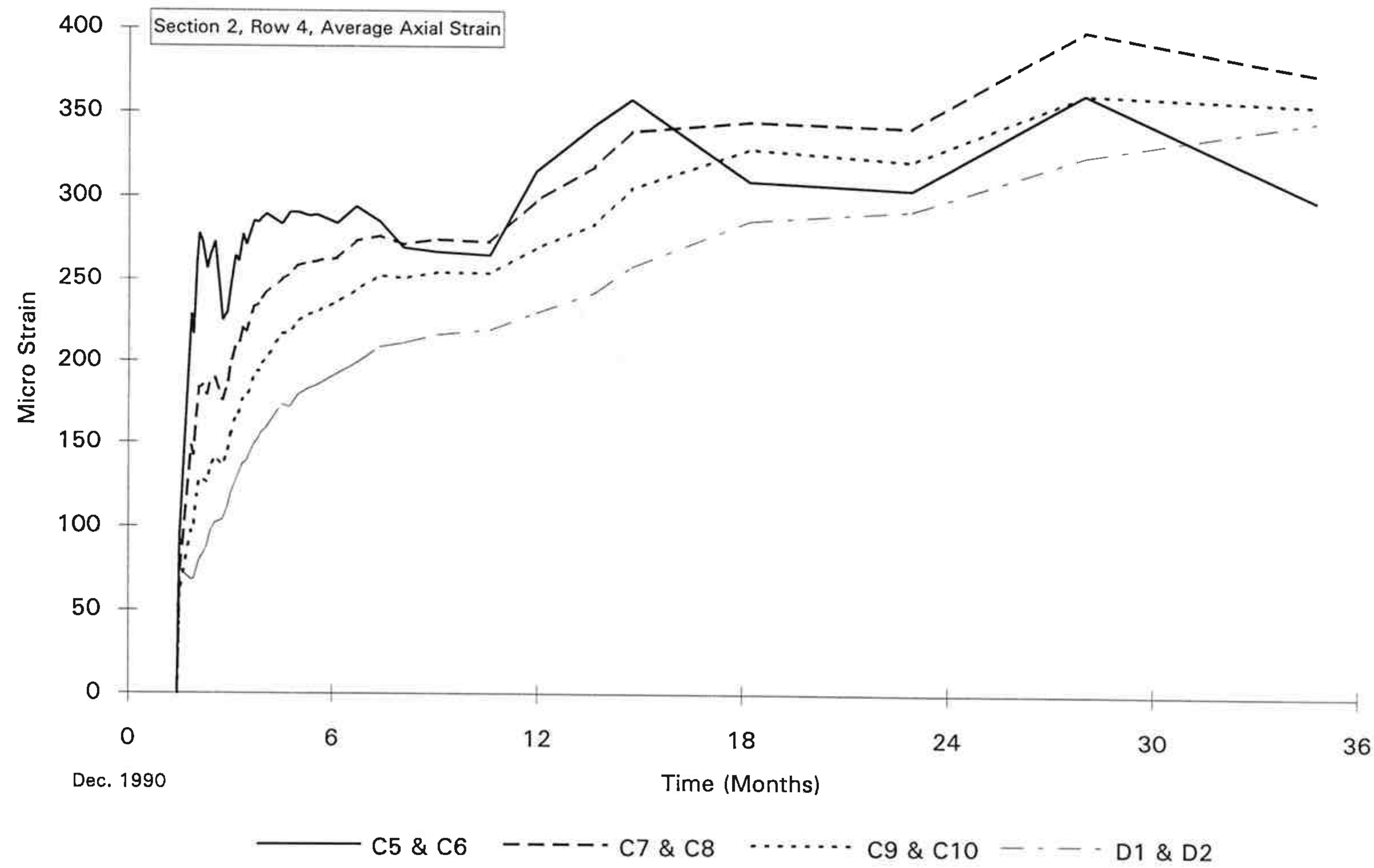


Fig. 117 - Long Term Performance

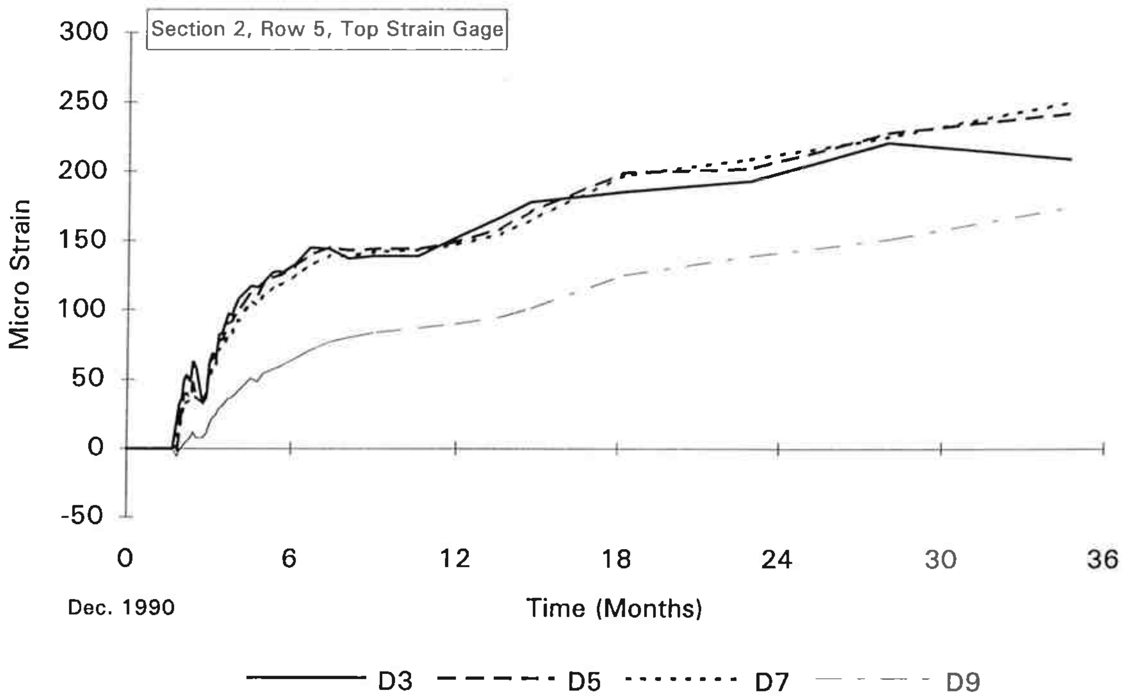


Fig. 118 - Long Term Performance

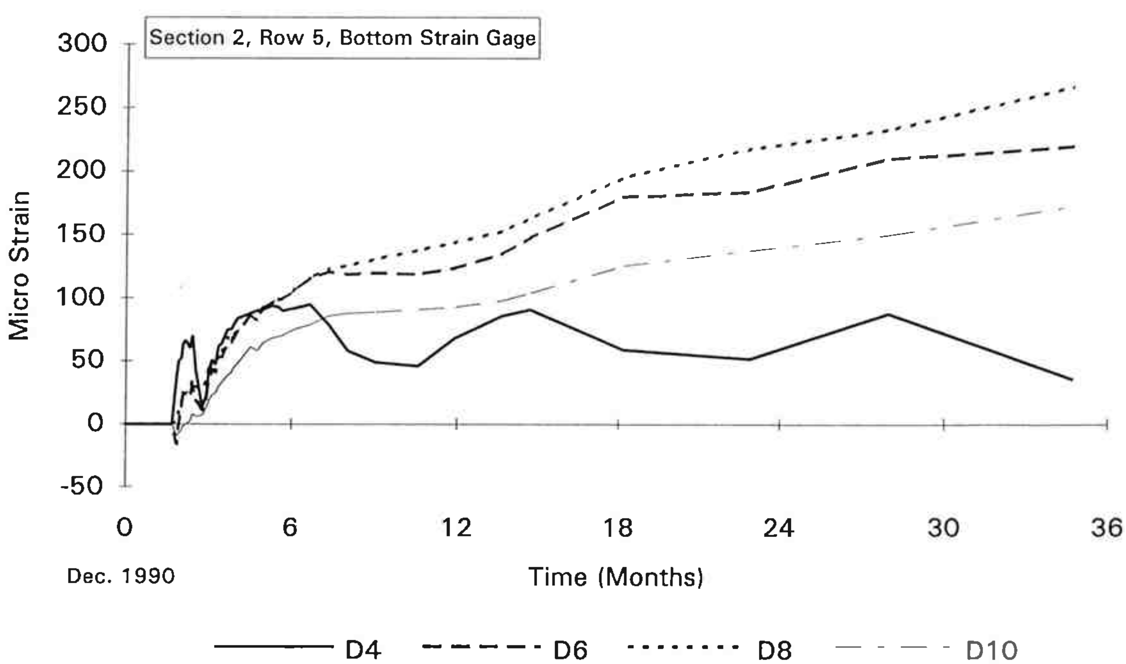


Fig. 119 - Long Term Performance

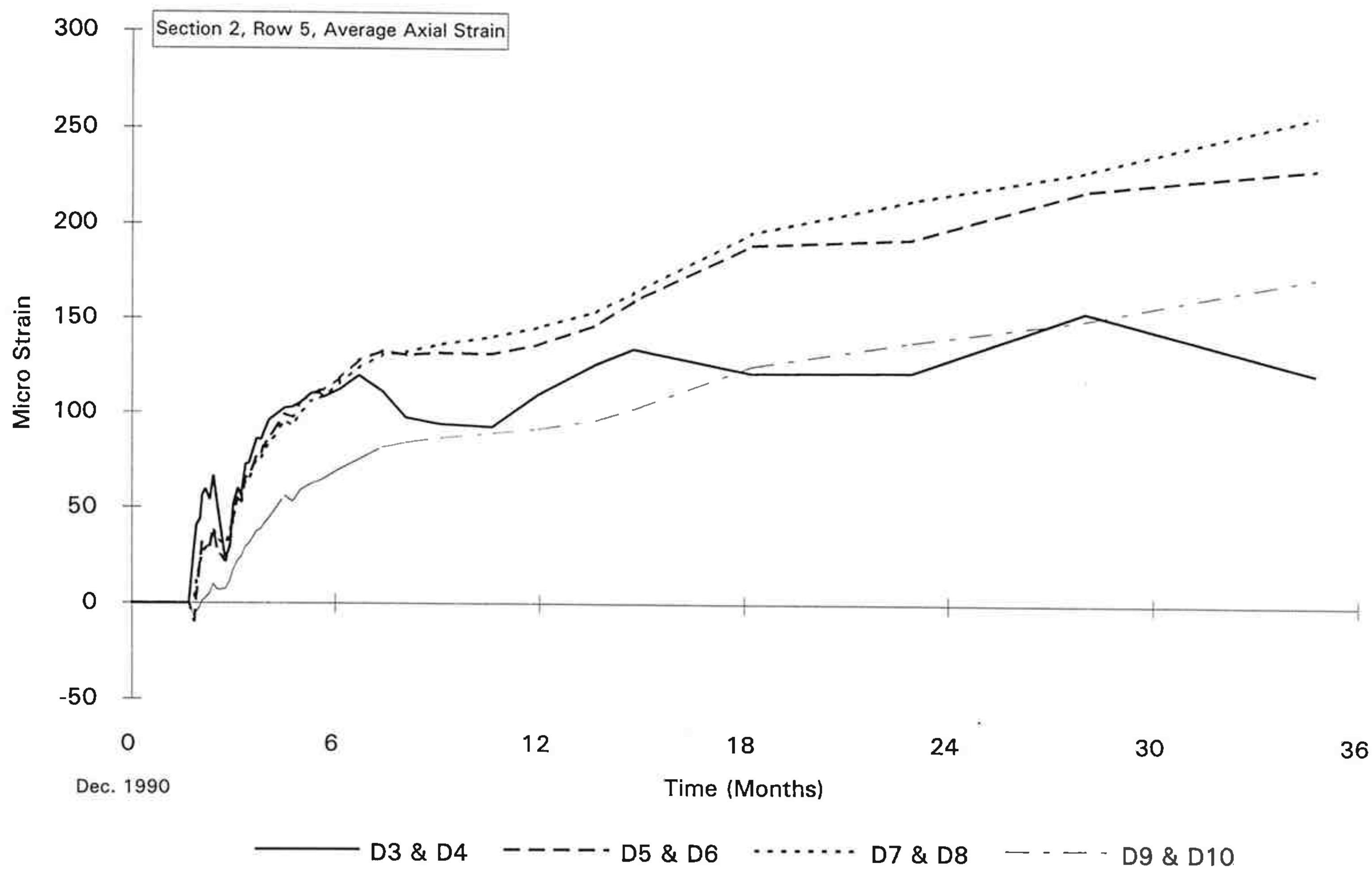


Fig. 120 - Long Term Nail Strain vs. Log Time at Section 1, Row 1

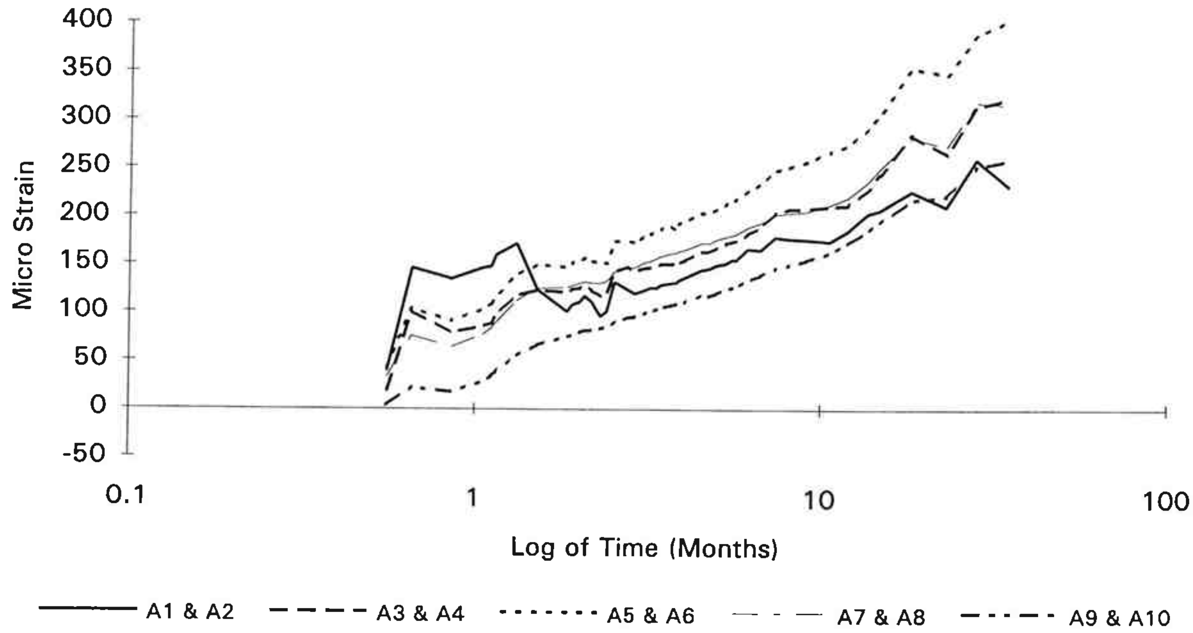


Fig. 121 - Long Term Nail Strain vs. Log Time at Section 1, Row 2

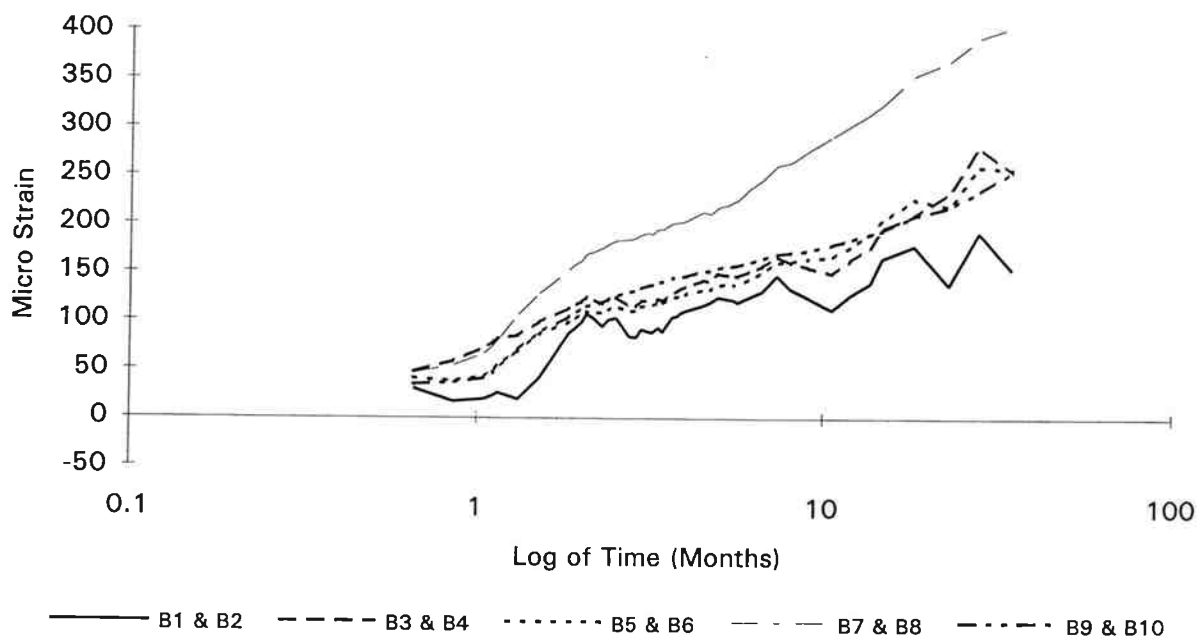


Fig. 122 - Long Term Nail Strain vs. Log Time at Section 1, Row 3

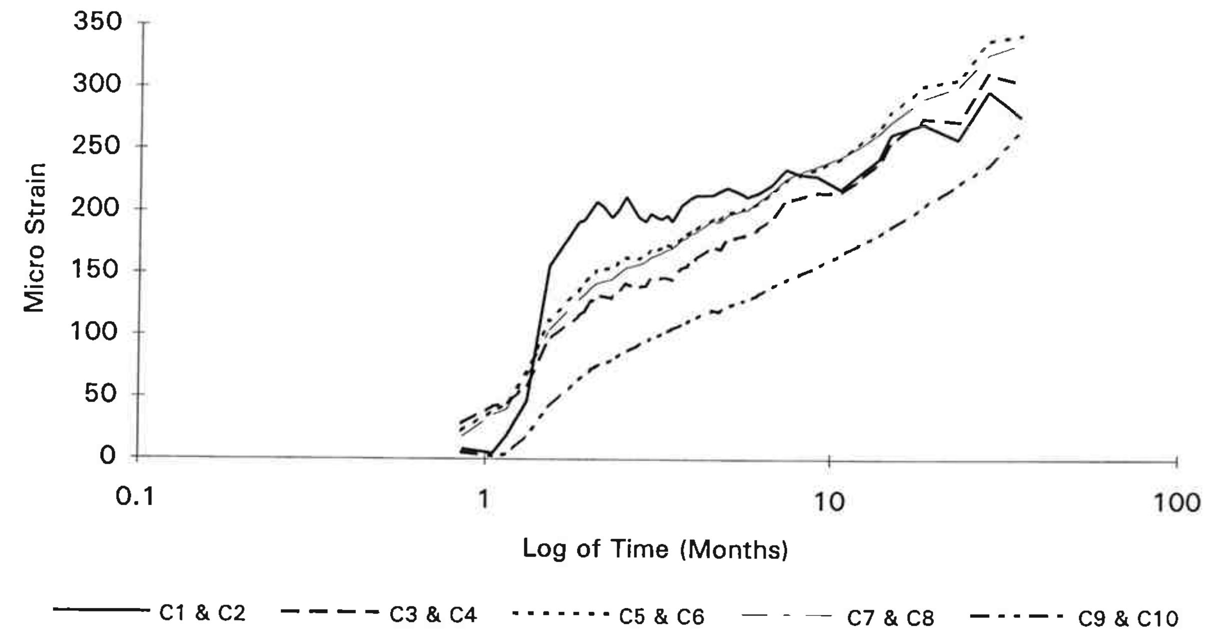
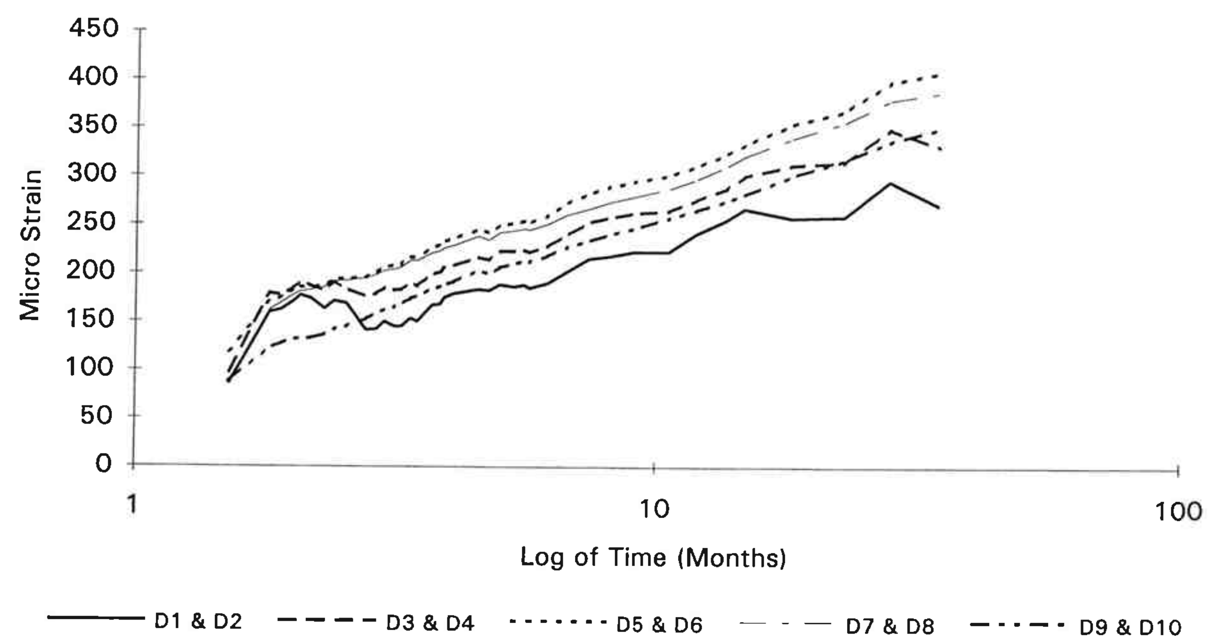


Fig. 123 - Long Term Nail Strain vs. Log Time at Section 1, Row 4



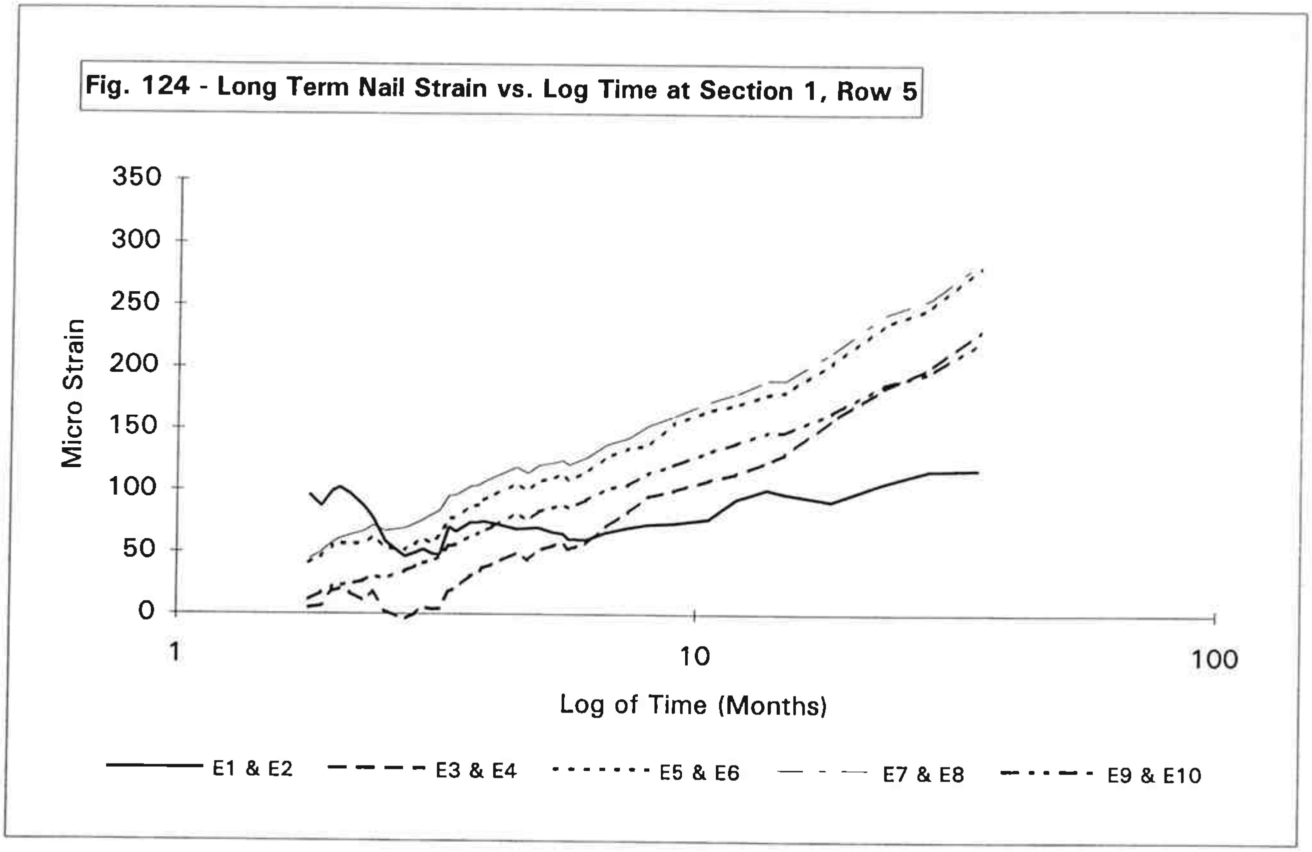


Fig. 124 - Long Term Nail Strain vs. Log Time at Section 1, Row 5
Micro Strain
350
300
250
200
150
100
50
0
1
10
100
Log of Time (Months)
E1 & E2
E3 & E4
E5 & E6
E7 & E8
E9 & E10

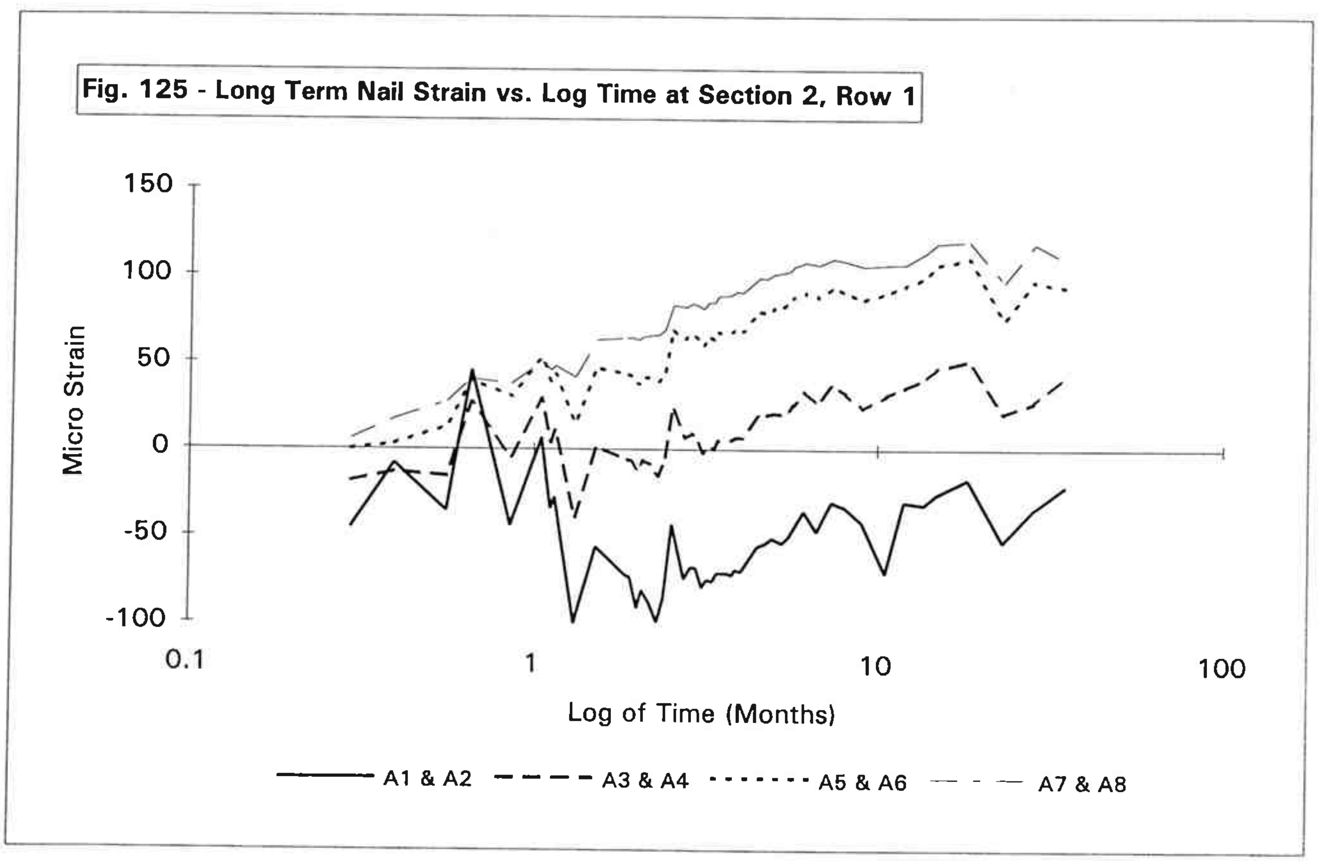


Fig. 125 - Long Term Nail Strain vs. Log Time at Section 2, Row 1
Micro Strain
150
100
50
0
-50
-100
0.1
1
10
100
Log of Time (Months)
A1 & A2
A3 & A4
A5 & A6
A7 & A8

