"FINAL"

THE LONDON AVENUE SITE SPECIFIC LOAD TEST

A Report by the Hurricane Protection Office of the US Army Corps of Engineers

and

The St. Louis District Corps of Engineers

Final Version, August 2008

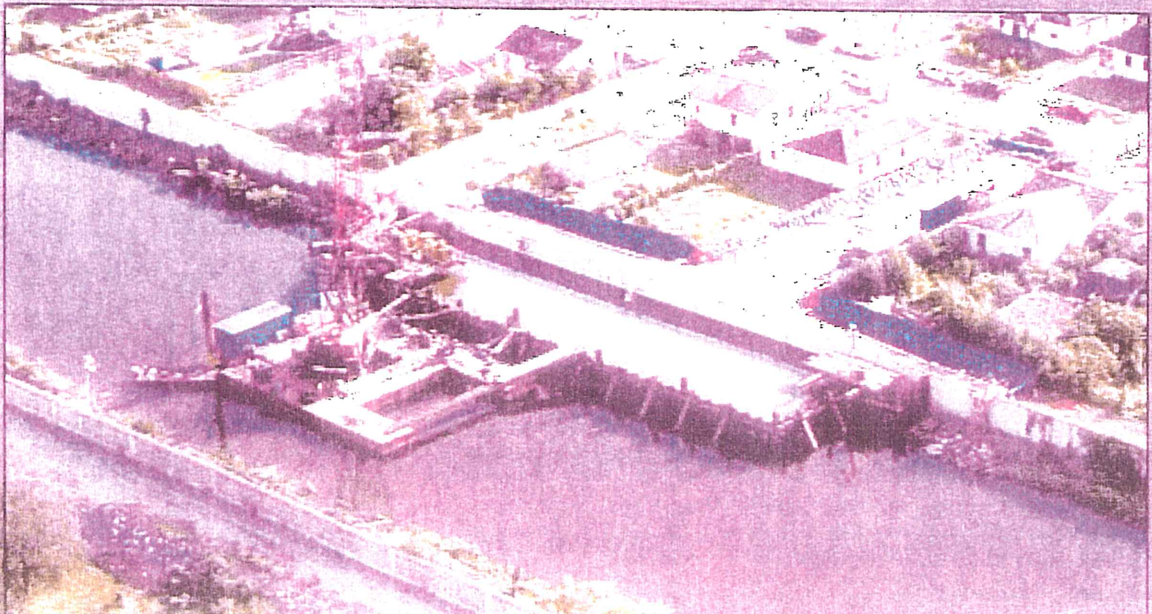


TABLE OF CONTENTS:

INTRODUCTION.

This report describes a full scale load test completed on an active portion of the Hurricane Protection System in the New Orleans Metropolitan Area. The test was completed on a 150-foot long section of concrete I-wall located on the east bank of the London Avenue Outfall Canal approximately 1000-feet south of Robert E. Lee Boulevard. The test began on August 18, 2007 and was concluded on August 28, 2007.

The test was designed by the St. Louis District Corps of Engineers (CEMVS) for the Hurricane Protection Office (HPO) of the New Orleans District Corps of Engineers. The Project Manager was Mr. John A. Ashley, P.E. (HPO) and the Project Engineer was Mr. Patrick J. Conroy, P.E., of the Geotechnical Branch of CEMVS. MAJ Nicholas Nazarko served as the Officer in Charge (OIC) during the Load Test.

Structural Engineering and Geotechnical Engineering was provided by the Engineering and Construction Divisions of the St. Louis and New Orleans Districts Corps of Engineers. These engineering divisions also provided Independent Technical Review and Bidability, Constructability, Operability, and Environmental (BCOE) review.

The Corps assembled a Technical Review Team consisting of Dr. Thomas L. Brandon, Ph.D., P.E., Virginia Polytechnic Institute (VA Tech); Mr. Noah D. Vroman of the Geotechnical and Structures Laboratory of the Engineering Research and Development Center of the Corps of Engineers (CEERD-GS-E); and Mr. Neil T. Schwanz, P.E. of the Geotechnical Section of the St. Paul District Corps of Engineers (CEMVP-EC-D). The Technical Review Team provided an Independent Technical Review of the data collected during the load test and a review of this document.

Ray E. Martin, PhD, P.E. and Robert Bachus, PhD, P.E. provided External Peer Review of the load test procedures and this report. Drs. Martin and Bachus were present during the load test observing all facets of the operation.

Mr. Richard Pinner and Mr. Frank Vojkovitch, of the Geotechnical Branch of the New Orleans District Corp of Engineers (CEMVN-ED-FS) provided invaluable advice and technical counsel during the design and execution of the load test.

EXECUTIVE SUMMARY

The Hurricane Protection Office of the US Army Corps of Engineers (HPO) successfully conducted a site specific load test on an active portion of the I-wall flood protection system that flanks the London Avenue Outfall Canal. This test was completed by constructing a sheetpile cofferdam against the canal-side face of the I-wall and carefully loading the wall by filling the cofferdam with water in slow, deliberate steps. The test began on August 18, 2007 and was concluded on August 23, 2007.

All aspects of the test were designed by the Corps of Engineers who retained complete technical control and maintained full coordination with its sponsors and stakeholders. Throughout the design process, the Corps remained transparent, answering all questions, allowing scrutiny of all design assumptions, and incorporated many of its sponsor and stakeholder concerns into the final design. The Corps cooperated with an External Peer Review group who had complete access to all design computations, design assumptions, and data recorded during the load test.

An Automated Data Acquisition System (ADAS) was specifically designed for this load test, installed, and operated to monitor the response of the I-wall, its foundation, and the groundwater regime. The ADAS functioned as designed, providing accurate, reliable data to the technical review team in near-real-time mode.

The technical review team was on-site for the entirety of the test, monitoring the contractors operations, and reviewing data obtained from the ADAS system. The technical review team consisted of subject matter experts from within the Corps of Engineers (St. Paul, St. Louis, and New Orleans Districts), from the Engineering Design and Research Center at Vicksburg Mississippi, from Virginia Tech University, and from the stakeholder's Peer Review Group. The technical review team reviewed data from the load test within 15-minutes of its being acquired by the ADAS and was able to confidently recommend adding the next additional load increment. The technical review team also used the data from the ADAS to determine when to terminate the load test.

The purpose of the London Avenue Site Specific Load test was to provide necessary information and data to assist the Corps of Engineers Hurricane Protection Office in its on-going analyses of the safe water elevation in the London Canal. Within weeks of the load test completion, the HPO issued its recommendation that the safe water elevation could be increased by one-foot without compromising the integrity of the flood protection system.

CHAPTER 1. GENESIS OF THE SITE SPECIFIC LOAD TEST

GENERAL

This chapter provides a brief description of the events leading to the Corps' decision to conduct a full-scale load test on a portion of the floodwall along the London Avenue outfall canal.

BACKGROUND

Hurricane Katrina caused failure of the hurricane protection system in two locations along the London Avenue Outfall Canal (hereafter referred to as 'the Canal'). The first of these was the I-wall on the west side of the Canal immediately South of Robert E. Lee Boulevard. The second was at the I-wall located on the east side of the Canal immediately North of Mirabeau Avenue. The Final Report of the Interagency Performance Evaluation Task Force (IPET) dated June, 2007 and titled '*Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System;- Volume V- The Performance - Levees and Floodwalls*' discusses the forensic investigations related to these two sections and the extent of these failures. Figure 1.1 indicates the locations of these failures.

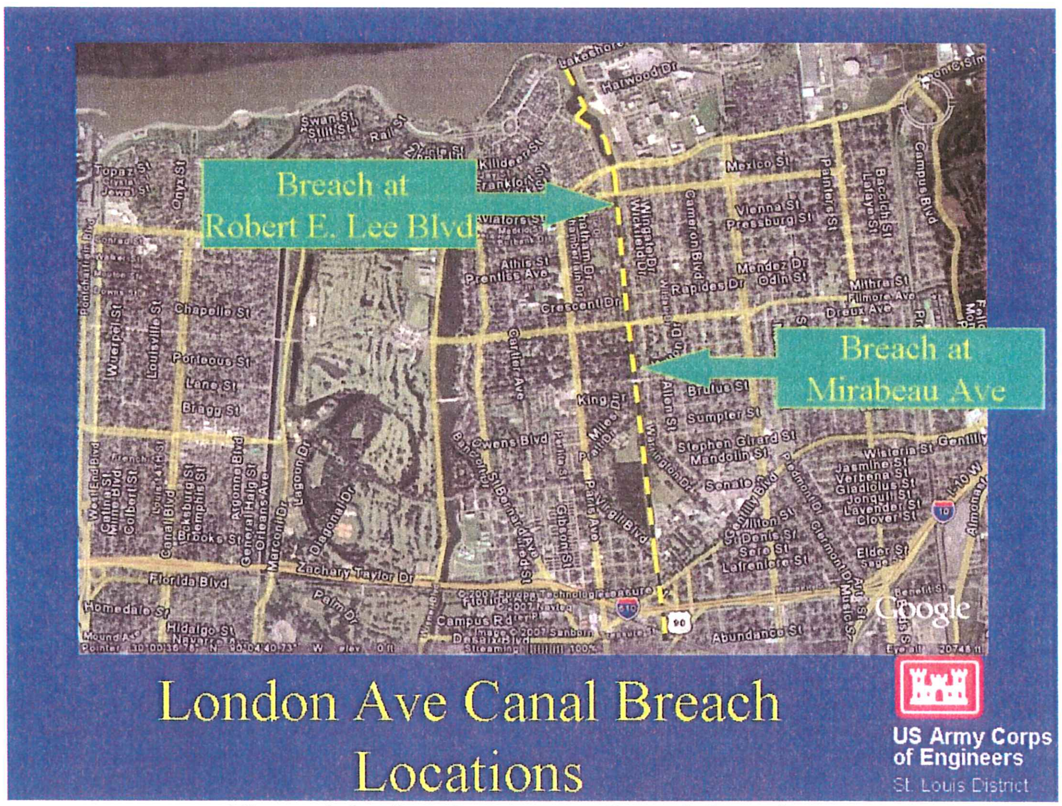


Figure 1.1 – London Avenue Breach Locations.

In the aftermath of the storm, the Corps of Engineers established Task Force Guardian and made it responsible for the recovery and repairs of the hurricane protection systems protecting the New Orleans Metropolitan Area. In June 2006, Task Force

Guardian was dissolved and the Hurricane Protection Office (HPO) assumed control of the hurricane protection systems including the canal. In August 2006, the HPO lowered the Safe Water Elevation (SWE) in the canal to +4.0-feet (NAVD88) as a precautionary measure.

The Sewerage and Water Board of New Orleans (S&WB) is charged with preventing or limiting interior flooding in the New Orleans Metropolitan Area. It does so with an intricate system of drainage ditches and pumping stations that collects storm water runoff from the low lying areas and pumps it into the Mississippi River, Lake Pontchartrain, and the various outfall canals, including the London Avenue Canal. The S&WB operates two pumping stations within the London Canal and has the pumping capability to raise the surface water elevation in the canal to levels well above +4.0-feet.

After lowering the SWE in the canal, the HPO recognized that a maximum operating elevation of +4.0-feet placed a substantial limitation on the amount of water that the S&WB could pump into the Canal, resulting in the increased potential for interior flooding. The S&WB counseled that raising the safe water elevation by only 1 foot (i.e., to +5.0-feet) would increase their allowable pumping capability by 30% and thereby decrease the chance of interior flooding, especially during a heavy rainfall or tropical event. The HPO considered site-specific and full-canal load test concepts as a means to substantiate any technical analyses used to support increasing the SWE. The HPO requested that the Corps of Engineers Engineering Research and Design Center (ERDC), IPET, and the Mississippi Valley Division (CEMVD) review the load test concepts.

