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FIELD STUDY OF SURCHARGE EFFECTS ON A STEEL SHEET PILE BULKHEAD

by

Thomas D. Dismuke

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CERTIFICATE OF APPROVAL

This thesis is accepted and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

September 16, 1968
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This thesis presents the results and analysis of an experimental investigation to determine the surcharge effect on a steel sheet pile bulkhead.

The investigation was carried out by the Bethlehem Steel Corporation at its Sparrows Point, Maryland Plant.

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ABSTRACT

This thesis presents the results and analysis of a field study conducted to determine surcharge effects on a steel sheet pile bulkhead. Slope indicator and strain gage data were used to determine the loads, shears, moments and deflections at one instrumented pile location. These results are compared to the results of theoretical calculations.

Field information was obtained through the use of strain gages, slope indicators and transit and tape surveys. The performance and accuracy of the instrumentation and data are reviewed with respect to the effect on the results.

Large axial loads and moments which developed on one of the instrumented sheet piling due to driving into soil were observed.

The development of soil arching or other means of reducing load on the bulkhead was evident.

It is concluded that the effects of wall friction should be included when flexible bulkheads with granular surcharges are designed. In addition, this study corroborated the results of previous field studies with respect to observed moment reduction (compared to the theoretical) for bulkheads when normally loaded by backfilling.

I. INTRODUCTION

1.1 Objective and Scope of Program

The objective of the test program was to determine the effect of surcharge on a steel sheet piling bulkhead. In recent years, a number of prototype steel sheet piling bulkheads have been instrumented (1, 2, 3, 4, 5, 6, 7). In addition, a number of model and theoretical studies have been made in order to develop realistic design methods for bulkheads (8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19). These tests and studies have produced much valuable information and serious attempts to develop better design criteria and methods have resulted; however, there is little field information on the effect of surcharge on flexible bulkheads.

The scope of this investigation, while experimentally limited to the determination of surcharge effects on a bulkhead, includes results of backfilling the bulkhead prior to placing the surcharge.

1.2 Methods of Bulkhead Design

Common design methods in general use in this country include free earth support, fixed earth support, equivalent beam and Tschebotarioff's modification of the equivalent beam. The free earth support method has been modified and updated by using Rowe's flexibility procedure. In all of these methods the pressure against the bulkhead is usually calculated by the classical Rankine-Coulomb methods. Lateral bulkhead pressure due to surface loads other than bulk materials are generally calculated by Spangler's adaptation of the Boussinesq formula (20).

Rowe's flexibility method of bulkhead design gives results similar to those developed by Tschebotarioff's method when steel sheet pile bulkheads are used. For stiff bulkheads, such as those made of concrete, Rowe's method gives more conservative results.

Subgrade reaction methods (21, 22, 23, 24) of analysis have been proposed and appear to more nearly reflect the effect of the loads on bulkheads, but the difficulty in using these methods lies in the almost impossible task of defining soil response from existing soil test data.

Many bulkheads have been greatly overloaded (as defined by existing design and analytical methods) principally by surface surcharges such as iron ore and other bulk materials (27). No bending failures due to overloading of steel sheet piling bulkheads have been reported. Therefore, it is evident that the computations of bulkhead loads and bending stresses (not anchor loads) due to any cause by existing methods are conservative.

1.3 Experimental and Theoretical Procedure

There are several methods by which the effects of surcharge on bulkheads may be determined experimentally. These include direct methods such as placing pressure cells at the soil-bulkhead interface and/or indirect methods such as measuring sheet piling strain or curvature (slope) changes. The reliability of strain and slope change measurements appeared better than other types of measurements. The experimental procedure consisted of attaching strain measuring devices and receptacles for inserting slope measuring devices to the sheet piling before the piling was driven, establishing instrumentation zeros, and taking measurements of the strain and slope changes, after placement, due to surcharge changes.

The theoretical procedure for finding the pressure was to determine moments from slope changes, shears from the change of moment between two points and pressures by the change in shear between two points. In addition, differentiation was used for checking the above by taking successive slopes of the moment distribution curve, which are ordinates of the shear curve, and so on to the pressure curve. These pressure curves are then compared with the theoretical bulkhead pressures imposed by the surcharges.

II. WHARF DESIGN, CONSTRUCTION AND INSTRUMENTATION

2.1 Wharf Design

The bulkhead on which the test program was conducted was the 450-ft extension to the existing coal field wharf at the Sparrows Point Plant of Bethlehem Steel Corporation. The plant is located on the upper reaches of Chesapeake Bay. Construction of the wharf started in January 1962 and was completed in October 1962.

The coal wharf is used for receiving and storing coal. Coal is brought in on barges and unloaded by any one of three bridges. A conveyor also runs on that portion of the wharf immediately behind the bridge rails.

2.1.1 Site Conditions

The site is sheltered and has a normal tidal variation of 1.5 feet. At times, weather conditions have raised the height of the water to +7.0 feet and lowered it to -4.0 feet. From 1935 to date the lowest water elevation has been -2.0 feet. The normal mean low water level is -1.5 feet.

Fig. 1 shows the soil profile along the bulkhead of the 450-ft wharf extension. The water depth was approximately 20 feet. The soil from the top down consisted of a stratum of sandy silts, clayey sand and sandy clay. The locations of the borings along the bulkhead are shown, as are the blows per foot of the 2" O.D. sampling spoon. The standard spoon was driven one foot, with a 140-lb weight dropped 30 inches.

The silt was removed from the site area, including the coal field, and fill consisting of plant refuse (miscellaneous material such as brick, slag, and other similar plant-generated refuse) was placed.

No organic material is mixed with the refuse. Dry weight of the plant refuse is approximately 120 pcf.

With the exception of the silt, the soil is dense and soil conditions are excellent for bulkhead construction.

2.1.2 Wharf Geometry

Figures 2 and 3 show the plan and elevation of the coal field and wharf. The sheet piling serves as a cut-off wall behind the batter- and coal-bridge-support piling.

This type of wharf was used so that the support system could be placed in front of the sheet piling rather than in the coal field where bridge buckets may damage tie-backs.

The original portion of the wharf was constructed of concrete sheeting and tie-backs. The sheeting was placed immediately behind the fender system.

Since barges are constantly increasing in size -- consequently drawing more water -- the wharf was designed to handle barges drawing 25 feet. The soil in front of the piling would then have to be dredged to -27.5 feet, but for the present the dredged depth is limited to -21.5 feet.

2.1.3 Loading on Wharf

Figure 4 shows the estimated loads applied to the bulkhead. Moments for the backfilled and surcharged conditions for a dredged depth of -21.5 feet are also shown. The fixed earth support method was used in the calculations.

A comparison of the support load, maximum moment, and location of the maximum moment below the support are given in Table 1 for the several design methods.

2.2 Wharf Construction

2.2.1 Procedure

The construction procedure consisted of five steps. The first step was to dredge the area of the soft, silty material in the channel as well as in the coal field area. The second step consisted of making a fill behind the bulkhead to an elevation of +2 feet, using miscellaneous fill. This fill was constructed by trucks. The third step consisted of removing the top of the fill by dragline to an elevation of -5.0 feet so that a floating pile rig could operate immediately behind the bulkhead in order to drive the piling. The fourth phase consisted of driving the piling and applying the concrete cap and deck. The fifth step included the placing of fill behind the bulkhead after completion of the concrete work. While the fourth phase was in progress, the remainder of the coal field was being filled to Elevation +5.0 feet, the elevation for the coal field. Fig. 5 shows the sequence of steps at Bent 18.

2.2.2 Pile Driving

Pile driving started from the end of the existing wharf and progressed outward. Driving records of the ZP 38 paired sheet piling show that the number of blows per last foot of penetration varied from 48 to 250 using a McKiernan-Terry 11B3 hammer, and from 40 to 70 using the Vulcan 0 hammer. The instrumented piling was driven by the Vulcan 0 hammer. The resistance to driving of the instrumented sheeting and H-piling indicated that all of the penetrated soil was dense and afforded excellent support for the wharf structure.

2.3 Instrumentation

2.3.1 Philosophy

The number of measurable quantities in this particular field study is quite large. These include surcharge (horizontal) pressure, pore, intergranular and total soil pressures, bulkhead pressures, movement, strain (stresses) and shape, batter pile strains, etc. All of the above measurements involve the determination of a change in length and may be measured by various transducers. Many existing transducers are not suitable for use in a soil-water environment. This is usually due to the fact that most transducers or connections are not adequately constructed to be protected from physical damage such as may occur during pile driving or from short-circuiting and/or disbonding when immersed in water.

For this study it was decided that minor emphasis should be placed on direct measurements in the soil phase of the 2-phase system. Redundancy of measurement systems on the sheet pile bulkhead was desirable as other investigators had experienced difficulties with SR4 gages.

A study of this type necessarily involves the time factor. Surcharge changes result in changes in bulkhead configurations. Since these changes can occur over short periods (hours) or many days, manual or, preferably, automatic recording of data is necessary.

The placement of the transducers influences the accuracy of the results as well as the length of time the instrumentation is effective. Whenever possible, the strain gages were placed above the water line.

2.3.2 Strain Gages

SR4 gages were mounted on the flanges of three ZP 38 sheet piling as shown in Figs. 6 and 7. An external bridge at each point of measurement was formed so as to eliminate the effect of the shielded lead wire resistance. Similar gage configurations were installed on the two anchor H-piling on either side of the instrumented sheet piling and the bridge support piling.

The number and location of the SR4 gages would permit recording of adequate data from which the axial and bending stresses in the H-piling could be determined. This information could then be used to indirectly establish the intensity and distribution of the lateral loads on the sheet piling caused by the coal field surcharge.

Calibration of the SR4 gages on the sheet piling was accomplished by interlocking two ZP 38 sections together and forming a simple beam by placing them horizontally on two wood supports. Known loads were placed at the centerline of the span and strain readings were taken. A return to the initial strain reading was accomplished after several loading cycles.

The SR4 gages for the H-piling were installed on short sections. These sections were then spliced to the H-piling when they were very close to the termination of driving. Following the splicing of the short instrumented sections, the H-piling was driven several feet to complete the driving.

Details of the shielded SR4 gage lead wires and protective channel are shown in Fig. 8. The severe conditions, driving and water immersion, which the SR4 gages were subjected to dictated the care used for installation of the gages.

2.3.3 Slope Indicators

Slope indicator pipes were fastened at the neutral axis of the same sheet piling to which SR4 gages were attached. Figs. 9 and 10 show the details of the installation. The Wilson slope indicator has been described in the literature but will also be briefly described here. It is a device to determine the slope of a member for a standard length element. An internal pendulum swings along a calibrated resistance to give a resistance reading. The maximum angle from vertical position that the model used in this investigation could make is 8 degrees. A plastic tube with 4 grooves spaced at 90° in the tube interior was placed in the steel pipe after the sheet piling was driven and the concrete cap placed. Sand was vibrated into the annular space between the plastic tube and steel pipe.

The grooves in the plastic tube guide the slope indicator wheels (180° apart) and are so oriented that one set of grooves is parallel to the bulkhead line and the other set perpendicular. As the pendulum swings in line with the guide wheels, the perpendicular and parallel profiles of the sheet piling may be determined when the slope indicator is used in both sets of guide grooves. The slope indicator is a rugged instrument and is well suited for its purpose. Since its operation is manual, the cost of using the instrument is higher than an electrical type of transducer which can have various types of automatic recording devices attached to it.

2.3.4 Surcharge Pressure Gages

Pressure gages were placed at several locations in the coal field behind the instrumented piling as shown in Fig. 11. These gages were to be used to determine the vertical surcharge pressures. The equipment consisted of small capacitor type transducers, oscillator, bridge detector and amplifier. A recorder was connected to the indicating equipment using a stepping relay, 4-pole, 10-position round and round type with timer for sequentially selecting each channel.

A device for recording the surcharge pressure was particularly desirable since coal is transferred in and out of the field on a continuous basis. At times, particular piles of coal were not touched for several days or more, but the amount and location of coal and time of movement are neither predetermined nor recorded.

III. EXPERIMENTAL RESULTS

3.1 Post Driving Stresses

Post driving stresses were determined by taking the difference in gage readings between the no-load condition and immediately after driving. The zero load condition for the sheet piling was taken as those gage readings recorded while the piling was hung from a crane.

Fig. 12 shows the axial loads and bending moments along the sheet piling due to driving the pile into the soil. Only those stresses where SR4 gage readings were obtained on both flanges are shown. Readings were obtained at gage locations 6 and 10 on the south flange.

Unfortunately, during the driving of the instrumented sheet piling at Bent 26 the lead wires became entangled with adjacent piling and pulled the wires away from all gages. The only SR4 gage data on the sheet piling after backfill was placed were obtained at Bent 18.

3.2 Lateral Loading

The method used to construct the wharf is similar to that of a "fill" bulkhead. The concrete cap was placed after the sheet and H-piling were driven. As a result, construction activity around the slope indicator pipe prevented installation of the plastic tube in the pipe until after the concrete was placed and backfilling had been completed at Bent 18.

3.2.1 Backfill

Because of the procedure outlined above at Bent 18 no slope indicator data were available to determine the effect of the backfill on the bulkhead. Also, with the exception of gage 10 at Bent 18, all sheet piling strain gages that survived pile driving became inoperable a few days after driving and before the backfill was placed. The resistance of all of the Bent 18 sheet piling strain gages, except at location 10, dropped to very low values.

Table 2 gives the SR4 gage data at location 10 noted above.

Fig. 13 shows the horizontal component of the batter pile loads for the backfilled and surcharged load conditions at Bents 26E and 26W. Only one SR4 gage on each flange of the batter piles at Bent 18 remained operable a few days after installation; therefore, axial loads could not be determined.

3.2.2 Surcharge

The slope of the driven sheet piling and deflection changes due to the surcharge loadings on the bulkhead are given in Figs. 14, 15 and 16. Three complete sets of observations were made. These data include results of manually cross-sectioning the coal surcharge adjacent to the bulkhead, strain gage, slope indicator and surcharge pressure readings. A number of slope indicator readings were made without cross-sectioning the coal surcharge simultaneously. These data did not indicate any anomalies with other slope indicator data, so they are not included. The first slope indicator readings taken on July 12, 1962 are used as the base readings. In the Analysis and Discussion section the July 26, 1962 readings were used as the base readings. Both sets of data were taken before surcharge was placed in the field.

The profile of the driven sheet piling is shown in the north-south and east-west directions. This slope reflects the effect of backfilling the bulkhead at 26E and 26W, but not at Bent 18.

As noted in Section 3.2.1, the horizontal component of batter pile loads due to surcharges at Bent 26 is given in Fig. 13.

3.3 Wharf Movement

The movement of the top of the slope indicator wells is shown in Fig. 17. The bulkhead base line was not established until September 28, 1962. This information, established by optical means (transit and tape), is compared to that of the slope indicator considering the lowest reading as a fixed point. The lowest point varied, depending on where the well broke away from the piling, or where the grooves in the plastic pipe did not line up as at 26E.

3.4 Surcharge Measurements

The manual surcharge measurements were necessary because the output of the capacitor transducer did not give consistent results. The surcharge loading slowly and continuously changes; therefore, a monitor giving only relative results would have been helpful. Data from the capacitor transducer are not included.

IV. ANALYSIS AND DISCUSSION OF RESULTS

4.1 Comparison of Experimental and Theoretical Results

Table 2 and Fig. 4 show the theoretical and experimental results for the sheet piling and anchor loads. For the backfill condition the prototype anchor loads, as expected, are equal to or higher than calculated values. In addition, the maximum bending moment below the cap is 76 percent lower than the lowest theoretical value. Fig. 18 gives the complete data concerning the bending of the sheet piling under backfill conditions at Bent 18. Since the backfill at Bent 18 was placed prior to taking the initial slope indicator readings, the strain gage bending moment data after driving were taken as the unloaded condition. Then the initial slope indicator data were corrected to reflect the change due to backfilling. The slope data from Elev. -10.0 up could not be corrected because after driving the piling an additional length of 4" diameter slope indicator pipe was welded to the driven pipe to bring the top of the pipe to Elev. +12.0. Apparently, the top of the driven pipe was bent to make the extension into the concrete forms, thereby altering the pipe position. From Fig. 18 it may be seen that the points of maximum bending and zero active load are lower than expected.

Fig. 19 shows the reactions of the flexible wall at Bent 18 to the surcharge loadings. In general, the derived imposed loads below the cap show that they are considerably less than those computed by classical methods. In contrast, the experimentally derived loads just below and presumably above the cap are generally much higher than can be accounted for by any method. This results in high sheet piling moments at the cap.