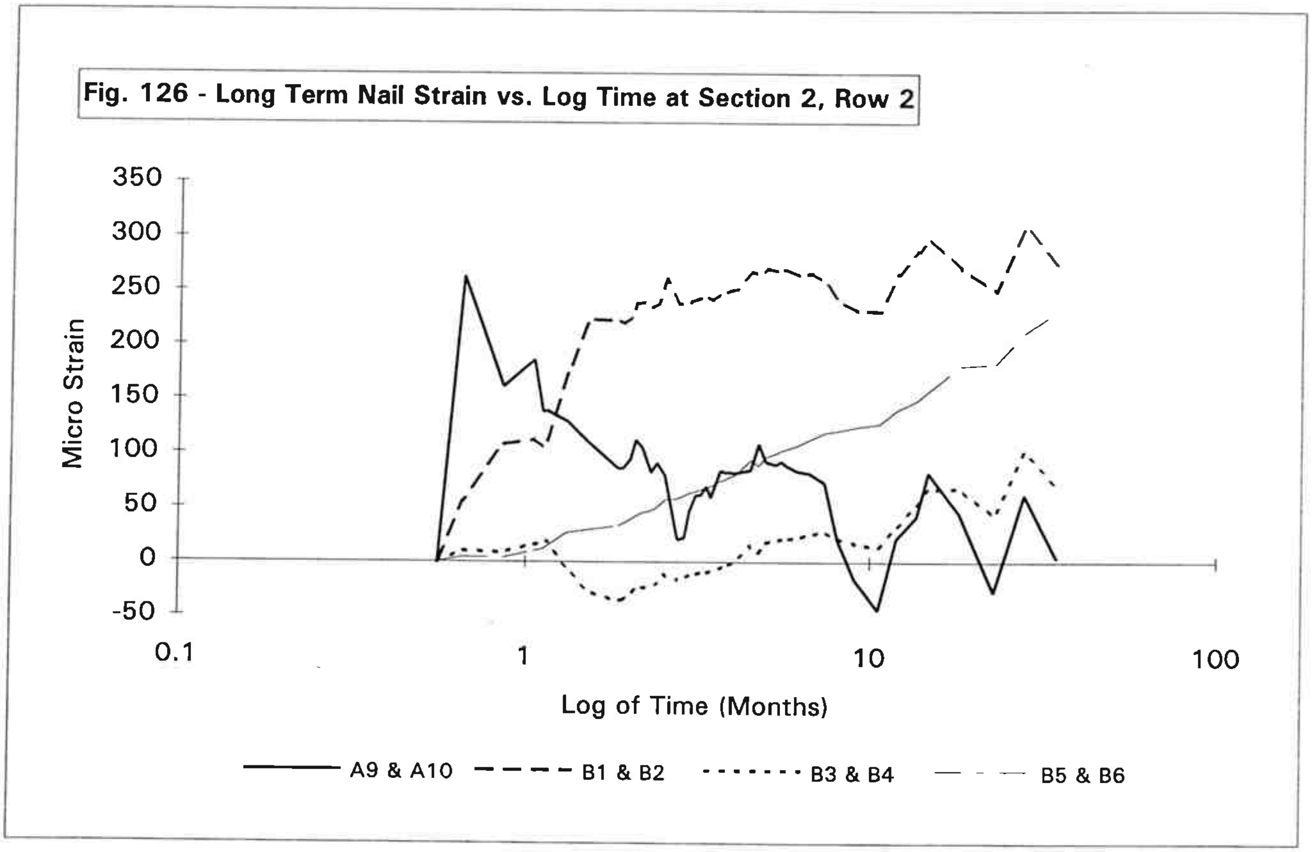


Fig. 126 - Long Term Nail Strain vs. Log Time at Section 2, Row 2
350
300
250
200
150
100
50
0
-50
Micro Strain
0.1
1
10
100
Log of Time (Months)
A9 & A10
B1 & B2
B3 & B4
B5 & B6

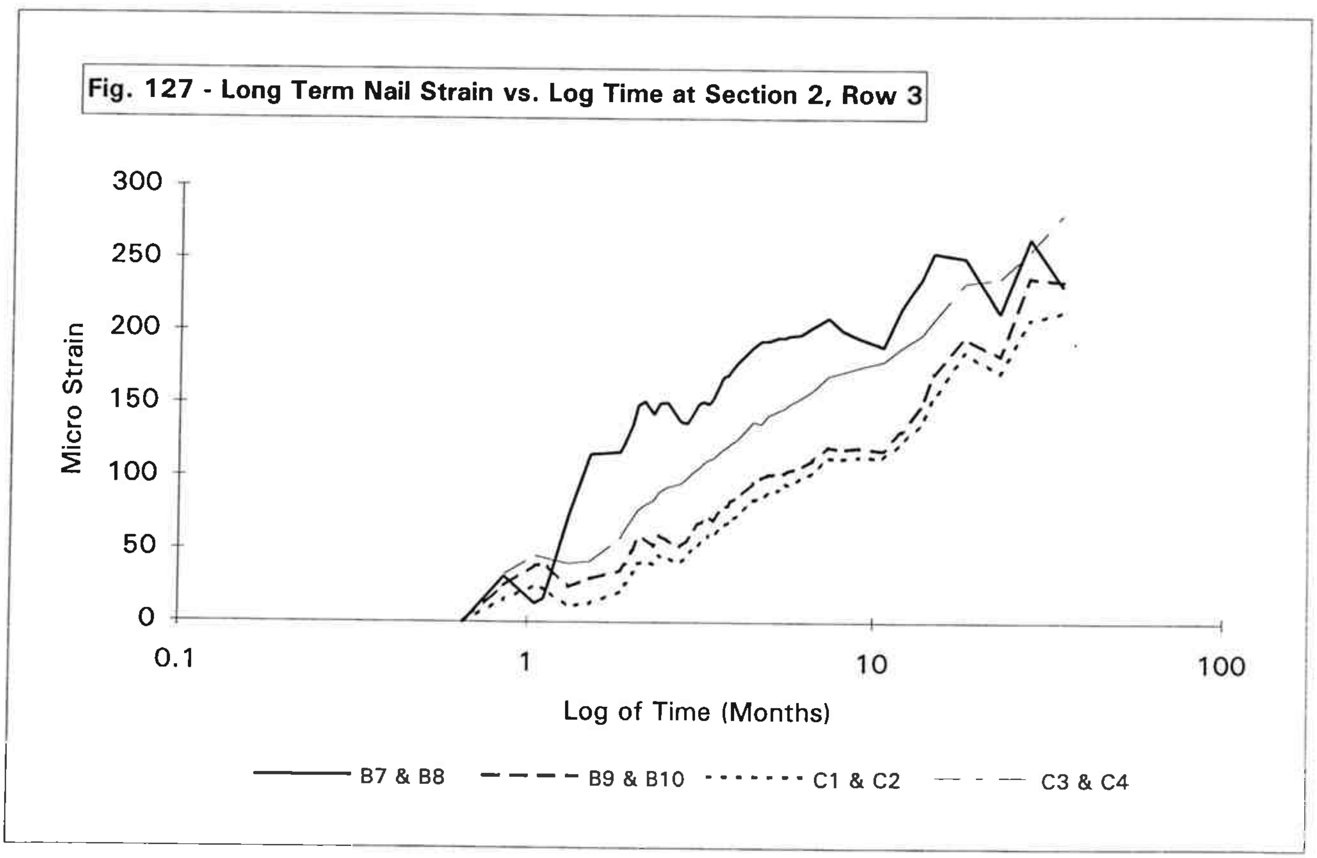


Fig. 127 - Long Term Nail Strain vs. Log Time at Section 2, Row 3
300
250
200
150
100
50
0
Micro Strain
0.1
1
10
100
Log of Time (Months)
B7 & B8
B9 & B10
C1 & C2
C3 & C4

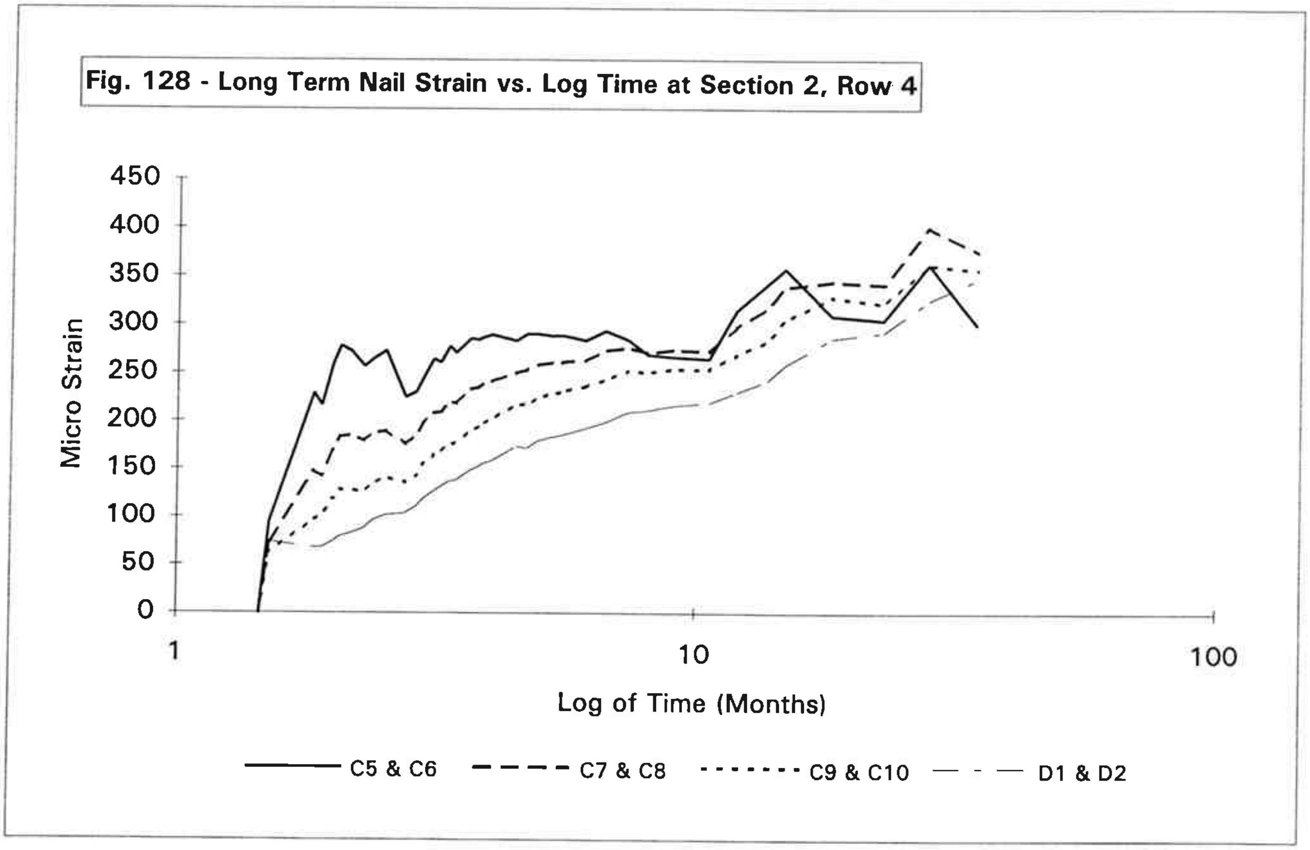


Fig. 128 - Long Term Nail Strain vs. Log Time at Section 2, Row 4
450
400
350
300
250
200
150
100
50
0
Micro Strain
1
10
100
Log of Time (Months)
C5 & C6
C7 & C8
C9 & C10
D1 & D2

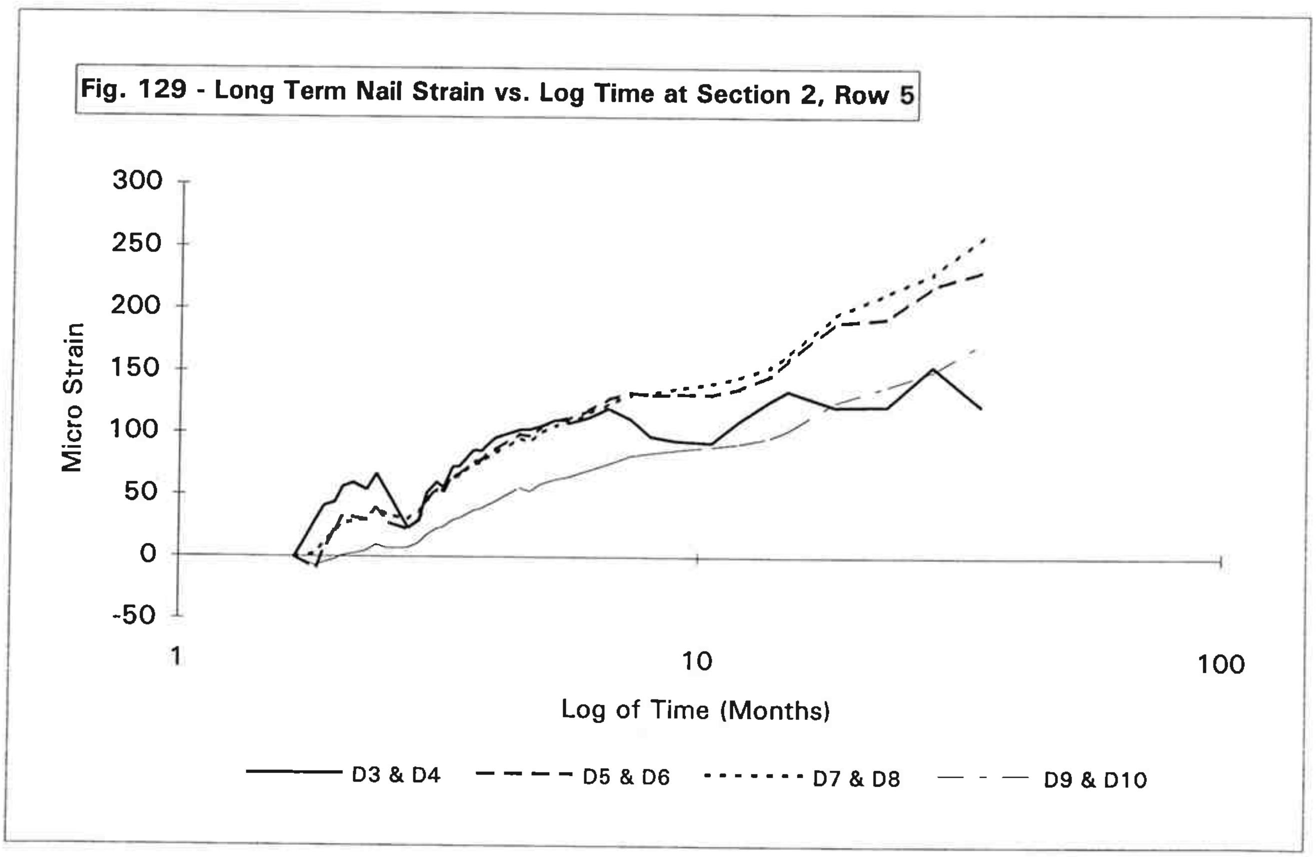


Fig. 129 - Long Term Nail Strain vs. Log Time at Section 2, Row 5
300
250
200
150
100
50
0
-50
Micro Strain
1
10
100
Log of Time (Months)
D3 & D4
D5 & D6
D7 & D8
D9 & D10