The Corps of Engineers Task Force Hope approved the site specific load test concept in November 2006. From November 2006 to January 2007, the USACE Mississippi Valley Division geotechnical community met to assess viable options, ideas, and concepts for such a load test. The geotechnical community of practice routinely deals with field tests ranging from simple in-situ soil density and moisture testing to very complex aquifer pumping tests and pile load tests. This group was very comfortable with the idea of this load test. From these meetings, numerous concepts were developed and carefully considered and a final conceptual design strategy emerged from the group in January 2007. It was at this time that the HPO requested that the Geotechnical Branch of the St. Louis District (CEMVS-EC-G) provide technical leadership in the planning and execution of the London Avenue Canal Site Specific Load Test described in this document. CEMVS agreed and detailed analysis, planning, and design began.

ORGANIZATION OF THIS REPORT. This report includes chapters that described the genesis of the site specific load test, planning and design efforts, construction aspects, and development of the load test protocols and execution details. Subsequent chapters present detailed measurements and discussions of piezometric data, wall and embankment movements, and inclinometer data.

CHAPTER 2 – PLANNING, ENGINEERING, AND DESIGN

GENERAL

This chapter describes the Planning, Engineering and Design that was completed prior to beginning the load test. The planning phase included desktop studies of the IPET report, as-built documents on hand in the New Orleans District Corps of Engineers, coordination with the local sponsors and stakeholders, additional field investigations, and site selection. The engineering phase included study of the existing geotechnical stratigraphy, slope stability and seepage analyses of sites along the London Avenue Outfall Canal, and additional field investigations. The design phase included structural and geotechnical design of the cofferdam, the instrumentation system, and the load test protocols.

GOALS OF LONDON AVENUE CANAL SITE SPECIFIC LOAD TEST

The primary goal of the load test was to compile data that could be used to support the technical assessment of the SWE in the London Avenue Outfall Canal (hereinafter 'the canal'). To this end, the following specific objectives were identified:

- characterize existing subsurface to identify potentially critical locations;
- define specific conditions at selected cross-section;
- assess response of I-wall and surrounding levee during service and extreme loading conditions;
- assess potential for flood side gap and excessive underseepage mechanisms identified in the IPET Report
- replicate potential worst-case subsurface conditions during load tests; and
- design a comprehensive performance monitoring system to provide sufficient monitoring results to fully characterize I-wall response and to calibrate numerical analysis models.

Other important goals emerged that were given great prominence. These additional goals were identified during the subsequent meetings with stakeholders

- cause no damage to public or private property; this goal was established to address reported concerns that the Corp was planning to fail the floodwall as part of the load test;
- bolster public trust in the actions and activities of the Corps; this goal was established to gain the support of the stakeholders, private property owners, and neighborhood organizations located immediately adjacent to the load test work site;
- complete the load test before the height of hurricane season.

COORDINATION WITH STAKEHOLDERS

The Corps considered it paramount to establish and maintain communication with its stakeholders. To this end, the HPO and CEMVS identified and maintained close contact with its stakeholders during the planning, engineering and design stages of the London Canal Site Specific Load Test. The stakeholders included the following:

- Sewerage & Water Board of New Orleans
  - Ms. Marcia St. Martin, Executive Director

    - Mr. Joe Sullivan, General Superintendent
- Southeast Louisiana Flood Protection Authority – East Bank
    - Mr. Tom Jackson, President
- Orleans Levee District
    - Mr. Steven Spencer, Director Hurricane & Flood Protection.
- Louisiana Department of Transportation & Development
    - Mr. Edmond Preau, Jr., Asst Secretary – Public Works
    - Mr. Michael Stack, District Engineer/Administrator

The Corps met with these stakeholders on numerous occasions to update them on project status and progress, solicit their input, answer questions, and allay concerns. The Corps attempted to remain "transparent" in the eyes of its stakeholders by allowing scrutiny of all planned activities and by identifying all planning and design details.

Part of Corps policy is to include critical internal peer review of its documents and work products. To compliment the local Corps activities, the Corps established a Technical Review Team consisting of Dr. Thomas Brandon, PhD (Virginia Tech), Mr. Noah Vroman (ERDC), and Mr. Neil Schwanz (CEMVP), to perform analyses, provide critical assessment of the load test program, and to review load test results. The Technical Review Team participated in several meeting with the Corps and prepared stability and underseepage analyses, finite-difference based deflection models, site-selection studies, and partially penetrating slotted pipe analyses. These analyes are detailed later in this chapter.

The stakeholders also requested that that the Corps allow an external peer review of all information related to the planning, engineering, and design of the London Canal Site Specific Load Test. The stakeholder's Peer Review team consisted of Dr. Ray Martin, P.E., (formerly Chairman of Schnabel Engineering Associates, Richmond, Virginia) and Dr. Robert Bachus, P.E., (Principal Engineer with Geosyntec Consultants, Atlanta GA). The Peer Review Team reviewed the planned load test program and the supporting analyses and documentation. The Peer Review team prepared its 'Peer Review Report' dated August 14, 2007. A copy of this report is included in Appendix A of this report. Recommendations provided in the Peer Review Report will be discussed in detail later in this chapter.

SUMMARY OF IPET STUDIES.

Volume five of the IPET report describes the two I-wall failures along the canal and identifies flood-side gap formation and excess underseepage as being contributory to both failures. This potential failure mechanism is summarized as follows. As the I-wall and supporting sheetpiling deflect toward the protected side in response to increasing water levels, the soil in the levee on the canal side of the I-wall looses its intimate contact with the I-wall superstructure and sheetpile. This results in the formation of a "gap" between the I-wall/sheetpile and the surrounding soils. The gap formation allows water from the canal to exert its full hydrostatic pressure on the vertical surface(s) of the I-wall and supporting sheetpile. The gap formation begins at the surface and propagates downward in response to the increased deflection of the system components. If the hydrostatic pressure in this gap exceeds the horizontal earth pressure in the levee soil mass, the gap may self-propagate deeper into the soil foundation. Centrifuge studies completed by the ERDC and presented in the IPET report confirm the potential for this gap formation mechanism in certain subsurface conditions.

If the gap propagates completely through the underlying marsh clay to the top of the underlying barrier beach sand layer, then a more critical underseepage condition will be created. Analysis results indicate that this is a potential critical condition as it could result in excessive seepage beneath the levee and failure of the levee itself. A canal-side gap located immediately next to the wall that extends from the surface, completely through the overlying blanket, and to the top of the barrier beach sand will cause an effective seepage entrance immediately canal side of the I-wall. This will create a much closer seepage entrance and a more aggressive underseepage condition.

ENGINEERING STUDIES COMPLETE BY CORPS OF ENGINEERS. After the decision was made to load test a portion of the I-wall along the canal, the St. Louis, New Oleans, and St. Paul Corps districts, the Corps Engineering Research and Development Center (ERDC), and Virginia Tech completed numerous engineering studies to provide the requisite characterization information needed for the analysis and design of the load test. These numerical analyses were performed to understand and clarify the failure mechanisms identified in the IPET report and to determine the response of the I-wall section to the test loading. Brief summaries of the key reports and findings follow.

General Geologic And Geotechnical Conditions.

The geological and geotechnical stratigraphy adjacent to the canal was described in great detail by *Design Memorandum #19a: London Avenue Outfall Canal, Volumes I and II*, dated January 1989. Figure 2-1a (from DM 19a) shows that the canal east bank stratigraphy consists of the levee embankment fill, underlain successively by marsh clays (lightweight organic and inorganic clays with interbedded layers of decomposed vegetation), barrier beach sand, (Pine Island Beach Sand), and Pleistocene clay deposit. The HPO completed additional exploration along the banks of the canal in September 2007 and produced a revised geologic profile. Figure 2-1b shows the revised profile. A detailed study of centerline borings presented in DM19a reveals that the bottom of the canal penetrates the top of the barrier beach sand in some locations and in other locations the canal bottom consisted of organic clay and vegetation, ranging up to 10-feet thick.

Underseepage Analyses (By Virginia Tech)

Virginia Tech completed underseepage analyses that demonstrated the effects of the canal side gap on the underseepage regime along the canal. A copy of the report "*Analyses of the London Avenue Canal Load Test – Impeded Drainage Analysis*" is included in Appendix B of this report. These analyses explored the impacts of the canal bottom conditions (free-draining or not) and the canal side gap (present or not) on the seepage regime. These analyses were completed on a typical I-wall section using the stratigraphy described in DM 19a and as shown on Figure 2.1.

These analysis results show that if the bottom of the canal is located in the top of the barrier beach sand, then underseepage and critical excess uplift pressures will occur at the landside levee toe at certain threshold water surface elevations regardless of the gap formation. In this case, the gap makes a critical condition slightly worse.

These results show that if the bottom of the canal is an aquitard, and no gap is present, the seepage and uplift at the landside levee toe present little or no concern. These results also show that if the canal bottom is an aquitard and a gap forms and propagates to the underlying coarse grained barrier beach sand, then underseepage and attendant uplift pressures at the landside levee toe may increase to critical levels for canal water levels between +4.0 to +6.0-feet.

Seepage Analyses (By CEMVS)

CEMVS evaluated the underseepage potential of the levees/floodwalls flanking both sides of the canal. per the guidance contained in the Corps Engineering Manual EM-1110-2-1914, *Design and Construction of Levees,*. These results identified critical areas on the east bank of the canal between Stations 83+00 to 89+00 (south of Fillmore Ave.); and between Station 104+00 to 112+00 (south of Robert E. Lee Blvd).

Stability Analyses.

Virginia Tech completed slope stability analyses to demonstrate the effects of the canal side gap on the I-wall global stability along the canal. A copy of the report "*Analyses of the London Avenue Canal Load Test – Seepage and Stability Analysis*" may be found in Appendix C of this report. These analyses explored the impacts of the previous underseepage analyses and the presence of the canal side gap on the stability of the I-wall. These analyses were completed on an I-wall section considered to be representative for the east bank of the canal between project stations 107+00 to 114+00. Results show that the factors of safety reduce to unity at canal water surface elevations of 6.2-feet with the gap present and the critical underseepage assumptions described above.

I-Wall Deflection Studies

The St. Paul District Corps of Engineers (CEMVP) completed 2-dimensional, finite difference analyses to assess deflections of a typical I-wall at various canal loading scenarios. A copy of the report "*Analyses of the London Avenue Canal Load Test – Soil Structure Interaction Analysis*" may be found in Appendix D of this report. Results show that deflections of 0.90 to 5-inches at the top of the I-wall should be expected for canal water surface elevations of +5.0 NAVD88. These analyses assume that the top of earthen levee exists at elevation of +2.5-feet NAVD88, and were based on a starting cofferdam water surface elevation of 0.0-ft. These analyses used soil stiffness properties developed by IPET based on pressure-meter tests and triaxial soil testing.

Pine Island Barrier Beach Sand Pumping Test

CEMVS completed three pumping tests of the barrier beach sand in 2006. One of these was located 700-feet south and 800-feet east of this site specific load test, on the west side of the canal in a place known as Pratt Park. The computed horizontal permeability based on measurements made during the pumping test was set at $156*10^{-4}$ cm/sec. The "*London Pump Tests*" report may be found in Appendices K1, K2, and K3.

IMPACTS OF ENGINEERING STUDIES ON LOAD TEST PLANNING. These engineering studies indicated that:

- gap formation could happen quickly;
- critical underseepage conditions could be encountered for certain combinations of gap formation, canal bottom conditions, and water surface elevation;
- critical slope stability conditions could be encountered for certain combinations of gap formation, canal bottom conditions; and water surface elevation.
- excess, permanent wall deflection could occur if the wall were overloaded.