Since analysis of the slope indicator data suggests high loads for the portion of the wall above Elev. 0.0, it would be expected that the anchor pile loads would be even higher than those given by model and field studies. Fig. 17 shows that the horizontal components of the axial load on the batter piles are close to the calculated values for the surcharged condition; however, the bridge support piling could provide considerable resistance to lateral movement in either the north or south direction. The strain gage data from the bridge batter piling at Bent 26 are not complete as only one gage on the outboard flange of outboard pile functioned; therefore, the bending and axial loads could not be determined.

On Figs. 18 and 19 the points of zero load were considerably lower than had been expected from results of model studies (8). The results from the Port of Toledo tests by Hakman and Buser (5) indicated lower hinge (or zero load) points than expected. Because of this and the fact that maximum bending moments were lower than the theoretical values, they concluded that the active pressure was considerably less than assumed. In this study the lower hinge and maximum moment points were probably partially due to fixity of the sheet piling in the pile cap. Several feet of ZP 38 or H-pile embedment into pile caps produces a joint capable of considerable moment transfer (25). Fig. 19 illustrates this point. The highest moments were at the pile cap, but complete fixity was not obtained because the slope curve was not zero at the cap line. The slope at the cap increased with each observation. The point of maximum moment on a beam with uniform loading, which has one end fixed and the other end simply supported, is one-eighth the span closer to the simple support than that of a uniformly loaded simple beam span.

The deflection of the bulkhead at Bent 18 due to the surcharge, as calculated from the smoothed slope curves, are shown in Fig. 19. The deflection of the bulkhead on October 19, 1962, calculated directly from the slope data, is compared to that derived from the smoothed slope curves. Figs. 14, 15 and 16 show the initial slope of the sheet piling and deflection changes at all three sheet pile observation points. The initial slope readings were used as the base readings for these figures. The difference between the initial reading and the July 26, 1962 deflection curve at Bent 18 was not due to a change in load conditions at the bent. The July 26, 1962 deflection shows an almost straight line from the cap down. This is most likely due to the fact that the cap, retaining wall and walkway form an extremely stiff structure with regard to lateral movement. Backfilling was proceeding toward Bent 26 after the July 13, 1962 (initial) reading was made at Bent 18. Therefore, the top of the wharf at Bent 18 moved outward, carrying the sheet piling with it. At Bent 26 the two instrumented sheet piling (26E and 26W) are 6 feet apart. Somewhat similar results were expected because of this close spacing. Fig. 15 shows that the July 26, 1962 slope reading revealed a sheeting deflection after the backfilling was completed, as well as slight movement at the anchor point. The surcharge deflection readings at 26W are qualitatively and quantitatively similar to those at Bent 18. The similarities consist of an apparent southward movement of the cap with sheeting deflection, lower than theoretical points of maximum deflection, and the relative position of the sheeting at each observation date.

The change of deflection in Fig. 16 for instrumented pile 26W shows a decided difference from either Bent 18 or 26E. Apparently, the slope indicator pipes tore away from the sheeting (at Bent 18: -43.0' and at 26W: -40.0') due to driving into hard material. However, the slope indicator well at 26W was on the south (soil) side of the sheeting, while at Bent 18 and 26E the wells were on the north or water side of the sheeting. The east-west profiles of the sheeting show that at Bent 18 and 26E the sheeting is leaning to the east, but at 26W it is leaning toward the west. Normal lean of the sheeting would be toward the west as the driving progressed from east to west, although this is not always the case. In any event, the slope indicator data from 26W are sufficiently different from the data shown for Bent 18 and 26E to indicate that some problem apparently has developed and extreme care must be exercised in using these data. No further analytical use will be made of data from 26W in this study.

4.2 Slope Indicator Well Movement at Elevation +10.8

It was stated above that the concrete structure above the sheet and H-piling provided considerable resistance to relative lateral movement. Also, it very likely resisted rotation above Elev. +2.0 and, due to resistance of the batter piling to northward movement, actually forced the top of the bulkhead to move southward when the sheeting was deflected due to active loads. This is illustrated in Figs. 14 and 15, showing that southerly movement occurred at the anchor point. Fig. 17 shows the comparison between the deflections at Elev. +10.8 determined by the deflections calculated from slope indicator data and the ground survey control. Although the base readings of the two methods of determining well movement differ, the relative movements at Bent 18 and 26E are similar. These movements are in direct contrast to model and other field studies made on flexible walls with tie-back systems. The above-described wall movement may result in high bending moments at the cap and much higher anchor loads and bending moments in the batter piling.

Table 2 gives the stress in the south flange of the sheet piling at Bent 18. From all available data, the stress in the piling at Elev. -45.0 should be very small as the pile at this elevation is much deeper than required for the fixed earth condition. The field data indicate that such is the case, although only information from one flange is available.

4.3 Post-Driving Stresses

The post driving stress condition of the instrumented piling at Bent 18 (as given in Fig. 12) is somewhat surprising. The data were taken within several hours after the pile was driven. The wave equation used in determining stresses while the piling was being driven shows that compressive and tensile stresses do exist during driving. It is possible that at the termination of driving the pile stresses could vary. Another possible cause of moments in the pile is the fact that the instrumented piling is interlocked with the adjacent piling. The adjacent piling may be deformed somewhat due to driving, and each pile driven thereafter would roughly conform to the slope of the preceding piling, thereby introducing moments into the piling. It is assumed that in time the axial loads will dissipate. The axial load at Elev. -55.0 is quite high considering that the piling was driven to Elev. -56.0.

4.4 Discussion of Experimental Data Results

4.4.1 SR4 gages

The SR4 gage installations were intended to produce reliable data for an extended time period -- hopefully more than a year. The zero drift technique was used and gage resistance checks were made (26). As a result, the SR4 readings are satisfactory until the resistance of the SR4 gages falls below 100 megohms.

Sources of error exist when the plane at which measurements are to be made is not accurately laid out, and gages are not placed on the extremities of the shape when axial loads and bending moments are to be determined.

The SR4 gage data used in this study are believed to be as accurate as information obtained from normal structural laboratory studies, assuming proper orientation of the sheet piling. Large errors which can occur result from twisting of the sheet and H-piling during driving. This is a common occurrence and can be detected by the use of rosettes.

4.4.2 Slope Indicator

Slope indicator results are claimed to be satisfactory in determining the stress within a range of 1000-2000 psi when satisfactory tangents can be drawn from the slope curve data (5). In this study, as with the Port of Toledo study, the accuracy of the moment (stress) determination was better where the slope curve did not change rapidly. From the cap down to Elev. -10.0 the accuracy of the smoothed slope curve is probably one-half that between Elevs. -12.0 and -22.0. Regardless of this quantitative inaccuracy, the qualitative indications such as slope direction are unmistakable.

It is possible that when the plastic inner guide casing was inserted into the wells some rotation may have occurred. This could, of course, result in considerable inaccuracy if the rotation were more than a few degrees. In this study Ottawa sand was vibrated into the annular space between the casing and the 4" diameter pipe. If the sand did not completely fill the void a limited accuracy problem may result because the outside diameter of the plastic casing is 3-1/4" which leaves little room for movement within the 4" diameter pipe.

The use of a torsion device to determine the rotation of the inner casing may be necessary to eliminate this problem.

4.4.3 Surcharge pressure gages

The malfunction of these gages experienced in this study are probably due more to inadequate gage preparation than anything else. The small surface of the transducers (1 cm^2) will react to pressure from one sand grain. Since intergranular pressure, as determined from a few sand grains, is rarely indicative of the pressure on a square foot, a much larger sensing element should have been used.

4.4.4 Ground control

Ground controls were not established for the slope indicator wells until the backfilling was complete. The control consisted of placing a transit near the shore (east) end of the coal field extension along the walkway in front of the bulkhead retaining wall and sighting on a point at the west end of the bulkhead. The point at the west end of the bulkhead was established by chaining a set distance from the west end of the southerly retaining wall to a point in front of the bulkhead retaining wall. The chained distance was determined by using the standard methods of constant chain tension and temperature correction.

These measurements are probably accurate to 1/16".

V. SUMMARY AND CONCLUSIONS

This study, conducted on a somewhat unusual type of wharf, shows that information developed over the past 20 years on model and full-scale tie-back type bulkheads is far from complete. In keeping with results of other recent tests, the sheet piling moments are considerably reduced and anchor loads increased for backfilled condition compared to those calculated by older methods. There are indications of very high loads behind the concrete retaining wall and cap, possibly due to the movement of the concrete wall into the surcharge and fill, even though the batter pile loads under surcharge did not show this. Additional slope indicator readings will be made in the near future to determine the changes that may have taken place since 1963.

The combination of strain gages and slope indicators provides an excellent system for enabling investigators to describe bulkhead loads, moments and movements. Pressure gages against the bulkhead-soil interface are desirable, as are methods for determining soil movement in front of and behind the bulkhead.

Results of this and all other field studies in granular soils show that wall friction should be used in the computation of active bulkhead pressures.

This study has produced some meaningful information. Only several field tests have been conducted to determine bulkhead resistance without surcharges. Since this is the only known study concerning surcharge effects on bulkheads, additional analysis of the slope indicator data at 26E and 26W will be made.

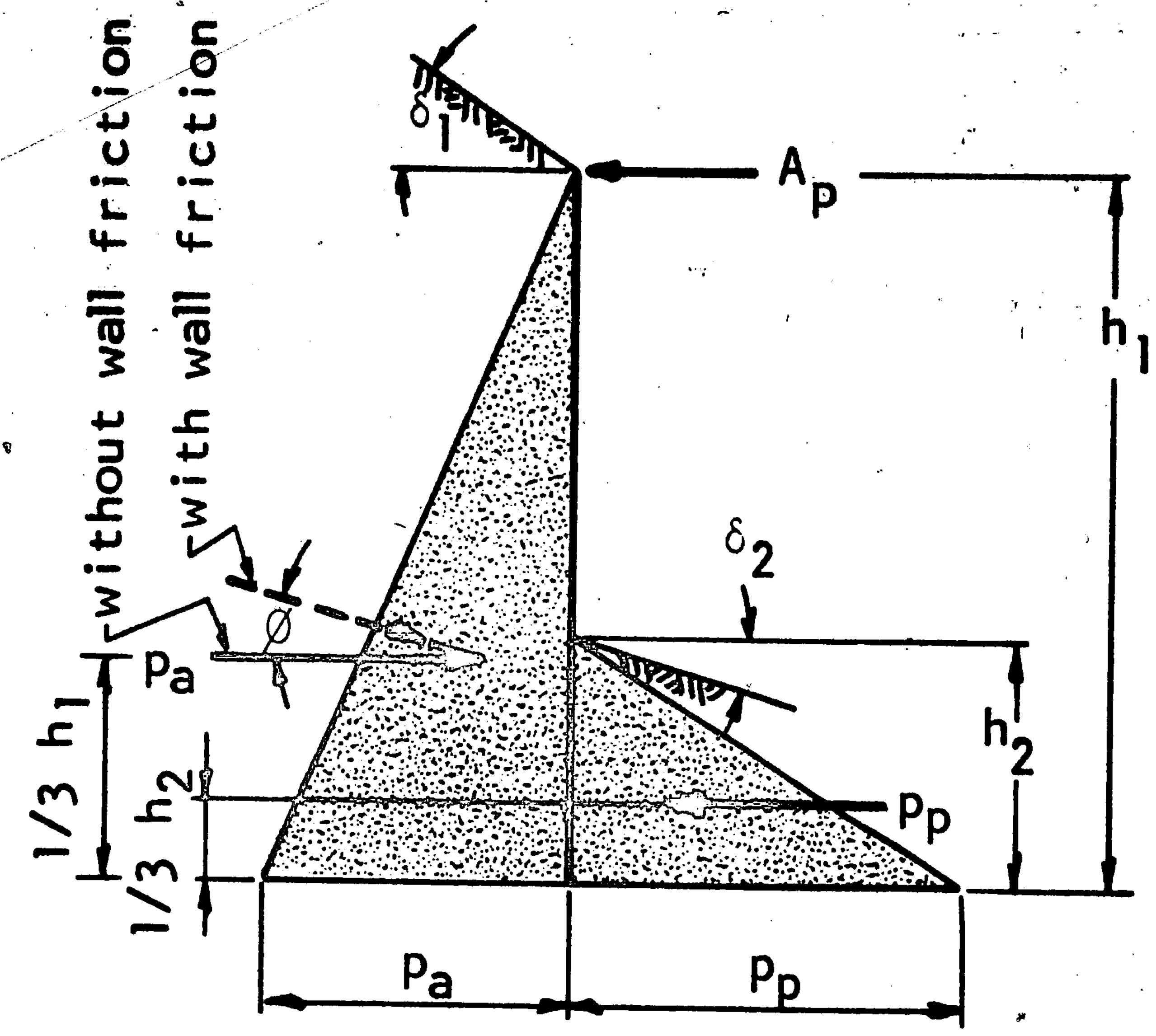
In conclusion, it is felt that Rowe's and Tschebotarioff's work on bulkheads is only the beginning in the effort to design economical bulkheads. The nonlinear subgrade reaction method of design, along with satisfactory soil property data, may be a proper tool to use in the development of design methods.

VI. NOMENCLATURE

θ	∠ of internal friction
ϕ	∠ of external or wall friction
δ_1	∠ of surcharge
δ_2	∠ of sloping soil in front of bulkhead
ρ	Rowe's flexibility number
Δ	Deflection of sheet piling
A_p	Anchor load
a	Distance from anchor or top of active pressure to bottom of active pressure
b	Distance from bottom of active pressure to bottom of sheeting
c	Distance from dredge line or soil in front of bulkhead to bottom of active pressure
E	Young's modulus
I	Moment of inertia
M.L.W.	Mean low water
M_{design}	Maximum positive moment for design of sheet piling
M_{max}	Maximum positive moment in sheet piling computed by free earth support method
p_a	Unit active pressure
p_p	Unit passive pressure
P_a	Total active pressure
P_p	Total passive pressure
R	Reaction at bottom of active pressure
t	Distance between top of bulkhead at the anchor and tangent to bottom of bulkhead
w	Unit weight of soil
w_s	Submerged unit weight of soil

VII. APPENDIX

7.1 Rankine-Coulomb Formulas for Earth Pressures in Granular Soils



pa = unit active pressure

pp = unit passive pressure

δ_1 = ∠ of surcharge

δ_2 = ∠ of sloping soil in front of bulkhead

θ = ∠ of internal friction

Ø = ∠ of wall friction

w = unit weight of soil

P_a = total active pressure

P_p = total passive pressure

h_1 = total height of soil in active zone

h_2 = total height of soil in passive zone

A_p = anchor

Unit Active Pressures

For Level Top Surface: $\delta_1 = 0$

Without wall friction: $$p_a = wh_1 \left(\frac{\cos\theta}{1 + \sin\theta}\right)^2 = wh \tan^2(45° - \frac{\theta}{2})$$

With wall friction: $$p_a = wh_1 \frac{\sin^2(90° - \theta)}{\cos\varnothing \left[1 + \sqrt{\frac{\sin(\theta + \varnothing)\ \sin\theta}{\cos\varnothing}}\right]^2}$$

For Surcharge:

Without wall friction: $$p_a = wh_1 \frac{\sin^2(90° - \theta)}{\left[1 + \sqrt{\frac{\sin\theta\ \ \sin(\theta - \delta_1)}{\cos\delta_1}}\right]^2}$$

$$= wh_1 \left[\frac{\cos\theta}{1 + \sqrt{\sin\theta(\cos\theta - \cos\theta\ \tan\delta_1)}}\right]^2$$

With wall friction: $$p_a = wh_1 \frac{\cos^2\theta}{\cos\varnothing \left[1 + \sqrt{\frac{\sin(\theta + \varnothing)\ \sin(\theta - \delta_1)}{\cos\varnothing\ \ \cos\delta_1}}\right]^2}$$

Unit Passive Pressures

For Inclined Surface in Front of Bulkhead:

Without wall friction: $$p_p = wh_2 \left[\frac{\cos\theta}{1 - \sqrt{\sin\theta(\sin\theta - \cos\theta \tan\delta_2)}}\right]^2$$