Fig. 130 - Long Term Tensile Nail Loads at Section 1, Row 1

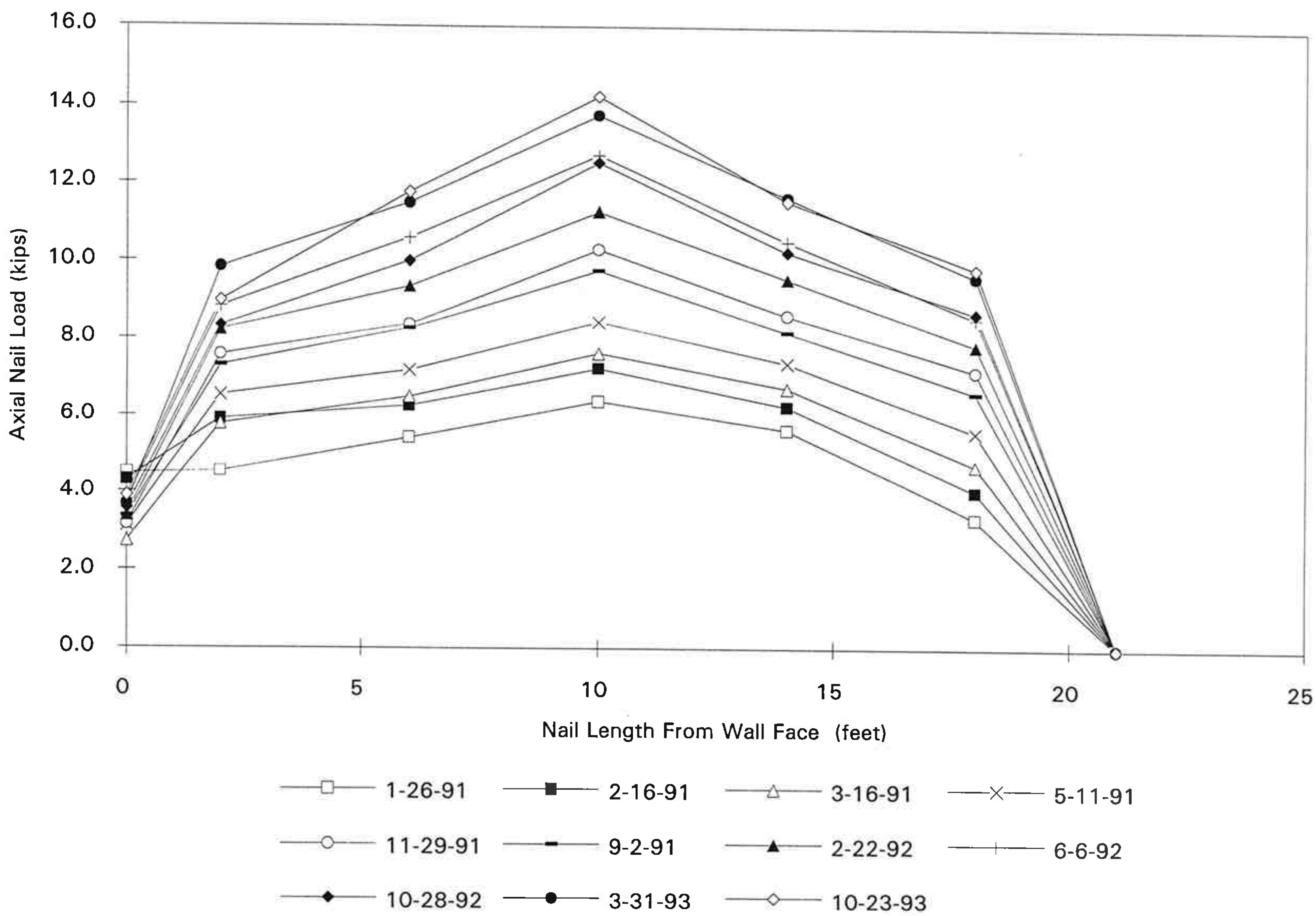


Fig. 131 - Long Term Tensile Nail Loads at Section 1, Row 2

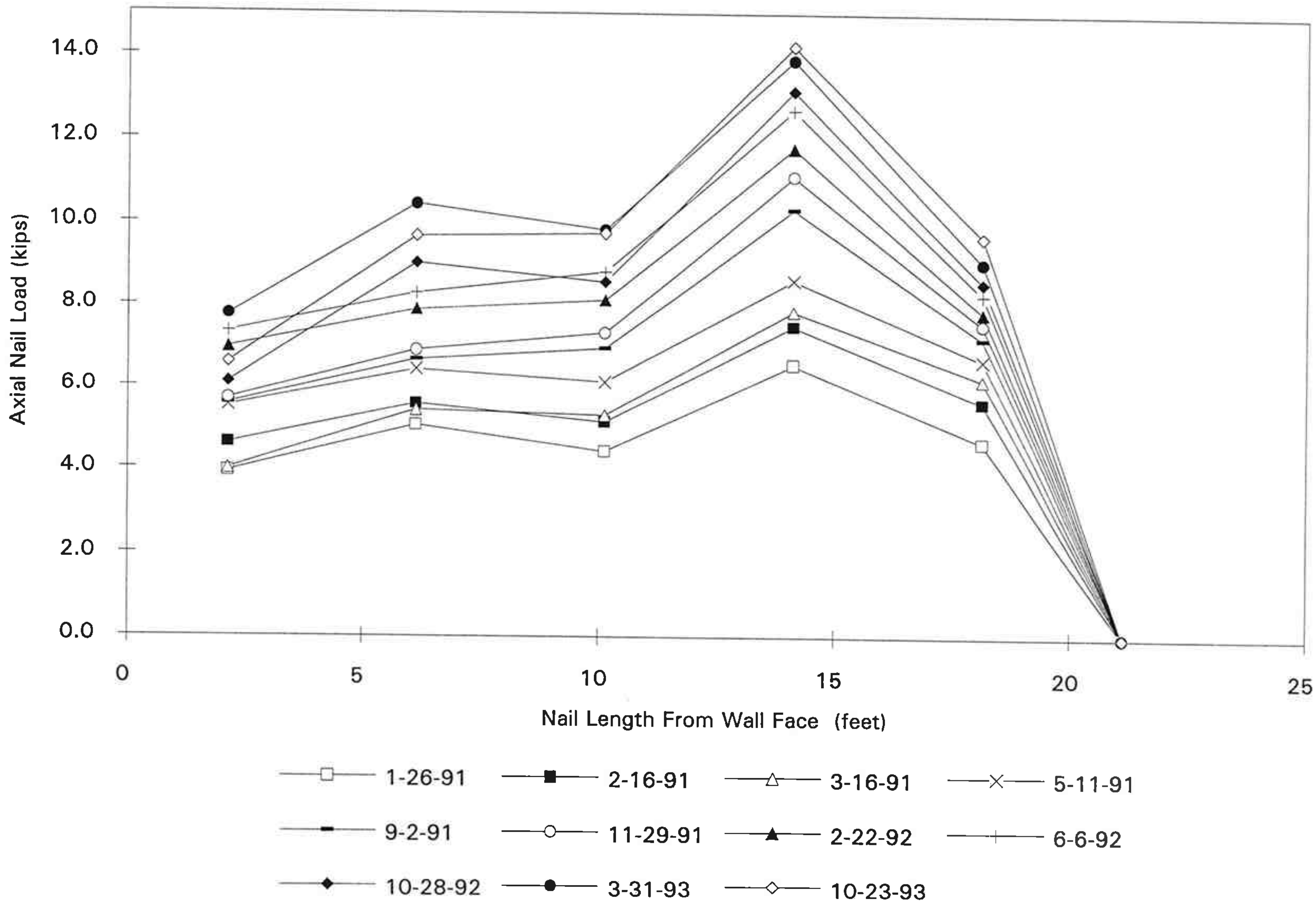


Fig. 132 - Long Term Tensile Nail Loads at Section 1, Row 3

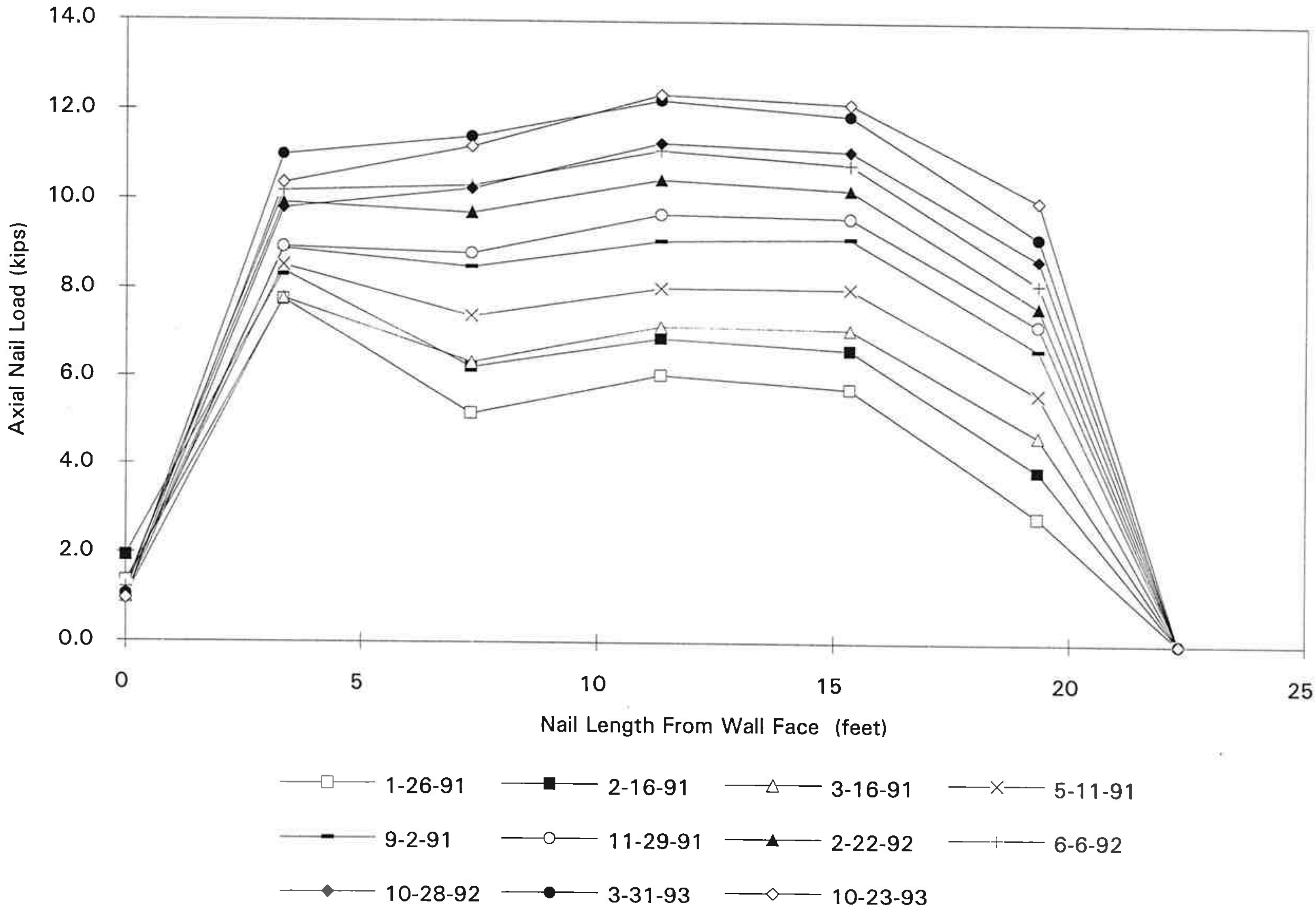


Fig. 133 - Long Term Tensile Nail Loads at Section 1, Row 4

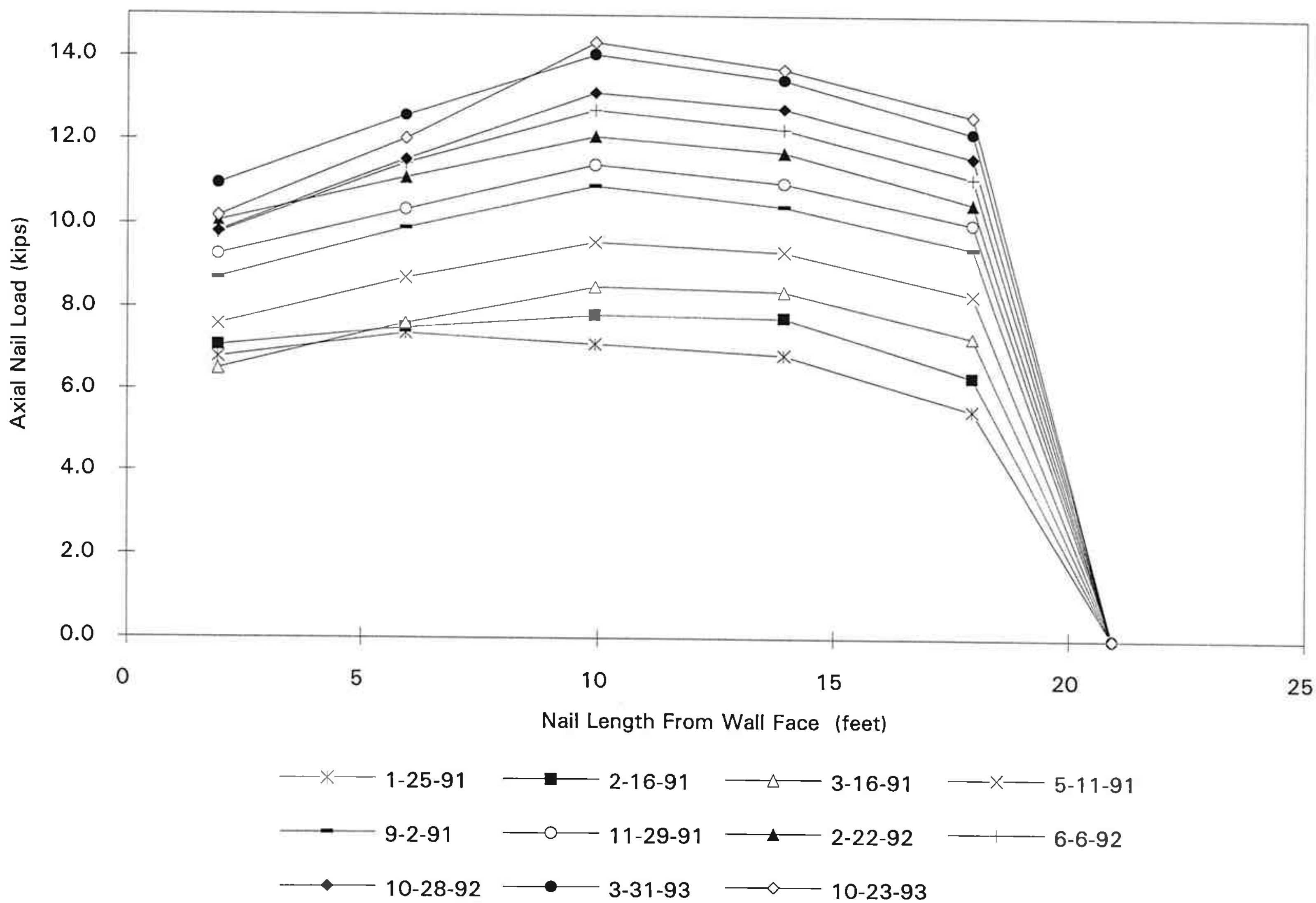


Fig. 134 - Long Term Tensile Nail Loads at Section 1, Row 5

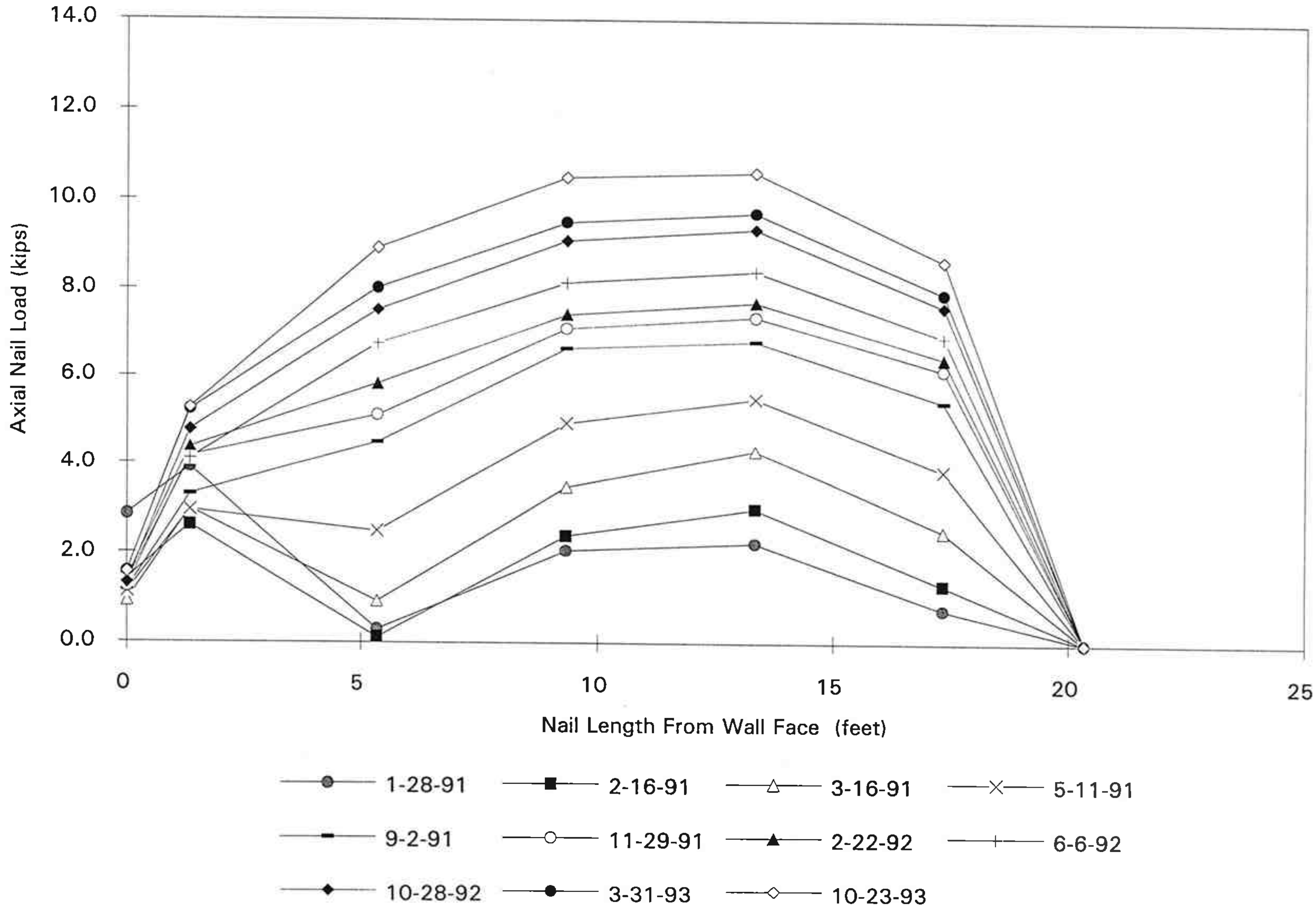


Fig. 135 - Long Term Tensile Nail Loads at Section 2, Row 1

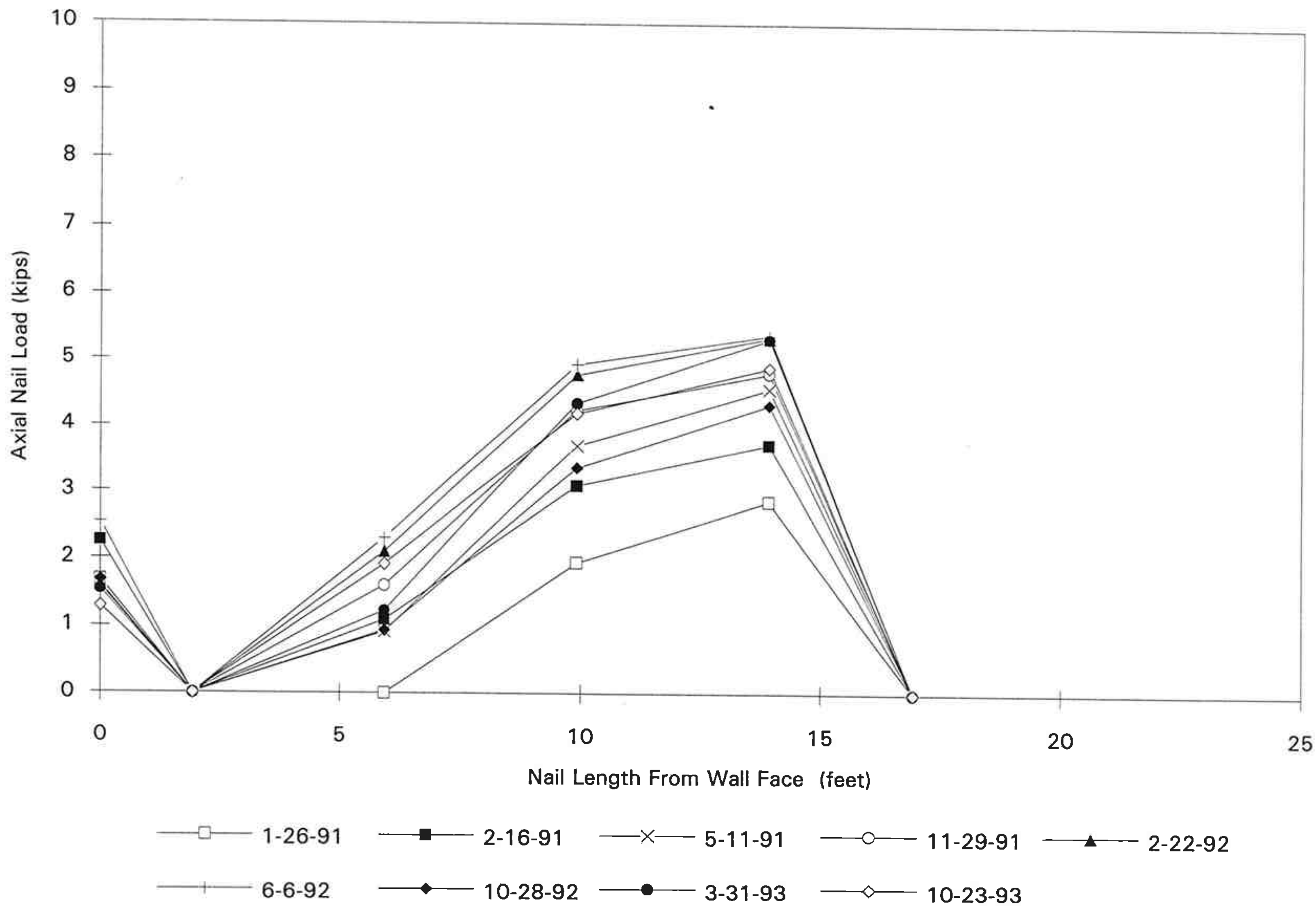


Fig. 136 - Long Term Tensile Nail Loads at Section 2, Row 2

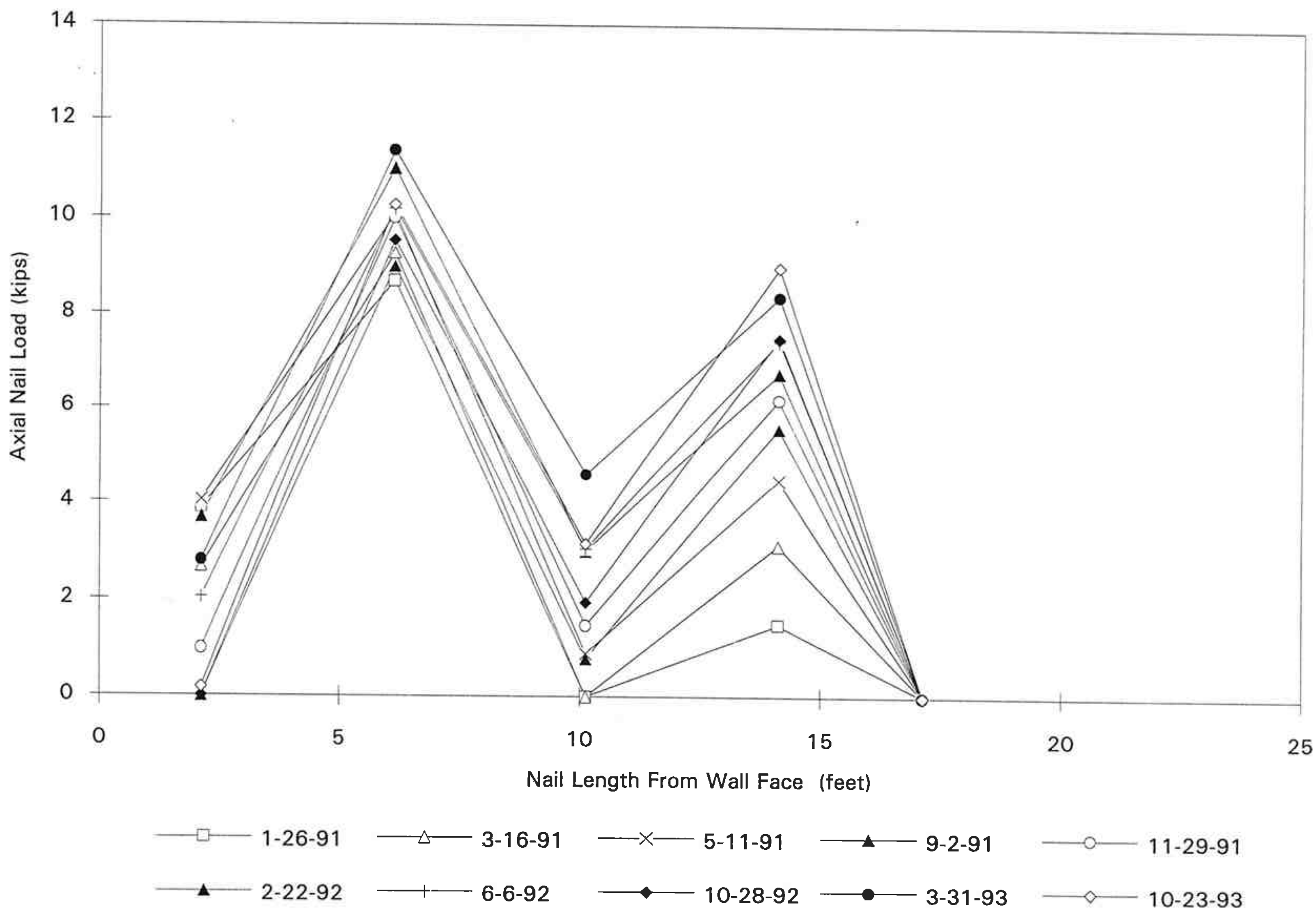


Fig. 137 - Long Term Tensile Nail Loads at Section 2, Row 3

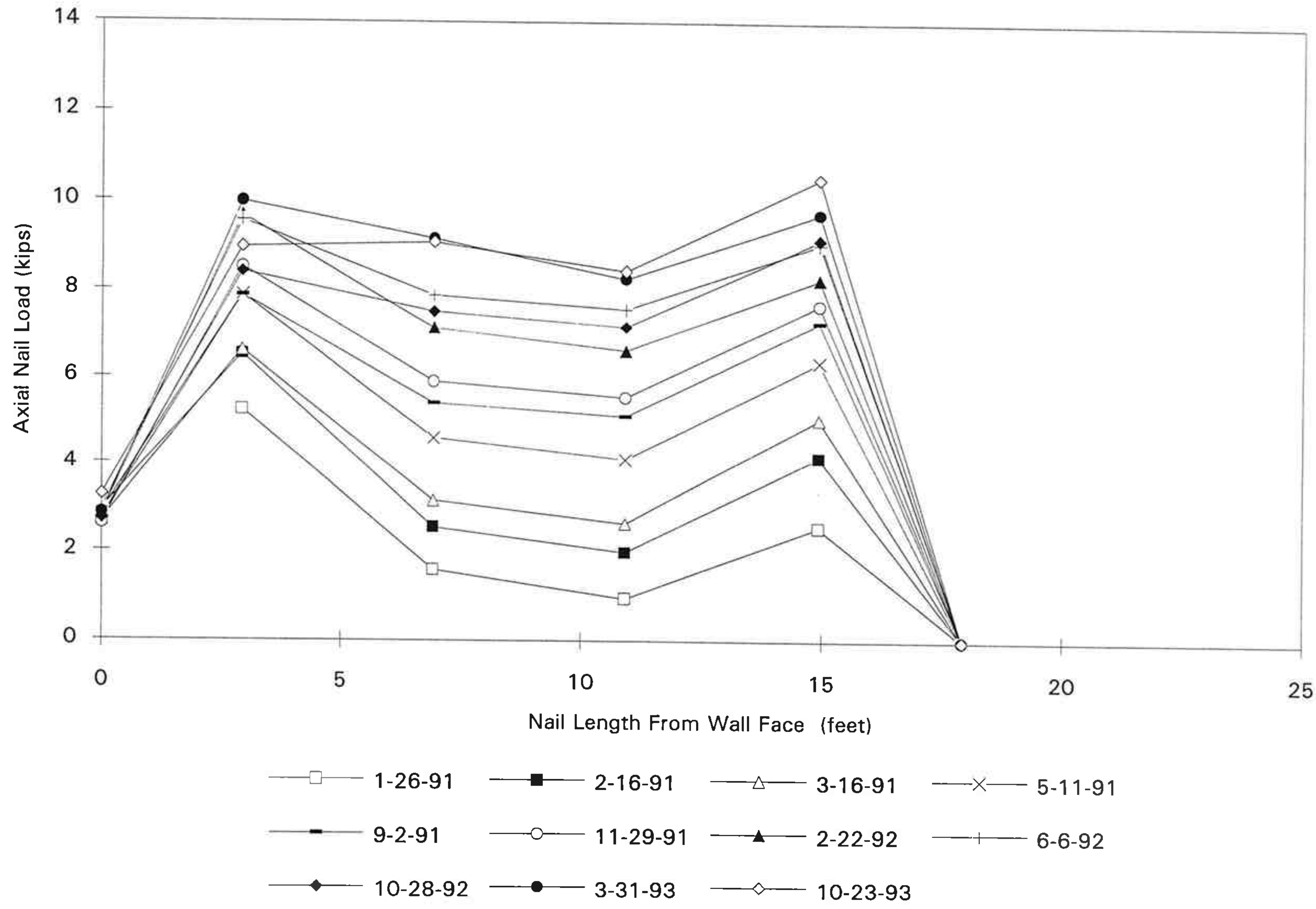


Fig. 138 - Long Term Tensile Nail Loads at Section 2, Row 4

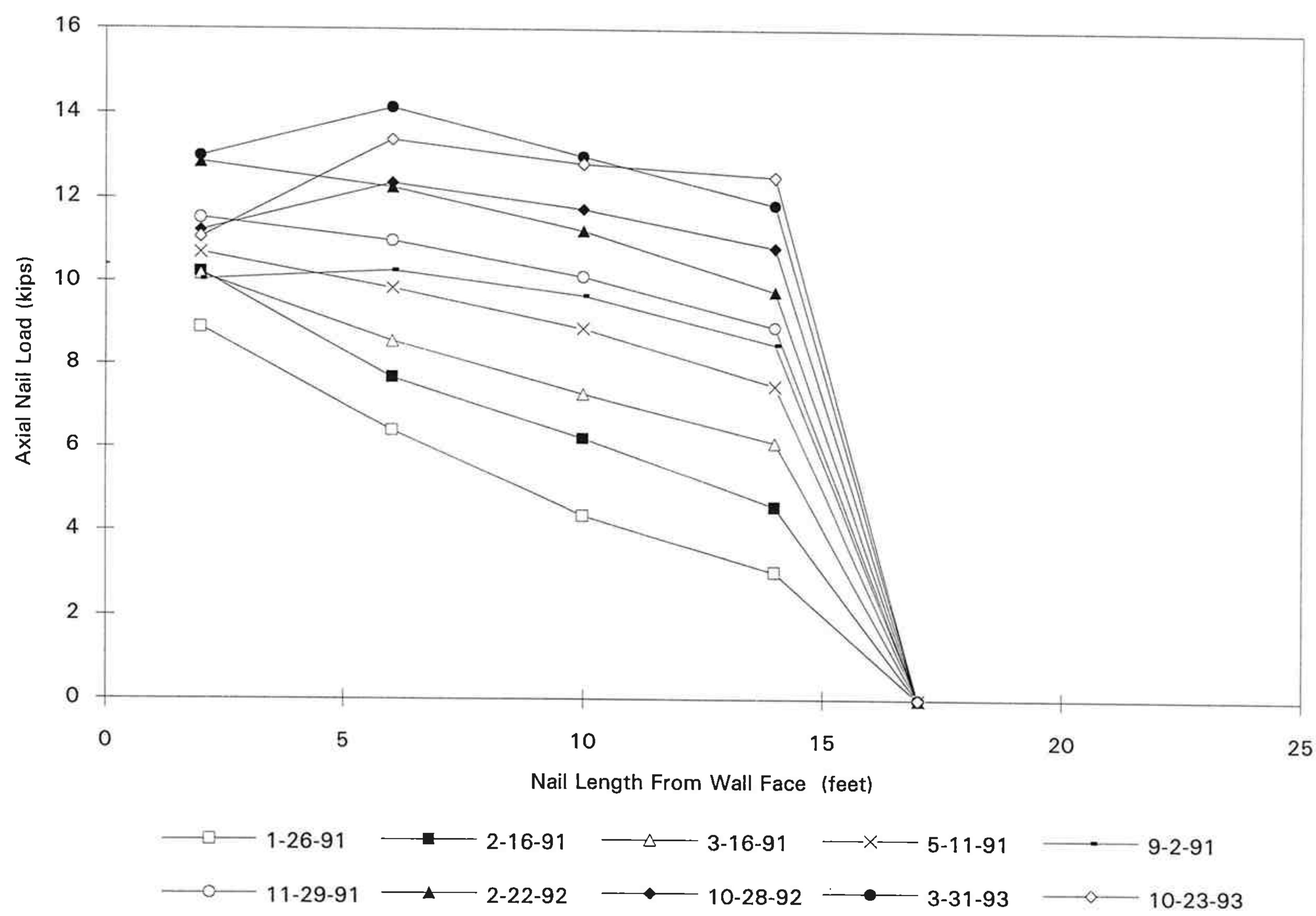


Fig. 139 - Long Term Tensile Nail Loads at Section 2, Row 5