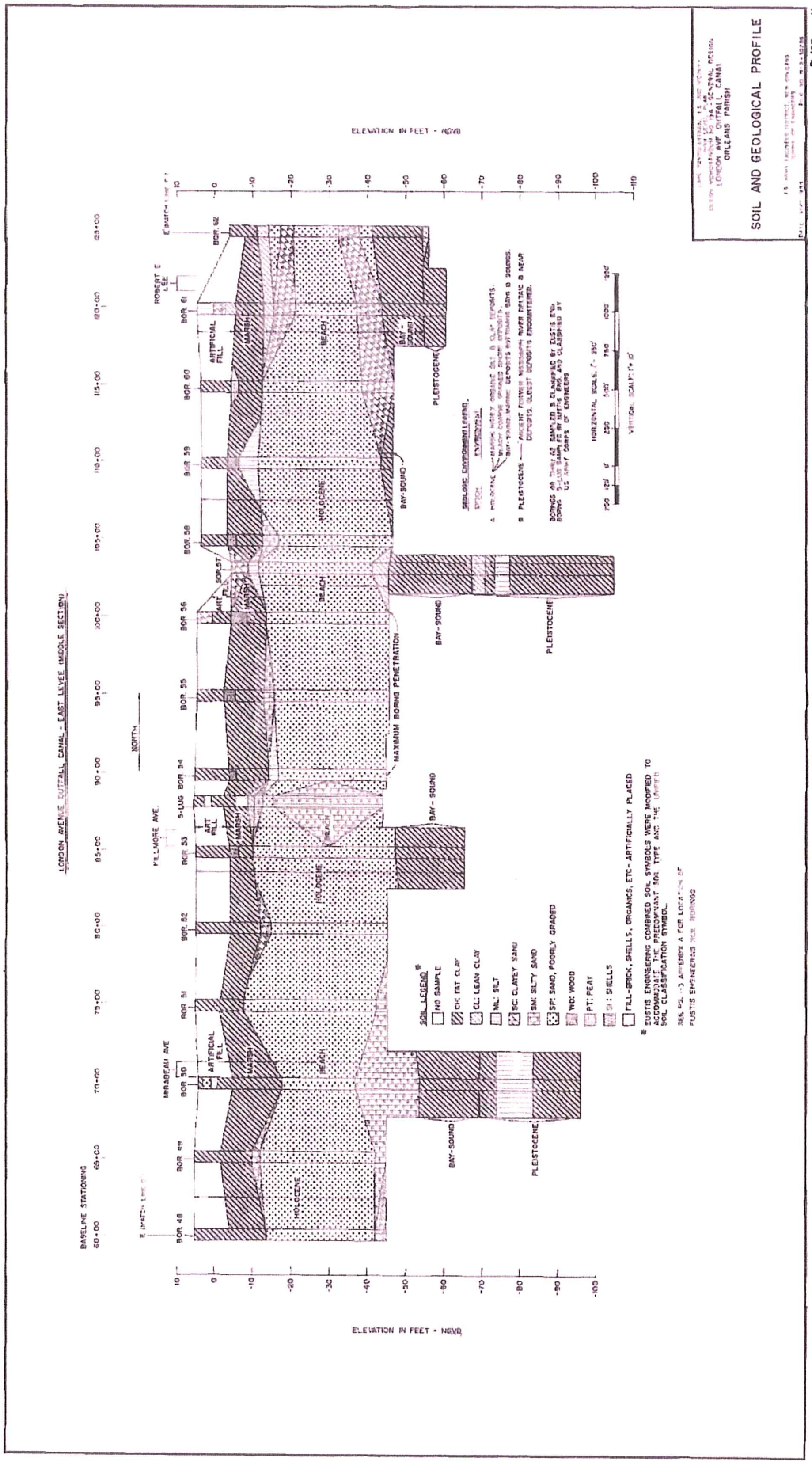


Figure 2.1a – Stratigraphy East Bank of London Canal (Elevations in NGVD)

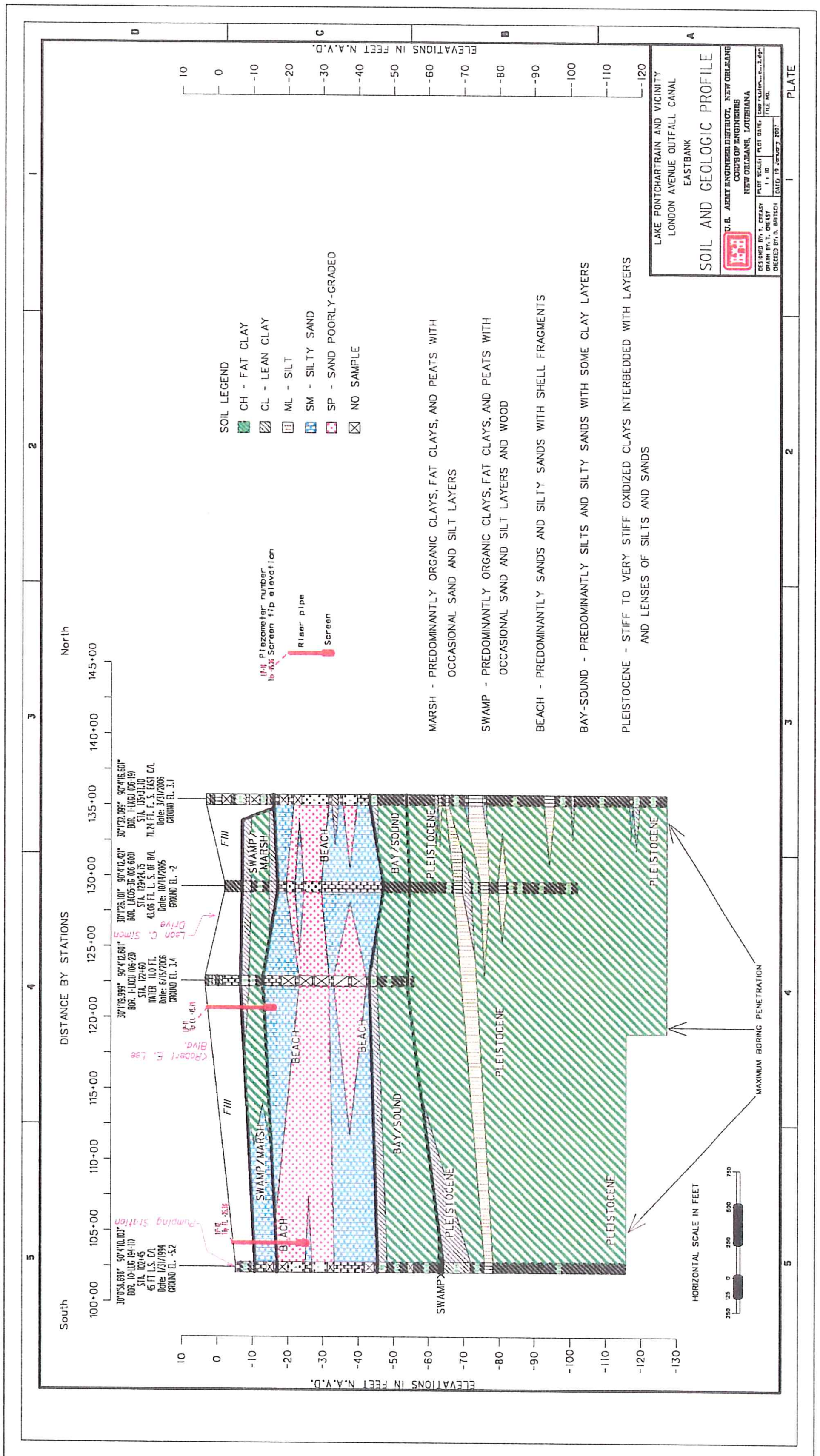


Figure 2.1b – 2007 Revised Stratigraphy East Bank of London Canal
(Elevations in NAVD88)

The Corps tempered the results of these analyses with the known performance of the remaining flood wall during the Katrina Hurricane. Volume 5 of the IPET report estimates that the high-water elevation in the canal during the storm of up to 8.2-feet. Knowing that this part of the wall had already resisted these water surface elevations and knowing that the maximum water level during the load test would remain below these elevations, and knowing that the wall would be closely scrutinized during the test, the Corps was confidant that the test could be performed safely.

The Corps would design and execute the load test to confirm the assumptions implicit in the previously completed engineering studies, especially the phenomena related to canal side gap propagation. The load test would also be designed to gather the necessary information to support raising the SWE to +5.0-feet.

The engineering studies confirmed that underseepage, wall deflection, and landside embankment deflection must be closely monitored during any load test on an existing I-wall and that any load should be placed carefully and slowly against the wall. These results affirmed the Corps decision to include a highly accurate automated data acquisition system (with redundant levels of measurement) in the plans for a load test.

SITE SELECTION. The Corps decided to locate the load test at the "most critical" location along both sides of the canal. ERDC and CEMVP completed an analysis of the subsurface conditions beneath the East and West banks of the canal to identify the most critical location based on available borings, spaced generally at intervals of 500 feet along the canal levee centerline. The report titled *"London Avenue Canal Load Test – Small Load Test Site Selection"* may be found in Appendix E. Since the failures that occurred on the canal east and west banks were apparently caused by pore-pressure induced instability, the most critical section was perceived as having the thinnest marsh clay thickness over the sands at the protected-side levee toe, having the smallest levee embankment section in terms of height and width, and having the thinnest cover over the sands in the bottom of the canal. This assumes that a thinner marsh clay at the protected side toe of the levee and beyond results in lower effective stresses in the barrier beach sand and lower factors of safety against heave. In addition, a smaller levee embankment section in terms of height and width above the canal water elevation will increase the chances of the formation of a gap down to the barrier beach sand (providing a connection of the canal water to the beach sand) at a lower canal water level. The thinner soil cover over the sands in the canal will increase the potential for underseepage beneath the levee/sheetpiles. The results of this study identified the following three critical locations:

- Station 111+00 East Bank (south of Robert E. Lee Blvd).
- Station 84+00 East Bank (south of Fillmore Ave.).
- Station 57+00 East Bank (south of Mirabeau).

Although the each of the three sites were considered potentially critical, the site near station 111+00 was considered to be the most critical because it exhibited the lowest Standard Penetration Test (SPT) N-values in the underlying barrier beach sand and it had the lowest levee crown elevation (+2.5-feet NAVD88). This site was also identified as a critical location by the CEMVS seepage analyses. The Corps proposed to perform the load test near station 111+00 on the East Bank. The site was attractive because of the vacant lot on the protected side of the wall. The site location and conditions are shown in Figures 2.2 through 2.4. The test site is just south of boring #59 shown on Figure 2.1.

SITE SPECIFIC EXPLORATION. The team laid-out and completed a site specific exploration program at the proposed load test site consisting of land based borings, land-based cone-penetrometer borings, and over-water borings. An initial round of exploration was completed using an existing exploration contract with Eustis Engineering (Metairie, LA) and consisted of a small-footprint drill rig obtaining 1-inch diameter samples of the levee embankment and marsh clay materials. A drill crew from the New Orleans District Corps of Engineers (CEMVN) completed a second round of exploration completing borings on the protected side of the I-wall obtaining 5-inch diameter, undisturbed samples of the levee embankment and marsh clay materials and SPT samples of the underlying barrier beach sands. The CEMVN drill crew also completed flexi-float based, overwater drilling, obtaining 1-inch diameter samples of the materials on the bottom of the canal. Finally, the Vicksburg District completed cone-penetrometer borings on the protected side of the I-wall. All exploration sites are shown on Figure 2.5 and a site-specific geotechnical, centerline profile is shown on Figure 2.6. Detailed boring logs may be found in Appendix F.