7.2 Methods of Calculating Sheet Pile Penetration and Moments

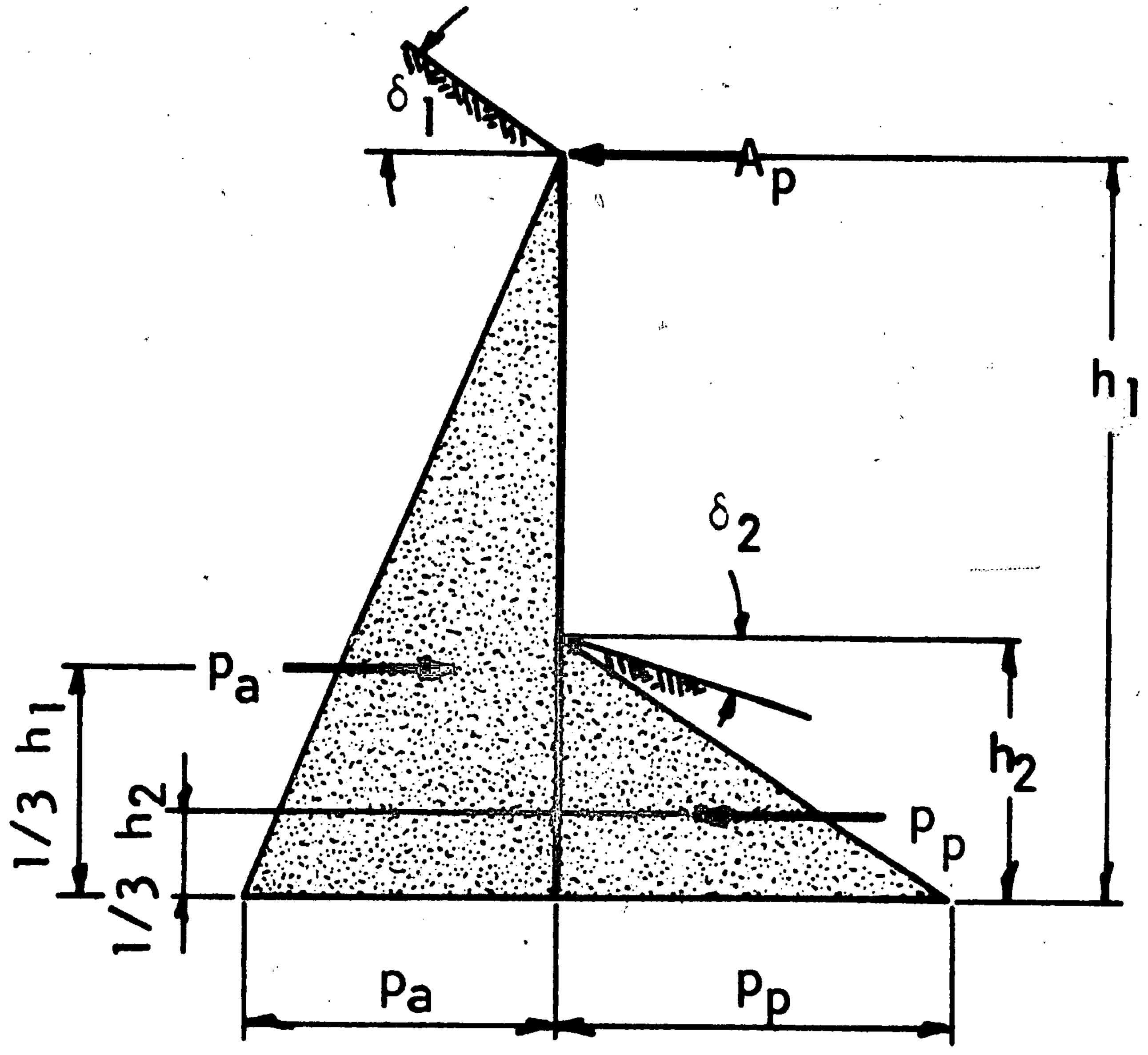
Free Earth Support:

Take moments about A_p for minimum bulkhead length.

$$2/3\ h_1\ P_a = (h_1 - 1/3\ h_2)P_p$$

Anchor load $A_p = P_a - P_p$

Maximum moment at point of zero shear

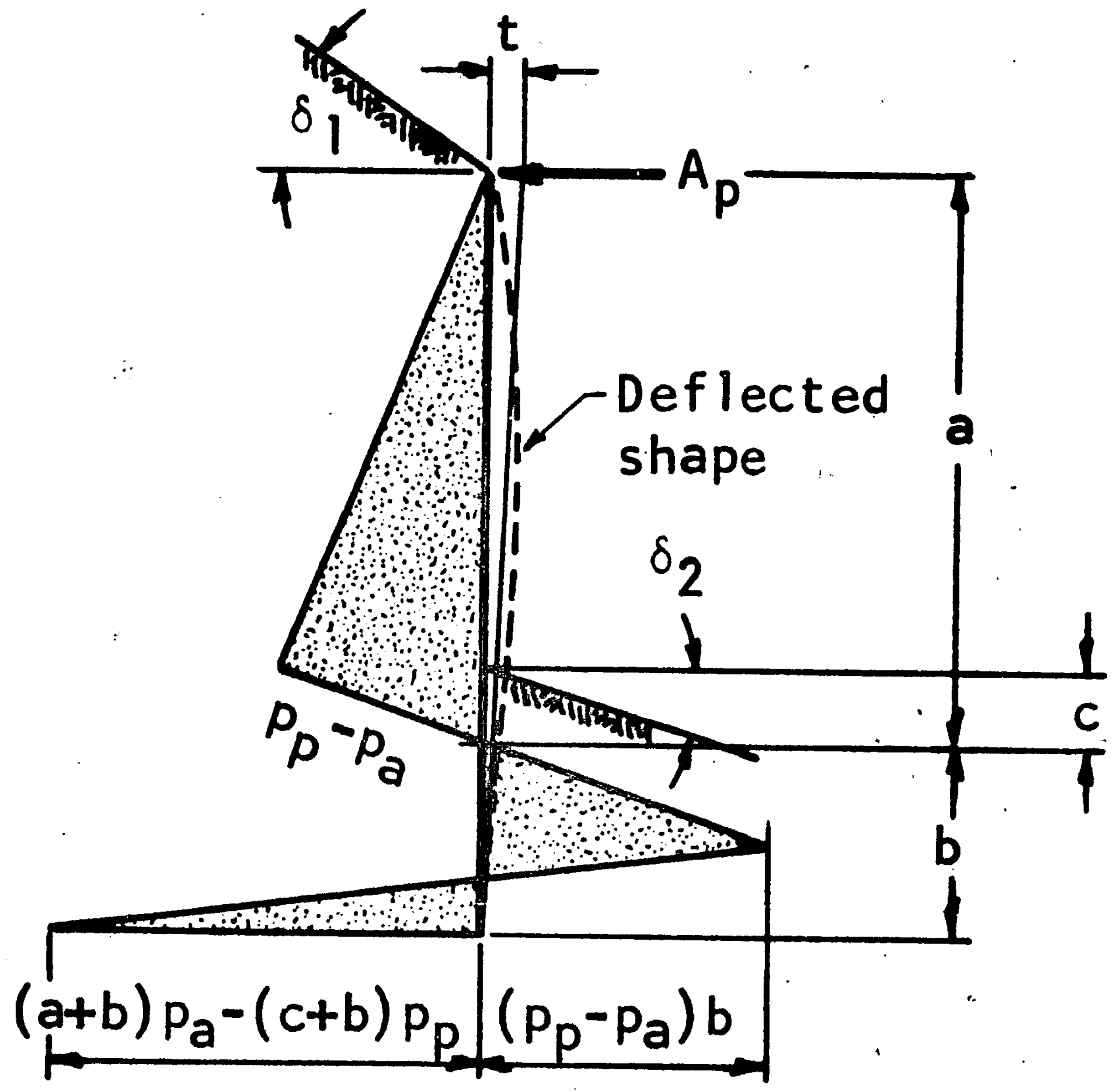


Fixed Earth Support:

Most economical depth when t = 0 (completely fixed)

Anchor load A_p = difference between active and passive pressures

Maximum moment at point of zero shear



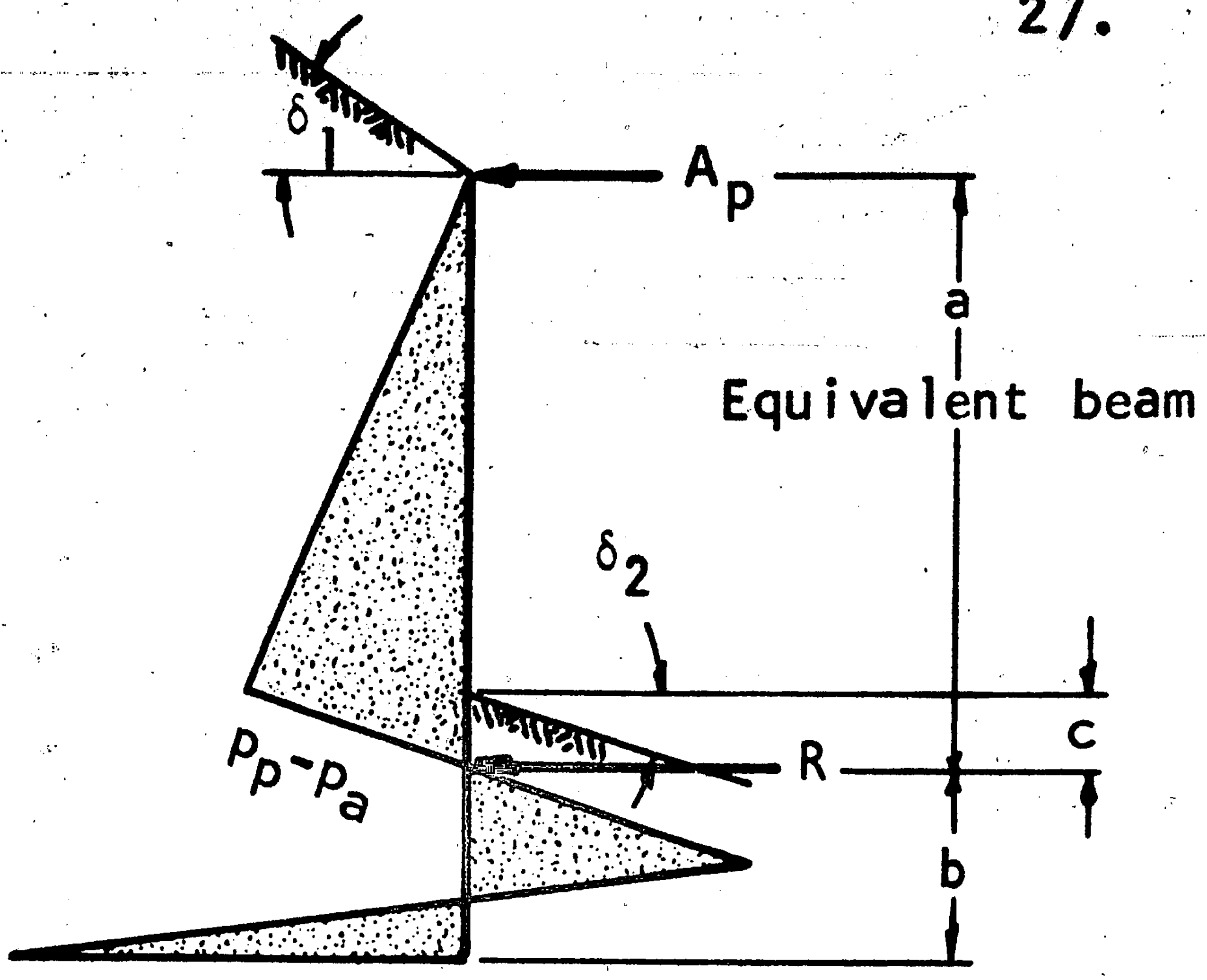
Equivalent Beam:

Most economical depth when

$$b + c = 1.1\left(c + \sqrt{\frac{6R}{P_p - P_a}}\right)$$

Anchor load A_p = active load - R

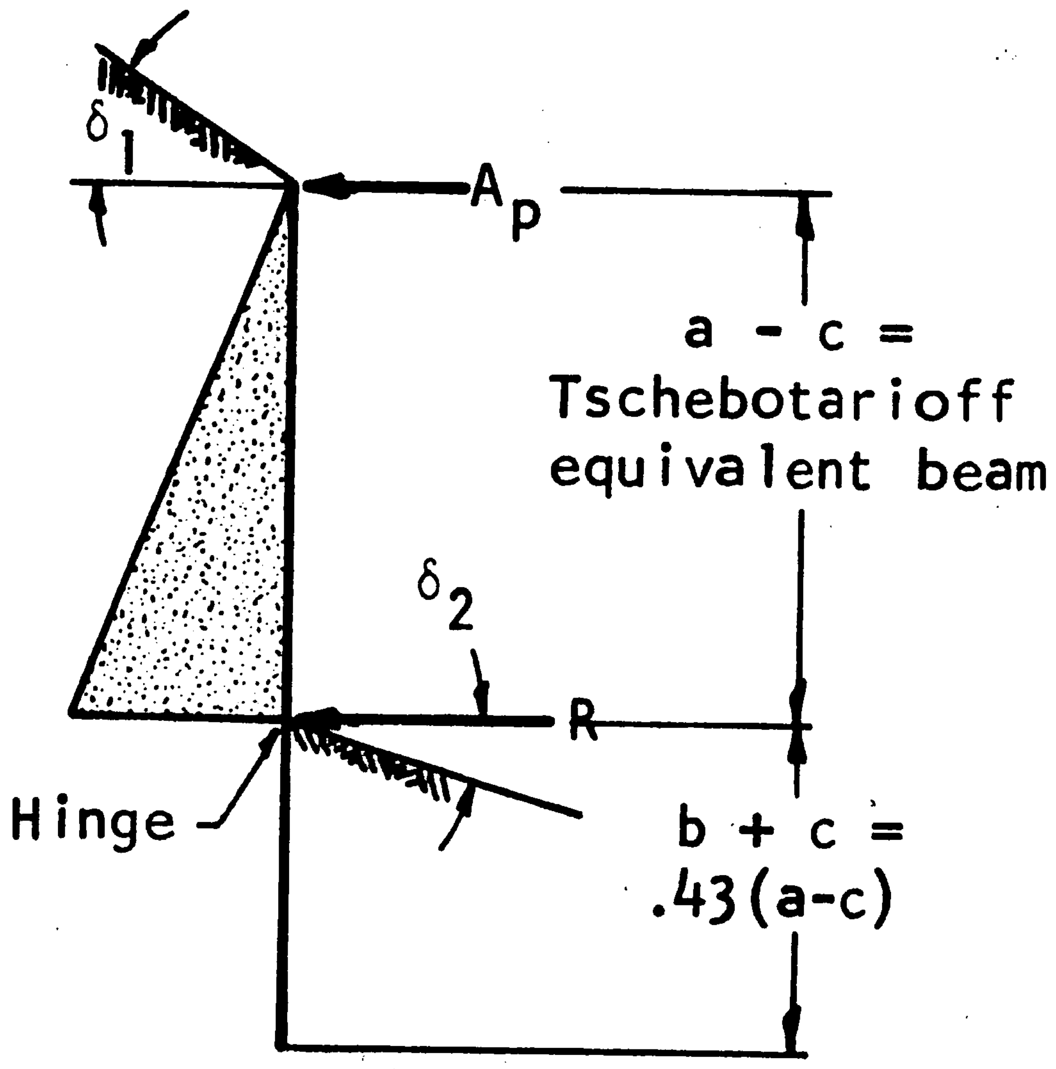
Maximum moment at point of zero shear



Tschebotarioff Equivalent Beam:*

Span taken as distance from anchor (A_p) to dredge line for moment determination.