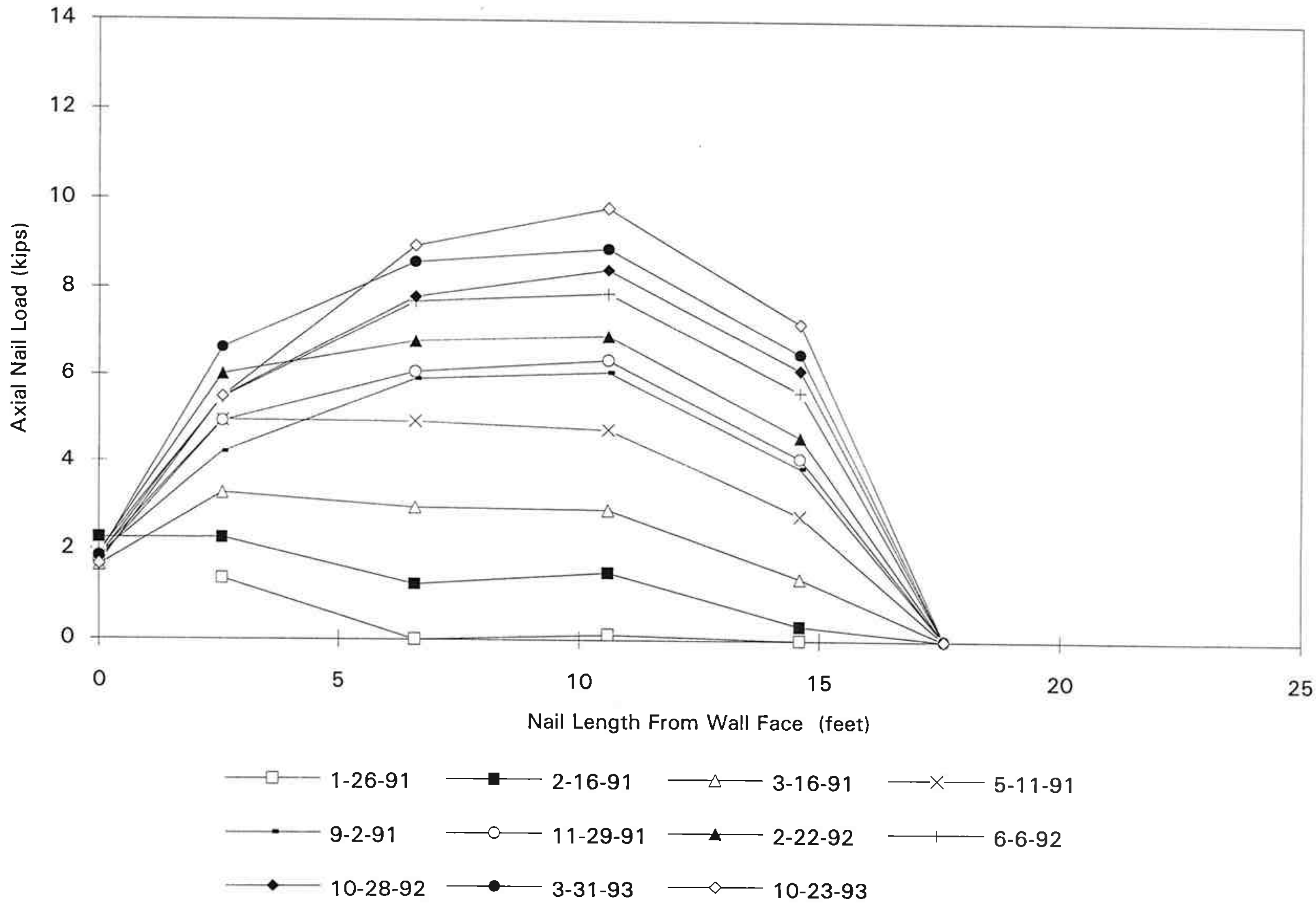


Fig. 140 - Comparison of Nail Force Distribution at Section 1

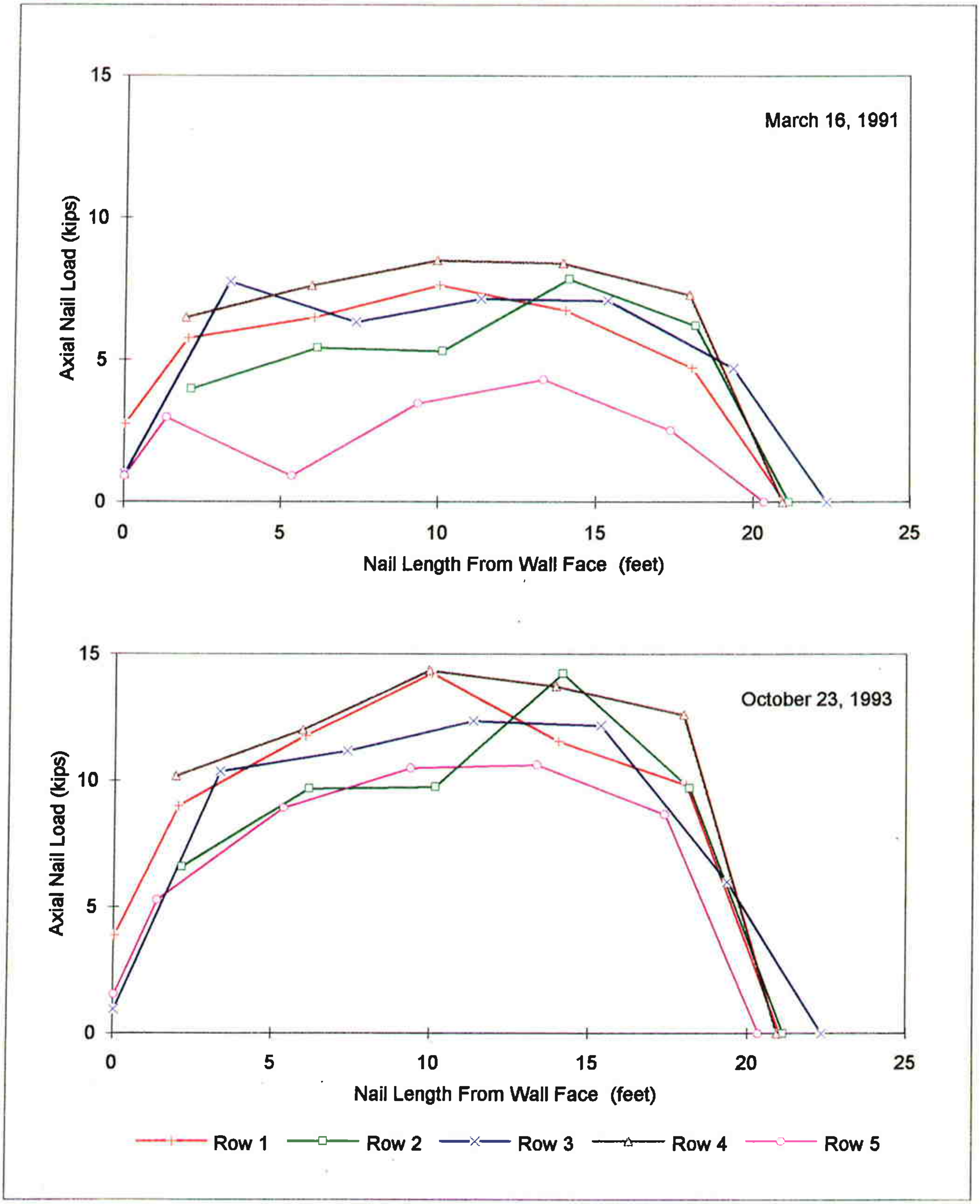


Fig. 141 - Comparison of Nail Force Distribution at Section 2

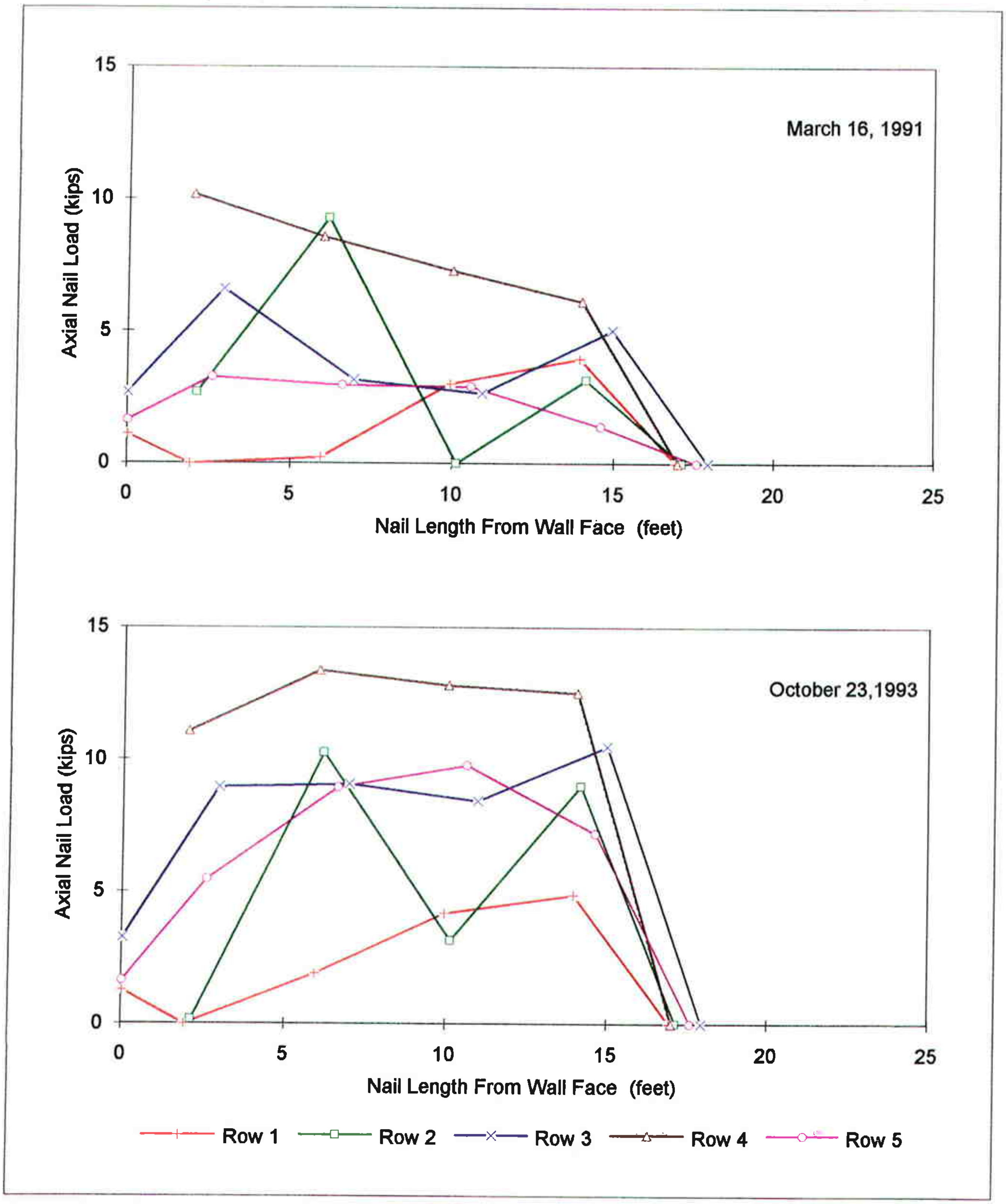


Fig. 142 - Load Cell Readings

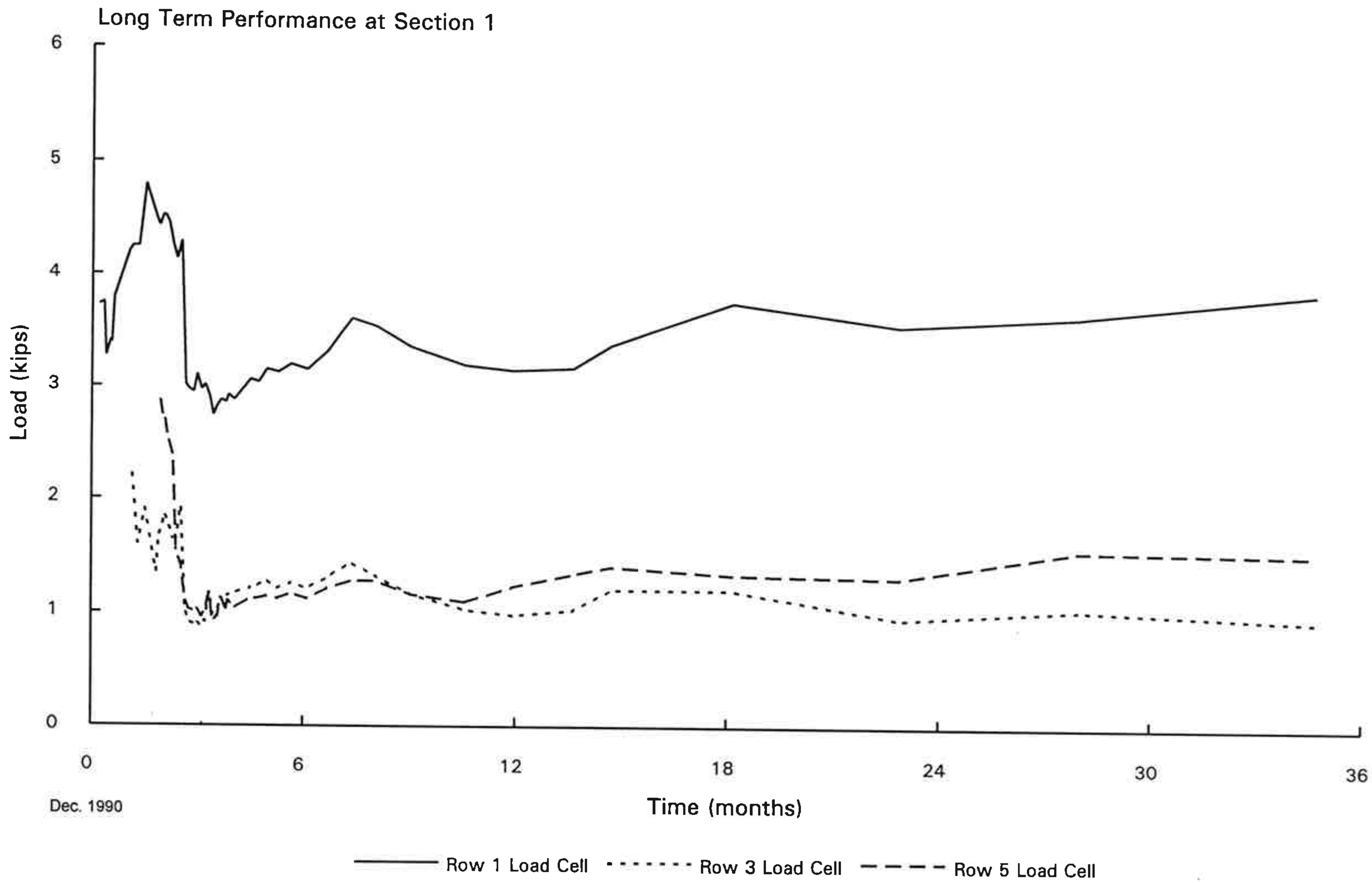
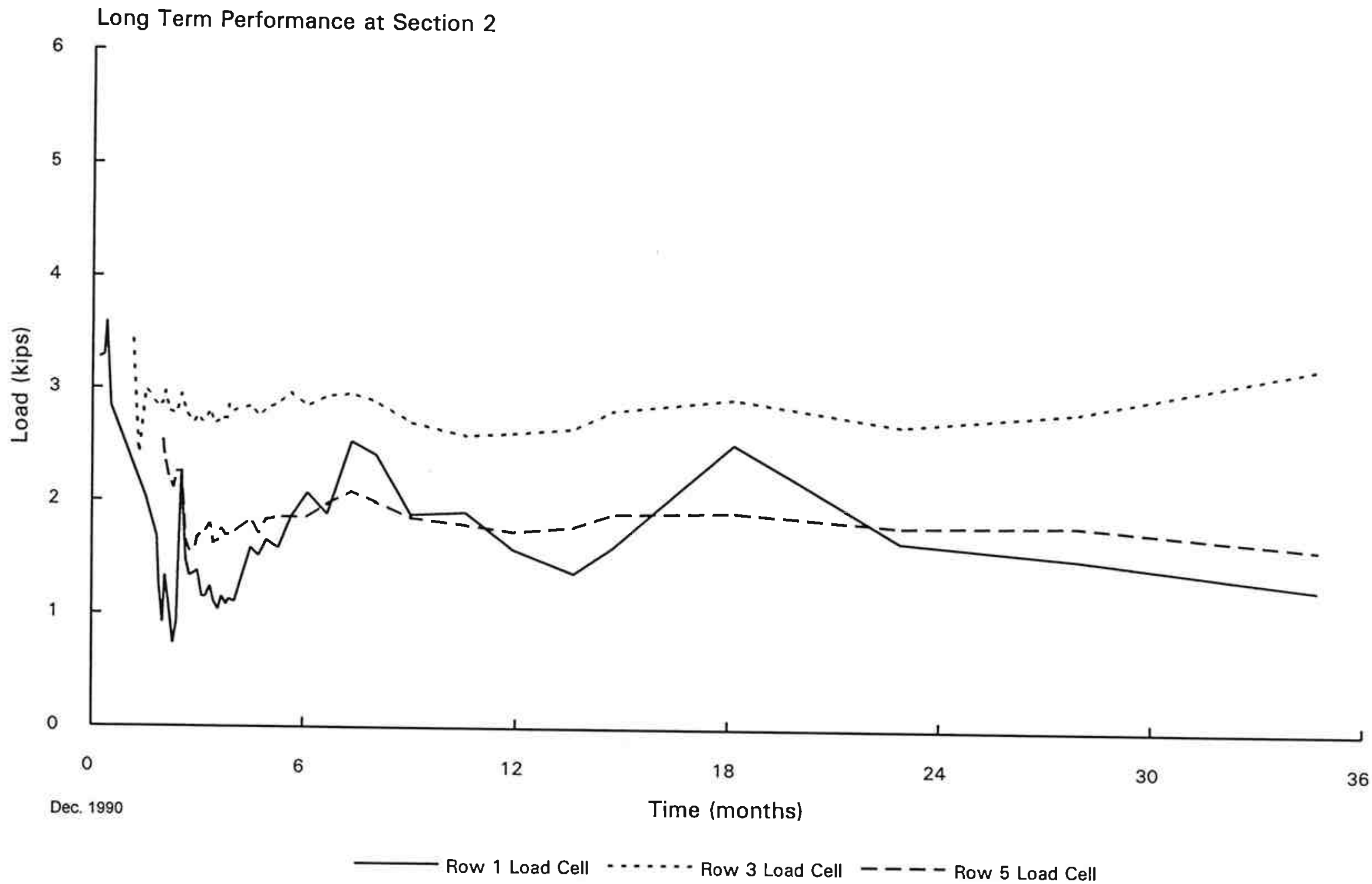


Fig. 143 - Load Cell Readings



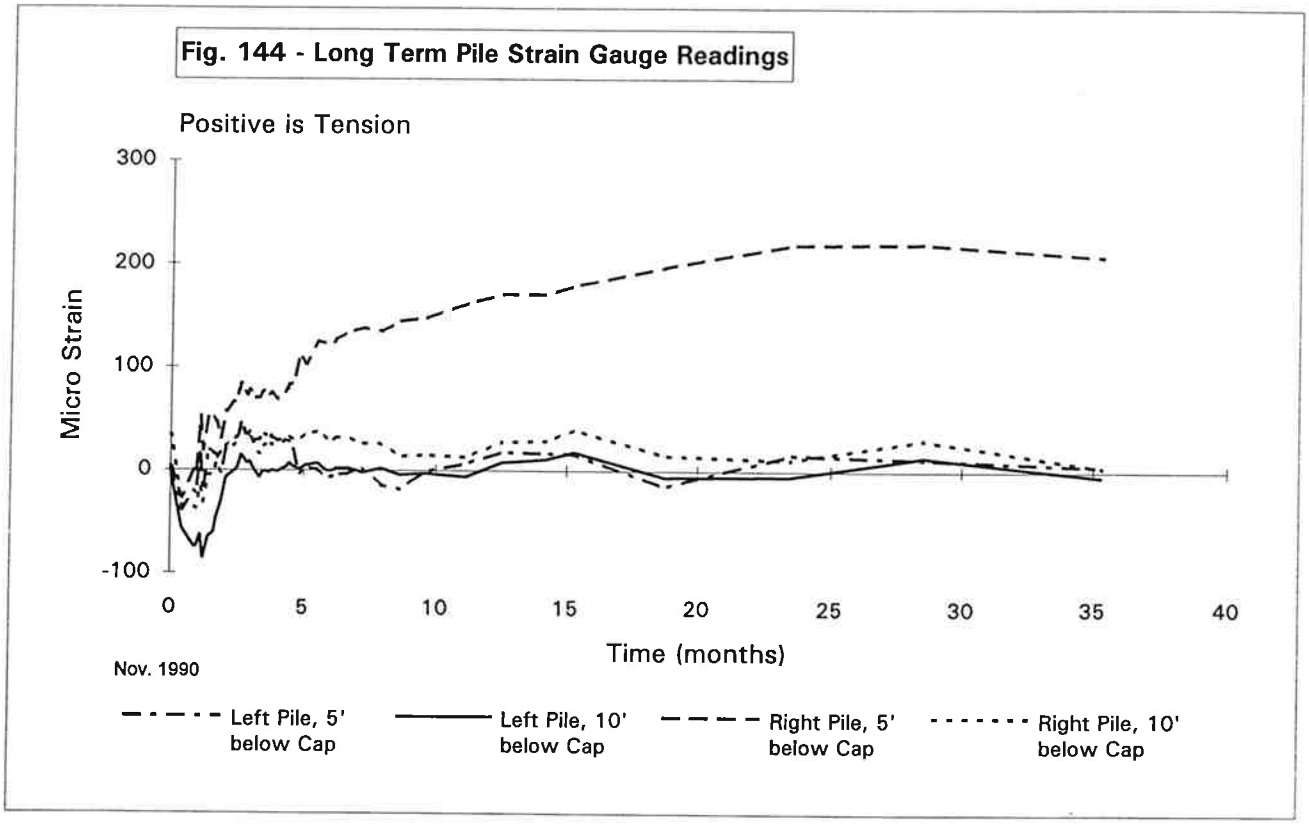


Fig. 144 - Long Term Pile Strain Gauge Readings
Positive is Tension
300
200
100
0
-100
Micro Strain
0
5
10
15
20
25
30
35
40
Time (months)
Nov. 1990
Left Pile, 5' below Cap
Left Pile, 10' below Cap
Right Pile, 5' below Cap
Right Pile, 10' below Cap

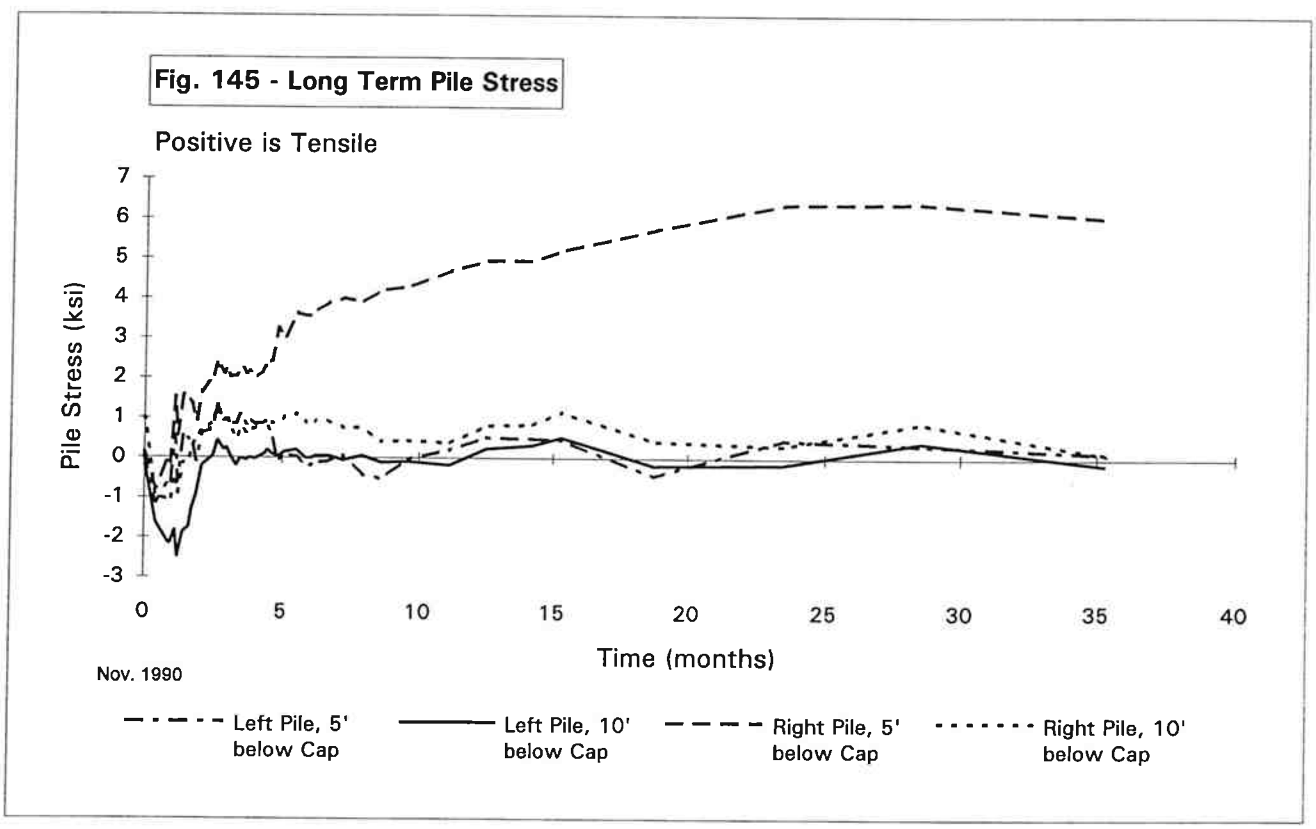


Fig. 145 - Long Term Pile Stress
Positive is Tensile
7
6
5
4
3
2
1
0
-1
-2
-3
Pile Stress (ksi)
0
5
10
15
20
25
30
35
40
Time (months)
Nov. 1990
Left Pile, 5' below Cap
Left Pile, 10' below Cap
Right Pile, 5' below Cap
Right Pile, 10' below Cap