The site specific exploration identified a major difference from the DM 19a stratigraphy: Specifically, these new borings confirmed that the upper 10-feet of the barrier beach sand consisted of silty sands rather than sand with low fines content. This difference impacted the piezometric pressures measured during the load test and is further discussed in Chapter 5.

Figure 2.2 – Site Location

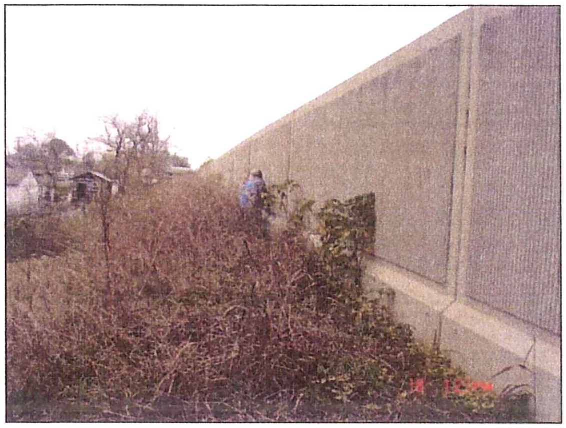


Figure 2.3 – Protected Side View of I-Wall

Figure 2.4 – Protected Side Vacant Property

DETAILED DESIGN. The design teams (St. Louis and New Orleans Districts) considered many issues during its design of the load test and some of these are described below. The full set of load test contract drawings may be found in Appendix G.

Size of Cofferdam. The as-built configuration of the steel sheetpile cofferdam was nominally 150-feet (parallel to the canal centerline), by 35-feet (perpendicular to the canal centerline), and designed to withstand water to +7.5-feet NAVDD-88. The structural and geotechnical analyses/design of the cofferdam was done assuming the use of PZ-35 sheetpile and HP 14 x 73 H-piles. Adequate quantities of these piling were owned by the government (previously used on other Corps of Engineers projects) and were provided to the contractor as government furnished property.

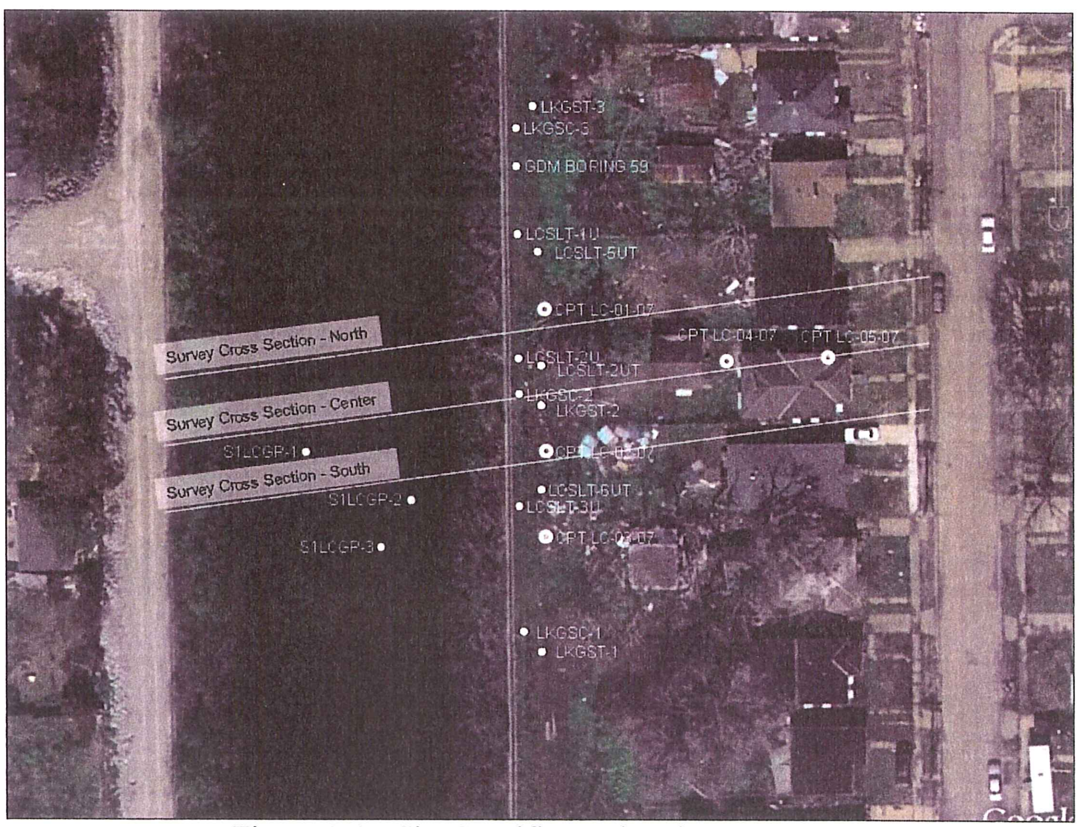


Figure 2.5 – Site Specific Exploration Locations

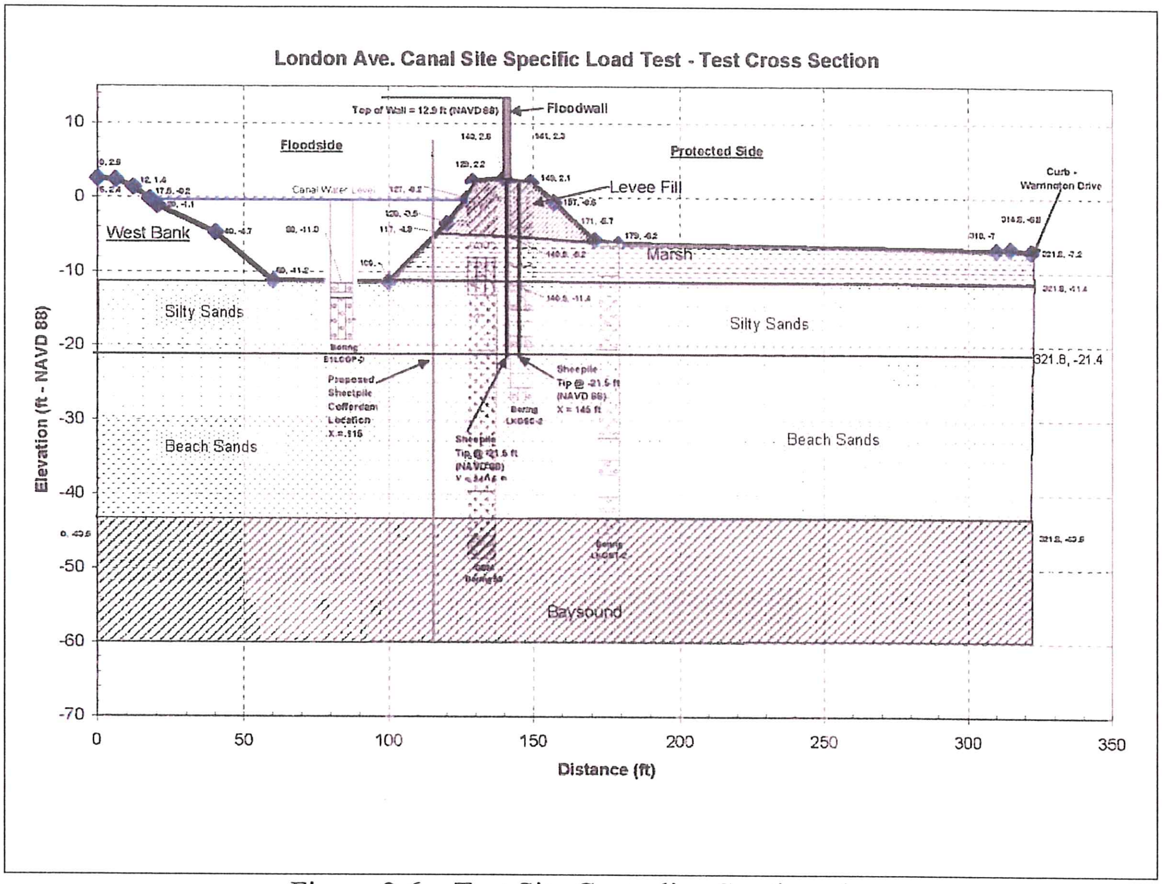


Figure 2.6 – Test Site Centerline Stratigraphy

Cofferdam Location. The cofferdam was sited at the critical location described by the site location study. The cofferdam's centerline lay on the center of an existing I-wall monolith and was located approximately 1000-feet south of Robert E. Lee Boulevard on the east bank of the canal and approximately 100-feet south of boring B-59 shown on Figure 2.1. This location placed the centerline of the cofferdam within the vacant property at 5772 Warrington Drive shown on Figure 2.4.

Cofferdam Length. The cofferdam length (150-feet) was based on the thickness of the underlying barrier beach sand and the length of the existing I-wall monoliths. Virginia Tech presented analysis results to the design team that indicated 3-dimensional (3-D) seepage effects would occur where the cofferdam tied-into the existing I-wall. CEMVS sized the cofferdam so that the north and south tie-ins would be at least twice the barrier beach sand thickness from the cofferdam centerline to minimize the 3-D effects at the cofferdam centerline, and given the aquifer thickness of 32-feet, this distance is 64-feet. Since the tie-ins would occur at monolith joints, and the monoliths were approximately 30-feet long (i.e., 29-feet, 4-inches), the nearest monolith joint that met or exceeded the 64-foot distance was located 75-feet from the centerline. This resulted in a 150-foot (approximate) long cofferdam encompassing 5 existing I-wall monoliths. Figure 2.7 is a view of the cofferdam interior looking north.

Cofferdam Width. The Corps originally anticipated a cofferdam that would extend 75-feet to the centerline of the canal. The Corps reasoned that capturing one-half of canal bottom inside of the cofferdam would replicate the seepage conditions encountered during a high water event in the canal. The stakeholders objected to this initial plan, stating a belief that this large of a cofferdam would severely impact the canal's conveyance capability and artificially increase the water surface elevations of the canal upstream of the cofferdam for a flow rate in the canal during a reasonably anticipated storm event. Hydraulic modeling by HPO and CEMVN confirmed this and subsequently showed that a 25-foot wide cofferdam would remain transparent to the flow regime in the canal. As described previously, this width was subsequently revised to 35 feet in response to the external Peer Review Team comments. Additional HPO hydraulic modeling results confirmed that the 35-foot width cofferdam would have a minor effect on the canal flow regime at high flow rates. The Corps completed additional planning and discussions with its contractor to determine what portions of the cofferdam would need to be removed to minimize the impacts on the canal flow should there be a high flow rate event during the load test.

Figure 2.7 – View of Partially Completed Cofferdam (Looking north)

Cofferdam Height. The results of the soil-structure-interaction deflection analyses (presented in Appendix D) showed that substantial I-wall deflections should be expected for canal water surface elevations of +6.0 to +6.5-feet NAVD88 against the wall. The Corps used this result as a guide in setting the top elevation of the cofferdam. The top elevation of the cofferdam sheetpiles were set at +8.00-feet NAVD88, and the maximum test water surface elevation in the cofferdam was assumed to be +7.5-feet NADV-88 with a corresponding canal water surface elevation of +1-feet NAVD-88.

Sheetpile Embedment. CEMVS completed sheetpile analyses using the Corps of Engineers CAGE program CWALSHT. Repeated analyses utilizing the various analyses methods available within the program showed that cantilever or anchored sheetpile fixity was achieved with the tips driven to between elevations -28 and -32 feet NADV88. Based on comments from the external Peer Review Team, the sheetpile tips were driven completely through the beach sand layer and into the underlying clays. The Peer Review team stated that there would be less unknowns involved in the system if the beach sand aquifer were completely cut-off by the cofferdam sheetpile.