Anchor load A_p = active load above dredge line - R



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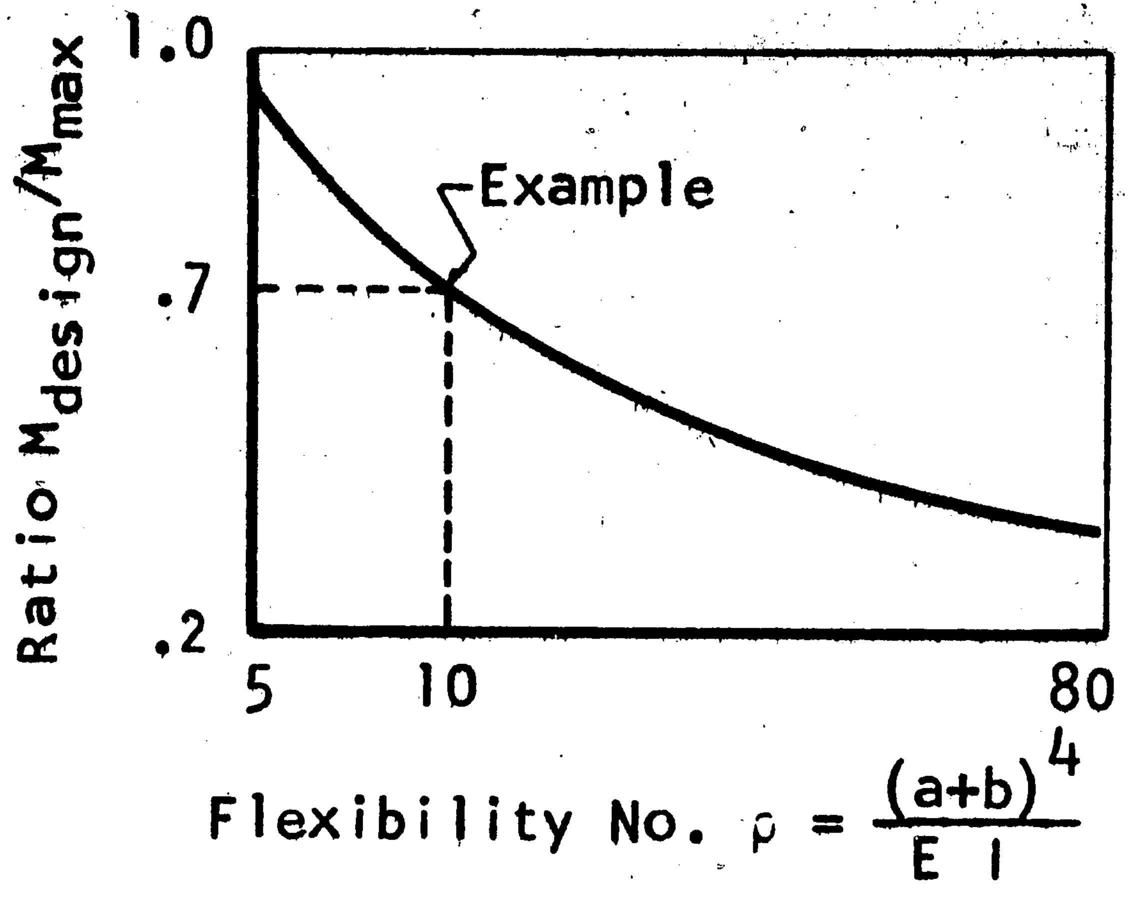
* Tschebotarioff used a level dredge line in his work and the position of R for inclined dredge line may have to be moved down slightly.

Rowe's Flexibility Method:

Calculate bending moments and anchor load by free earth support method and then reduce the moment due to sheet piling flexibility according to chart at right.



Example:

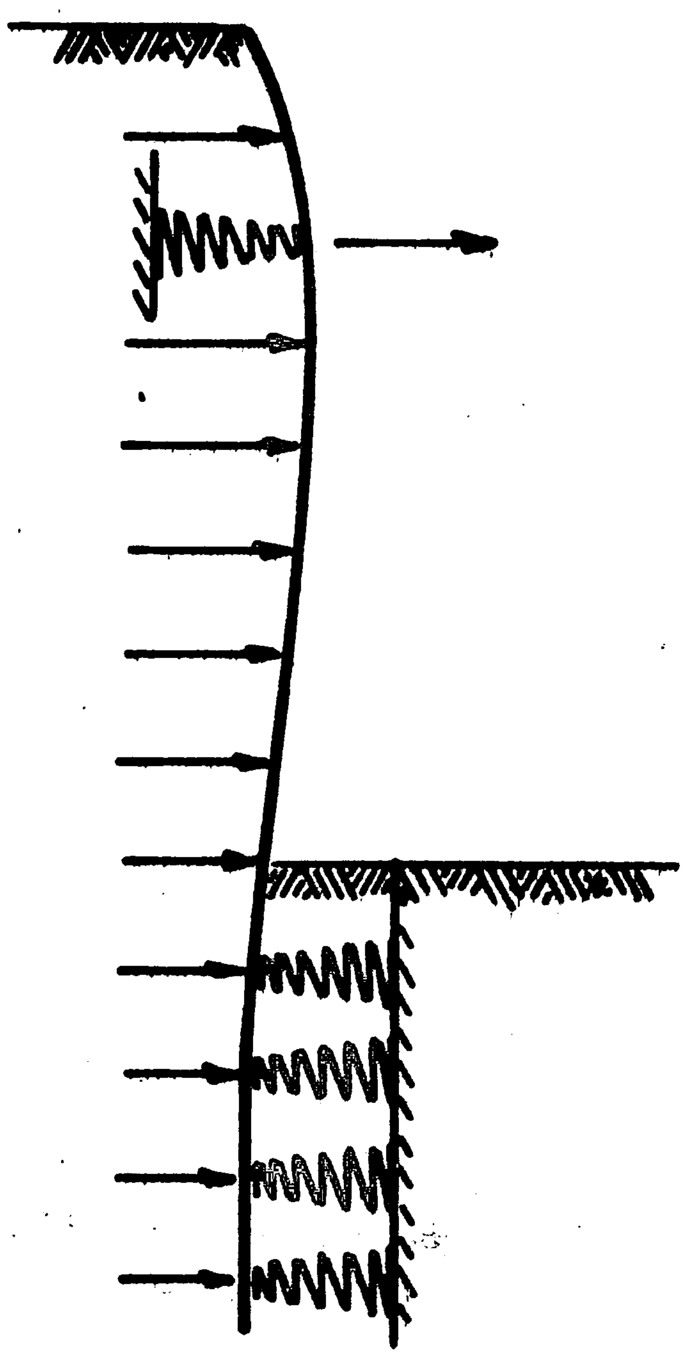
If $\frac{M_{design}}{M_{max}} = .7$

then $M_{design} = .7\ M_{max}$

Nonlinear Subgrade Reaction Method:

Differential equation for a beam supported by soil:

$$E\ I\ \frac{d^4y}{dx^4} - P_a + P_p = f(x_1y)\ *$$



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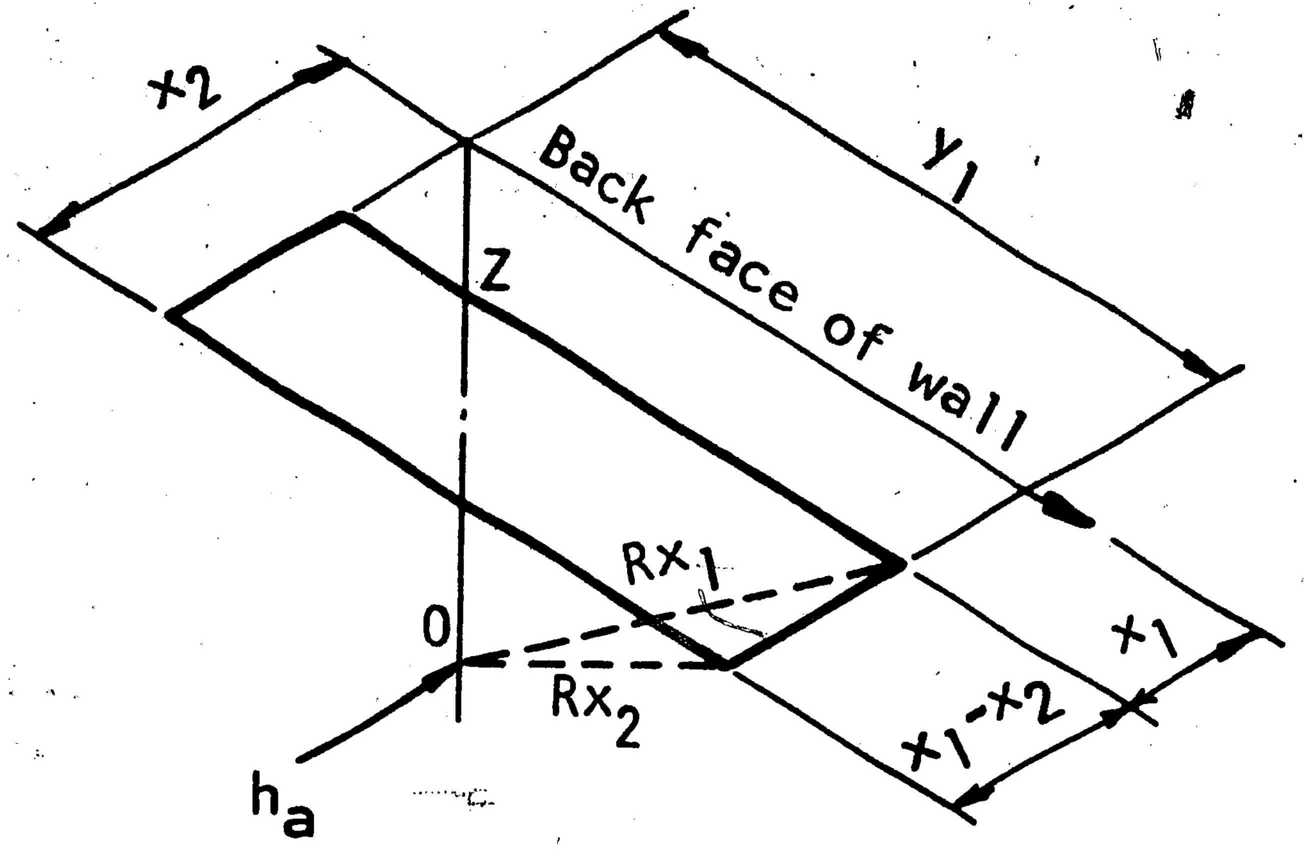
* This requires a digital computer using numerical procedures to solve the 4th order difference equations and proper definition (nonlinear) of P_a and P_p

7.3 Spangler (Boussinesq) Equation for Area Load of Finite Length

At point 0:

$$h_a = \frac{d}{3}\left[\arctan\frac{x_2\, y_1}{Z\, Rx_2} - \frac{x_2\, y_1\, Z}{(x_2^2 + Z^2)\, Rx_2} - \arctan\frac{x_1\, y_1}{Z\, Rx_1} + \frac{x_1\, y_1\, Z}{(x_1^2 + Z^2)\, Rx_1}\right]$$



Cross section of surcharge is broken up into sections of $x_1 - x_2$ width and y_1 length. Pressures from each element are added together to get total pressure at any distance and below surcharge.

7.4 Calculation of Moment of ZP 38 Sheet Piling from Slope (Dial Unit) Curve

$$m = E\, I\, \frac{d^2y}{dx^2}$$

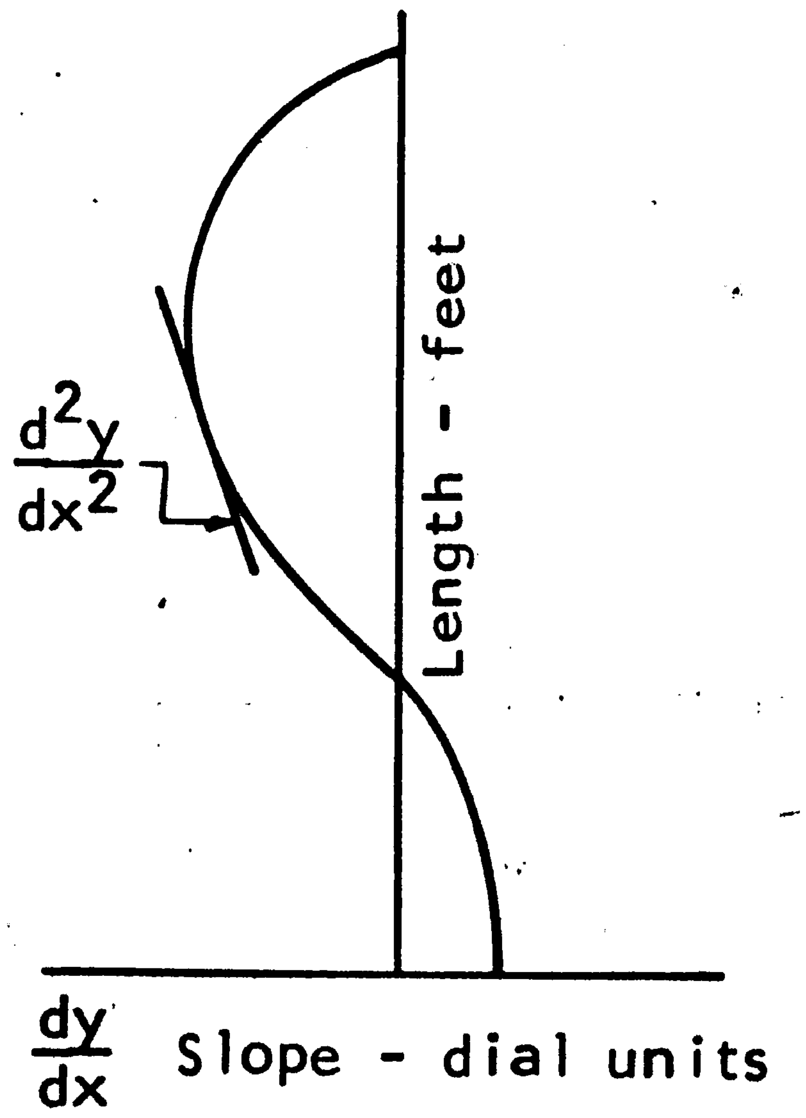
$$= \frac{30 \times 10^6 \times 421.2}{1.5 \times 12 \times 1000 \times 7200}\ \frac{d^2y}{dx^2}$$

(slope of slope curve)

$$= 97.5 \times \frac{d^2y}{dx^2} \text{ ft-kips/ft of wall}$$

where 7200 is an instrument constant and slope curve is laid out on dial unit (abscissa) and foot (ordinate) basis.

Typical Slope Curve



7.5 Calculation of Deflection from Slope Indicator Data

Average distance between slope readings is 20 inches.

Calculations referenced to the bottom of the sheeting and dial readings are summed from the bottom to the top.

$$\Delta = \frac{\Sigma \text{ dial readings} \times 20}{7200}$$

VIII. TABLES AND FIGURES

TABLE 1

THEORETICAL AND EXPERIMENTAL MAXIMUM BULKHEAD MOMENTS BELOW CAP AND ANCHOR LOADS

(determined by various methods of calculation)

	Fixed Earth Support	Free Earth Support	Rowe's(1) Modified Free Earth Support	Equivalent Beam	Experimental
Maximum Moment at Bent 18 Below Cap (ft-kips/ft)					
Backfilled	16.4 at - 9.3'	23.3 at -10.8'	15.8	17.3 at - 9.5'(2)	12.8 at -11.3'
Backfilled and surcharged	48.5 at -11.3'	84.2 at -14.0'	56.4	54.6 at -11.6'	
Surcharge only	33.1 at -13.0'			35.1	14.3 at -20.0'
Anchor Loads (kips/ft)					
Backfilled	2.7	3.2		2.7(3)	3.1 to 5.2(4)
Backfilled and surcharged	10.9	13.3		11.3	

(1) Flexibility No $\rho = \frac{(a + b)^4}{E\ I} = \frac{[25 + 20]^4 \times 12^4}{30 \times 10^6 \times 280.8} = 10$ in.^{2}/lb

(2) If wall friction included, moment is 13.4 ft-kips

(3) If wall friction included, anchor load is 2.2 kips

(4) At 26E and 26W

TABLE 2

STRESS CHANGE ON SOUTH FLANGE OF ZP 38 SHEET PILING AT ELEV. -45.0 BENT 18

Date	Load Condition	Stress (psi)
5-4-62	Suspended from crane	0
5-23-62	After driving	+ 30
7-16-62	Bulkhead filled, no surcharge	- 208
7-26-62	Bulkhead filled, no surcharge	- 209
3-19-63	Surcharged	- 1400
5-17-63	Surcharged	0