Fig. 146 - Long Term Pile Cap Deflection Measured by Single Point Extensometer

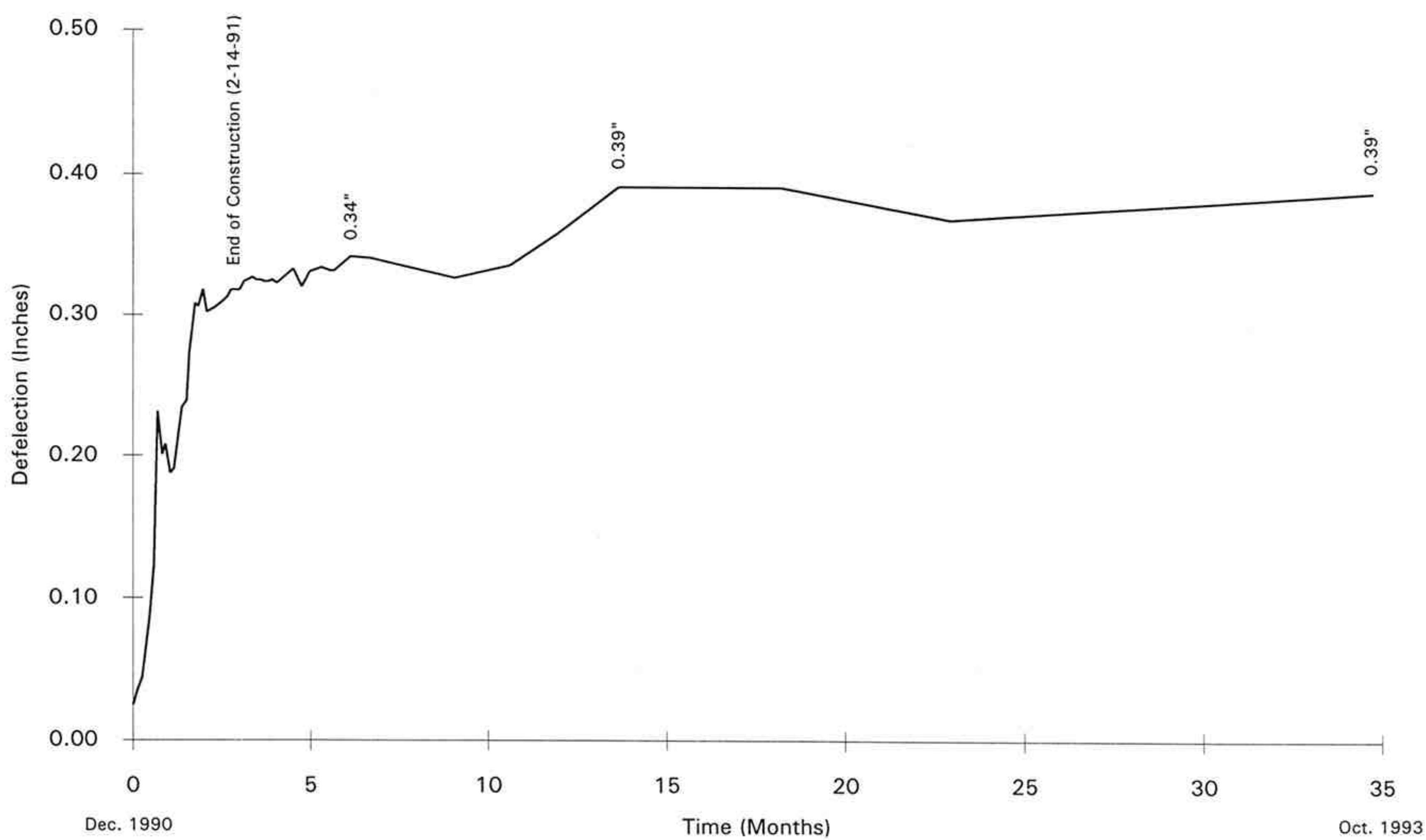


Fig. 147 - Long Term Deflections - Slope Inclinometer SD129

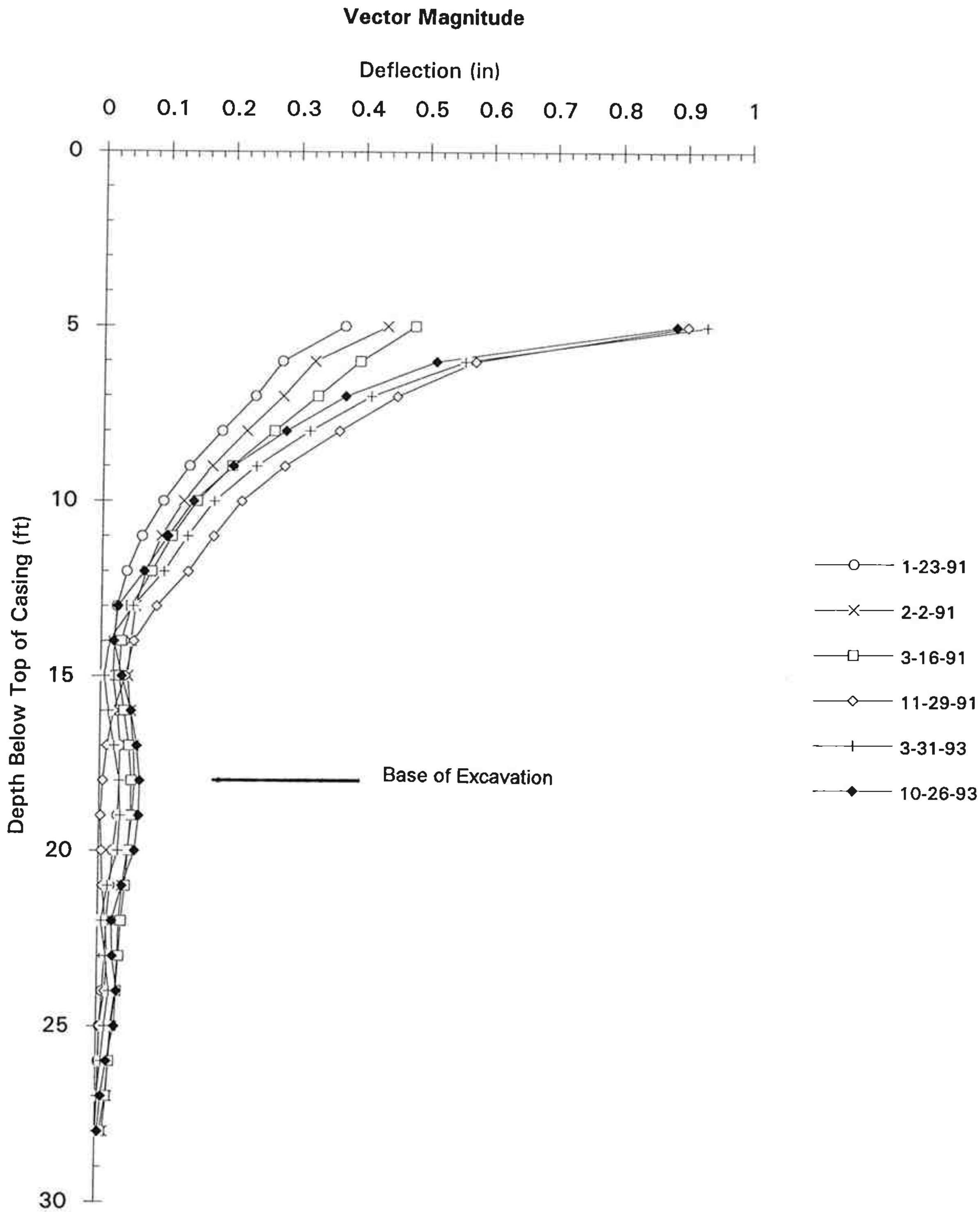


Fig. 148 - Long Term Deflections - Slope Inclinometer SD130

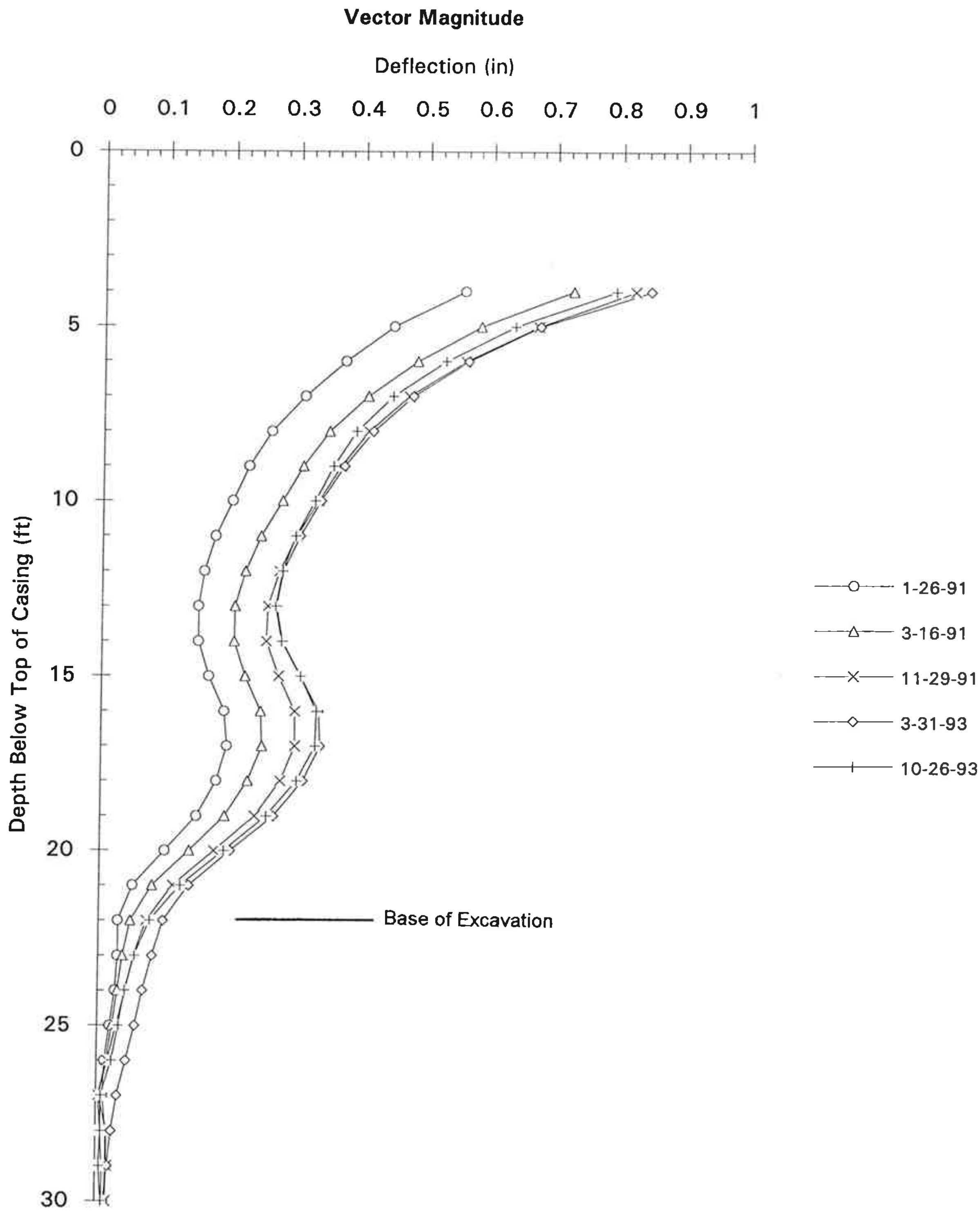


Fig. 149 - Long Term Deflections - Slope Inclinometer SD131

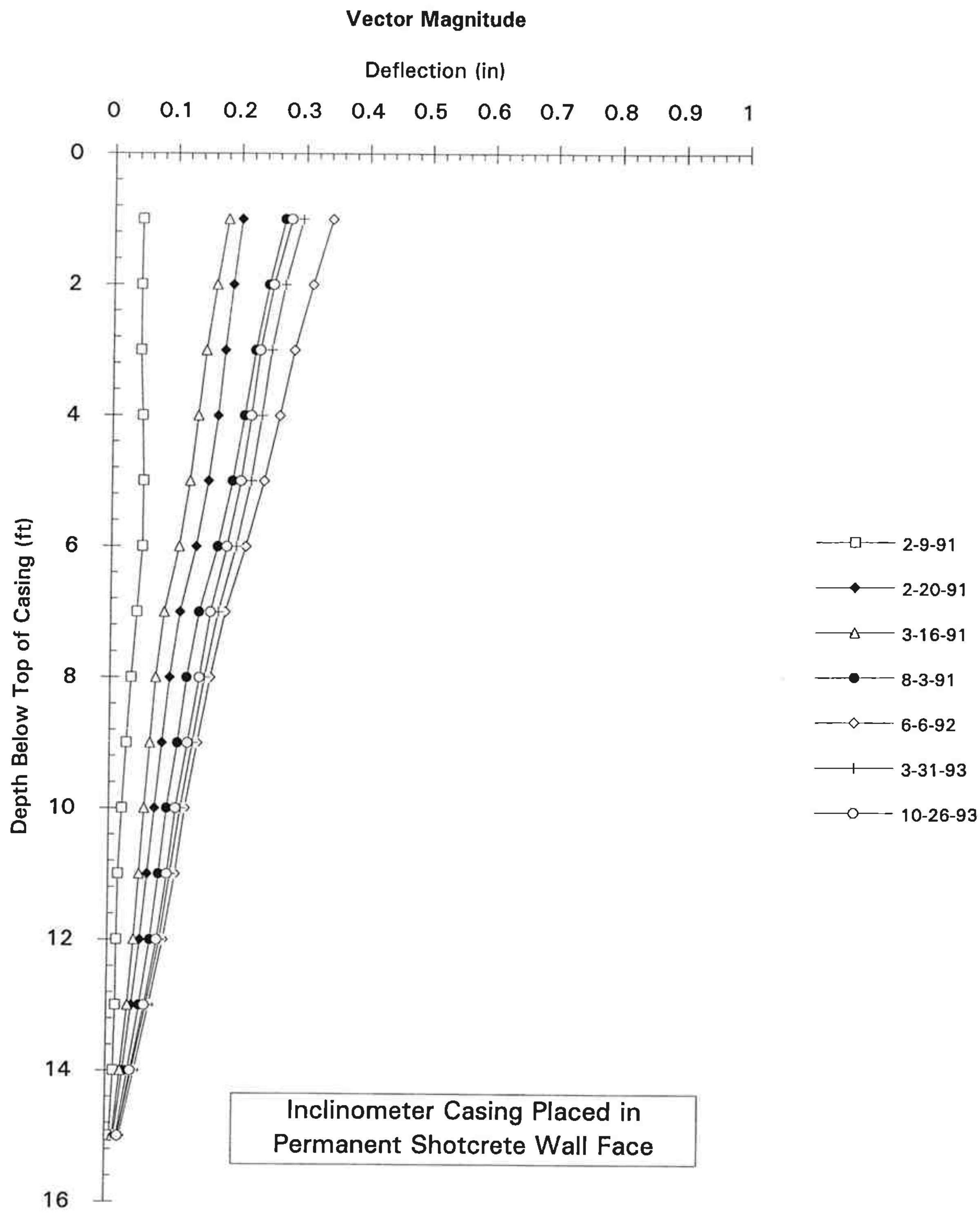


Fig. 150 - Long Term Deflections - Slope Inclinometer SD132

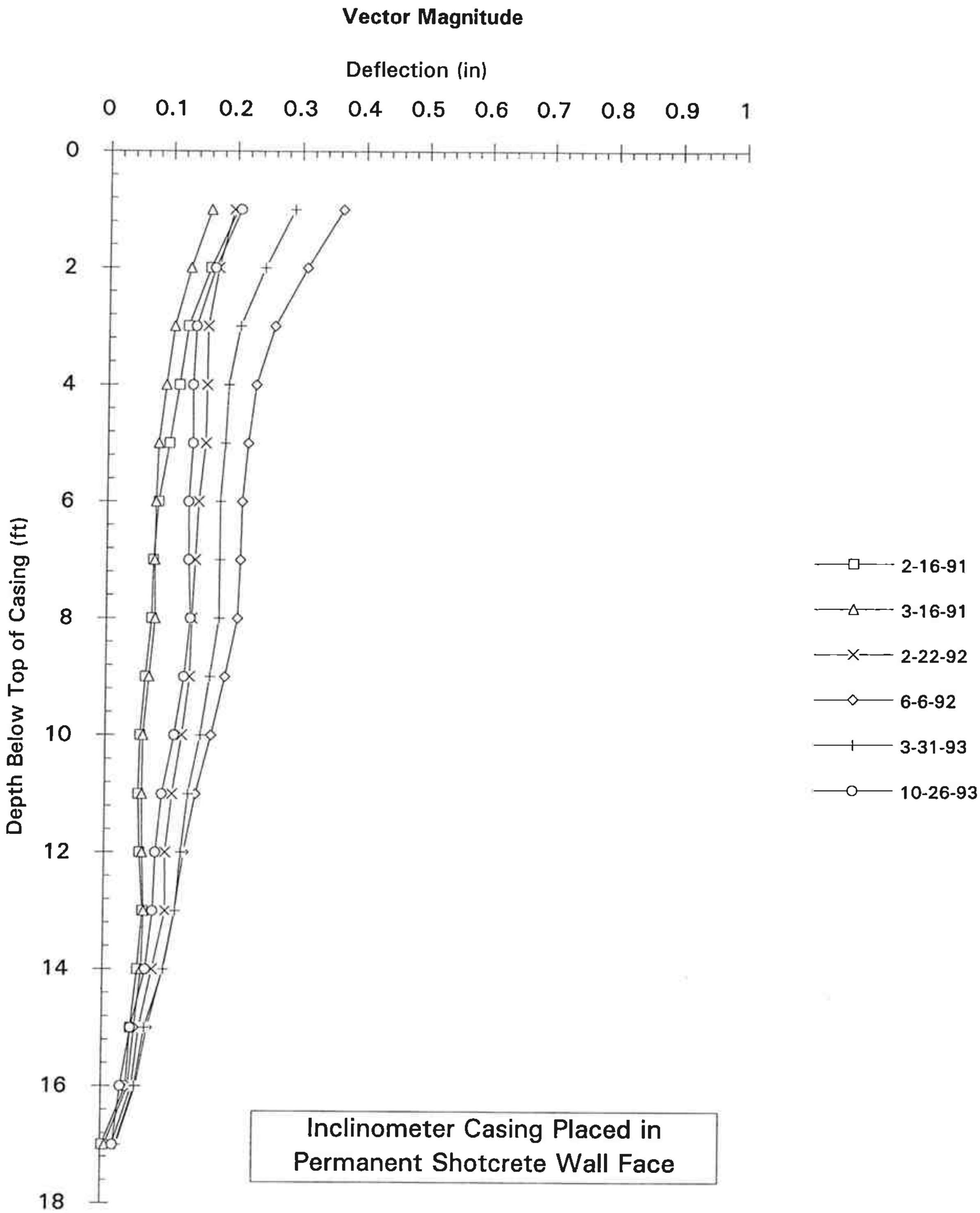


Fig. 151 - Maximum Measured vs. Theoretical (design) Nail Loads at Section 1

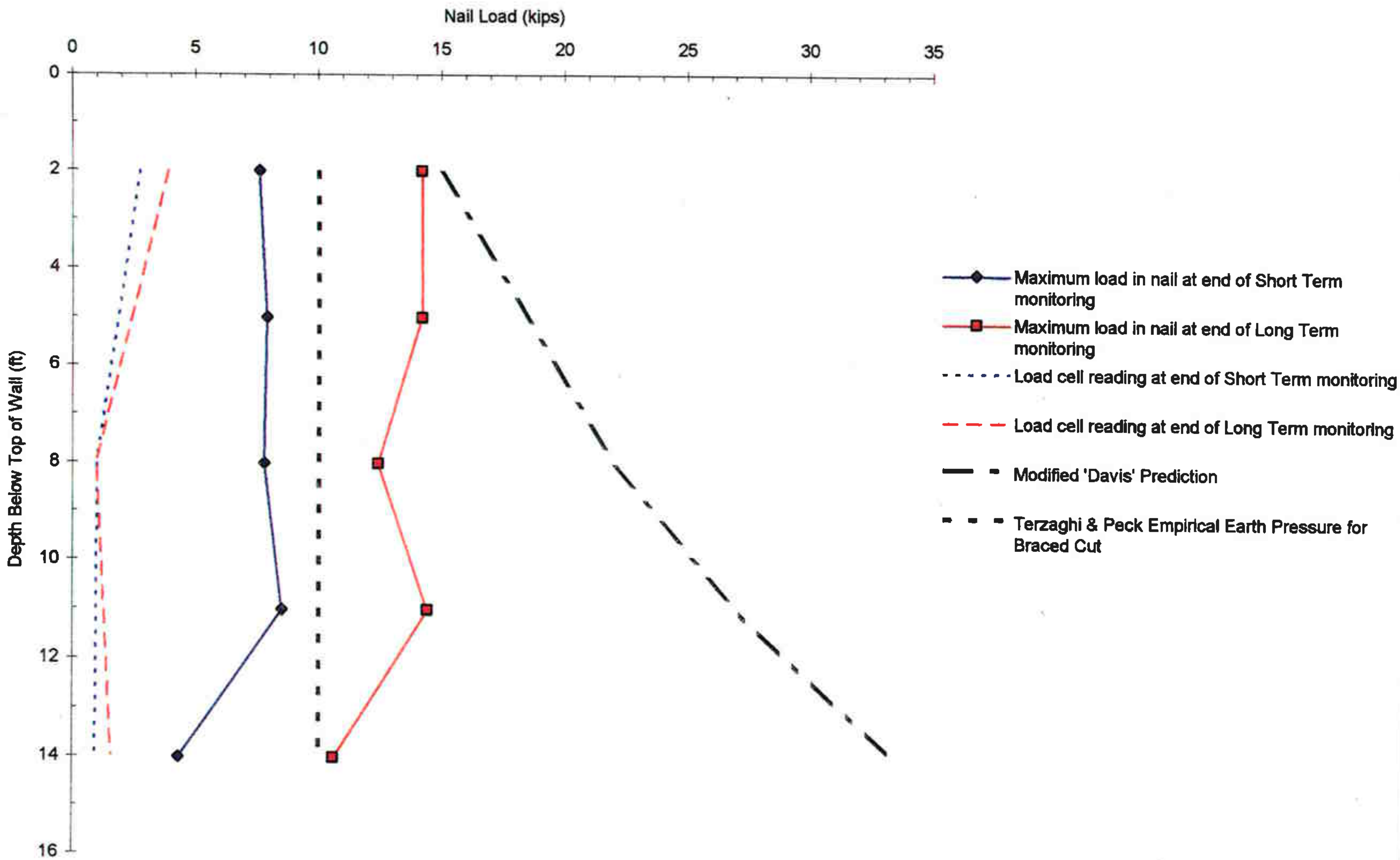
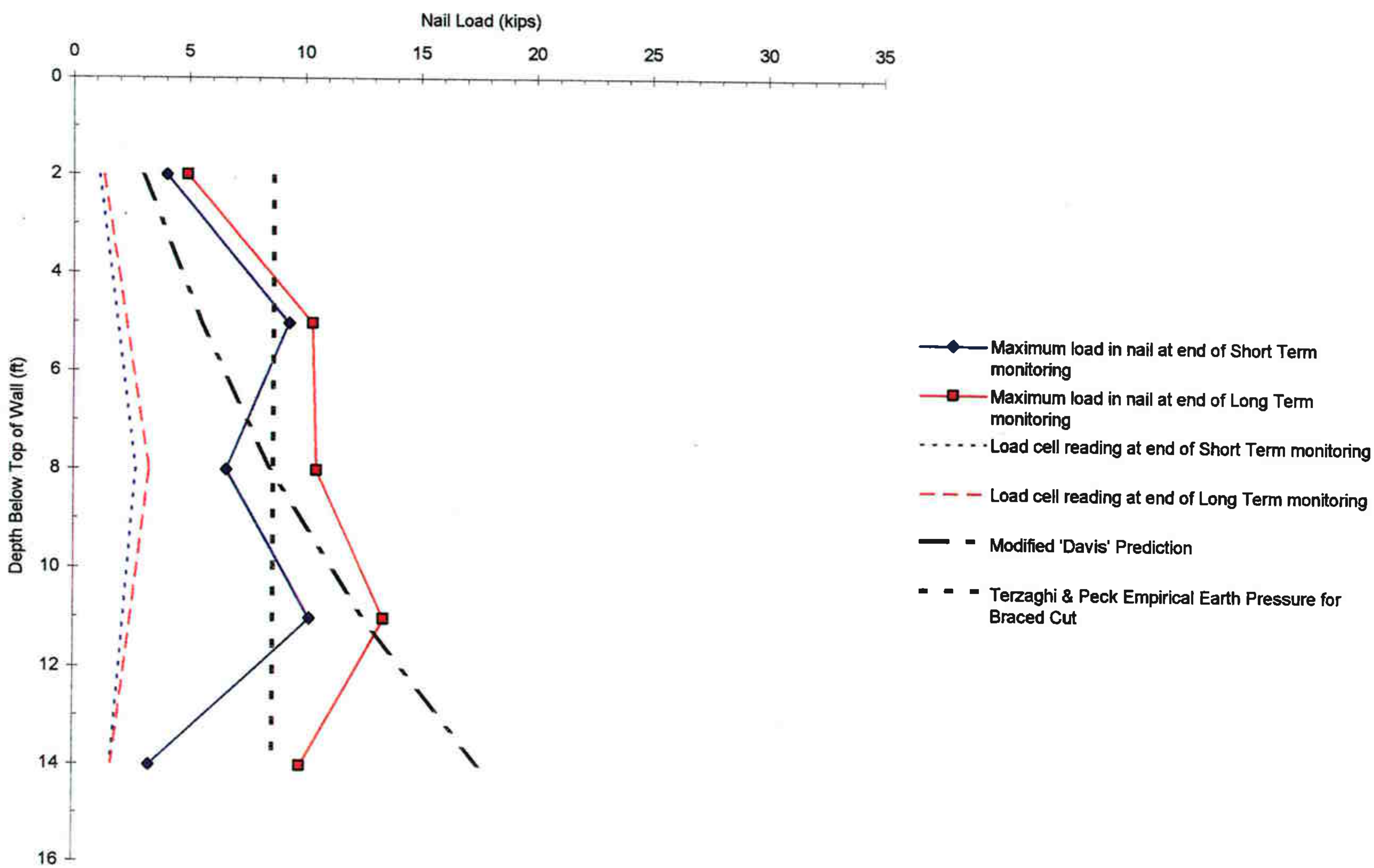


Fig. 152 - Maximum Measured vs. Theoretical (design) Nail Loads at Section 2



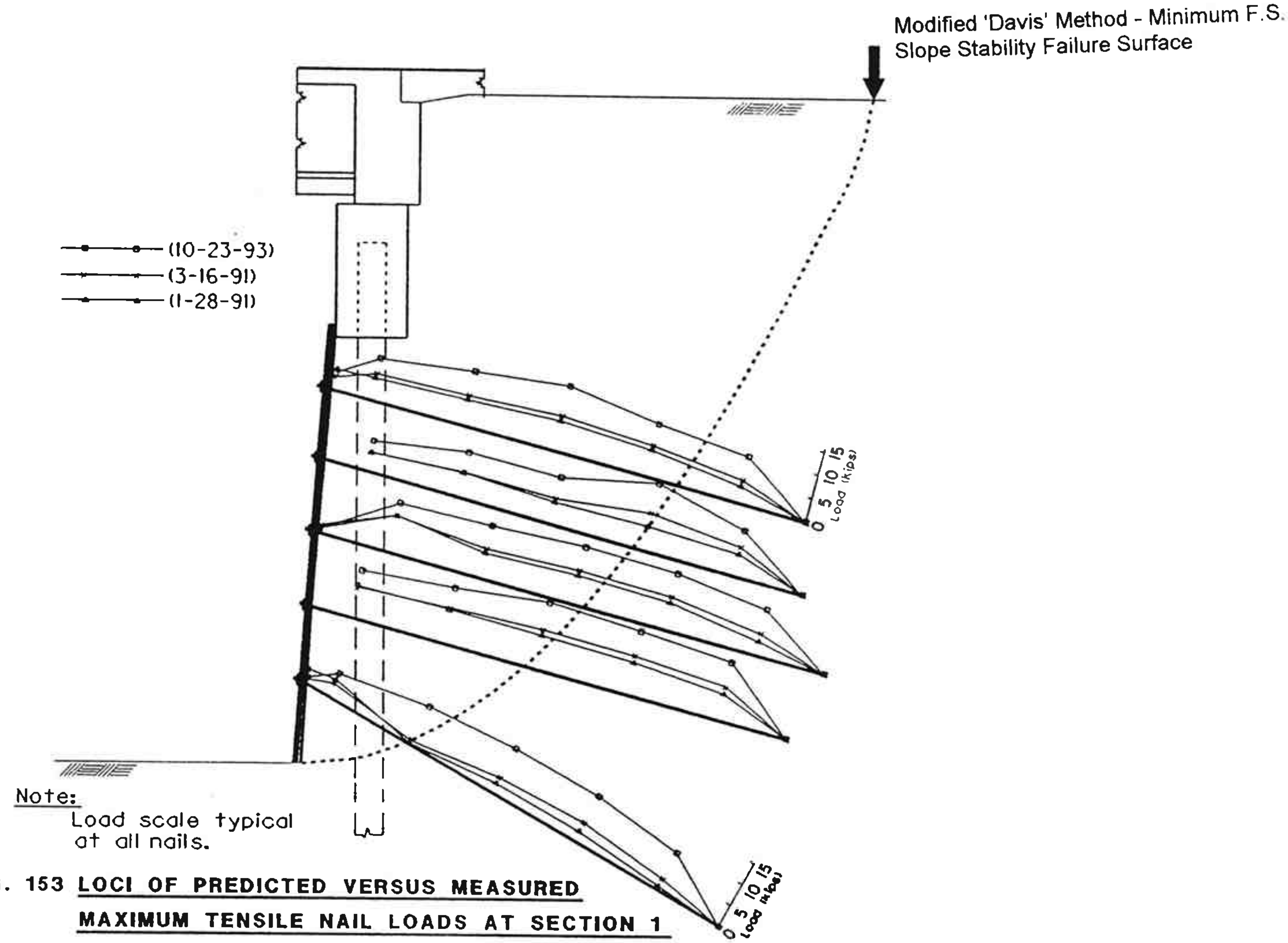


FIG. 153 <u>LOCI OF PREDICTED VERSUS MEASURED</u>

<u>MAXIMUM TENSILE NAIL LOADS AT SECTION 1</u>

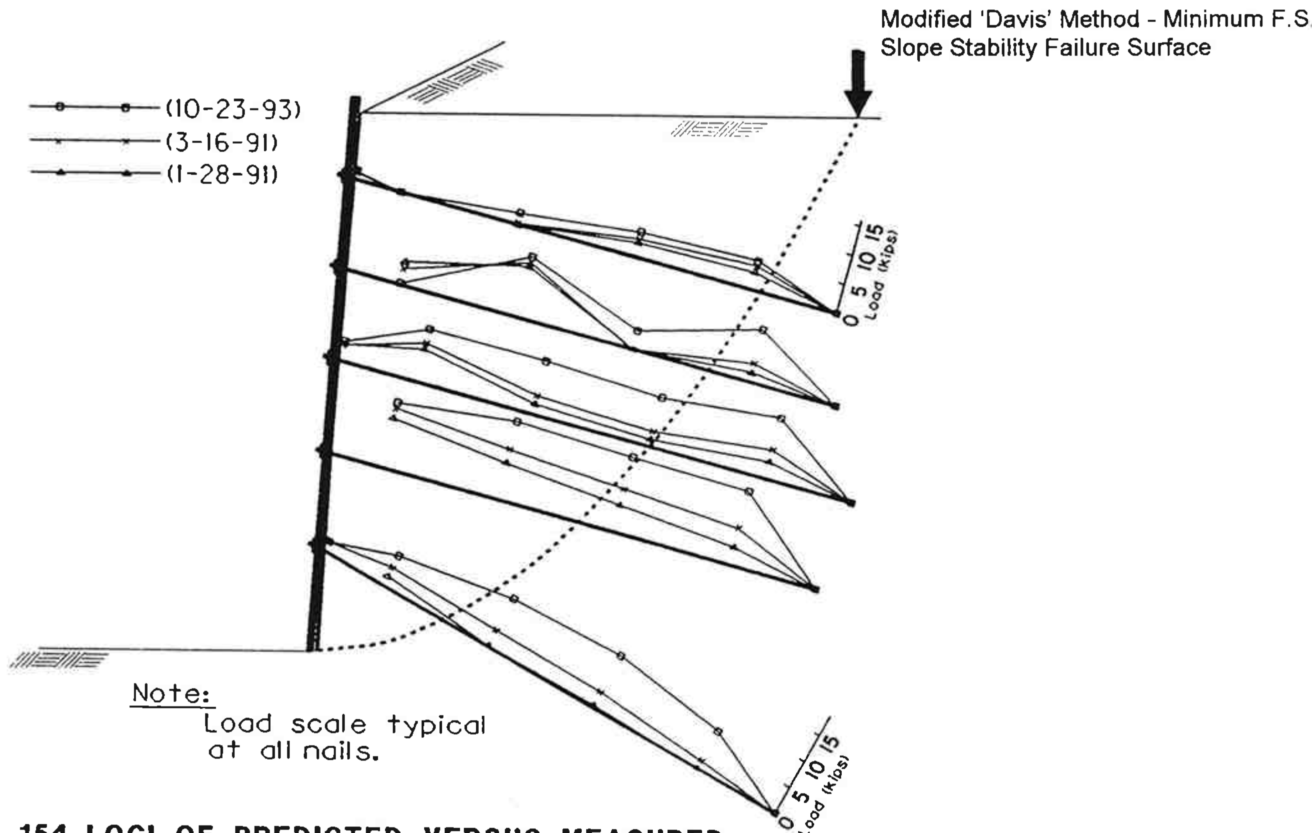
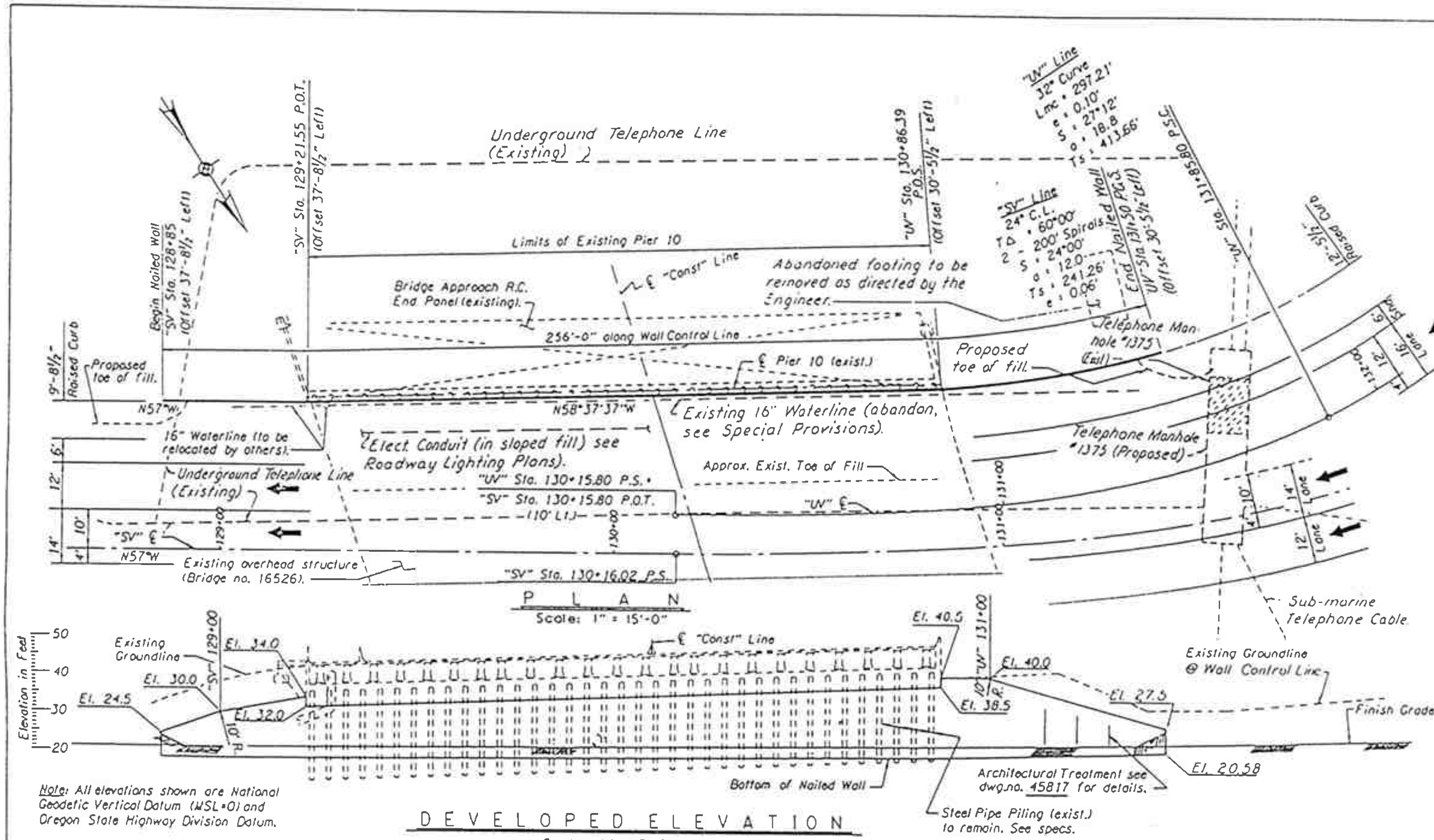
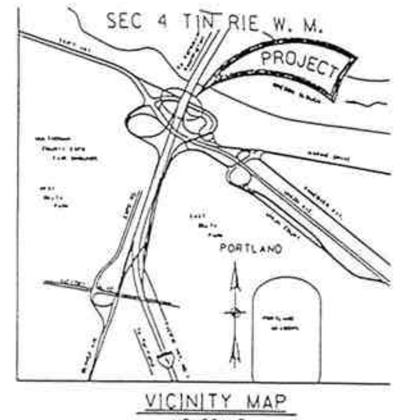


FIG. 154 LOCI OF PREDICTED VERSUS MEASURED MAXIMUM TENSILE NAIL LOADS AT SECTION 2

APPENDIX B: SWIFT-DELTA SOIL NAIL WALL CONTRACT PLANS AND GENERAL SPECIFICATIONS





VICINITY MAP
NO SCALE

GENERAL NOTES:

All materials and workmanship shall conform to be the Standard Specifications for Highway Construction of the Oregon State Department of Transportation, Highway Division.