Buttressed Sheetpiles. Analysis results showed that the estimated deflections at the top of the cantilevered sheets along the 150-foot long cofferdam wall would exceed allowable limits. The Corps changed the design from that of a cantilevered sheetpile to an anchored, or buttressed sheetpile. Based on additional CWALSHT analyses, CEMVS designed a horizontal whaler for a uniform load of 4000 lb/foot. Pile bents, consisting of one vertical HP 14x73 and one battered HP14x73, were located every 11ft-4in to resist the loads in the horizontal whaler. H-pile axial capacities (compressive and tensile) were determined assuming 110-foot long piles driven to elevation -92-ft NAVD88. The typical pile bent arrangement is illustrated in Section A of Sheet 3/8 in the construction drawings found in Appendix G. Figure 2.8 is a picture of the as-built pile bents and their spatial relation to the horizontal whaler and cofferdam sheetpile.

Connection Details – Cofferdam to Existing I-Wall.

The as-built drawings show that the existing concrete I-wall consists of a concrete cap astride CZ-101 sheetpile. The top of the concrete cap is at elevation +12.9 ft and the base of the concrete is at elevation +2.0. The CZ-101 sheetpile extend down to a tip elevation of -21.5 ft. The concrete wall monoliths are nominally 30-feet long units that are tied together to form essentially a monolithic structure; the sheetpiles similarly provide continuous coverage along the length of the canal. A typical section of the I-wall may be found at the end of the construction drawings in Appendix G.

The connection between the cofferdam and the existing I-wall was considered to be a critical detail with two functions: It should cause limited impact on the response of the I-wall to the water loads imposed on it during the test and is should be water-tight. The need for a structurally transparent, yet water tight, connection was achieved in the design by embedding the web of the last PZ-35 cofferdam sheetpile into the construction joint between the adjacent monoliths. As shown by Figure 2.9, the contractor did not embed the sheetpile web into the monolith joint. Rather, he chose to weld a flat piece of steel to the last sheetpile and embedded this into the monolith joint to achieve the same result. This joint was strengthened by backing it up with a partial circumference of pipe welded onto the canal-side of the sheet and filling it with non-shrink grout. Water-tightness was achieved by bolting neoprene sheets on both sides (i.e., canal side and interior) of the joint. These details are illustrated in Detail 1 of Sheet 5/8 in the construction drawings found in Appendix G. Figure 2.9 is a photograph of the completed joint at the north tie-in.

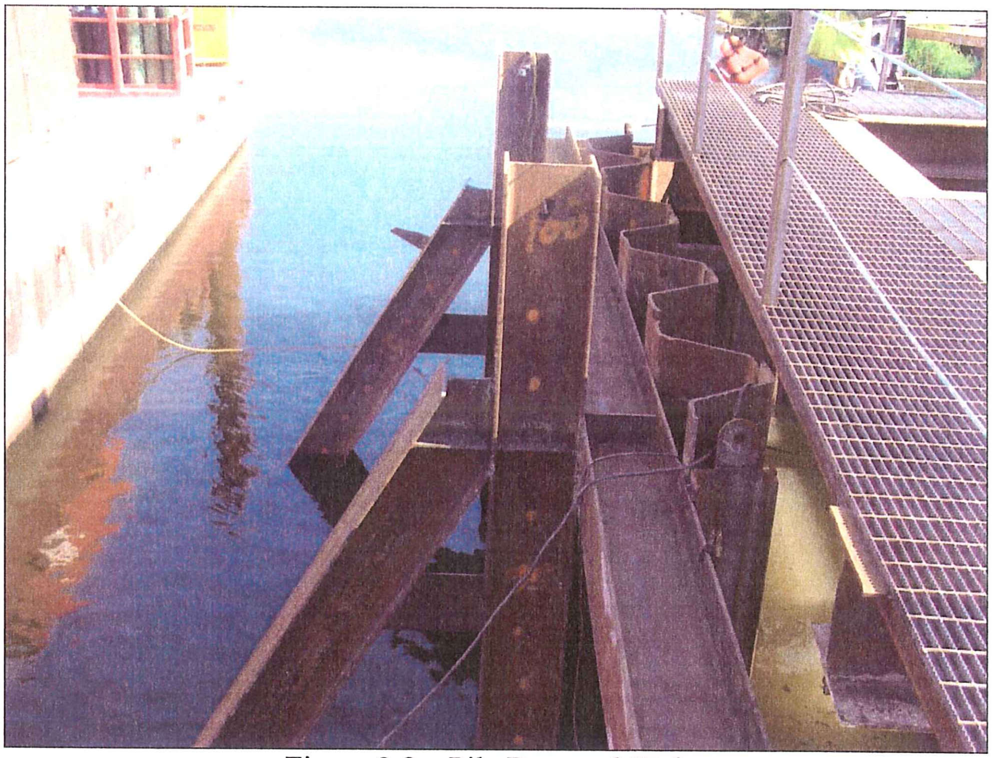


Figure 2.8 – Pile Bent and Waler

Figure 2.9 – Connection Between Cofferdam and I-Wall

Jet Grouting. Although the contractor was able to connect the sheetpile cofferdam to the concrete I-wall, there was no practical way to establish a structural connection between the cofferdam sheetpile and the existent CZ-101 sheetpile under the I-wall at the two tie-in points of the cofferdam. The Corps required the contractor to install jet-grout columns to "tie-together" the cofferdam sheetpile and the I-wall sheetpile. The contract required the contractor to install three jet-columns on a tight, triangular arrangement in the interior of the cofferdam at the north and south tie-ins. The jet grout columns extended through the levee embankment fill, the in-situ marsh clay, and penetrated slightly into the underlying barrier beach sand.

Emergency Dewatering - Pumping. The contractor was required to pump water into the cofferdam during the load test. As a part of the contract, the Corps also required that the contractor be able to dewater the cofferdam on an emergency basis during the load test, should the need exist. The Corps analyses involved determining a required pumping capacity to empty the cofferdam (from maximum elevation of +7.5-feet NAVD88 to +2.0-feet) in two minutes, evaluating a range of pumping systems that a contractor might install, sizing a supporting structure (pump stand) for a typical pump, and then including that information on the drawings (in limited detail) with the caveat that the final details were to be "contractor designed". The "Pump Support Plan" is shown on construction drawing 6/8 in Appendix G. Figure 2.10 shows the contractor-designed emergency pumping system. The contractor set two hydraulic driven pumps (30 and 40-inch diameter) on one pump stand in the north end of the cofferdam. The total capacity of the contractor emergency pumping system was 77,000 gpm which met the design requirement.

Emergency Dewatering - Sluice Gate. The stakeholder's desire for redundant safety measures prompted the Corps to include a second emergency dewatering system in the cofferdam to supplement the emergency pumping system. The Corps included a 7.75-ft wide by 6-ft tall sluice gate into the 150-foot long cofferdam wall that could be pulled to hasten the emergency dewatering of the cofferdam. Drawing 3/8 shows the gate location near the emergency pump stand and the gate is detailed on drawing 7/8. Figure 2.11 shows a picture of the sluice gate installed in the cofferdam. During the load test, an operator was at the controls of a crane ready to pull the sluice gate if directed to do so by the Officer in Charge.

Coordination Between the Corps and Contractor During the Test. The contract documents for the Site Specific Load Test consisted of conventional Corps civil works specifications for Selective Demolition, Clearing, Steel H-Piling, Excavation, Stone and Bedding Construction, Steel Sheet Piling, and Jet Grouting. The language that governs how the Corps would control the contractor's operations during the load test is included in paragraph 1.36 of Section 01100 – "General Provisions". In addition, Section 01500 – "Temporary Facilities, Equipment, and Services" defines the tangible resources that the contractor would need to provide and operate under the Corps' guidance. These sections may be found in their entirety in Appendix H. The terms of the Corps guidance to the contractor is defined in Chapter 4 – Load Test Protocols and Execution Details.

Figure 2.10 – Emergency Pumping System

Figure 2.11 – Emergency Sluice Gate

INSTRUMENTATION.

To conduct the load test on an active part of a flood protection system, the Corps concluded that an accurate and reliable Automated Data Acquisition System (ADAS) was required. This ADAS was required to monitor the response of the I-wall, sheetpiling foundation, levee embankment, and the groundwater regime to the water loads that were placed against it and report this information in "near-real-time". The information from the ADAS was used by the Corps to assess the condition of the I-wall-soil-groundwater system during loading. Results were used to control the incremental loading.

CEMVS contracted with the St. Louis Office of URS to complete the design, installation, and monitoring of an ADAS. The design and installation/operation of the ADAS was executed under two separate task orders that are summarized below. The final work products of both task orders may be found in Appendix I.

Under the first contract, URS completed a final design of an ADAS utilizing well understood and field proven instruments that could provide "near-real-time" measurements of: (i) lateral and vertical movements of the five monoliths within the limits of the cofferdam; (ii) movement of the protected side levee surface; (iii) movement of the I-wall foundation below the ground surface; and (iv) groundwater response. The first contract contained the following tasks:

Task 1: Review Existing Design Documents and Geotechnical Reports.
Task 2: One Day Site Visit.
Task 3: Select Instrument Types and Prepare Design Details/Specifications.
Task 4: Design ADAS/Telemetry System.
Task 5: Design Database Management/Graphics Reporting System.
Task 6: Prepare Detailed Equipment/Components List.
Task 7: Evaluate Two Emerging Technologies for Inclusion into the Load Test.
Task 8: Assist USACE to Develop Site Specific Load Test Plan.

Under a subsequent contract, URS acquired all the necessary instruments and provided the resources, manpower, and knowledge to install, operate, and remove the previously designed ADAS. This contract included the following tasks:

Task 1: Pre-Installation Activities:
Task 2: Provide ADAS Equipment, Installation and Pre-Test System Testing:
Task 3: Subcontract with Emerging Technology Companies
Task 4: Perform ADAS Monitoring During Site Specific Load Test:
Task 5: Remove All ADAS Equipment and Demobilize From Site:
Task 6: Prepare and Submit Final Report:

A plan view of the ADAS layout is shown on Figure 2.12 and the measurement systems included in the ADAS are summarized below:

- Survey monuments to detect and accurately measure movements (i.e., X, Y, and Z) of the I-wall and the protected side earth surface. Survey monuments were installed at the top and base of the center of each of the I-wall monoliths captured by the cofferdam as well as one additional monolith just beyond the north and south cofferdam tie-in points. Survey monuments were also installed at the top, mid-slope and toe of the protected side embankment in-line with the monuments installed on the wall. This installation served to create a range of five survey monuments associated with each I-wall monolith. Additional monuments were installed just beyond the levee toe and at the edge of the street on the protected side. Three additional monuments were installed to function as "control" back-sights. Two backsights were installed adjacent to the I-wall at distances of 200 and 400-feet away from the test site and the third was installed at the edge of the street. Figure 2.13 shows a typical survey prism..

- All survey monuments were read by a pair of robotic, Leica total stations running continuously during the test. One Leica instrument was set up to survey the primary prisms while the second Leica instrument surveyed the secondary prisms. The primary prisms were measured at 5 to10-minute intervals and the secondary prisms were measured at 10 to15-minute intervals. The primary prisms were defined as:
    - the survey prisms mounted on the three interior monoliths (SP-3 and SP-3A, SP-4 and SP-4A, SP-5 and SP-5A),
    - the three survey monuments mounted on the protected side embankment in front of the I-wall (SM-3, SM-4, and SM-5),
    - the three survey monuments mounted on the protected side embankment toe in front of the I-wall (SM-17, SM-18, and SM-19),
    - and the survey monument installed on the cofferdam centerline 50-feet away from the wall on the protected side (SM-23).