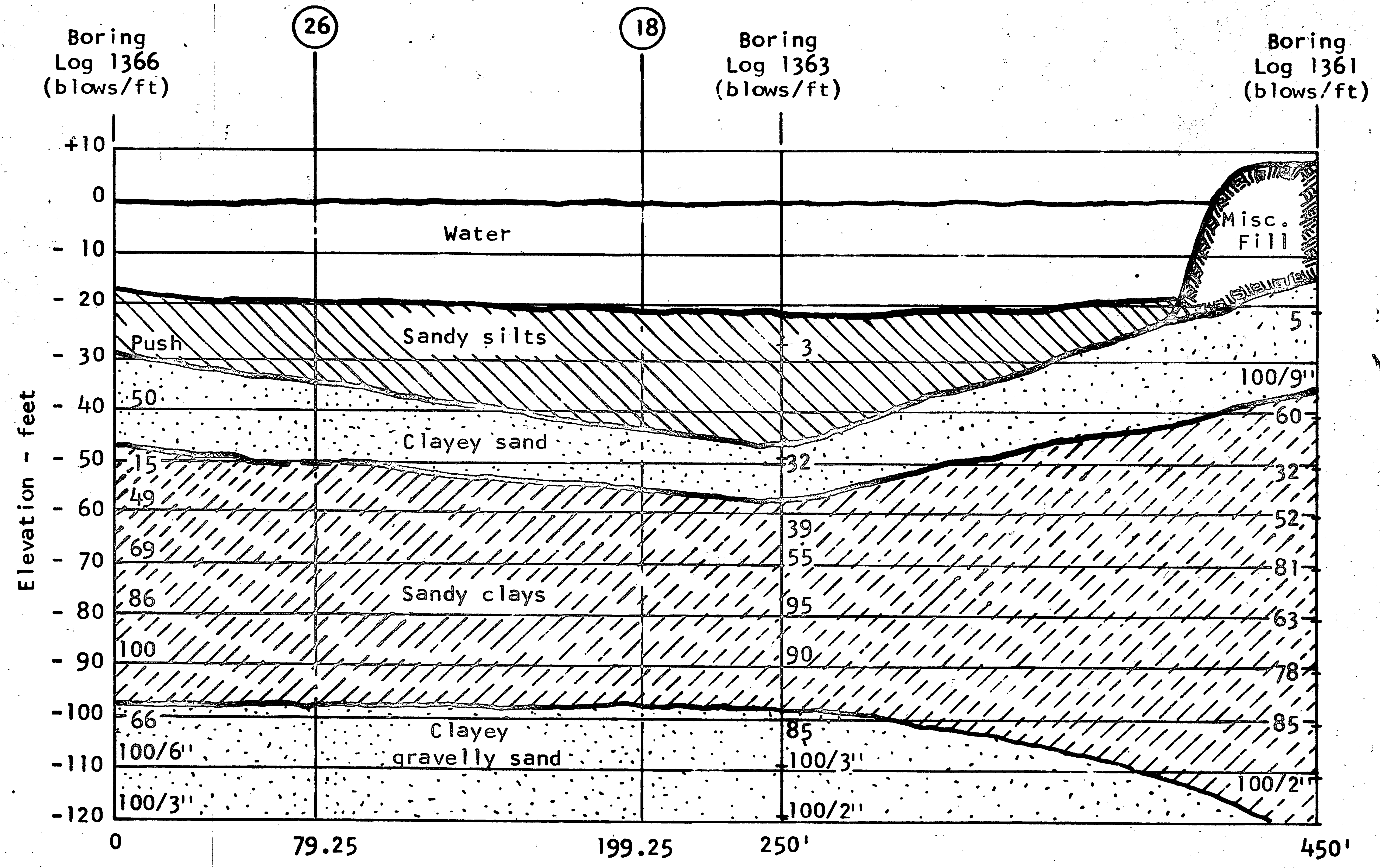


Fig. 1 - Soil Profile on Bulkhead Centerline

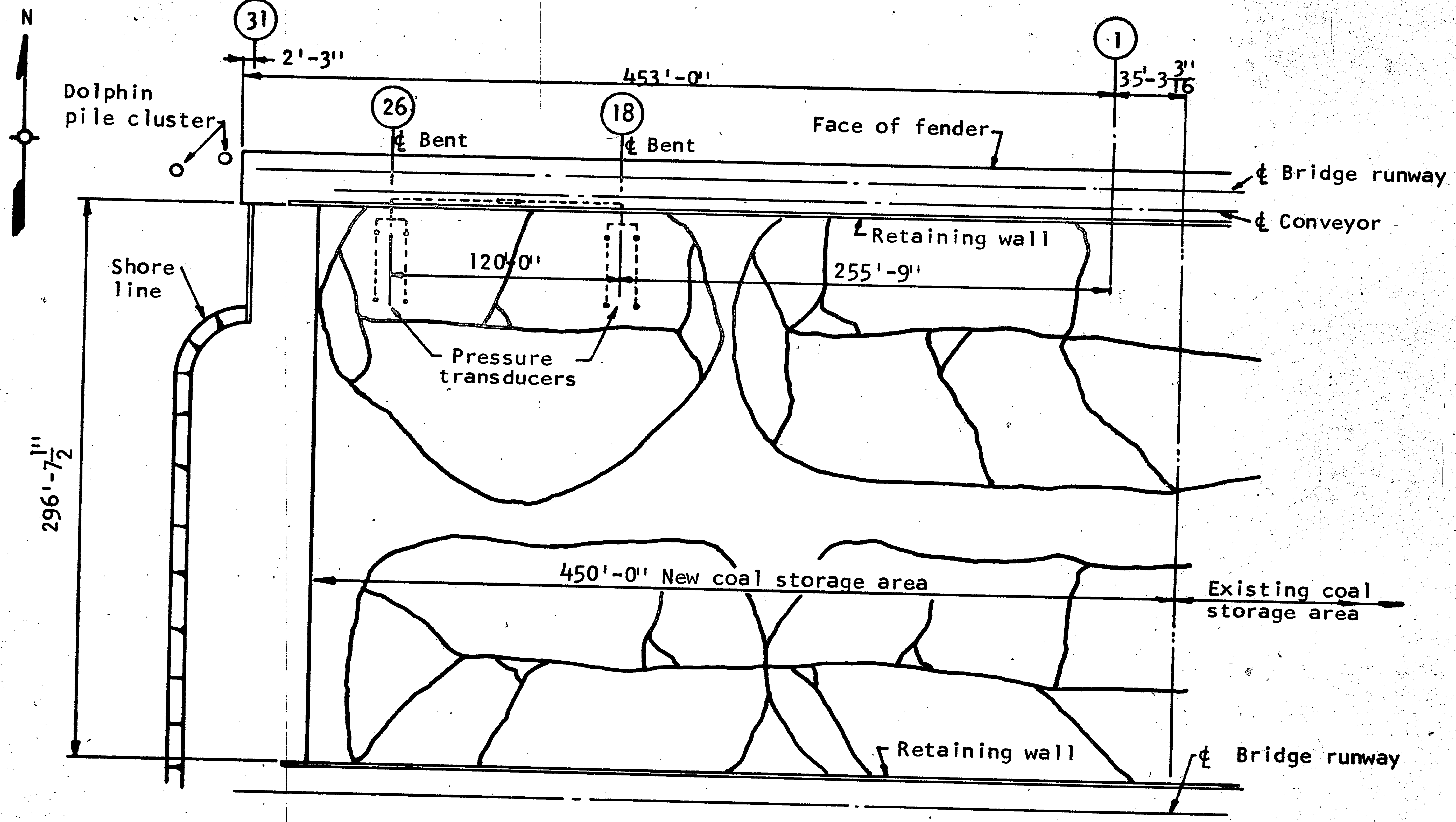
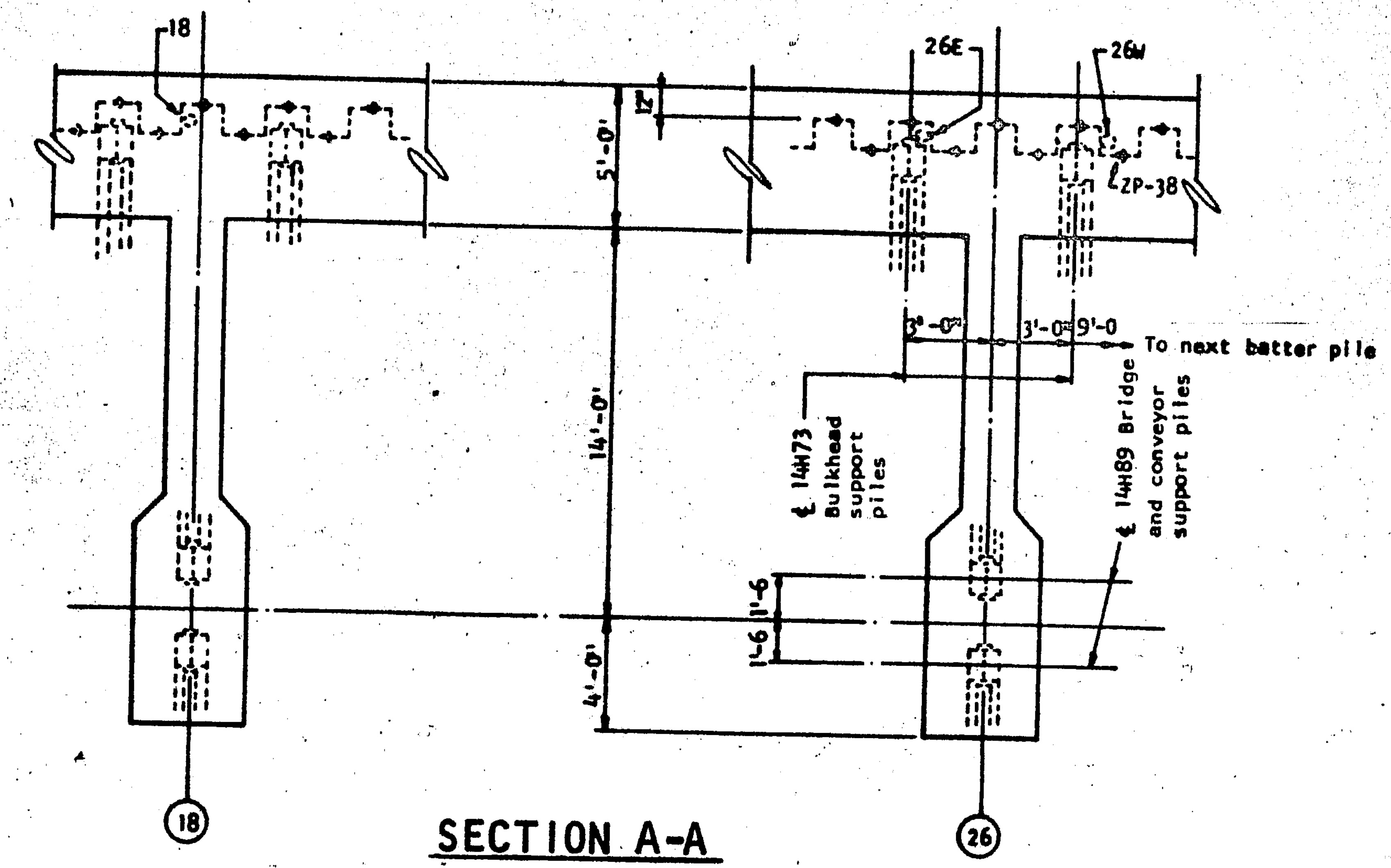