Concrete facing designed by Load Factor Design Method.

Concrete for Shotcrete Facing shall be Class 4000 - ½" (see Special Provisions).

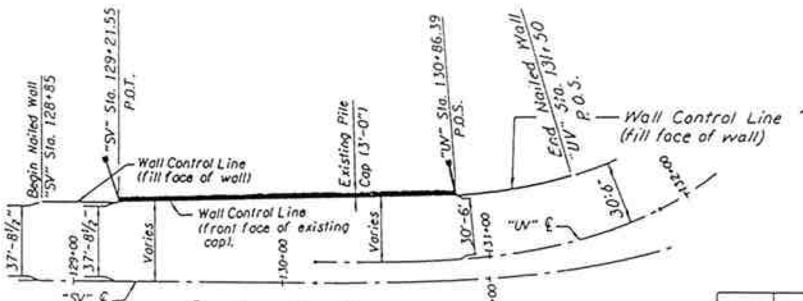
Anchor grout shall be capable of reaching a strength of 3500 p.s.i. in seven days (see Special Provisions).

All steel material shall conform to the following Specifications:
Epoxy Coated Soil Nails: ASTM A615, Grade 60
(Coat only non-threaded portion of the nail) the Exposed threaded portion of the nail, after tightening, shall be epoxy painted, as per Special Provisions.

Welded Wire Fabric: ASTM A497 or A185
Bearing Plates: ASTM A36
Nuts: ASTM A563 Grade B, Hexagonal
Washers: ASTM F436

All welded wire fabric, bearing plates, nuts, washers and threaded portion of the nail shall be hot-dip galvanized per specification after fabrication.

Soil Nails shall be installed to the minimum length shown on dwg. no. 45816.



PLAN
Scale: 1" = 30'-0"

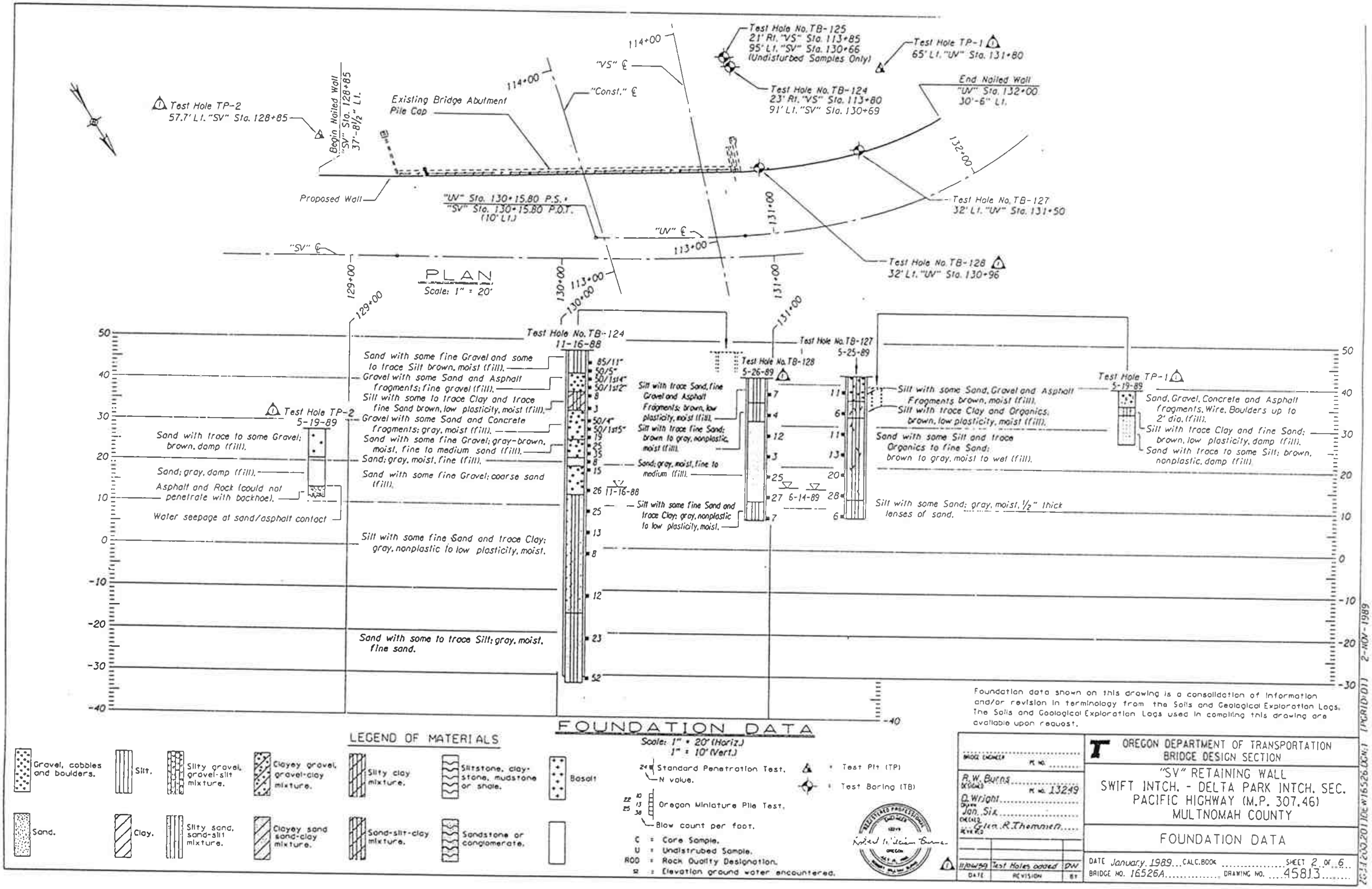
DATE	REVISION	BY

APPROVED:
BRIDGE ENGINEER
DESIGNED: CLAUDE SAAR
DRAWN: TOM HERNANDEZ
CHECKED: R. W. Bonn
REVIEWED: K.A. Kennedy 7-7-89
REVIEWED: J.E. Backstrand

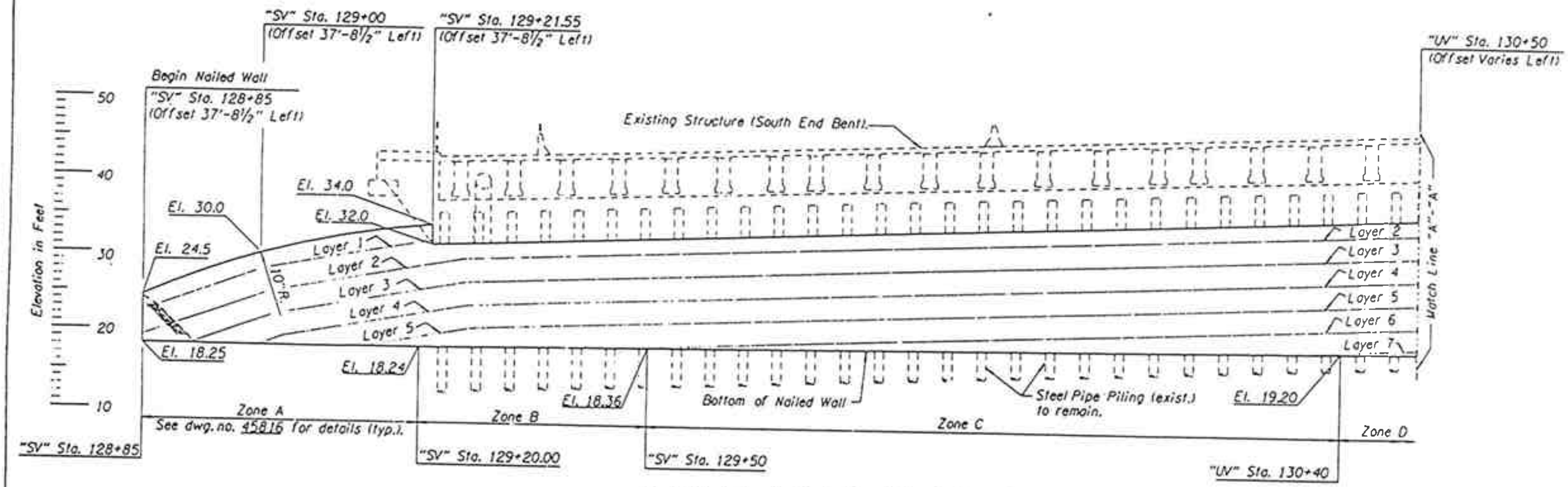
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL
Swift Intchge-Delta Park Intchge Sec.
Pacific Hwy.(MP 307.46) Multnomath Co.
PLAN AND ELEVATION

ACCOMPANIED BY DWGS. 45813 thru 45817
DATE April 1989 CALC.BOOK 2876 SHEET 1 OF 6
BRIDGE NO. 16526A DRAWING NO. 45812



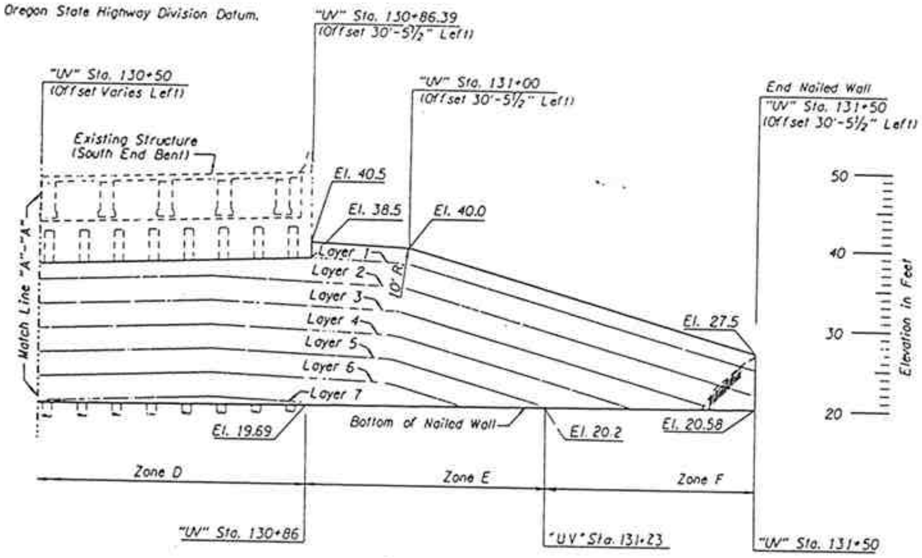
Test Hole TP-2
57.7' Lt. "SV" Sta. 128+85
Begin Nailed Wall
"SV" Sta. 128+85
37'-8 1/2" Lt.
Existing Bridge Abutment
Pile Cap
114+00
"Const." ℄
"VS" ℄
Test Hole No. TB-125
21' Rt. "VS" Sta. 113+85
95' Lt. "SV" Sta. 130+66
(Undisturbed Samples Only)
Test Hole No. TB-124
23' Rt. "VS" Sta. 113+80
91' Lt. "SV" Sta. 130+69
Test Hole TP-1
65' Lt. "UV" Sta. 131+80
End Nailed Wall
"UV" Sta. 132+00
30'-6" Lt.
132+00
Proposed Wall
"UV" Sta. 130+15.80 P.S. =
"SV" Sta. 130+15.80 P.O.T.
(10' Lt.)
"UV" ℄
113+00
131+00
Test Hole No. TB-127
32' Lt. "UV" Sta. 131+50
Test Hole No. TB-128
32' Lt. "UV" Sta. 130+96
"SV" ℄
129+00
130+00
PLAN
Scale: 1" = 20'
Test Hole No. TB-124
11-16-88
Sand with some fine Gravel and some to trace Silt brown, moist (fill).
Gravel with some Sand and Asphalt fragments; fine gravel (fill).
Silt with some to trace Clay and trace fine Sand brown, low plasticity, moist (fill).
Gravel with some Sand and Concrete fragments; gray, moist (fill).
Sand with some fine Gravel; gray-brown, moist, fine to medium sand (fill).
Sand; gray, moist, fine (fill).
Sand with some fine Gravel; coarse sand (fill).
Silt with some fine Sand and trace Clay; gray, nonplastic to low plasticity, moist.
Sand with some to trace Silt; gray, moist, fine sand.
85/11" 50/5" 50/1st4" 50/1st2" 8 3 50/4" 50/1st5" 19 25 35 8 15 26 25 13 8 12 23 52
11-16-88
Test Hole TP-2
5-19-89
Sand with trace to some Gravel; brown, damp (fill).
Sand; gray, damp (fill).
Asphalt and Rock (could not penetrate with backhoe).
Water seepage at sand/asphalt contact
Test Hole No. TB-128
5-26-89
Silt with trace Sand, fine Gravel and Asphalt Fragments; brown, low plasticity, moist (fill).
Silt with trace fine Sand; brown to gray, nonplastic, moist (fill).
Sand; gray, moist, fine to medium (fill).
Silt with some fine Sand and trace Clay; gray, nonplastic to low plasticity, moist.
7 4 12 3 25 27 7
6-14-89
Test Hole No. TB-127
5-25-89
11 6 11 13 20 28 6
Silt with some Sand, Gravel and Asphalt Fragments brown, moist (fill).
Silt with trace Clay and Organics; brown, low plasticity, moist (fill).
Sand with some Silt and trace Organics to fine Sand; brown to gray, moist to wet (fill).
Silt with some Sand; gray, moist, 1/2" thick lenses of sand.
Test Hole TP-1
5-19-89
Sand, Gravel, Concrete and Asphalt fragments, Wire, Boulders up to 2' dia. (fill).
Silt with trace Clay and fine Sand; brown, low plasticity, damp (fill).
Sand with trace to some Silt; brown, nonplastic, damp (fill).
50 40 30 20 10 0 -10 -20 -30 -40
FOUNDATION DATA
Scale: 1" = 20' (Horiz.)
1" = 10' (Vert.)
LEGEND OF MATERIALS
Gravel, cobbles and boulders.
Silt.
Silty gravel, gravel-silt mixture.
Clayey gravel, gravel-clay mixture.
Silty clay mixture.
Siltstone, claystone, mudstone or shale.
Basalt
Sand.
Clay.
Silty sand, sand-silt mixture.
Clayey sand sand-clay mixture.
Sand-silt-clay mixture.
Sandstone or conglomerate.
24 Standard Penetration Test. N value.
Oregon Miniature Pile Test.
Blow count per foot.
C = Core Sample.
U = Undistrubed Sample.
RQD = Rock Quality Designation.
= Elevation ground water encountered.
= Test Pit (TP)
= Test Boring (TB)
Foundation data shown on this drawing is a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Geological Exploration Logs used in compiling this drawing are available upon request.
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION
"SV" RETAINING WALL
SWIFT INTCH. - DELTA PARK INTCH. SEC.
PACIFIC HIGHWAY (M.P. 307.46)
MULTNOMAH COUNTY
FOUNDATION DATA
BRIDGE ENGINEER
R. W. Burns
P.E. NO. 13249
D. Wright
Jon. Six
Glen R. Thommen
11/04/89 Test Holes added DW
DATE REVISION BY
DATE January 1989 CALC. BOOK
SHEET 2 OF 6
BRIDGE NO. 16526A
DRAWING NO. 45813
2-NOV-1989



DEVELOPED ELEVATION

Scale: 1/8" = 1'-0"

Note: All elevations shown are National Geodetic Vertical Datum (MSL=0) and Oregon State Highway Division Datum.



DEVELOPED ELEVATION

Scale: 1/8" = 1'-0"

See dwg. no. 45817 for wall control line.

DATE	REVISION	BY

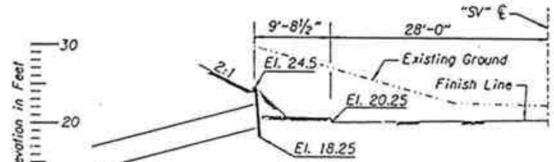
APPROVED: Bridge Engineer, PE No. 8891
Designed, PE No. 11112
TOM HERNANDEZ
Drawn: R.W. ...
Checked: K.A. ... 7-3-89
Reviewed, PE No. ...
Reviewed: J.E. Backstrand

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

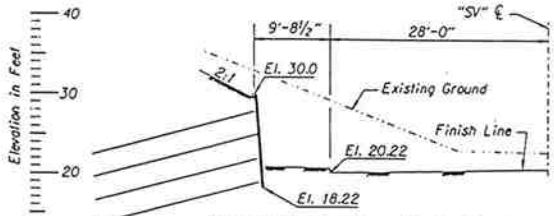
"SV" RETAINING WALL

NAILED WALL DETAILS

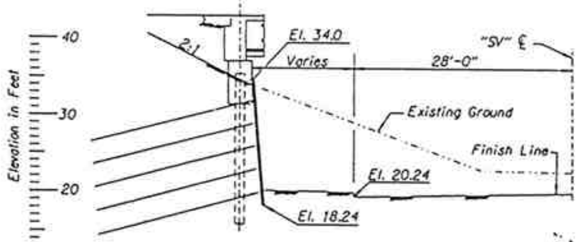
DATE April 1989 CALC.BOOK 2816 SHEET 3 OF 6
BRIDGE NO. 16526A DRAWING NO. 45814



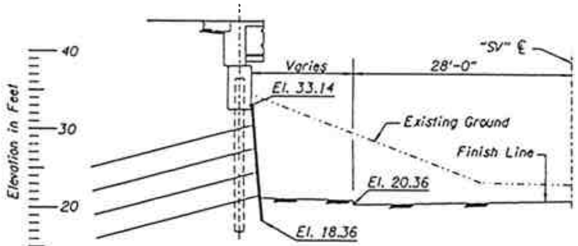
BEGIN NAILED SOIL WALL
(Sta. "SV" 128+85)
No Scale



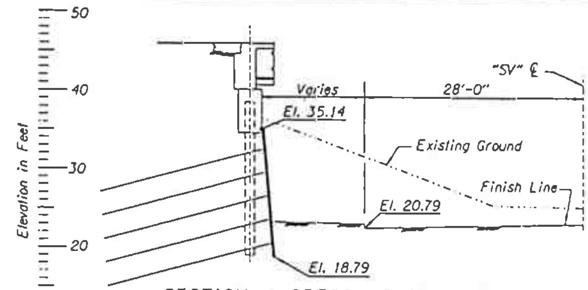
SECTION @ STA. "SV" 129+00
No Scale



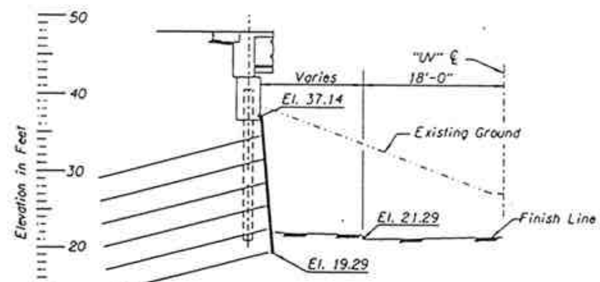
SECTION @ OREGON SLOUGH BRIDGE
(Sta. "SV" 129+21.55)
No Scale



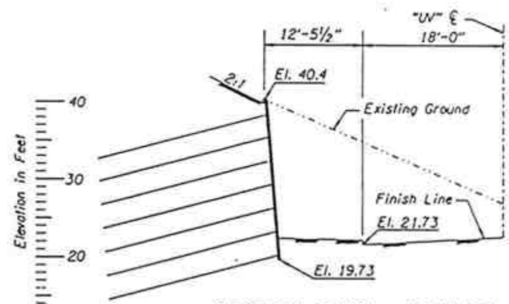
SECTION @ OREGON SLOUGH BRIDGE
(Sta. "SV" 129+50)
No Scale



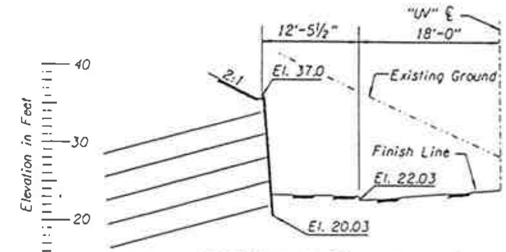
SECTION @ OREGON SLOUGH BRIDGE
(Sta. "SV" 130+00)
No Scale



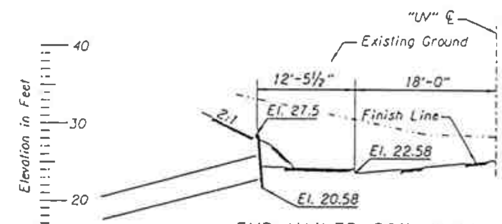
SECTION @ OREGON SLOUGH BRIDGE
(Sta. "UV" 130+50)
No Scale



SECTION @ STA. "UV" 130+90
No Scale



SECTION @ STA. "UV" 131+12
No Scale



END NAILED SOIL WALL
(Sta. "UV" 131+50)
No Scale

DATE	REVISION	BY

APPROVED: Thomas D. Lulay BRIDGE ENGINEER PE NO. 8491
DESIGNED [signature] PE NO. 11312
TOM HERNANDEZ
DRAWN R. W. [signature]
CHECKED / REVIEWED K. A. Konradi 7-3-89 PE NO. —
J. E. Brickstrand REVIEWED

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

TYPICAL WALL SECTIONS

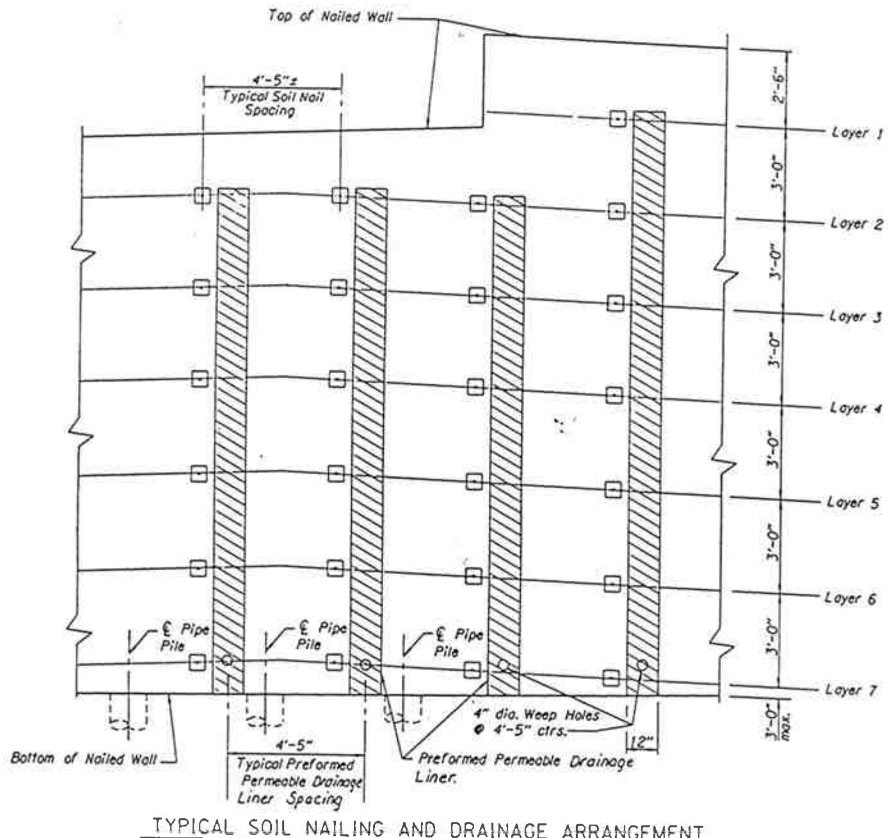
DATE April 1989 CALC. BOOK 2876 SHEET 4 OF 6
BRIDGE NO. 16526A DRAWING NO. 45815

LAYER NO.	PULL-OUT RESISTANCE AND UNGROUTED TEST LENGTH											
	PULL-OUT RESISTANCE (KIPS)						UNGROUTED TEST LENGTH (FT.)					
	ZONE											
	A	B	C	D	E	F	A	B	C	D	E	F
1	3	-	-	-	2.5	2	12	-	-	-	16.5	11.5
2	6	20	15	10	5	5	10	11.5	15	16	15	9.5
3	9	23.5	18.5	13	8	8.5	8.5	9.5	13	14	13	7
4	12	27	22	16.5	11	12.5	6	7.5	11	12	11	4
5	17	32	27	21.5	14.5	—	2	4.5	8	9	8.5	—
6	-	-	33.5	26.5	20	-	-	-	4	6	5	-
7	-	-	-	34.5	23.5	-	-	-	-	1	2.5	-

Note:
Pullout resistance to be developed within effective length.