  The secondary prisms were defined as every other survey prism or survey monument not included in the primary set.
- 2-dimensional, vibrating wire tiltmeters were attached to the protected-side center of each I-wall monolith captured by the cofferdam. The tiltmeters detect and measure rotation of the concrete stem of the I-wall. Figure 2.14 shows a typical tiltmeter installed under a thin, aluminum sun shield.
- Vibrating-wire, uniaxial crackmeters and manually-read Avongard gages were used to detect and measure differential movement between adjacent I-wall monoliths, between the I-wall and the cofferdam tie-ins, and between the I-wall and the protective casing surrounding the canal side in-place inclinometer. Figure 2.15 shows a typical crackmeter installation on top of the concrete I-wall.
- Three earth pressure cells were installed on the flood side of the I-wall, approximately 18-inches below grade, to indicate any sudden or drastic change in horizontal earth pressure, which would indicate the onset of gap formation. A manufacturer's catalog-cut on the pressure cells is included in the second URS work product entitled *"Structural and Foundation Response Measured During the Site Specific Load Test on the London Avenue Outfall Canal I-Wall/Levee"* in Appendix I to this report.
- Mechanical telltales, consisting of 5/8th inch diameter rods, set inside of a vertical, outer casing that was secured to the canal side of each of the I-wall monoliths captured by the cofferdam. The bottom end of the telltale rod set directly on the ground surface next to the wall and the top of the rod extended 5-feet above the I-wall. The rod was free to move inside of the supporting casing. If a gap began to form between the concrete I-wall and the soil as the wall deflected, the telltale rod would fall into the gap. Drop of the top of the rod provided a visual indication to indicate the on-set of gap formation. Figure 2.16 shows a typical telltale installation with the tip of the telltale bearing on the ground surface immediately adjacent to the concrete I-wall.

- Three in-place inclinometers were installed to measure movement of the I-wall foundation. One inclinometer was installed on the canal side of the wall inside the cofferdam and two were installed on the protected side of the wall: one adjacent to the I-wall and the second half-way down the landside levee slope. Each in-place inclinometer contained 6 accelerometers.
- Electronic pressure transducers installed in open system piezometers to measure the piezometric response in the barrier beach sand. The same pressure transducers were used to measure the canal and cofferdam water surface elevations. Figure 2.17 shows the Geokon pressure transducers used in this test.
- Staff gages were placed in the canal and in cofferdam interior. These devices were manually read. Figure 2.18 shows the staff gage installation for instrument SG-2 located on the cofferdam interior. SG-1 is located opposite SG-2 on the cofferdam exterior to monitor the canal water level during the test..
- Web cameras were installed to provide a constant video stream to monitors inside of the URS instrumentation trailer. The southern camera provided a north-looking video and the north camera a south-looking video.

The final URS report entitled *"Structural and Foundation Response Measured During the Site Specific Load Test on the London Avenue Outfall Canal I-Wall/Levee"* can be found in Appendix I of this report. It provides illustrations and technical descriptions of the instrumentation system. All electronic instruments were wired into the ADAS and were automatically monitored at specific time intervals. Redundancy was provided by multiple instrument types and reliability was increased through comparisons of mathematical correlations made between the various instrument types. Where and when possible, manual readings were obtained to verify the instruments. During each load increment, the results of key instruments were displayed in near-real-time in a centralized trailer under the scrutiny of URS and Corps personnel. The ADAS would not only measure pore pressure increases and deformations as they occur, but also alert the Technical Review Team should deformation or piezometric levels exceed safety-based threshold levels.

Depending on the type of electronic instrument, the ADAS scanned each at different time frequencies:

- The cofferdam and canal surface water level were scanned every 15 seconds.
- Each piezometer was scanned every 15 seconds.
- Each tiltmeter was scanned every 15 seconds.
- The robotic Leica total stations measured the primary prisms at 5 to 10-minute intervals and the secondary prisms every 10 to 15 minutes.
- The crackmeters were scanned every 60 seconds.
- Each in-place inclinometer was scanned every 6 minutes.

Every 15 minutes during the load test, URS compiled data packets containing instrumentation plots and supporting raw data and transmitted them to the Corps Technical Review Team. The team analyzed the data and made recommendations regarding application of the next load increment. Detailed plots and discussion of measurement are presented in subsequent chapters.

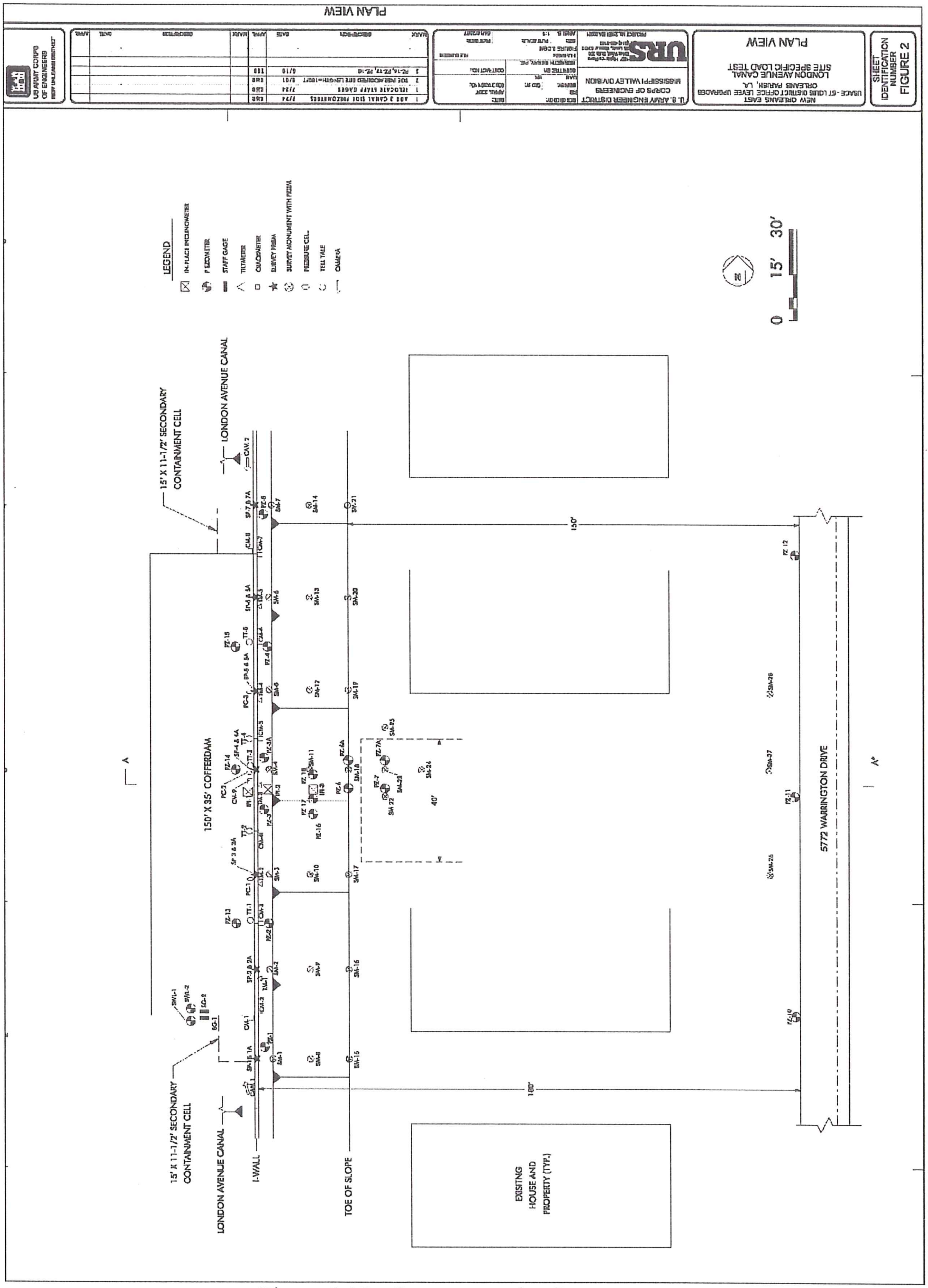


Figure 2.12 – Instrumentation Layout

Figure 2.13 - Survey Prism

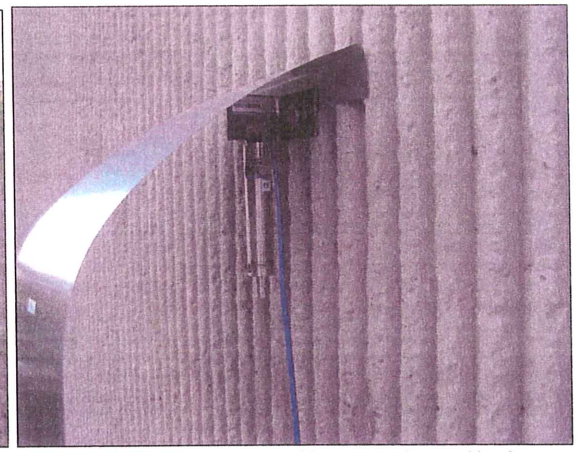


Figure 2.14 - Tiltmeter Installation

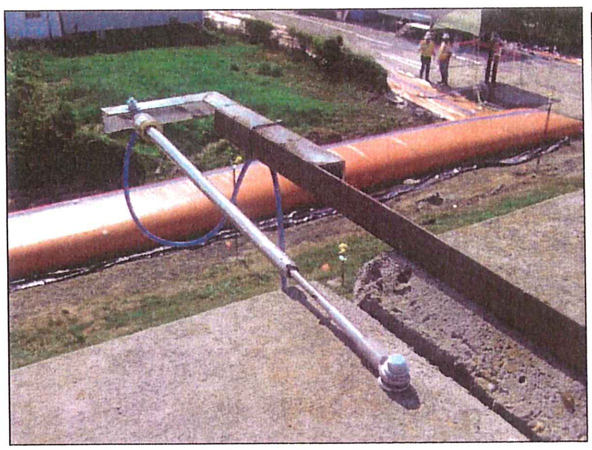


Figure 2.15 - Crackmeter

Figure 2.16 – Telltale Bearing on Soil

Figure 2.17 – Piezometer Pressure Transducer

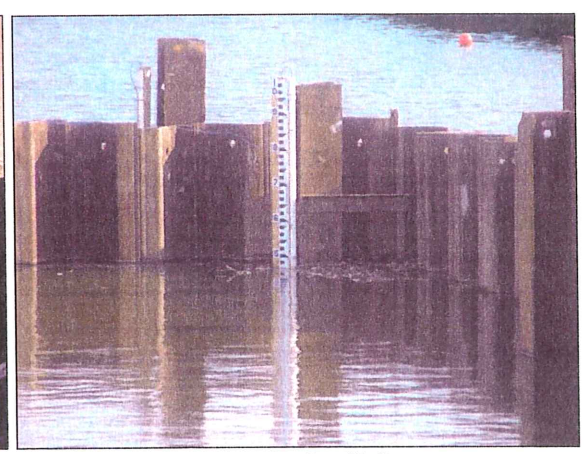


Figure 2.18 – Staff Gage

STAKEHOLDER REVIEW.

Landside Flood Protection. The stakeholders required the Corps to install a temporary flood protection system on the landside perimeter of the load test site. This landside system was designed to contain any water that might come through the floodwall and prevent any flooding of the properties adjacent to the test site in the unlikely event of damage to the I-wall system during the load test. The Corps met this requirement by erecting a 4-foot tall PortaDam™ system on the edge of the landside property and tying it into the landside levee slope. The PortaDam was built 2-feet taller where deployed immediately opposite and parallel to the floodwall at the protected side embankment toe. This extra height was included to contain wave runup in the unlikely event of a monolith failure. Figures 2.19 and 2.20 show the deployed PortaDam.