Fig. 2 - Plan of Coal Field Extension



SECTION A-A

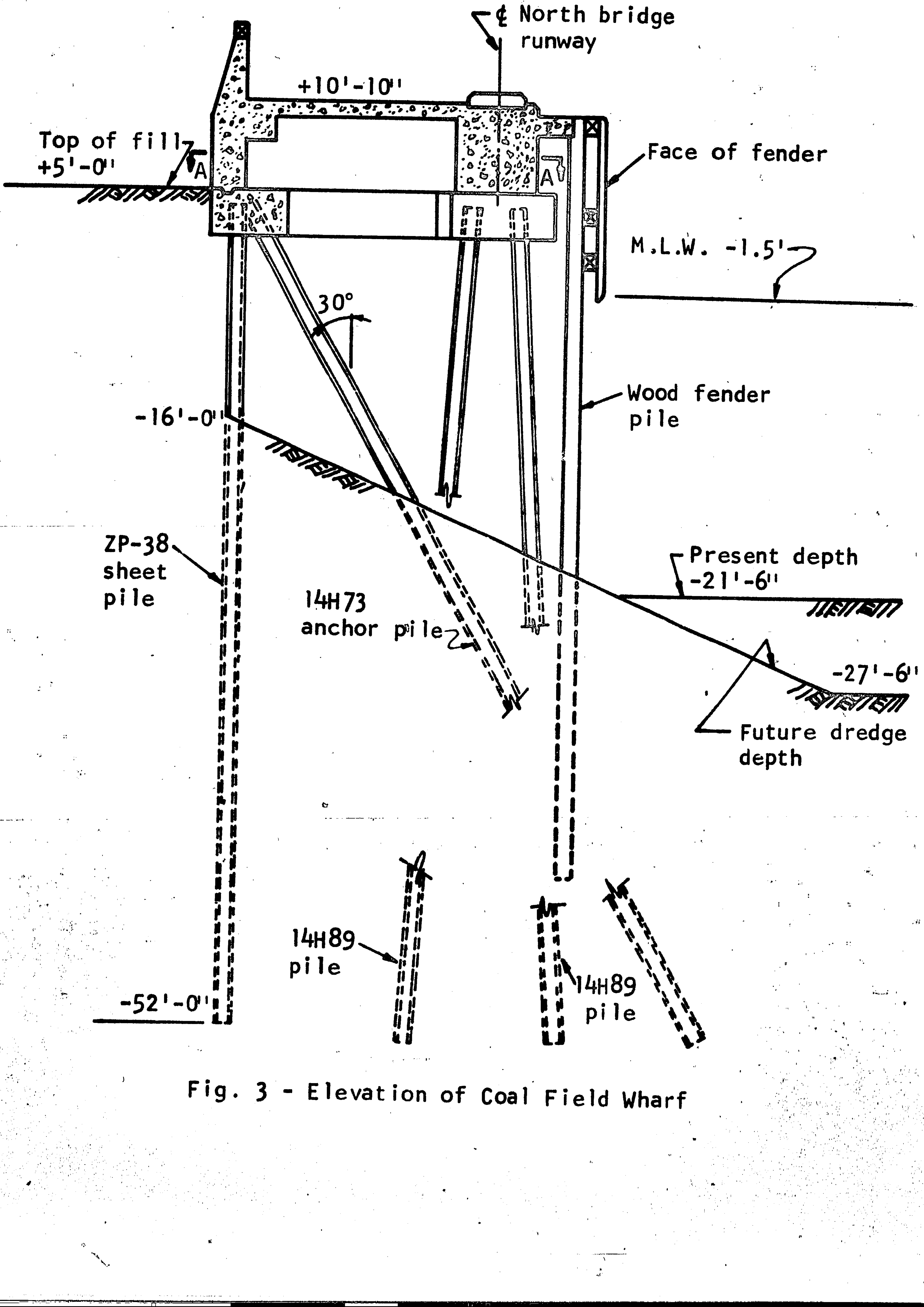


Fig. 3 - Elevation of Coal Field Wharf

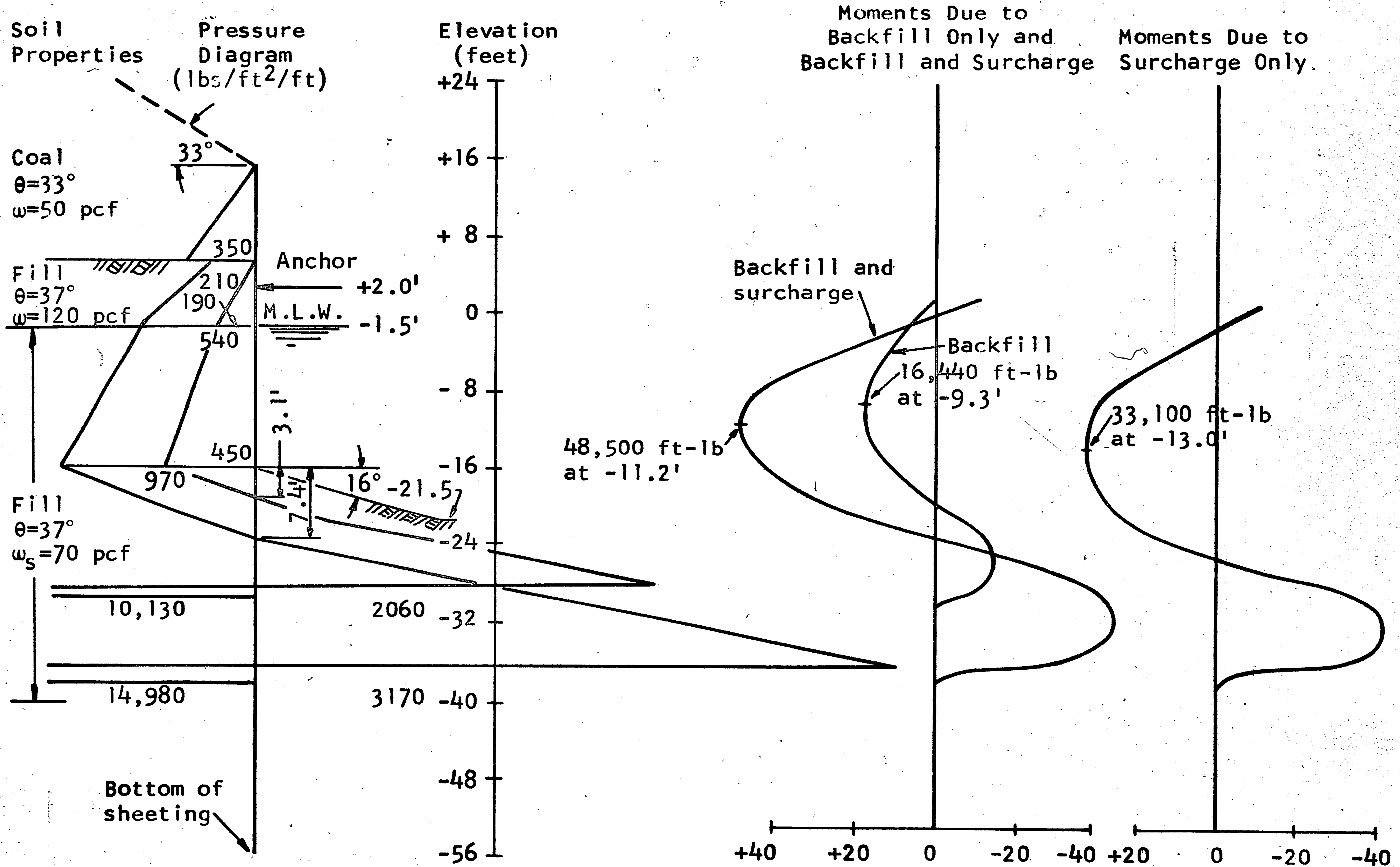
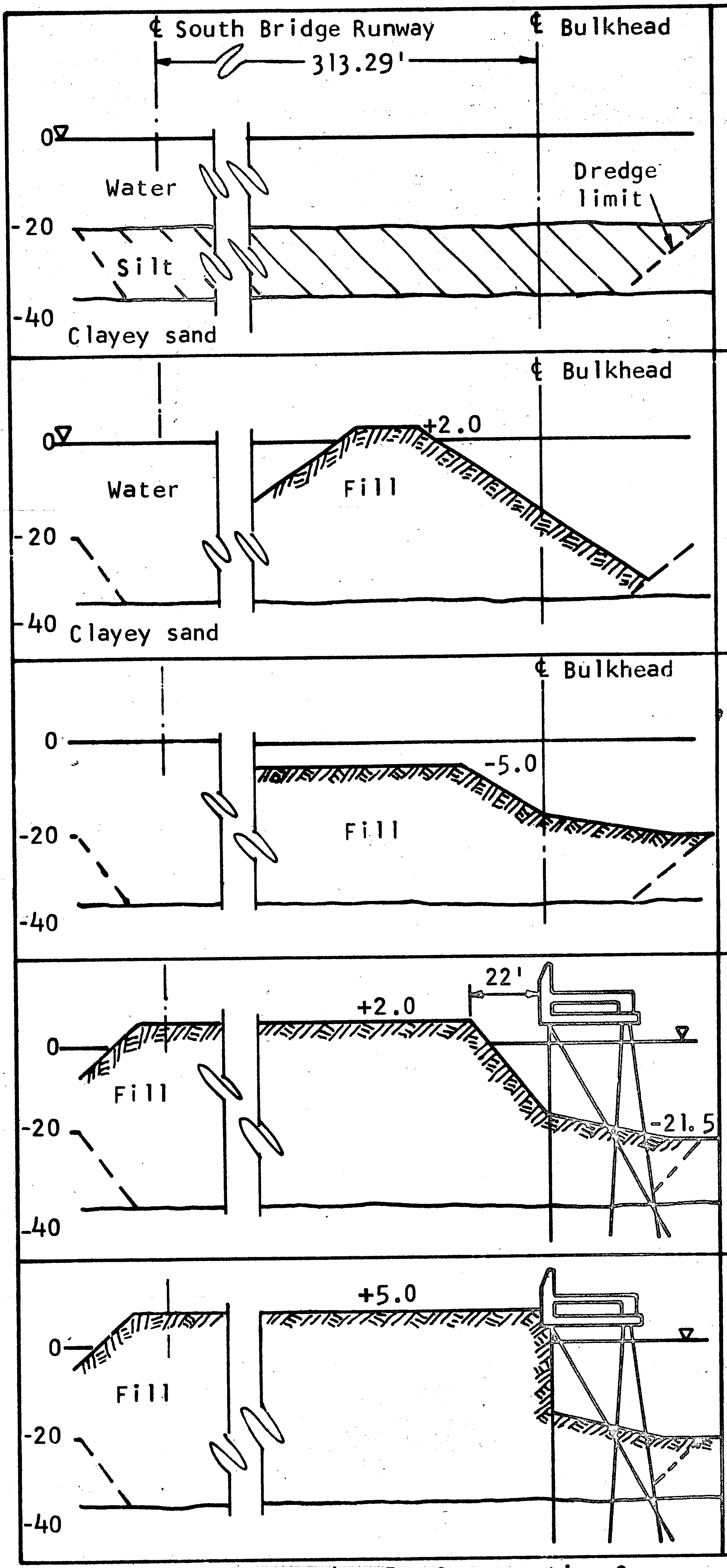


Fig. 4 - Theoretical Loads and Moments for Bulkhead



STEP 1

Mud is dredged from field, channel and bulkhead areas

STEP 2

Miscellaneous fill placed behind bulkhead centerline

STEP 3

Miscellaneous fill excavated to Elevation -5.0'

STEP 4

Piling driven, concrete cap, deck and wall placed. Fill brought up to Elevation +2.0'

STEP 5

Remainder of fill placed behind bulkhead and in coal field

Fig. 5 - Construction Sequence

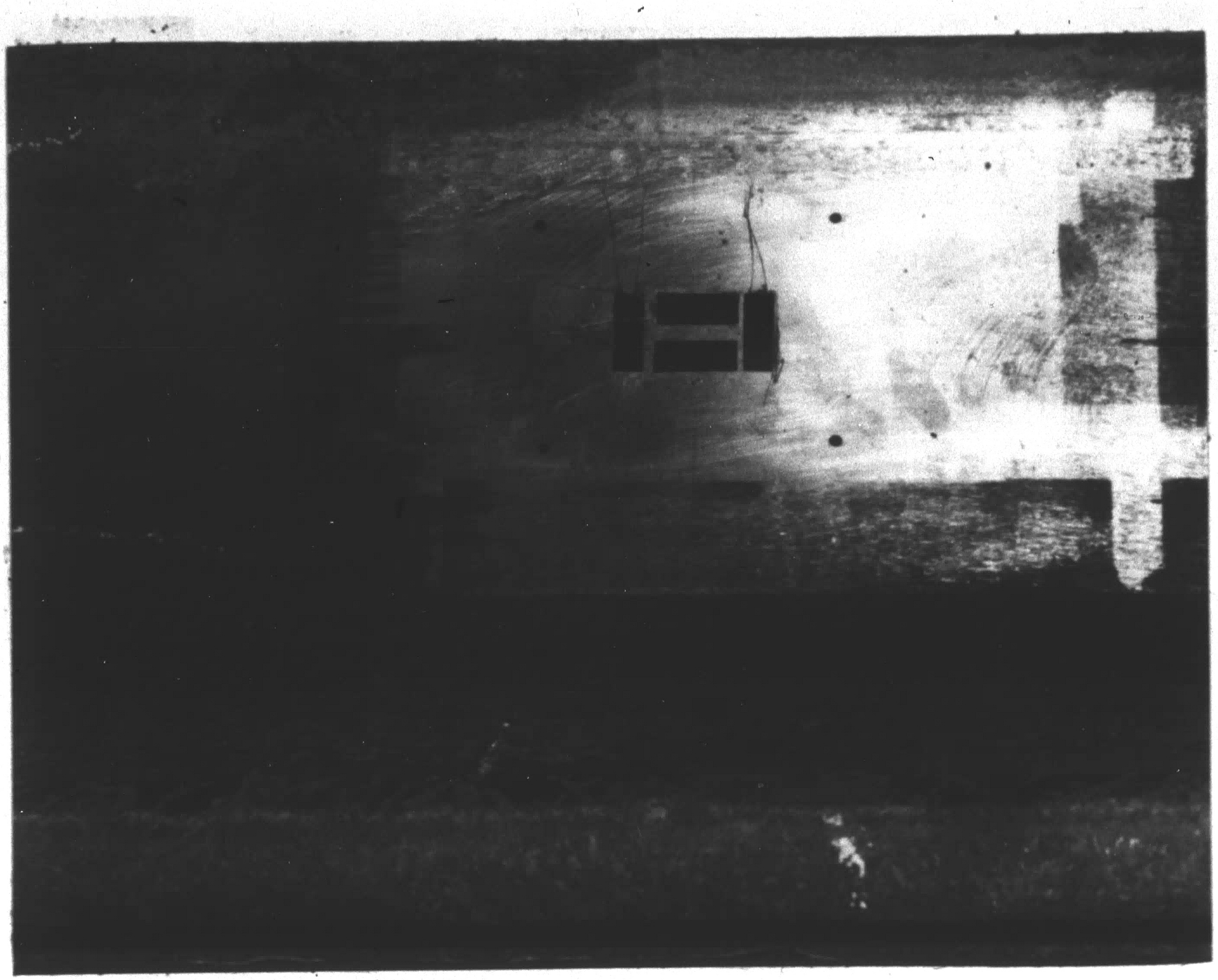
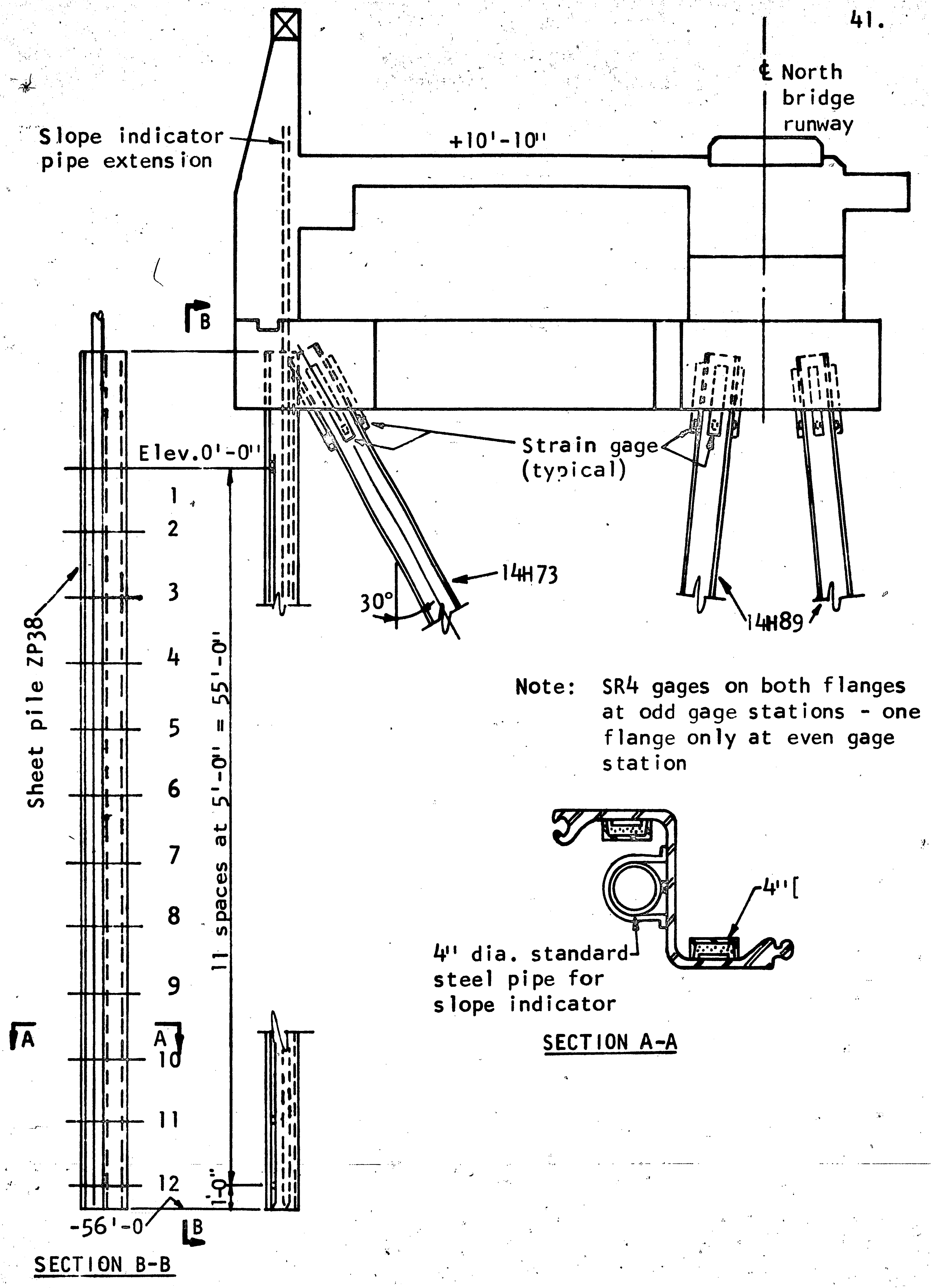


Fig. 6 - SR4 External Bridge on Flange of ZP 38 Sheet Piling

Fig. 7 - Waterproofing of SR4 Gages

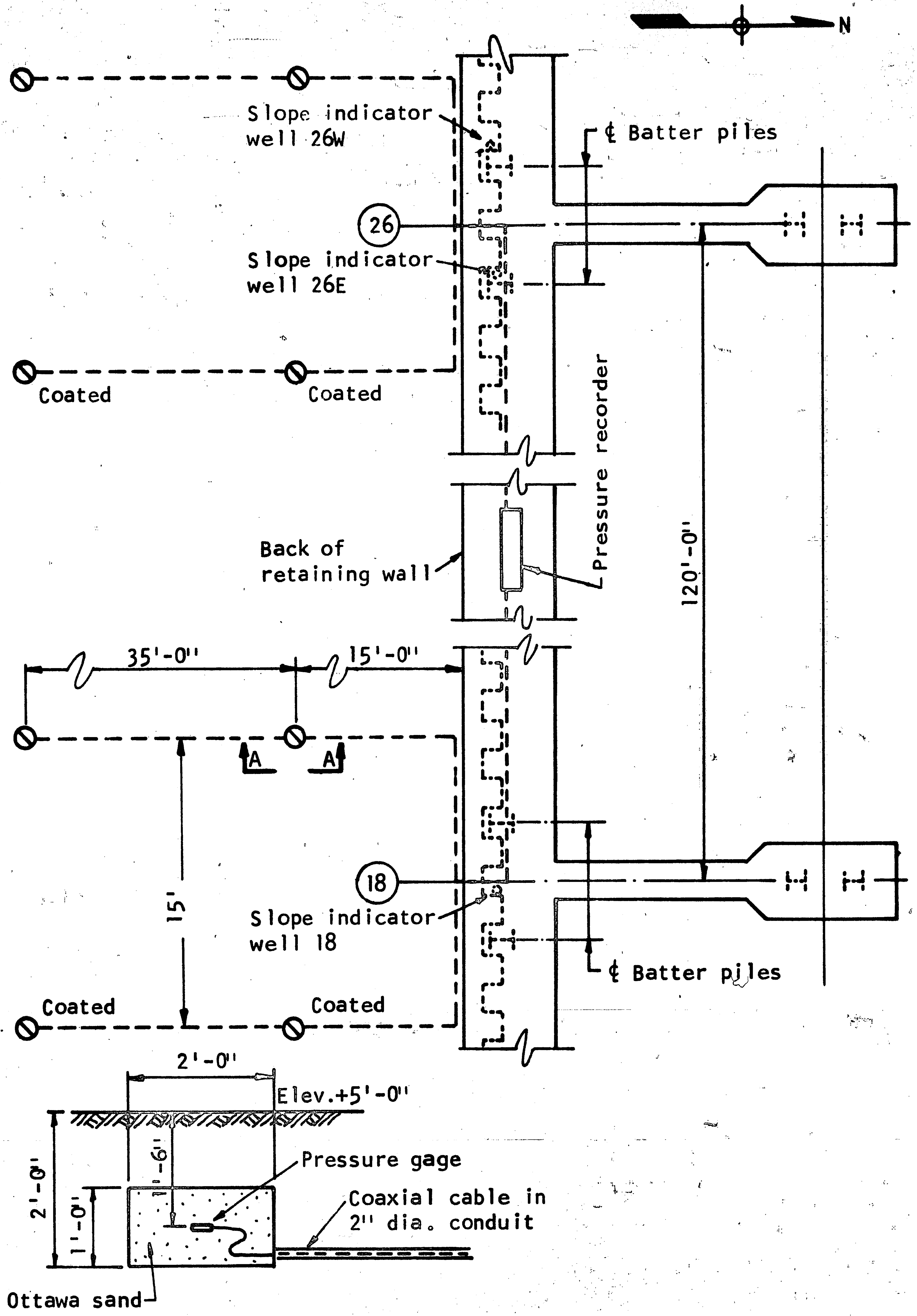
Fig. 8 - Clamped, Shielded SR4 Gage Lead Wires and Protective Channel