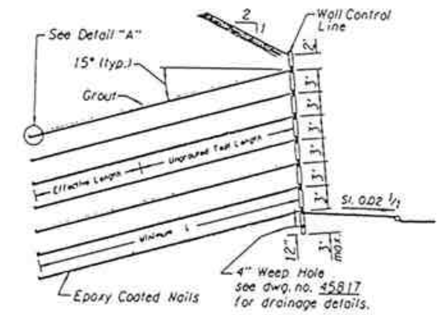
BAR SIZE (BY ZONE)					
A	B	C	D	E	F
#8	#9	#9	#9	#8	#8

MINIMUM "L" LENGTH (BY ZONE)					
A	B	C	D	E	F
14'	23'	24'	22'	18'	13'



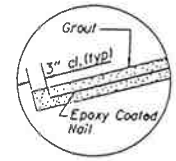
TYPICAL SOIL NAILING AND DRAINAGE ARRANGEMENT

Scale: 1/2" = 1'-0"



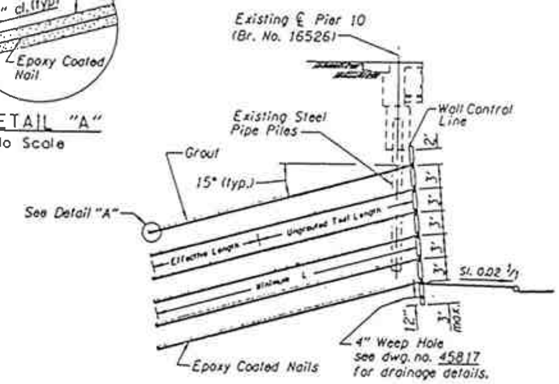
TYPICAL SECTION AWAY FROM BRIDGE

Scale: 1/8" = 1'-0"



DETAIL "A"

No Scale



TYPICAL SECTION @ OREGON SLOUGH BRIDGE

Scale: 1/8" = 1'-0"

DATE	REVISION	BY

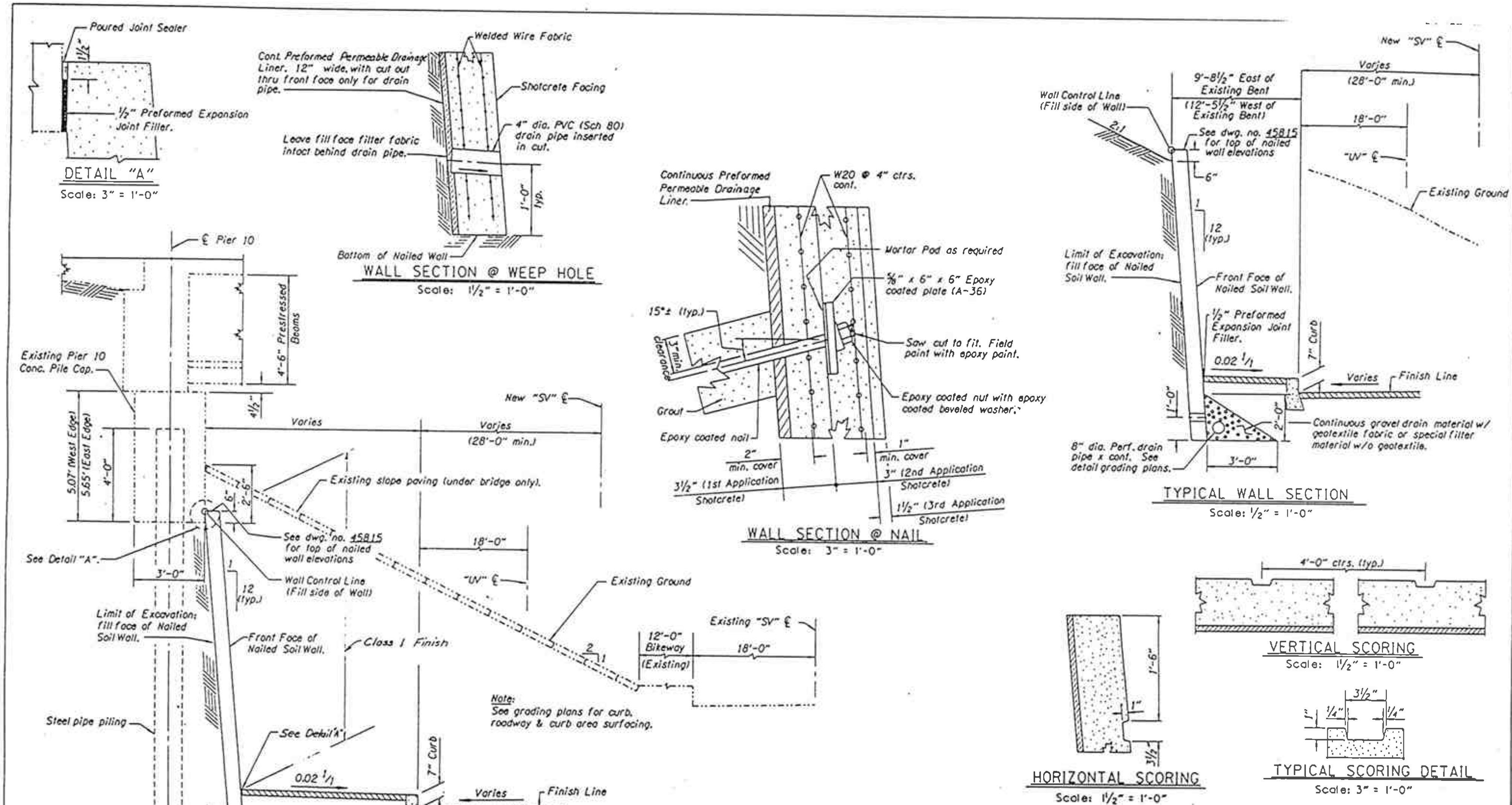
APPROVED: Thomas D. Linley — BRIDGE ENGINEER — P.E. NO. 8491
DESIGNED: P.E. NO. 13712
TOM HERNANDEZ — DRAWN
R. W. — CHECKED REVIEWED
K. A. Kornblit 7-3-87 — P.E. NO.
J. E. Burkhardt — REVIEWED

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

SOIL NAIL DETAILS

DATE April 1989 CALC. BOOK 2876 SHEET 5 OF 6
BRIDGE NO. 16526A DRAWING NO. 45816



APPROVED:	
BRIDGE ENGINEER	PE NO. 6191
DESIGNED	PE NO. 13112
DRAWN	TOM HERNANDEZ
CHECKED	R. W. Burns
REVIEWED	K. L. Kenmotsu 7-3-87
REVIEWED	J. E. Backstrand

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

TYPICAL WALL DETAILS

DATE April 1989 CALC.BOOK 2876 SHEET 6 OF 6
BRIDGE NO. 16526A DRAWING NO. 45817

DATE	REVISION	BY

SECTION 614 - NAILED SOIL RETAINING WALLS

614.01 Scope - This work shall consist of constructing soil nail walls in accordance with the Standard Specifications, these special provisions and in reasonably close conformity with the lines, grades and dimensions shown on the plans or established by the Engineer.

(a) Soil nailing - Soil nailing shall consist of staged excavation of the existing south end slope from the top down to the layer limits shown in the plans, placing preformed permeable drainage fabric and welded wire fabric, applying air blown structural shotcrete, drilling holes at the inclination shown in the plans, and placing and grouting steel bars (soil nails).

The Contractor shall select the nail installation method, the maximum hole diameter, and the grouting method. The Contractor shall install nails that meet the design requirements shown on the plans and the testing requirements specified herein. Once the Contractor selects a nail installation method, the Contractor shall not change the system without written approval of the Engineer.

(b) Shotcreting - This work shall consist of constructing a pneumatically applied shotcrete blanket on soil surfaces at locations shown in the plans or as directed by the Engineer. These specifications refer to premixed cement and aggregate pneumatically applied by suitable equipment and competent operators.

614.02 Prequalification of Contractor and Contractor's Personnel - The Contractor performing the work described in this special provision shall have at least 5 projects successfully completed in the last 3 years involving construction of earth reinforced walls using permanent soil nails. The Contractor's personnel shall meet the following requirements:

(a) Supervising engineer - The Contractor shall assign an engineer to supervise the work with at least 3 years of experience in the design and construction of permanently nailed structures.

(b) "On-site" supervisors - "On-site" supervisors shall have a minimum of 1 year of experience installing permanent soil nails with the approved Contractor.

(c) Drill operators - Drill operators shall have a minimum of 1 year of experience installing permanent soil nails with the approved Contractor.

(d) Foremen - The foremen shall have performed satisfactory work in similar capacities elsewhere for a sufficient length of time, as determined by the Engineer, to be fully qualified to perform their duties. Foremen shall have at least 2 years of experience as a structural shotcrete nozzleman.

(e) Nozzlemen - Nozzlemen shall have served at least 1 year of apprenticeship on similar applications as determined by the Engineer and with the same type of equipment. Prior to the start of shotcreting on this project, the nozzlemen shall in the presence of the Engineer, demonstrate their ability to apply shotcrete of the required quality on 2 test panels. Two satisfactory test panels, described under 614.33, shot in a vertical position for each mix used during the course of the work shall be the minimum qualification test for nozzlemen before they will be permitted to place shotcrete in permanent construction.

(f) Delivery equipment operators - The delivery equipment operators shall have performed satisfactory work in similar capacities elsewhere for a sufficient length of time, as determined by the Engineer, to be fully qualified to perform their duties.

The Contractor shall not use consultants or manufacturer's representatives in order to meet the requirements of this subsection.

614.03 Submittals - No later than the preconstruction conference, the Contractor shall submit, in writing, resumes documenting that the Contractor performing the work described in this Section and the Contractor's personnel, have the required experience as set forth in 614.02. For the Contractor performing the work, a brief description of each project and a reference shall be included for each project listed. As a minimum, the reference shall include an individual's name and current phone number. For the Contractor's personnel, the list shall contain a summary of each individual's experience and it shall be complete enough for the Engineer to determine whether or not each individual has satisfied the qualifications of 614.02.

The Engineer will approve or reject the Contractor's qualifications and staff within 15 working days after receipt of the submission. Work shall not be started on the nailed soil wall nor materials ordered until approval of the Contractor's qualifications are given. The Engineer may suspend the soil nailing work if the Contractor substitutes unqualified personnel for approved personnel during construction. If work is suspended due to the substitution of unqualified personnel, the Contractor shall be fully liable for additional costs resulting from the suspension of work and no adjustment in contract time resulting from the suspension of work will be allowed.

The Contractor shall submit, in writing, to the Engineer not less than 15 working days prior to start of wall excavation, the proposed schedule and detailed construction sequence; proposed method of excavation; proposed drilling methods and equipment; proposed hole diameter; grout and shotcrete mix designs; and nail steel corrosion protection details.

The shotcrete mix design shall be prepared, tested, and submitted for approval by the Engineer. The results of compatibility testing done in accordance with ACI 506.2 shall also accompany this submission to verify that any proposed admixtures to accelerate set are compatible with the cement to be used.

The Contractor shall submit certified mill test results and typical stress-strain curves along with samples from each heat, properly marked, for the nail steel to the Engineer for approval. The typical stress-strain curve shall be obtained by approved standard practices. The guaranteed ultimate strength, yield strength, elongation and composition shall be certified.

The Contractor shall submit the procedures for placing the grout to the Engineer for approval.

The Contractor shall submit detailed plans, as specified in 614.41, for the method proposed to be followed for the permanent soil nail testing to the Engineer for approval prior to the tests. This shall include all necessary drawings and details to clearly describe the methods proposed.

The Contractor shall submit to the Engineer for review and approval calibration data for each load cell, test jack, pressure gage, stroke counter on the grout pump, and master gage to be used. The calibration tests shall have been performed within 60 calendar days of the data submitted. Testing or work shall not start until the Engineer has approved the load cell, jack and pressure gage calibrations.

Materials

614.11 Materials:

(a) Reinforcement soil nails - Soil nails shall be epoxy coated for corrosion protection. Epoxy coating shall conform to AASHTO M 284 in accordance with 709.05(d) of these special provisions, found under Section 505. The coating thickness shall not be less than 14 mils or greater than 18 mils. Epoxy coat only nonthreaded portion of the nail. The exposed threaded portion of the nail shall be epoxy painted after the installation and the tightening of the nut according to 709.05 of these special provisions.

Soil nails shall be clean and free of oil, grease and other foreign substances that would destroy or reduce bond.

Soil nails shall be installed using plastic centralizers to keep the nails centered in the hole. Wood shall not be used. Centralizers shall be spaced no further than 10 feet apart. Any other method selected by the Contractor shall be approved, in writing, by the Engineer.

The soil nails shall not be spliced. Soil nails shall be threaded on one end a minimum of 6 inches. Soil nails shall be coarse threaded with a diameter 1/8-inch less than the nominal diameter of the bar.

If the resisting soil nails fail to develop the pullout resistance (kips) specified on the plans and, in the opinion of the Engineer, all work conformed with the best general practices, lengthening of the nails by approved mechanical splicers will be allowed. The splicing shall conform to the provisions in 505.35 "Splicing" of these special provisions. Nail splicers shall develop the ultimate tensile strength of the bars without evidence of failure. All work associated with furnishing, placing, grouting and joining of the spliced length of the soil nail shall be done at the Contractor's expense.

The bearing plate shall be as shown in the plans, conforming to ASTM A 36.

The nuts shall conform to ASTM A 563, Grade B Hexagonal. The nut shall be fitted with a special washer such that the nut will bear uniformly on the plate.

(b) Welded steel wire fabric - Unless shown otherwise in the plans, welded steel wire fabric shall be galvanized meeting the requirements of ASTM A 185. The welded wire fabric shall be clean and free from loose mill scale, rust, oil, or other coatings interfering with bond.

Welded deformed steel wire fabric of equal or greater diameter and yield strength may be substituted for the welded steel wire fabric. Welded deformed steel wire fabric shall conform to the specifications of ASTM A 497.

Fabric shall be overlapped at least 2 mesh dimension at all seams. Tie wires shall be bent flat in the plane of the fabric and shall not form large knots.

(c) Grout - The grout to be used for soil nailing shall consist of a pumpable mixture of Types I, II, or III portland cement, sand and water. Chemical additives shall not be allowed.

Cement should be fresh and should not contain any lumps or other indications of hydration. Water for mixing grout should be potable.

The grout shall be capable of reaching a cube strength of 3500 psi in 7 days as per AASHTO T 106.

(d) Shotcrete - Shotcrete shall be composed of portland cement, fine and coarse aggregate, and water. Wet-mix shotcrete shall be used. The shotcrete shall be reinforced with welded wire fabric.

Shotcrete shall comply with the current requirements of the American Concrete Institute's ACI 506R, "Guide to Shotcrete", and ACI 506.2, "Specifications for Materials, Proportioning, and

Application of Shotcrete". The ACI specifications and recommendations are hereby made a part of this Section as specification requirements except as modified herein.

Materials in the shotcrete shall conform to the following requirements of the Standard Specifications modified and/or supplemented as follows:

Portland Cement (Type I, II or III)	701.01
Air-Entraining and other Chemical Admixtures	701.03
Curing Materials	701.05
Water	701.02
Fine Aggregate	703.01(d)
Coarse Aggregate	703.02(d)

(d-1) Prepackaged product - Premixed and prepackaged concrete product specifically manufactured as a shotcrete product may be provided for "on-site" mixed shotcrete if approved by the Engineer. The packages shall contain cement and aggregate conforming to the materials portion of this specification.

(d-2) Admixtures - Admixtures shall not be used without permission of the Engineer. If admixtures are used to entrain air, reduce water-cement ratio, retard or accelerate setting time or accelerate the development of strength, they shall be used at the rate specified by the manufacturer and must be compatible with the cement used. Use of calcium chloride accelerating agent will not be permitted. When used, admixtures shall be dissolved in water before introduction into the mixture. Wet-mix shotcrete shall have 7.5 (plus or minus 1) percent air meeting the requirements of 701.03.

(d-3) Water - In addition to the requirements set forth in 701.02, the water used in the shotcrete mix shall be free of elements which cause staining.

The Contractor shall be responsible for the design of shotcrete mixes and for the quality of shotcrete placed in the work.

(e) Aggregate - Aggregate used in shotcrete shall have a combined gradation of fine and coarse aggregates meeting the following gradation requirements:

Sieve Size	Percent Passing by Weight
1/2"	100
3/8"	90-100
No. 4	70-85
No. 8	50-70
No. 16	35-55
No. 30	20-35
No. 50	8-20
No. 100	2-10

(f) Preformed permeable drainage liner - The preformed permeable liner shall consist of 12-inch wide MIRADRAIN 6000, AMERDRAIN 200, or approved equal, fully wrapped with filter fabric.

Should the fabric on the preformed liner be torn or punctured, the damaged section shall be replaced completely or repaired by placing a piece of fabric that is large enough to cover the damaged area and is at least 6 inches on each side of the damaged area.

Construction

614.31 Construction Sequence:

(a) General - As-built plans for the existing pier 10 are available in the Project Manager's office. The Contractor shall verify the actual location of the pipe piles in the field prior to any drilling operations. Any damage to the existing piles shall be remedied to the Engineer's satisfaction at the Contractor's expense.

(b) Excavation - Excavation for the nailed soil wall shall conform to the provisions in Section 251 and this subsection.

The excavation shall proceed from the top down in a horizontal lift sequence with the ground level excavated no more than 18 inches below the level of the next uninstalled row of nails. Only the amount of excavation that can be covered with shotcrete and nailed during a work shift shall be performed.

Each stage of excavation shall have all preformed permeable drainage liners, all soil nails and appurtenances installed, the required number of production nails tested and a 6.5-inch shotcrete cover placed over the excavation before excavation of the next lift is to begin. After a lift is excavated, the cut surface shall be trimmed to line and grade to provide adequate support and to assure the design thickness of the shotcrete. The cut surface shall be cleaned of all loose material, mud, rebound and other foreign matter that could prevent or reduce shotcrete bond.

The tolerance on the soil cut shall be such that overexcavation does not damage overlying shotcrete sections by undermining or other means. Costs associated with additional thickness of shotcrete due to overexcavation or irregularities in the cut face shall be borne by the Contractor.

(c) Drains - After each excavation lift, and before any shotcrete is placed, place the preformed permeable drainage liner against the exposed face at the required spacing shown in the plans. The drainage liner installed after each excavation lift shall be hydraulically connected with the drain installed in the previous lift.

Four-inch weep holes shall be installed at locations and at the spacing shown in the plans. Weep holes shall be protected during shotcrete application to prevent formation of a plug. A continuous drain pipe wrapped with gravel drain material shall be provided as shown in the plans.

(d) Inner welded steel wire fabric layer - After each excavation lift, place the inner welded wire fabric layer providing cutouts and markers at nail locations shown in the plans. The wire fabric shall be attached firmly in proper position to prevent vibration while the shotcrete is being applied. The fabric shall be positioned in such a manner that the fabric is not in physical contact with the nail once the nail is installed.

(e) Initial shotcrete layer - The sequence of wall construction is based on short duration of soil standup. After each excavation lift and the placement of the preformed permeable liner and steel welded wire fabric, place the initial shotcrete lift to the lines and grades shown in the plans. The Engineer may allow an alternate sequence of construction if the Contractor can demonstrate sufficient duration of soil standup is achievable with the construction methods, soil/groundwater and weather conditions to allow nail installation prior to placing initial shotcrete layer.

(f) Nail installation - After placement of the initial shotcrete layer, holes shall be drilled through the initial shotcrete layer. The method used for drilling the holes shall be chosen by the Contractor and approved, in writing, by the Engineer. Subject to the Engineer's approval, the Contractor may place blockouts at nail locations prior to placing the initial shotcrete layer. The location, length and minimum diameter of the holes shall be as shown in the plans. Holes shall be cleaned to remove all material resulting from the drilling operation or any other material that would impair the strength of the nails. Water or other liquids shall not be used to flush cuttings, but air may be used. Subsidence, damage to the shotcrete face or any other detrimental impact from drilling shall be cause for immediate cessation of drilling and repair of all damages at the Engineer's direction and the Contractor's expense.

After drilling, the nail shall be installed in the hole, any casings used to stabilize the hole shall be removed during the grouting operation. Each soil nail shall be secured with a steel plate as shown in the plans conforming to ASTM A 36. Each plate shall be fastened to the soil nail with a nut and shall be secured wrench tight with a minimum 100 ft.-lbs. torque after the initial shotcrete layer has set sufficiently to provide bearing for the plate.

(g) Intermediate shotcrete layer - After installing the nails, placing the plates and tightening the nuts, place the outer welded steel wire fabric layer. The fabric shall be positioned in such a manner that the fabric is not in physical contact with the nail. The wire fabric should be held firmly in proper position while shotcrete is applied. Apply the intermediate layer of shotcrete to the lines and grades shown in the plans.

(h) Subsequent excavation lifts - Further excavation shall not start until the shotcrete on the preceding lift has reached 25 percent of its required 28-day minimum compressive strength and the production nails, in the preceding lifts, tested as specified in 614.41. Each excavation lift shall be completed using the sequence outlined in steps (a) through (g) above.

(i) Final shotcrete layer - The final shotcrete layer shall be placed full height after the wall excavation is completed to grade using the sequence outlined in steps (a) through (h) above. Place the final shotcrete layer a fraction beyond the guide pins and wires. Excess material shall be trimmed to the true lines and grades shown in the plans, the guide pins and wires removed and their impressions covered.

(j) Architectural finish - The exposed shotcrete face shall be given a Class I finish and an architectural treatment as shown on Drawing 45817.

614.32 Nailing Application:

(a) Preproduction testing - The preproduction testing is performed on nails installed with the proposed production drilling and nail installation system.

The purpose of the test is to verify the Contractor's procedures, hole diameter, grouting method and design assumptions.

Preproduction test nails shall be sacrificial and shall not be incorporated in the production nail scheme. Drilling and installation of production nails shall not be permitted unless preproduction testing has been completed and approved by the Engineer, using the same equipment, hole diameter and installation methods proposed for the production nails.

Any changes in the installation or drilling method may require additional preproduction testing as determined by the Engineer and will be done at the Contractor's expense.

The Contractor shall submit detailed plans describing his proposed preproduction nail testing method to the Engineer for approval prior to the tests.

Three successful preproduction tests are required. Test nail locations shall be selected by the Engineer.

The intent is to stress the bond between the grout and the surrounding soil. The soil shall be loaded to a total load equal to the pullout resistance (kips) shown in the plans. This test requires a no load zone (ungrouted test length) and a bond zone (grouted length). The bonded length of the preproduction nail should be equivalent to the effective length of the closest production nail within the control area for that level of wall.

After the effective length is grouted and the grout has gained sufficient strength to withstand the test load, the test nail shall be loaded in increments of 25 percent of the pullout resistance (kips) to a total load equal to the pullout resistance (kips) shown in the plans.

Each load increment shall be held for at least 1 minute except for the final load. The load-hold period shall start as soon as the test load is applied.

The final load shall be held for 10 minutes. Measurement of nail movement with respect to a fixed reference point shall be obtained and recorded at 1 minute, 2, 3, 5, 6, and 10. The preproduction test will be considered successful and concluded if the test nail meets the criteria for a preproduction tested nail in subsection 614.41(a-3a) of these special provisions.

If a final load equal to the pullout resistance (kips) cannot be maintained for 10 minutes with less than 0.04-inch of movement between 1 minute and 10 minutes, the load shall be maintained for an additional 50 minutes. The preproduction test will be considered successful and concluded if the test nail meets the criteria for a preproduction test of nail in 614.41(a-3.b) of these special provisions.

The Engineer will evaluate the results of each preproduction test, make a determination of the suitability of the test and the Contractor's proposed production nail design and installation system. Tests which fail to meet the design criteria will require retesting or an approved revision in the Contractor's proposed production nail design and installation system. The soil nail shall be unloaded and completely grouted, only after completion of the test.

(b) Location and length - The location and length of the nails shall be as shown in the plans or as directed by the Engineer. The Contractor shall locate the holes within 3 inches of the predetermined location and in such a manner that the nail is not in physical contact with the welded wire fabric.

(c) Drilled hole diameter and length - The Contractor shall determine the maximum diameter of the hole. Minimum hole diameter shall provide a 3-inch clearance around the outer surface of the soil nail. Holes shall be drilled to a depth sufficient to provide the minimum embedment length (L) shown in the plans.

(d) Nail capacity - The nail capacity shall equal or exceed the pullout resistance (kips) shown on the plan. Embedment lengths for nails shall in no case be less than the minimum shown in the plans.

(e) Nail handling - Nails shall be handled and stored in such a manner as to avoid damage or corrosion. Damage to the nail steel as a result of abrasions, cuts, nicks, welds, and weld splatter will be cause for rejection by the Engineer. The nail steel shall be protected if welding is to be performed in the vicinity. Grounding of welding leads to the nail steel will not be allowed. Nail steel shall be protected from dirt, rust, and foreign substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the Engineer shall reject the affected nails.

(f) Nail installation - The nail shall be inserted in the hole to the required depth without difficulty. If the bar cannot be completely inserted, the Contractor shall remove the bar and clean or redrill the hole to permit insertion. Partially inserted nails will be rejected.

(g) Grouting - The grout shall be injected at the lowest point of each drilled hole and the hole filled in a continuous operation. The grout may be pumped through grout tubes, casing, or drill rods. The grout shall be placed after insertion of the nail. The quantity and pressure of the grout shall be carefully controlled and recorded. The grout equipment shall produce a uniformly mixed grout free of lumps. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gage which can measure at least twice the intended grout pressure and a stroke counter. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

(h) Installation of plate and nut - After the first layer of shotcrete and the grout have had time to gain the specified strength, the plate shall be placed as shown in the plans and the nut secured wrench tight with a minimum 100 ft.lbs. of torque.

614.33 Shotcrete Construction:

(a) General - Shotcrete test panels shall be prepared by each crew on vertically supported molds. The material used to form the back and sides of the molds shall be rigid, nonabsorbent and be nonreactive with cement. The shotcrete placement in vertical molds shall be accomplished utilizing the same equipment, shotcrete mix, air and water pressure, and nozzle tip as used for the actual placement of shotcrete on production surfaces. The panels shall be constructed at the project site in the presence of the Engineer. The panels shall be left undisturbed and protected at the point of placement for at least 24 hours or until the final set has taken place.

(b) Preproduction testing - Each crew shall prepare at least two test panels for each mix design for testing. The test panels shall be a minimum of 24 inches by 24 inches and shall be fabricated to the same thickness as in the proposed application. Material to form the sides shall be 3/8-inch hardware cloth.

One of the two panels shall be reinforced with the same welded wire fabric as in the proposed application. The Contractor shall saw the completed panel into at least 6 pieces to allow a visual inspection of the shotcrete density, void structure and coverage of the reinforcement. This panel shall have 3 applications of shotcrete as shown in the plans. The other test panel shall be constructed without reinforcing and the Contractor shall extract at least six 3-inch diameter cores from this panel in the presence of the Engineer for compressive strength testing by the Engineer in accordance with ASTM C 42.

The test panels shall be cured using the proper curing compound in a manner similar to the anticipated field conditions. The Contractor shall provide the Engineer with a copy of the mix design at least 5 working days prior to starting any production work. Production shotcrete work shall not begin until satisfactory test panel results are obtained.

(c) Deficient shotcrete - If any shotcrete produced by the Contractor fails to meet the requirements of these special provisions, the Contractor shall immediately modify procedures, equipment or system, as necessary and as approved by the Engineer, to produce specified material. All substandard shotcrete already placed shall be repaired to the satisfaction of the Engineer at the Contractor's expense. Such repairs may include removal and replacement of all effected materials, or placement of additional thickness, as determined by the Engineer.

(d) Equipment - The pump system utilized to convey premixed shotcrete ingredients shall deliver a uniform and uninterrupted flow of material, without segregation or loss of the ingredients.

The air compressor shall be capable of maintaining a supply of clean air adequate for maintaining sufficient nozzle velocity for all parts of the work and for the simultaneous operation of a blow pipe for clearing away rebound.

Batching and mixing shall be done according to ASTM C 94. Aggregate and cement shall be batched by weight. Mixing and placing equipment shall be capable of continuous operation and shall deliver a uniform and uninterrupted flow of material without segregation or loss of any ingredients. Ready-mixed shotcrete may be delivered in transit mixers which comply with AASHTO M 157.

The delivery equipment shall be capable of discharging the premixed materials into the delivery hose and delivering a continuous stream of uniformly mixed material to the discharge nozzle. Recommendations of the equipment manufacturer shall be followed on the type and size of nozzle air hoses and supplies to be used, and on cleaning, inspection and maintenance of the equipment.

(e) Application - Immediately prior to shotcrete application, soil surfaces shall be cleaned of loosened material. Areas where raveling develops shall be immediately shotcreted. Shotcrete should not be placed on any surface which is frozen, spongy, or where there is free water. The surfaces to be shot shall be damp but have no free-standing water. Thickness, method of support, air pressure and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete.

The thickness of the shotcrete blanket shall be controlled by installing noncorrosive guide pins, nails or other gaging devices normal to the face, such that they protrude the required shotcrete thickness outside the face. These pins shall be placed on a maximum 5-foot square pattern. A minimum cover of shotcrete shall be placed over the welded wire fabric as shown on the plans.