Figures 2.19 and 2.20 - Landside PortaDam ™ System

Secondary Cofferdams. The stakeholders also required the Corp to build an additional, secondary cofferdam beyond the point where the primary cofferdam tied-into the I-wall. The secondary cofferdam extended approximately 15-feet beyond the north and south ties-ins, was 11.5-feet wide, and built to the same elevation as the primary cofferdam. The secondary cofferdam would provide continued protection in the catastrophic case of an I-wall failure at either of the primary tie-in points. Figure 2.21 shows an excerpt of contract drawing 3/8 (from Appendix G) that details this secondary cofferdam.

PEER REVIEW. The external Peer Review Team reviewed supporting analyses and documentation for the load test and issued a "*Peer Review Report – London Avenue Canal I-Wall Load Test*" dated August 14, 2007. A copy of this report is included in Appendix A of this report. A summary of the report's recommendations follows.

- The reviewers recommend that the load test be performed in two stages to obtain information regarding the development and progression of the gap, as well as under-seepage beneath the levee. The first stage of the load test would be performed to evaluate the development and propagation of the gap, while the second stage would be focused on assessing the potential for under-seepage beneath the levee. Given the stratigraphy beneath the canal, the reviewer's believed that the second stage would simulate a potential worst case seepage condition beneath the levee.

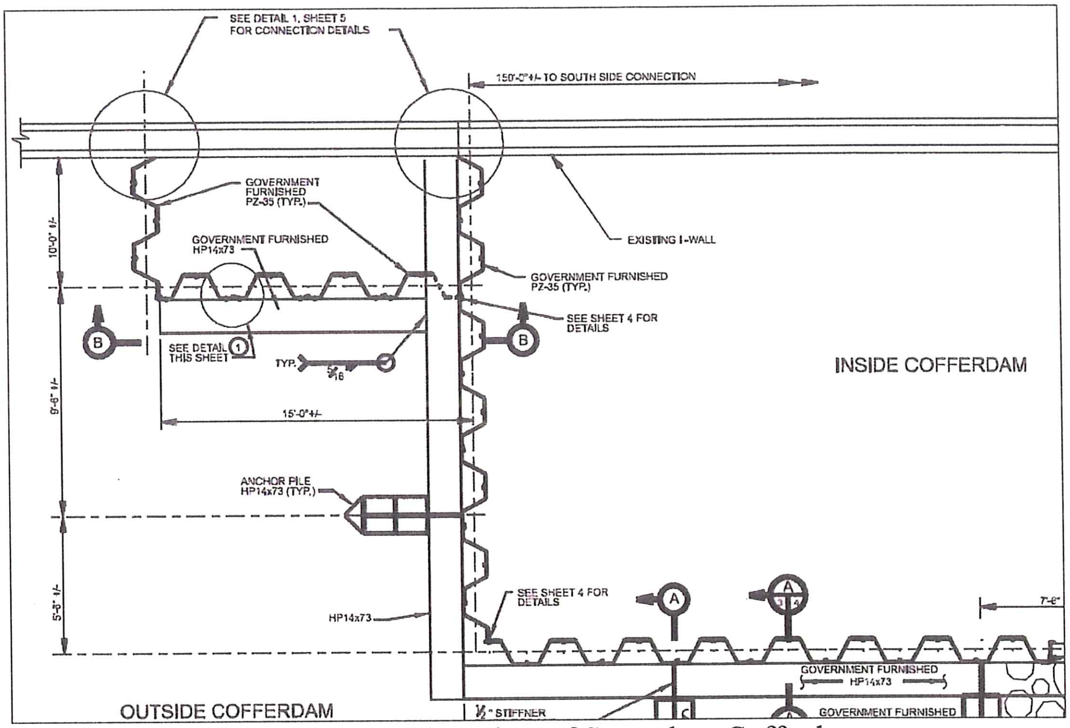


Figure 2.21 - Plan View of Secondary Cofferdam

- The reviewers concur with load test configuration modifications proposed by USACE and agree that these modifications will facilitate the evaluation of the second stage of testing. The proposed USACE modification extends the width of the cofferdam by only 10 ft into the canal and includes installing infiltration pipes into the underlying barrier beach sand beneath the bottom of the canal. It is also recommended that that the depth of the sheetpiles used for construction of the cofferdam be extended into the clays underlying the sand stratum to facilitate data evaluation during the load test. Again, the entire team believed that these details would lead to potential worst case seepage conditions.
- Several additions/modifications were recommended regarding the original load test program prior to commencement of the program. These changes include obtaining additional strength and stiffness characterization data in the subsurface soils on the landside of the levee, as this information will help in the analysis of the results from the load test program, as well as assisting future activities related to extending the results of the load test program to other reaches of the levee.
- It was recommended that the data collection activities during the load test focus on monitoring pore pressures in the underlying sands in lieu of monitoring physical deformations of the I-wall gap, as the increase in water pressure is likely the most important end result of wall deformations.
- It was recommended that the originally proposed load testing protocols be adjusted to allow stabilization of the pore water pressure and wall deflection readings within predetermined limits prior to incrementally increasing the hydrostatic loads. This recommendation would also allow assessing the time-dependent nature of pore pressure and wall deformations during sustained loading.

- The reviewers believed it beneficial to have individuals from USACE, ERDC, and Virginia Tech (Technical Review Team) who were responsible for the post-Katrina canal numerical analyses maintain an on-site presence during the performance of the load test so that their analysis models can be updated in real-time using the actual performance monitoring test results.
- The SLFPA requested that the external Peer Review Team include comments regarding the interpretation of the load test results and the extension of the load test results to the evaluation of the SWE in the entire canal.
- It was recommended that results of the two-stage hydrostatic load test be coupled with additional subsurface characterization tests performed along the canal and pore water pressure monitoring results along select portions of the canal during service operations be used to assess the reasonableness of any proposed increase in the SWE in the canal. Although the reviewers felt that it was premature at the time of their report to identify a specific framework for assessing the results of the load test and the additional subsurface characterization testing and pore pressure monitoring, the reviewers envision several potential outcomes, including: (i) the use of additional subsurface characterization testing to help identify conditions along the canal and to assess whether they are similar or different than the conditions in the vicinity of the load test and to subsequently identify locations for installing additional instruments (e.g., piezometers); and (ii) the use of installed piezometers along the canal to monitor pore pressures during service conditions and to compare their response. This latter recommendation would use the piezometer response during the load test in such a way to allow the load test results to be used to help estimate the response time for underseepage under increased water elevations, possibly allowing the duration of loading to be used to control the SWE.

Based on the Peer Review Comments, the Corps of Engineers changed its contract documents and load tests procedures as follows:

- The cofferdam width was increased to 35-feet and ERDC and Virginia Tech completed the necessary studies to estimate the diameter, length, and spacing of the slotted pipes needed to create similar impacts on the groundwater regime as the condition of the barrier beach sand in the bottom of the canal. The results of this study entitled *"Analyses of Injection Wells for London Avenue Canal Test Section"* may be found in Appendix J.
- The Corps revised its load test program, breaking it into Phase I and Phase II. Phase I would be run assuring that the bottom of the cofferdam was tightly sealed so that impacts on the groundwater regime were minimized. Three days were scheduled between Phase I and Phase II to allow the contractor time to install the slotted pipes. Phase II would be run with the installed slotted pipes operational.
- The Corps revised its construction drawings to require the sheetpiles to be driven completely through the barrier beach sand and 5-feet into the underlying Pleistocene clays.
- The Corps completed additional exploration obtaining 5-inch diameter, undisturbed samples of the levee embankment materials and marsh clay.

- The Corps changed its instrumentation plan to include three additional piezometers located at mid-slope of the landside embankment. These are piezometers PZ-16, PZ-17, and PZ-18.
- The Corps revised its anticipated loading protocols to include waiting until all measured I-wall movement and all measured piezometric changes stabilized (became constant with time) under the given load.
- The Corp assigned responsibility to individuals commensurate with their overall responsibility in the load test. A "Load Test Decision Template" that addresses this concern may be found on Figure 4.3 of Chapter 4. The HPO arranged for the Technical Review Team to be on-site during the test. Mr. Frank Vojkovitch (CEMVN) and Mr. Patrick Conroy (CEMVS) were also required to be on-site for the entirety of the load test.
- The Corps did not completely agree to the last recommendation of the Peer Review Team. This recommendation included coupling the results of the two-stage hydrostatic load test with additional subsurface characterization tests performed along the canal and pore water pressure monitoring results along select portions of the canal during service operations to assess the reasonableness of any proposed increase in the SWE in the canal. USACE recognizes that the currently installed piezometers along the canal can be monitored during service conditions and their responses compared to the piezometer response during the load test.

CHAPTER 3 – CONSTRUCTION

General: This chapter will describe some of the general contracting provisions and construction aspects that were established for constructing the load test cofferdam.

Advertising and Award. To meet an accelerated schedule, the HPO issued the contract documents to prospective bidders and requested submittals within approximately one-week. During this period, the interested bidders were invited to a pre-bid meeting and site visit. On 6 June, 2007, the HPO awarded the contract to Allen Wright Enterprises. The award was for $2.7M and required the contractor to have the cofferdam constructed and ready for load test commencement on or before 17 August, 2007

Contract Submittals. Per the contract requirements, the contractor submitted shop drawings and plans detailing the following major cofferdam items. All work was completed over-water, from the canal side of the wall using barge mounted equipment.

- Sheetpile Installation. The contractor installed the sheetpiles using an MKT V-20B vibratory hammer powered by an HP 325B power pack. The contractor fashioned various driving templates by driving vertical H-piles and attaching horizontal guide beams across them. To ensure successful installation, the contractor completed a narrow, 2-foot deep excavation along the path of the sheetpile to clear all debris that might impede sheetpile installation. Significant amounts of heavy debris were cleared by this operation. Only two sheetpiles were unable to be driven to the design depth due to deep, underground obstacles. Figures 3.1 shows the contractor beginning to drive sheetpile on the established template. And Figure 3.2 shows the sheetpile cofferdam well underway.

Figure 3.1 – Establishing Sheetpile Template

- H-Pile Installation. The contractor used a Vulcan-06 and a Conmacco 65E5 hammer to drive the H-piles. The H-piles were supported in swinging leads. Figure 3.3 shows the contractor's pile driving system.

Figure 3.2 – Sheetpile Cofferdam Underway

Figure 3.3 – H-Pile Driving Hammer in Swinging Leads

- Sluice Gate Details. The contractor provided detailed shop drawings of the sluice gates, its supporting frame and sealing mechanisms.

- Cofferdam Emergency Emptying System. The contractor submitted a plan that provided a total pumping capability of 77,000 GPM using two individual pumps (i.e., 42-inch diameter with 54,000 GPM, and 30-inch diameter with 23,000 GPM). These pumps were hydraulic, vertical turbine pumps driven by diesel engines.

- Jet Grouting Procedures. The contractor submitted plans and procedures to install jet grout columns on the cofferdam interior where the primary sheetpile cofferdam was joined to the existing concrete I-wall construction joint. Figure 3.4 shows the jet grouting system installing jet grout columns at the southern end of the cofferdam.

Figure 3.4 – Jet Grouting Operation

Cofferdam Completion. The contractor completed the cofferdam and all attendant features per the contract schedule. Figure 3.5 shows the substantially completed cofferdam.


Figure 3.5 – Almost Completed Cofferdam

Cofferdam Modifications for Phase II.

After the completion of the Phase I portion of the load test (subsequently discussed in this report) and before the beginning of the Phase II portion, the contractor installed 29-slotted pipes along the inside face of the 150-foot long cofferdam wall. The purpose of the slotted pipes was to allow the 35-foot wide cofferdam to simulate a section of the London Canal that had the barrier beach sand in direct contact with the bottom of the canal. As described before, this was intended to simulate a potential worst case condition related to under-seepage beneath the levee.