Fig. 9 - Channels Over SR4 Gages and Slope Indicator Pipe at Bottom of Instrumented Sheet Piling



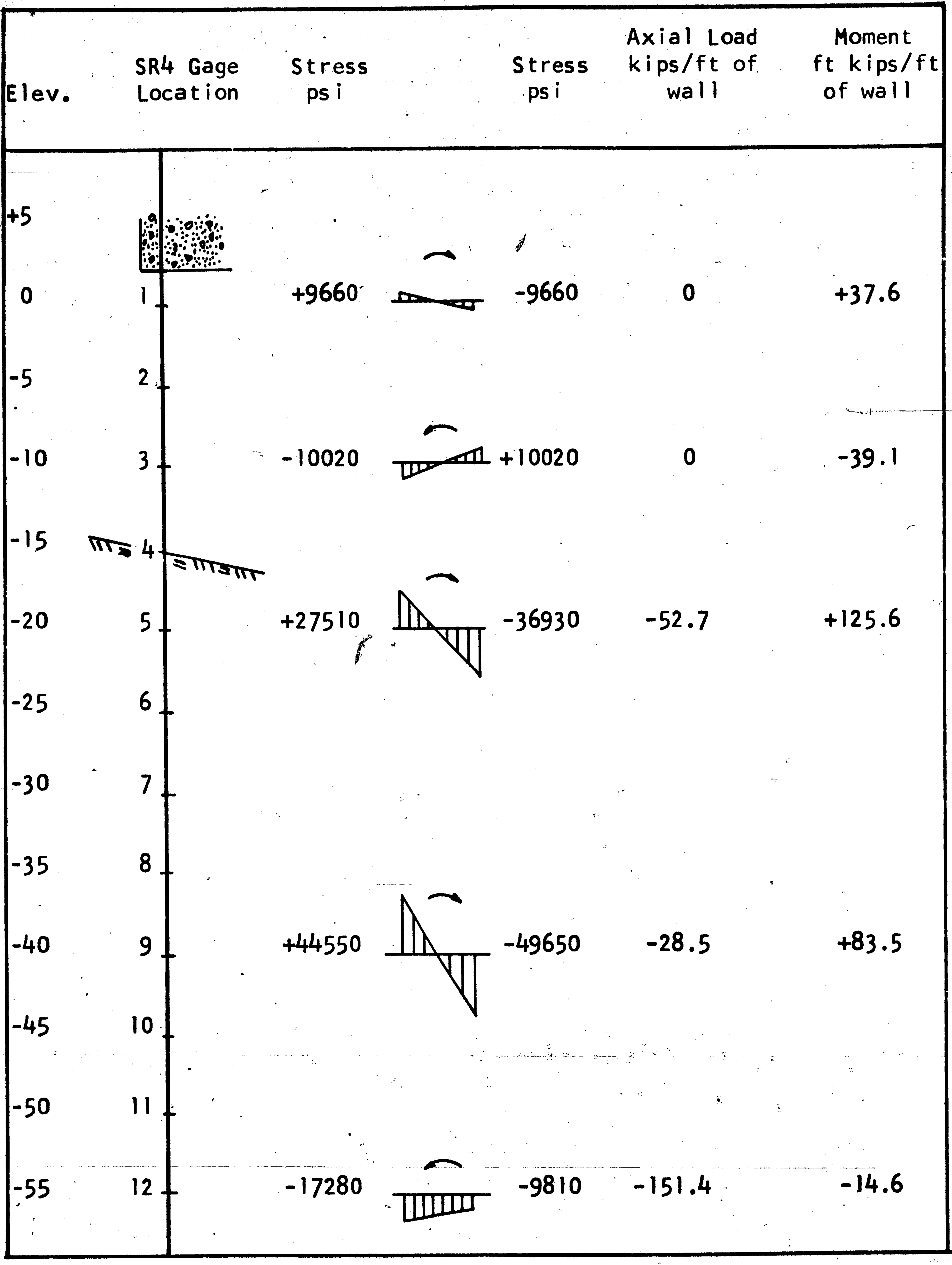
Note: SR4 gages on both flanges at odd gage stations - one flange only at even gage station

Fig. 10 - Location of Instrumentation on Sheet Piling



SECTION A-A

Fig. 11 - Location of Surcharge Pressure Gages



Elev.	SR4 Gage Location	Stress psi	Stress psi	Axial Load kips/ft of wall	Moment ft kips/ft of wall
+5					
0	1	+9660	-9660	0	+37.6
-5	2				
-10	3	-10020	+10020	0	-39.1
-15	4				
-20	5	+27510	-36930	-52.7	+125.6
-25	6				
-30	7				
-35	8				
-40	9	+44550	-49650	-28.5	+83.5
-45	10				
-50	11				
-55	12	-17280	-9810	-151.4	-14.6

Fig. 12 - Axial Loads and Bending Moments in Sheet Piling at Bent 18 After Driving

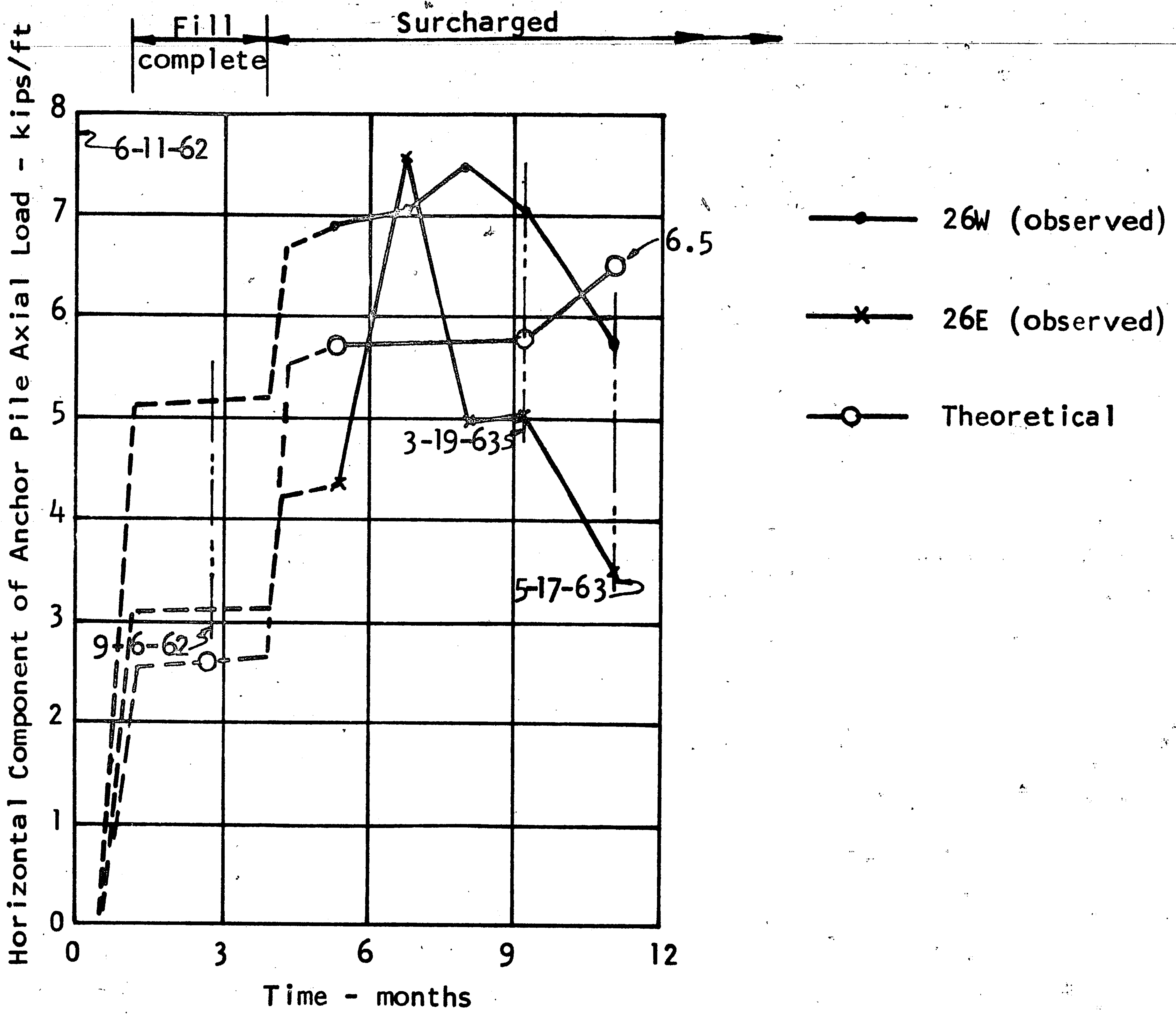


Fig. 13 - Horizontal Component of Batter Pile Loads at 26E and 26W Sheet Piling

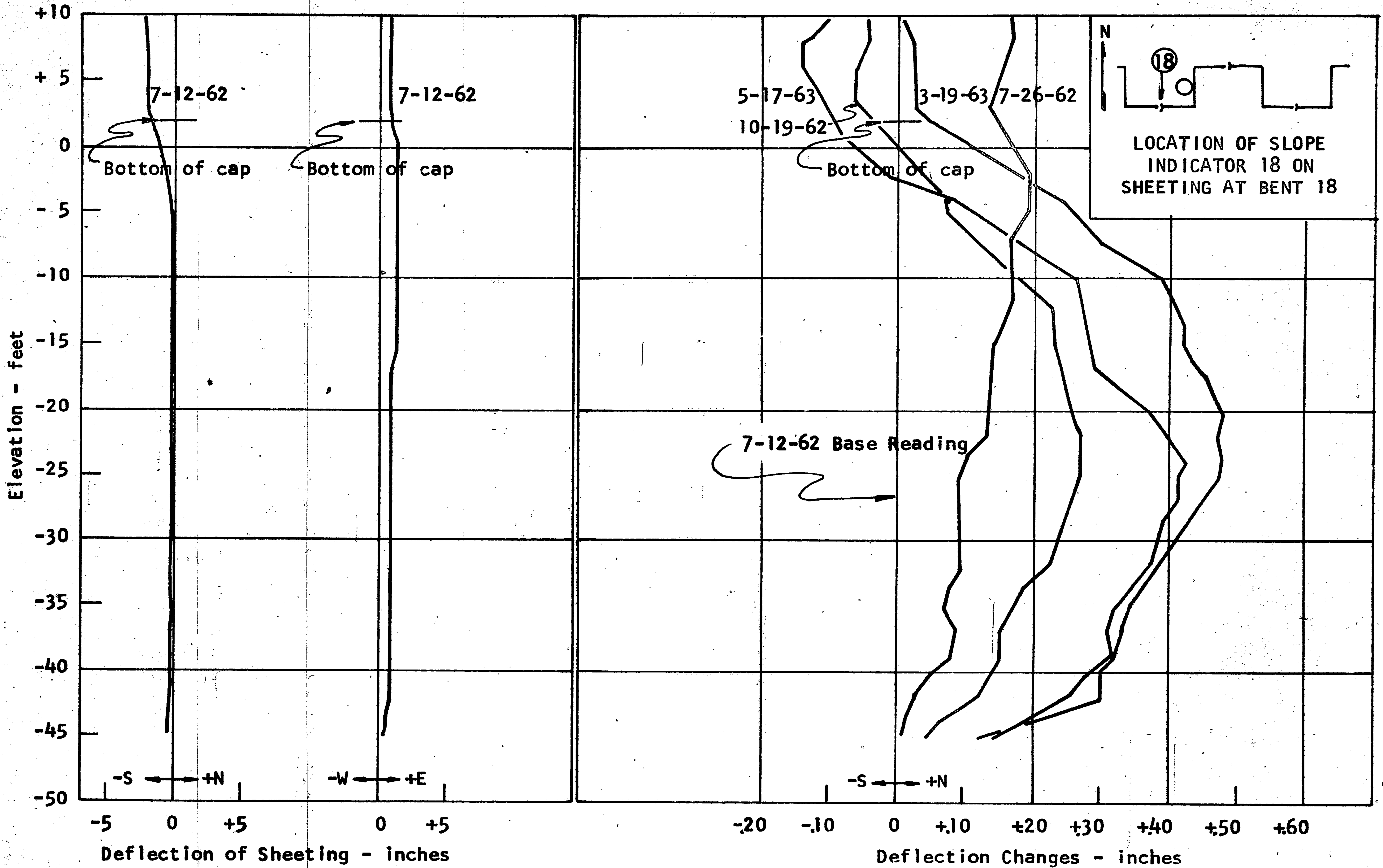


Fig. 14 - Deflection of ZP 38 Sheet Piling at Bent 18 After Backfilling and Surcharging

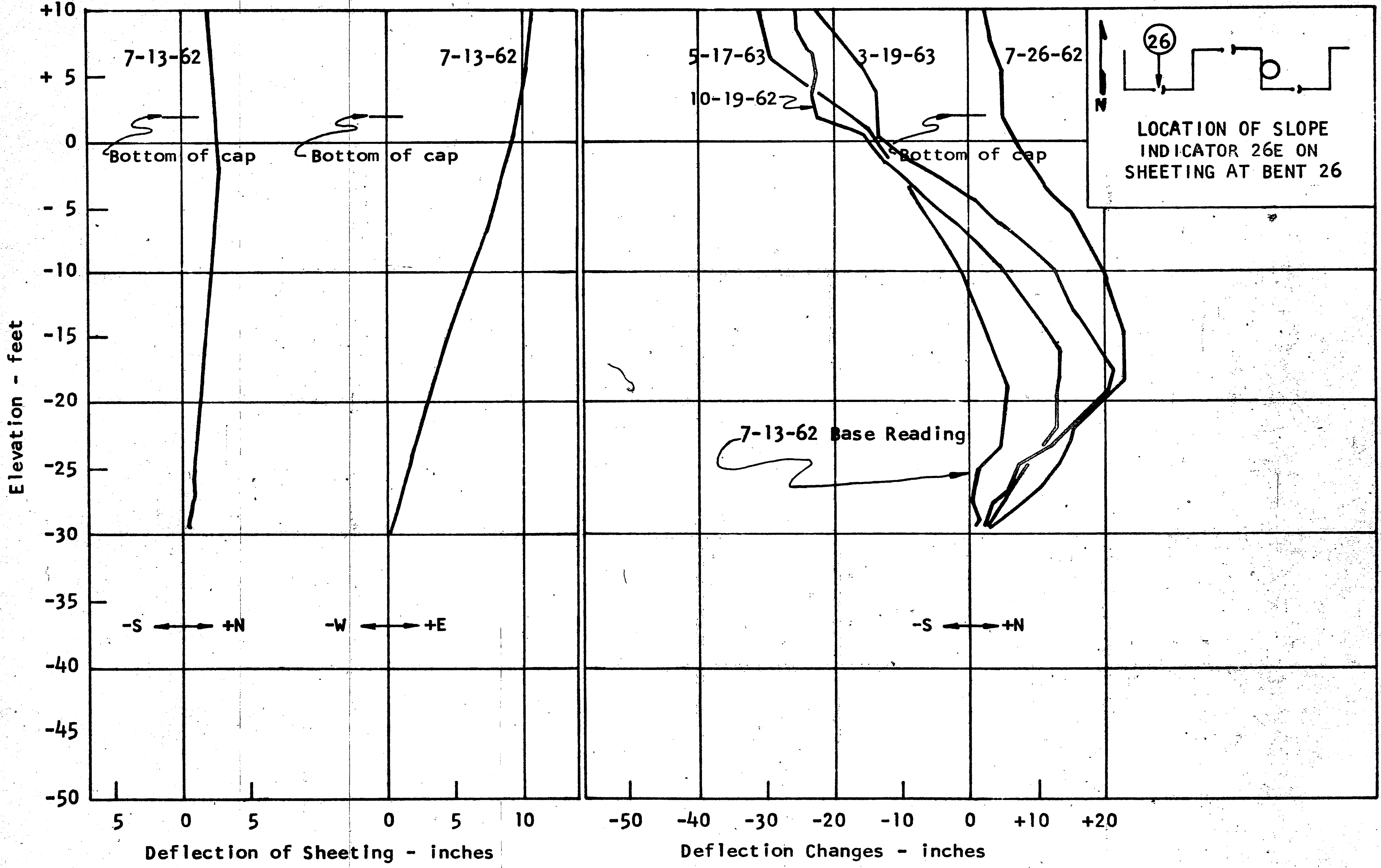


Fig. 15 - Deflection of ZP 38 Sheet Piling at Bent 26 After Backfilling and Surcharging