The shotcrete shall be applied from the lower portion of the area upwards so that rebound does not accumulate on the portion of the surface that still has to be covered. The nozzle shall be held at a distance and at an angle approximately perpendicular to the working face so that rebound material will be minimal and compaction will be maximized. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. When, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until steady flow resumes. A helper equipped with an air blowout jet shall attend the nozzleman at all times during the placement of shotcrete, to keep the working area free from rebound.

Rebound material shall not be worked into the finished product. Rebound is defined as the shotcrete constituents which fail to adhere to the surface to which shotcrete is being applied. It shall not be salvaged and included in later batches.

Shooting shall be suspended if:

1. High wind prevents the nozzleman from proper application of the material.

2. The temperature is below 40°F.

3. External factors, such as rain, wash cement out of the freshly placed material or cause sloughs in the work.

Construction joints shall be tapered over a minimum distance of 12 inches to a thin edge, and the surface of such joints shall be thoroughly wetted before any adjacent section of mortar is placed. Square construction joints shall not be permitted.

Surface defects shall be repaired as soon as possible after initial placement of the shotcrete. All shotcrete which lacks uniformity, which exhibits segregation, honey combing, lamination, or which contains any dry patches, slugs, voids or sand pockets shall be removed and replaced with fresh shotcrete at the Contractor's expense and to the satisfaction of the Engineer. If the wire fabric reinforcement is damaged or destroyed by such repairs, the damaged area shall be replaced by properly lapped and tied additional wire fabric.

Where a layer of shotcrete is to be covered by succeeding layers, it shall first be allowed to take its initial set. The initial layer shall be cleaned of all loosened material prior to placing succeeding layers.

(f) Curing - Air placed shotcrete shall be cured by applying a clear pigmented, liquid membrane-forming compound as specified in 701.05 of the Standard Specifications. The curing compound shall be applied immediately after gunning. The air in contact with shotcrete surfaces shall be maintained at tempertures above freezing for a minimum of 7 days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cement finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials. All hot and cold weather shotcreting procedures shall conform to ACI 506.2 except as modified herein.

Acceptance Testing

614.41 Acceptance Testing - Acceptance testing shall comply with the following:

(a) Nail load testing - Preproduction sacrificial nails and a percentage of the production nails shall be load tested to check the capacity of the proposed system to sustain the minimum pullout resistance (kips) shown on the plans for the service life of the wall. The Contractor shall supply all material, equipment, and labor to perform the tests. The Engineer will record all required test data. The cost of all nail testing is considered incidental and shall be borne by the Contractor.

Load testing of the preproduction nails shall be performed against a temporary bearing yoke which bears directly against the existing soil. Temporary bearing pads shall be kept a minimum of 12 inches from the edges of the drilled holes.

(a-1) Testing equipment - A hydraulic jack and pump are used in testing to apply the load. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total nail length at the maximum test load plus 1 inch. The jack shall be independently supported and centered over the nail so that the nail does not carry the weight of the jack.

The elongation of the nail shall be measured with a dial gage or vernier scale fixed to a tripod or some other support device independent of the structure. The dial gage should permit reading to a maximum accuracy of 0.001 inch. The ram travel of the dial gage shall not be less than the theoretical elastic elongation of the total nail length plus 1 inch. The axis of the dial gage ram shall be aligned to within 5 degrees from the axis of the nail. A pressure gage attached to the hydraulic pump shall be used to measure the applied load. The pressure gage dial face shall be graduated in 100 psi increments or less and the full scale range shall not be greater than twice the pressure required for the maximum load to be applied.

The hydraulic jack and the pressure gage shall be calibrated as a set by an independent testing laboratory. Proof of calibration must be submitted before use. The loads on the nails during the tests shall be monitored with an electronic load cell. The Contractor shall provide the electronic load cell and a readout device. Care should be taken that the axis of the nail and the load cell are parallel to prevent eccentric loading. The stressing equipment shall be placed over the nail in such a manner that the jack, bearing plates, load cell and stressing anchorage are in alignment.

(a-2) Production testing - Ten percent of the nails in each shotcrete lift shall be tested to demonstrate that the minimum required pullout resistances (kips) shown in the plans are being developed. The location of the production nails to be tested shall be determined by the Engineer.

Production test nails can be either sacrificial or used as production nails. This test requires a no-load zone (ungrouted test length) and a bond zone (grouted length). The ungrouted length of the production nail shall be equivalent to the ungrouted test length shown on the plans. After the effective length is grouted, and the grout has gained sufficient strength to withstand the test load, the test nail shall be loaded to a total load equal to the pullout resistance (kips) shown on the plans. The ungrouted test length shall be grouted after testing if the nail is to be used as a production nail.

Applied test loads shall be measured with the pressure gage or load cell. Movement of the end of the nail, relative to a fixed reference, shall be measured and recorded to the nearest 0.001 inch.

Production testing shall be performed by loading the tested nail in increments of 25 percent of the pullout resistance (kips) to a total load equal to the pullout resistance (kips) shown in the plans.

The load shall be held for 1 minute between increments except for the final load which shall be held for 10 minutes. The load-hold period shall start as soon as the maximum load is applied.

Nail movements with respect to a fixed reference point shall be measured and recorded at one minute, 2, 3, 4, 5, 6 and 10 minutes.

If the change in movement between 1 and 10 minutes exceeds 0.04 inch, then the maximum test load shall be held for an additional 50 minutes. If the observation period is extended to 60 minutes, then the nail movements shall be recorded at 15 minutes, 20, 25, 30, 45, 50 and 60 minutes. If the nail fails in creep, retesting will not be allowed.

(a-3) Load testing acceptance criteria - Production testing of nails shall comply with the following requirements:

(a-3.a) A preproduction or production tested nail with a 10-minute load-hold is acceptable if:

- Nail carries the maximum test load with less than 0.04 inch of movement between 1 minute and 10 minutes; and

- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded test length.

- Total movement measured at the maximum test load does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length.

(a-3.b) A preproduction or production tested nail with a 60-minute load-hold is acceptable if:

- Nail carries the maximum test load with a creep rate that does not exceed 0.08 inch during the final log cycle of time and is a linear or decreasing creep rate; and

- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded test length.

- Total movement measured at the maximum test does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length.

(a-4) Replacement nails - If a production test fails, the Engineer may direct the Contractor to replace some or all of the installed production nails between the failed test and the adjacent production test nail that met the test criteria. Alternatively, nail length on the succeeding row may be lengthened to make up the additional capacity needed, additional design analysis by the Engineer would be required to determine the additional lengths required. The Engineer may also require additional testing. Costs associated with additional tests or installation of additional and/or longer nails and Engineer's redesign costs shall be at the Contractor's expense.

(b) Shotcrete - Acceptance testing of shotcrete shall conform to the following:

(b-1) Production testing - A minimum of 28 days after the initial and intermediate layers of shotcrete have been placed, the Contractor shall core 3 test specimens from each 2,000 square feet of shotcrete placed in the field at locations designated by the Engineer. These cores shall be 3 inches in diameter and the full thickness of the wall. This coring shall be done in the presence of the Engineer and the Contractor shall individually seal the cores in plastic bags and tag them for identification. The cores will be tested by the Engineer for compression strength in accordance with ASTM C 42. The holes resulting from the cores shall be sealed in a manner satisfactory to the Engineer.

The shotcrete shall be capable of attaining the following minimum compressive strength (f'c) as determined by ASTM C 42 testing of cores drilled for compressive strength determinations:

Age-days	Compressive strength-psi
28	4000

Shotcrete work will be accepted based on 28-day strengths. The Contractor may propose a method of expediting the work. The Contractor's proposal shall detail methods to assure that the 28-day strength is attained.

Measurement

614.81 General - The method of measurement for soil nailing payment shall be per square foot of shotcrete complete in place and accepted by the Engineer. It shall include furnishing soil nails, plates, washers, welded wire fabric, wire holding devices, centralizers, preformed permeable liner, weep holes, anchor grout and structural shotcrete as shown in the plans. Measurement shall include only those areas where the full thickness called for in the plans is in place.

Structure excavation will be measured in accordance with Section 251.

Drain pipe and drain backfill material associated with retaining walls will be measured for payment in accordance with Section 605 under "Roadwork".

Payment

614.91 General - The accepted quantity measured as provided above will be paid for at the contract unit price per square yard for the item "Nailed Soil Retaining Wall" which payment will be full compensation for furnishing all materials, labor, equipment, tools and incidentals necessary to complete the work as specified in this Section and detailed on the plans, with the exception of structure excavation which will be measured and paid for in accordance with Section 251.

Full compensation for cutting and removing the existing end slope shall be considered as included in the contract price paid per cubic yard for structural excavation and no additional compensation will be allowed.

Drain pipe and drain backfill material associated with retaining walls will be paid for under the "Roadwork" portion of the job, as set forth in Section 605.

APPENDIX C: SWIFT-DELTA SOIL NAIL WALL INSTRUMENTATION PLAN SPECIFICATIONS

NAILED SOIL WALL INSTRUMENTATION

Scope - This work shall consist of furnishing all instruments, tools, materials and labor and performing all tests necessary to install instruments in accordance with the plans and these special provisions. Station "UV" 130+40 shall hereafter be referred to as Section 1 and Station "UV" 131+12 as Section 2.

Each instrumented section shall have all instruments installed under this work and wired to a central control panel. Wiring to the control panel will be completed after installation of each instrument and is test proven by the Contractor to the satisfaction of the Engineer that the system is working in accordance to the manufacturer's specifications.

The Contractor shall install the instruments under the supervision of a qualified geotechnical instrumentation specialist having a minimum 3 years of experience in the design and installation of similar instrumentations.

Inclinometers will be furnished and installed by the Engineer. The Contractor shall cooperate in the installation of the inclinometers.

All instrumentation shall be protected by the Contractor during the term of the contract and shall be replaced or restored at the Contractor's expense if damaged by reason of his operations, to the satisfaction of the Engineer.

The Engineer will conduct research activities within the limits of the nailed soil wall structure. Visual observations and instrumentation readings will be made by the Engineer. Preconstruction readings will be taken immediately after the installation of the top strain gages to two existing concrete filled steel pipe piles at Section 1. Readings will be taken from the instrumentation after each instrumented nail is installed. Postconstruction readings will be taken monthly for the next 24 months.

Submittals - At least 5 weeks prior to start of nailed wall excavation, the Contractor shall submit in writing, 5 copies of a list of the instruments including instrument specifications, installation procedures and a wiring diagram detailing the wiring of the instruments to the central control panels. Also, at this time, the Contractor shall submit resumes' of those individuals responsible for instrument installation and testing. The list shall include references, including current telephone number, that can verify the experience requirements. Nailed soil wall construction shall not begin until the Engineer has approved instruments, installation procedures, personnel, and two adjacent piles at Section 1 instrumented with strain gages.

Materials

Instruments:

Strain Gages - Nails shall be instrumented to assess the load on the nails over the long term. Existing piles shall be instrumented to monitor any lateral load transfer and subsequently flexural stress build-up during the construction phase. Instrumentation to monitor loads shall consist of vibrating wire strain gages. The strain gages shall be weldable vibrating wire gage such as slope indicator part Nos. 52602100 and 52602200 or an approved equal. Strain gage accessories include Model Nos. 52602600, 52602300, 06700180, 06700019, 52606956, 52604100 and 52604110 or approved equal. The strain gages will be read using the Engineer's strain gage readout box Model No. 52669 manufactured by Slope Indicator Company.

Nail Load Cells - The nail load cells shall have an ultimate capacity not to exceed 50 tons. The load cells shall be center hole load cells with minimum hole diameter of 1.5 inches. Slope indicator parts Nos. 51301050, 56400800 and 51300960 or approved equal shall be used. The load cells will be read using a load cell indicator Model No. 51300900 manufactured by Slope Indicator Company or approved equal.

Earth Pressure Cells - The earth pressure cells shall exhibit an ultimate capacity not to exceed 50 tons. Slope indicator parts Nos. 51408200, 51417800, 51416900, 51421115, 51401510, 51400095 and 51407302 or approved equal shall be used. The earth pressure cells will be read using a readout box No. 51421100, 211 model 0.1 percent, manufactured by Slope Indicator Company or approved equal.

Tiltmeters - Tiltmeters shall be used to monitor the existing Bridge Pier 10 pile cap rotation. Ceramic tiltplates shall be slope indicator Model No. 50323 or Terra Technology Corp. Model No. TP-C or approved equal. The plates shall be mounted on the exposed face of the pile cap using Devcon UW No. 11800 bonding compound. The portable tiltmeter censor (english version) shall be slope indicator Model No. 50304400 or Terra-Technology Model No. TT-2 or approved equal. The censor will be read using the State's readout box Model No. 50309 manufactured by Slope Indicator Company.

Extensometer - One extensometer shall be used to monitor the pile cap deflection as excavation progresses. Slope indicator parts Nos. 51815800, 51815835, 51815855, 51815860, 51809600, 51703900, 51702701 and 517046FM or approved equal shall be used.

Central Control Panel - The central control panels shall be of sufficient size and capacity to handle the specified number of instruments outlined in these special provisions for each instrumented section. There shall be a channel for each individual instrument.

Installation

Strain Gages - The nails to be instrumented are located at Section 1 and Section 2. Three nails in Row Nos. 1, 3 and 5 in each respective section will be instrumented. Five pairs of strain gages shall be welded to each nail in Row Nos. 1, 3 and 5 at Section 1, and four pairs of strain gages shall be welded to each nail in Row Nos. 1, 3 and 5 at Section 2. The strain gages which are mounted opposite each other shall be micro-welded to the nail and the complete gage, sensor and wire assembly protected from moisture. Two pairs of gages shall be mounted 3 feet from the nail ends with the remaining three pairs at Section 1 and two pairs at Section 2 mounted and evenly spaced in between. The instrumented nails shall be installed in the drill holes with the strain gages aligned vertically. All wire connections shall be of an approved waterproof type. Installation and protection of the strain gage and connections shall be in accordance with the manufacturer's specifications.

The concrete filled steel pipe piles to be instrumented are located at Section 1. Two adjacent piles shall be instrumented with two strain gages each. The strain gages shall be micro-welded to the piles and the complete gage, sensor and wire assembly protected from moisture. The strain gages shall be mounted 5 feet and 10 feet below the bottom face of the pile cap.

Nail Load Cells - A total of six load cells (three in each instrumented section) shall be installed. The electric load cells shall be located in nail Row Nos. 1, 3 and 5. A 12"x12" blockout shall be provided in the shotcrete facing at the instrumented nail Row Nos. 1, 3 and 5 after the first application of shotcrete. The load cell shall be mounted on the nail between the bearing plate and the nut. The Contractor shall attach the cells and protect the connections according to the manufacturer's specifications. All wire connections shall be of an approved waterproof type. When the instrumentation program is completed, the Contractor shall remove the load cells and the blockouts, retighten the nuts, apply the second and final shotcrete layers to the true lines and grades shown in the plans, and apply a Class I finish as shown on Drawing 45817.

Earth Pressure Cells - A total of 12 earth pressure cells (six in each instrumented section) shall be installed. The earth pressure cells will be aligned vertically. Two earth pressure cells will be located in nail Row Nos. 1, 3 and 5. Locate one cell adjacent to the instrumented nail and the second cell midway between the instrumented section and an adjacent nail. The earth pressure cells will be installed at the interface between the soil and the first layer of shotcrete. The cells shall be positioned such that the lateral earth pressures bearing against the shotcrete wall will be monitored. The installation and protection of the earth pressure cells and their connections shall be in accordance with the manufacturer's specifications. All wire connections shall be of an approved waterproof type.

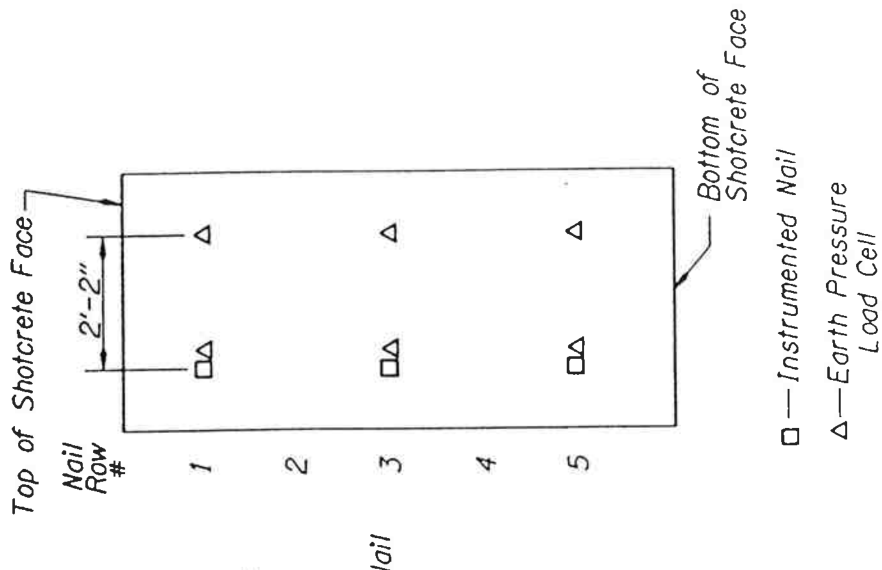
Central Control Panel - Two central control panels shall be installed at each instrumented section. The control panel at Section 2 will be attached to a steel or treated wooden post which is firmly secured in the soil. This control panel will be located 3 feet behind (south) the nailed soil wall. The control panel at Section 1 shall be installed at a location to be selected by the Engineer. The mounting details shall be submitted for the Engineer's approval. All instrumentation wiring to the control panels will be done in accordance with the manufacturer's specifications. The control panels must be sealed and completely waterproof. All above-ground wiring shall be enclosed in a steel conduit which is firmly attached to the control panels. All instrument-control panel wiring shall be done during instrument installation.

Measurement and Payment

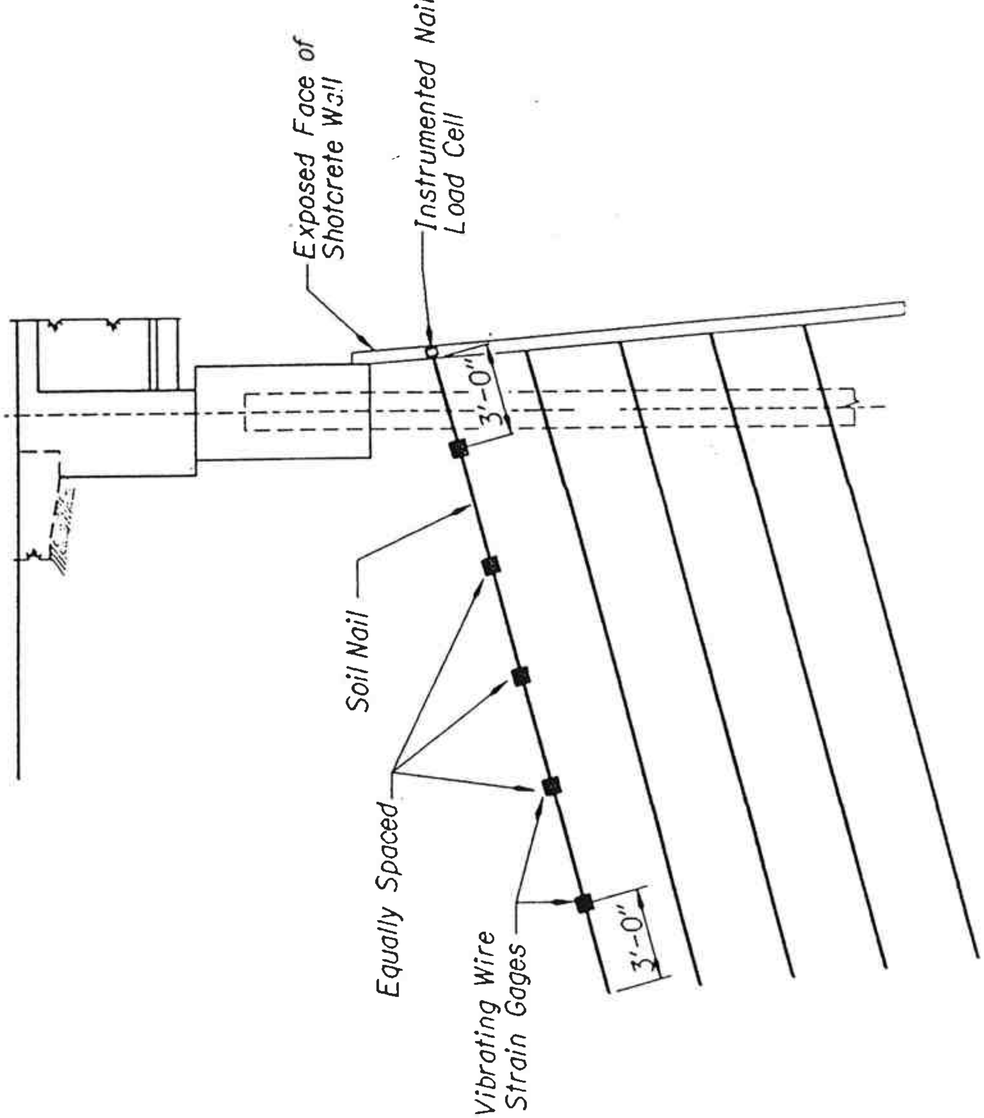
Measurement - No separate measurement will be made for the materials and work specified in this Section.

Payment - Soil nailing instrumentation will be paid for at the contract lump sum amount for the item "Soil Nailing Instrumentation" which payment will be full compensation for furnishing all materials, labor, equipment, tools and incidentals necessary to complete the work as specified in this Section.

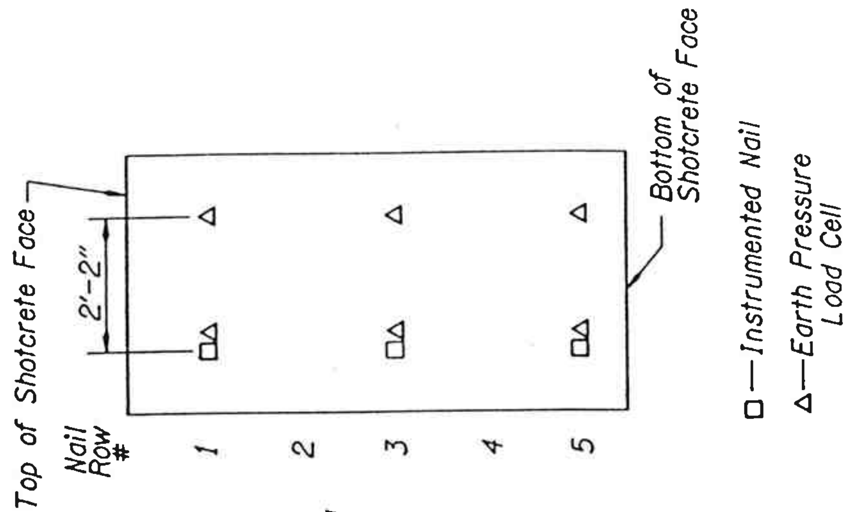
All instruments furnished and installed under this Section shall become the property of the Division.



FRONT VIEW
(Sta. "UV" 130+40.00)
No Scale



SECTION @ OREGON SLOUGH BRIDGE
(Sta. "UV" 130+40.00)
No Scale



FRONT VIEW
(Sta. "UV" 131+12.00)
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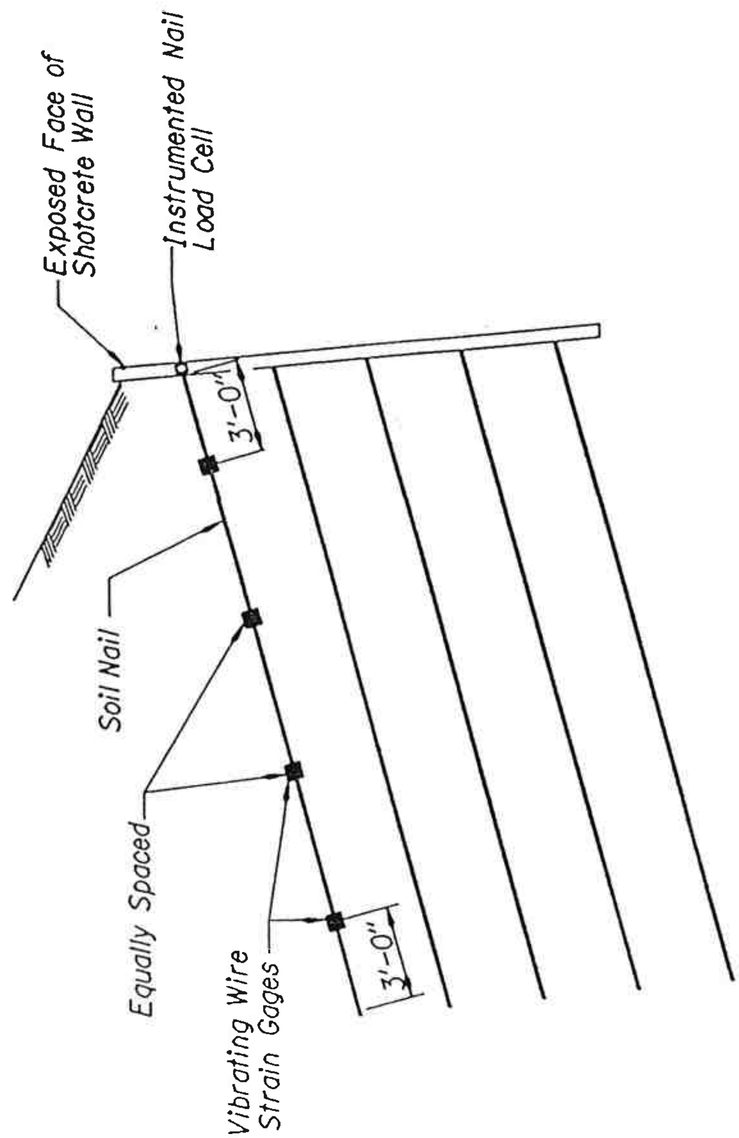

SECTION @ STA. "UV" 131+12.00
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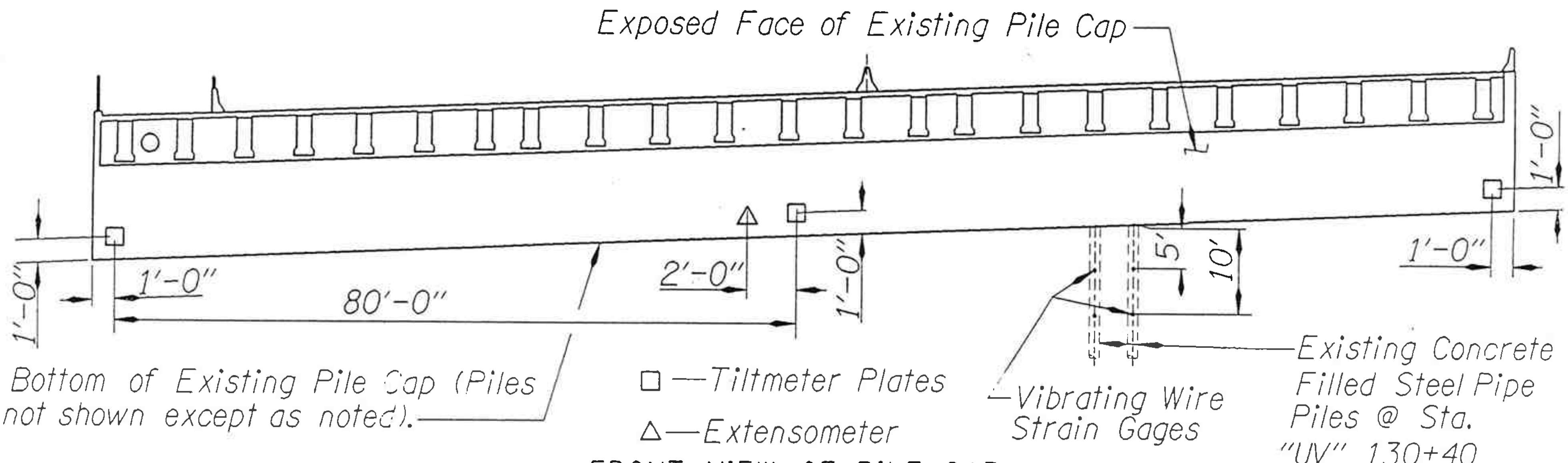

FRONT VIEW OF PILE CAP

No Scale

SOIL NAILING INSTRUMENTATION