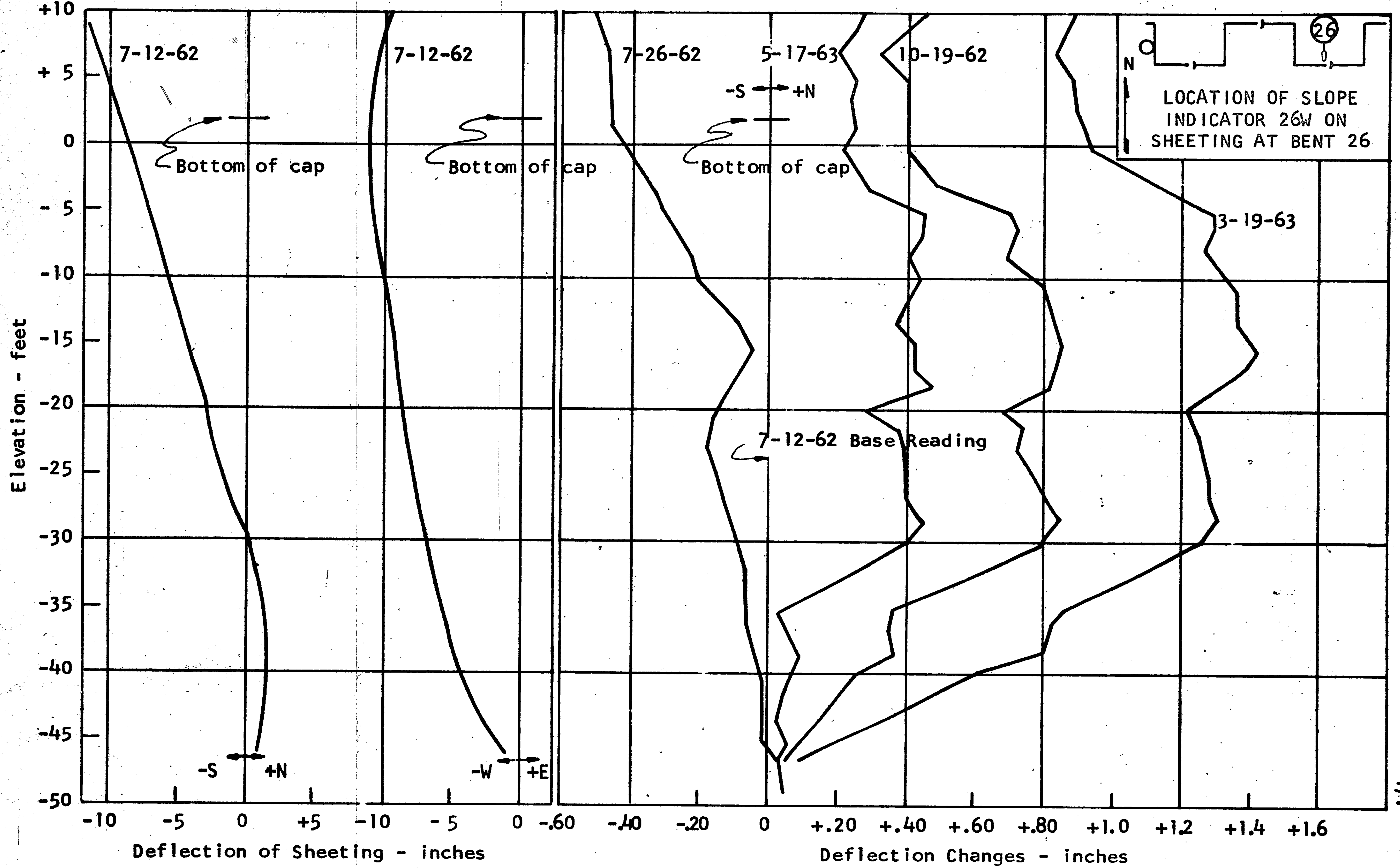


Fig. 16 - Deflection of ZP 38 Sheet Piling at Bent 26 After Backfilling and Surcharging

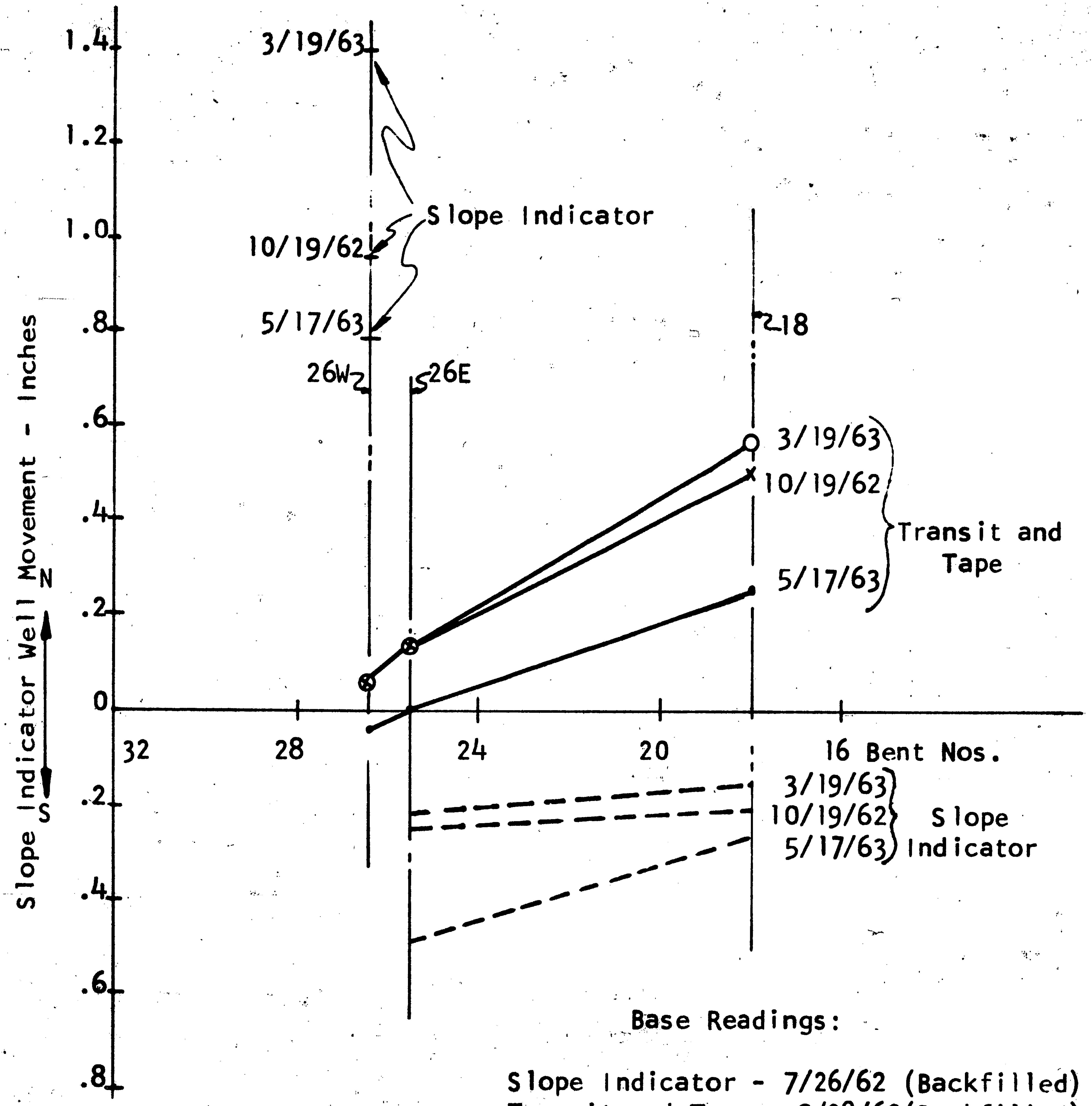


Fig. 17 - Slope Indicator Well Movement at Elev. +10.8'

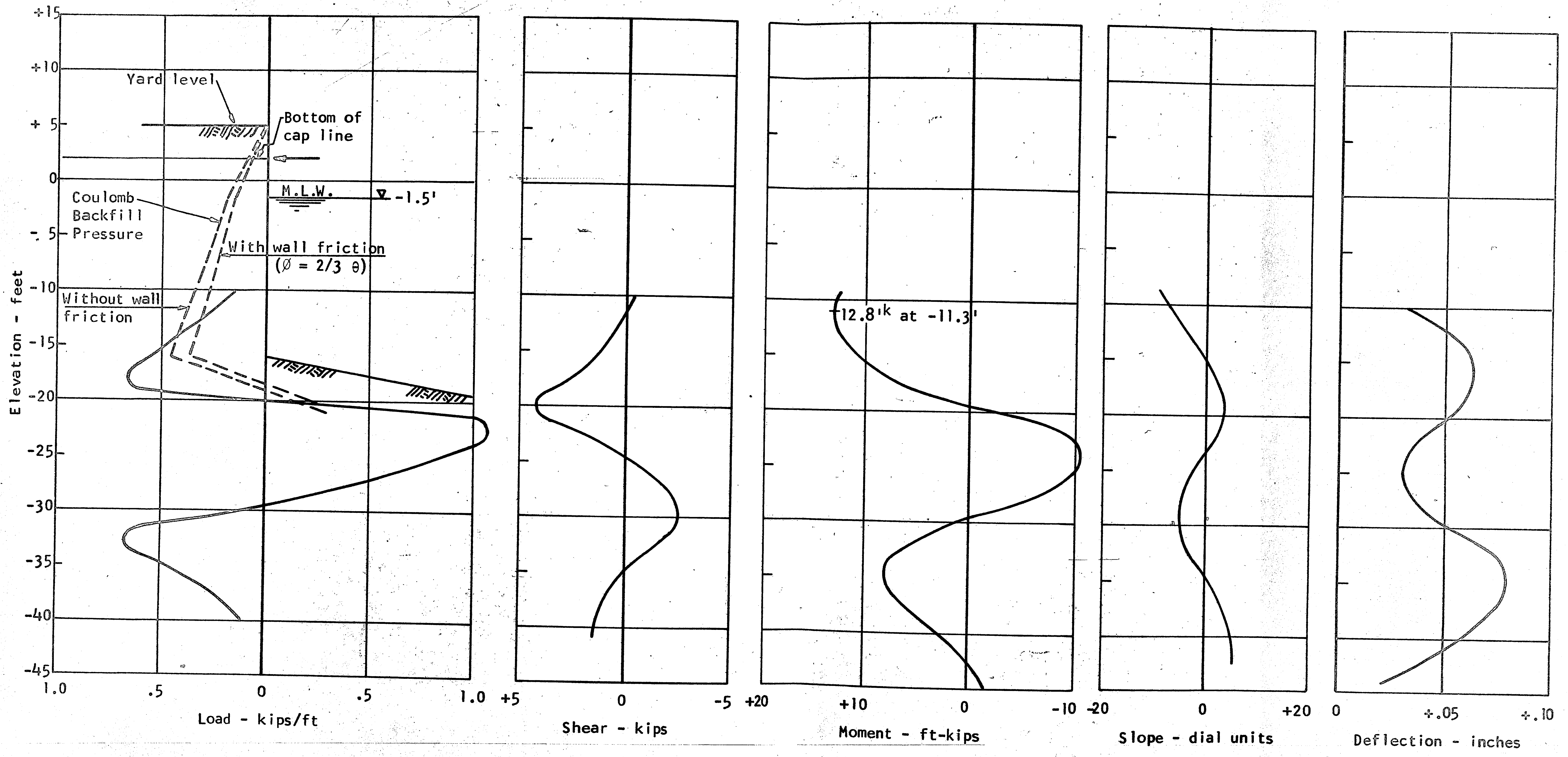


Fig. 18 - Diagrams Showing Results of Backfilling ZP 38 Sheet Piling at Bent 18

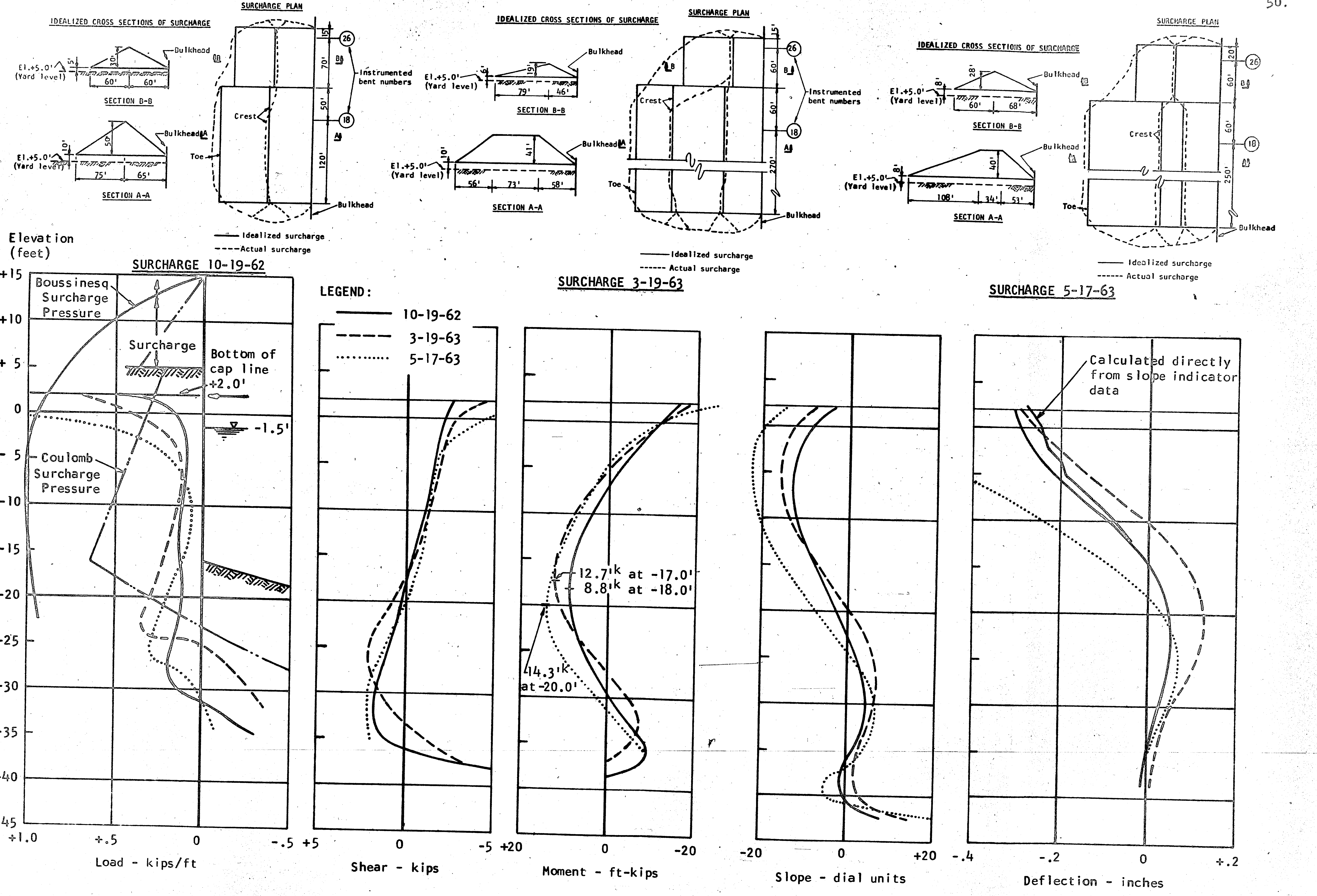


Fig. 19 - Diagrams Showing Results of Surcharging ZP 38 Sheet Piling at Bent 18

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