Each of these slotted pipes consisted of a 6-inch diameter, 10-foot length of slotted PVC pipe (Johnson #10-slot) attached to a 10-foot long length of solid, 6-inch diameter PVC pipe in turn attached to a 6-inch diameter, 10-foot length of slotted PVC pipe (Johnson #20-slot). Figure 3.6 shows an illustration of the slotted pipe. The #10-slotted end of each pipe was jetted into the barrier beach sand so that its tip was at elevation -22. A 4-inch diameter jet pipe with a single downward-facing, 1-inch diameter jetting orifice was located alongside the slotted pipe during its installation. The jet casing pipe was used to clear any soil that would have accumulated inside the 6-inch diameter PVC pipe during installation. Figure 3.7 shows the jetting process.

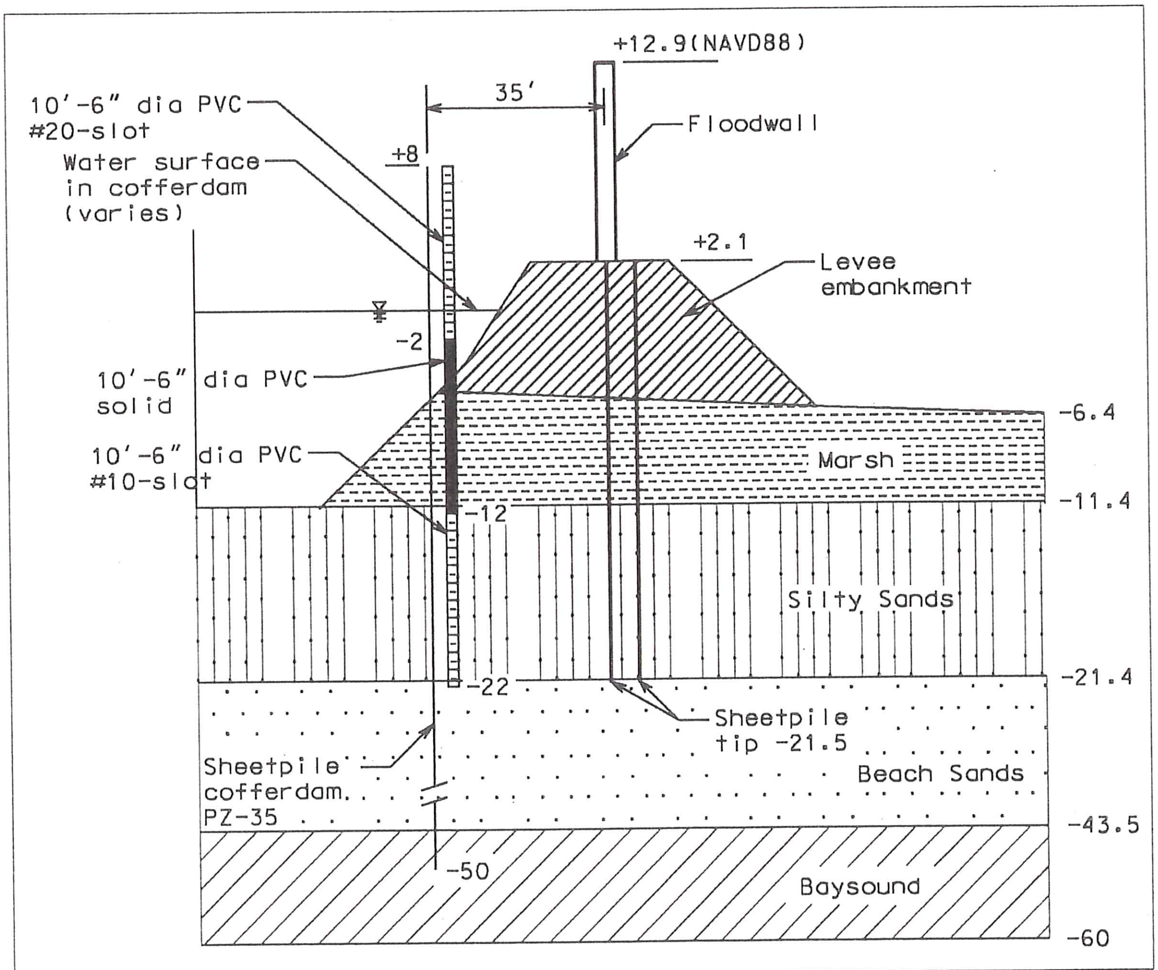


Figure 3.6 – Illustration of Slotted Pipe

Figure 3.7 – Slotted Pipe Installation Using Jetting Techniques

The upper 10-feet of the 6-inch diameter pipe (i.e., the end with the #20-slots) was open to the surface water in the cofferdam. The diameter, depth, and spacing of these slotted pipes allowed the surface water in the cofferdam to charge the groundwater regime in the barrier beach as if the canal bottom was in full contact with the beach sand.

A 20-foot length of 12-inch diameter PVC pipe was positioned in place as a "shut-off valve" over each piece of slotted PVC. By lowering the larger piece of PVC "shut-off" pipe (i.e., blue pipe in Figure 3.8) over the slotted PVC (i.e., white pipe in Figure 3.8), and embedding the larger "shut-off" pipe into the clay at the bottom of the cofferdam, the surface water in the cofferdam was prevented from entering the slotted PVC. This would effectively isolate the slotted pipe from the surface water in the cofferdam. The partially completed array of slotted pipes along the 150-foot long cofferdam wall (some with/without the shut-off pipes) is shown in Figure 3.9.

All of the blue "shut-off" pipes were installed. Then one by one, a single shut-off pipe was removed and the interior of that slotted pipe was cleaned by water-jetting to ensure that the slots were not clogged. When this cleaning process was completed, the shut-off pipe was reinstalled over the cleaned, slotted pipe, and the process was repeated on the next slotted pipe. This cleaning process was completed on every slotted pipe and is shown in Figure 3.10.

The effectiveness of four selected slotted pipes were measured by first running a 10-gallon slug test; then developing that pipe by air-lifting; and then running another slug test. The slug tests showed no appreciable improvement in permeability created by the air-lift development so the process was discontinued.

Figure 3.8: Shut-off Pipe In-Place over Slotted Pipe.

Figure 3.9 – Partially Completed Array of Slotted Pipes.

Figure 3.10 – Cleaning Each Slotted Pipe by Water Jetting.

Immediately before the beginning of the Phase II load test, all of the blue shut-off pipes were removed allowing the water in the cofferdam access to the slotted pipes. The shut-off pipe removal was done methodically, beginning with the center pipe, and then alternately removing a shut-off to the north and then a shut-off to the south. The Corps kept a close eye on the piezometric pressures while the shut-off pipes were removed. The first 6-inch increment of water was not added into the cofferdam until the groundwater regime achieved a steady state condition.

CHAPTER 4 – LOAD TEST PROTOCOLS AND EXECUTION DETAILS

GENERAL. This chapter will describe details about the load test including protocols, alert values, overnight hold elevations, multiple test phases (Phase I and Phase II), and control of cofferdam water surface elevation.

Loading Protocols.

In accordance with the established testing protocols, the Corps controlled the loading on the instrumented I-wall by instructing the contractor to pump water into the cofferdam, raising the water surface elevation in 6-inch increments. After each increment, the Corps' Technical Review Team would examine the measured data and decide whether or not to add the next load increment. Per the Peer Review group's recommendations, the next load increment would not be placed until the wall, its foundation, and the groundwater regime all reached equilibrium from the previous load.

Per the stated requirements of the Technical Review Team, URS extracted all desired data from the ADAS database at 15-minute intervals and sent it electronically to the Technical Review Team. The team tracked the response of the I-wall system by plotting wall deflections, inclinometer readings, and piezometer response against time. Only when the team reached a consensus that all metrics had reached steady state, and it would be safe to do so, did the team approved the application of the next load increment. The instruments receiving the closest scrutiny included

- Survey prisms on the five monoliths captured within the cofferdam including: SP-2 and 2a, SP-3 and 3a, SP-4 and 4a, SP-5 and 5a, and SP-6 and 6a. The technical review team closely monitored these prisms, watching the monolith movements and time trends for each monolith and comparing these to maximum allowable movements.
- Piezometers: PZ-6, PZ-6a, located on the cofferdam centerline at the landside toe and PZ-7, and PZ-7a located on the centerline 50-feet away from the wall. The technical review team closely monitored these piezometers, comparing the seepage pressures at and just beyond the protected side embankment toe to the maximum allowable pressures
- In-place inclinometers: IPI-1, IPI-2, and IPI-3 located on the cofferdam centerline. The technical review team closely monitored the movement of the entire soil mass to ensure that the computer deflections with depth looked reasonable.

If requested by the Technical Review Team, URS would extract measurement data from any other instrument in the ADAS and provide that to the team. In this way, the team could plot the data and observe the trends of any instrument that the team decided merited a closer look.

URS provided electronic data files of the processed instrumentation results (i.e., results in engineering units) in a comma separated variable (.csv) format to the team who imported the files into various established Excel spreadsheets that had been created prior to the start of the load test. Upon copying and pasting the data into the spreadsheet, the newest data point was automatically appended to the existing data file and plots were automatically updated. The work of data import and plotting was distributed among the Technical Review Team to expedite their analyses.

Amber and Red Alerts.

The Technical Review Team established Amber and Red alert levels for what were considered "critical indicators" and URS programmed these values into the ADAS. If any instrument exceeded the instrument- and location-specific predetermined amber or red alert level, an audible alarm would sound in the URS trailer and a large amber or red visual indicator would appear on the main ADAS console. An "Amber Alert" would remind the team that some aspect of the I-wall system response was approaching a critical level. A "Red Alert" would be cause for immediate test termination. Given the technical review team's close scrutiny of the measured data and the strong trends that developed during the test, the technical review team became proficient at anticipating the key indicator response with additional increases in the cofferdam water surface..

Some alert levels were pre-determined before the test and remained unchanged throughout the test. These absolute alerts were the measured horizontal deflection of the wall (top and bottom), the measured heave at the protected side embankment toe, wall rotation, and the measured piezometric elevation at the protected side embankment toe. The values of these alerts were set to:

Table 4.1 – Amber and Red Alert Values

Metric	Amber Alert Value	Red Alert Value
Base Wall Deflection	0.75-inch	1.50-inch
Protected-Side Mid-Slope Heave	0.25-inch	0.50-inch
Protected-Side Toe Heave	0.25-inch	0.50-inch
Tiltmeters	0.5-degree	1.0-degree
Protected-Side Toe Piezometric Elevation	-5.70-ft NAVD88	-4.00-ft NAVD88

An additional dynamic alert was developed and updated with each additional load based on the piezometric response of the underseepage regime. Based on its collective experience and results of numerous numerical analyses, the Technical Review Team concluded that the plot of piezometric elevation versus cofferdam water surface elevation would be a linear relationship with a relatively constant slope for any water surface elevation in the cofferdam. The slope of this correlation would remain approximately constant as long all seepage-related conditions remained unchanged until (potentially) a gap opened. The Team further assumed that if a canal side gap propagated to the top of the underlying barrier beach sand, the effective entrance condition for the underseepage would become more aggressive and that next piezometric data point would plot above the previously established trend. The Team revised the value of the dynamic alert for each subsequent load increment based on the observed trend of all previously measured data.

Multi-Part Load Test. The Peer Review group recommended that the load test be performed in two stages to obtain information regarding the development and progression of the gap, as well as under-seepage beneath the levee. The first stage of the load test would be performed to evaluate the development and propagation of the gap, while the second stage would be focused on assessing the potential for under-seepage beneath the levee. Given the stratigraphy beneath the canal, the reviewer's believed that the second stage would simulate a potential worst case seepage condition beneath the levee. Based on this comment, the Corps performed the load test in two separate, independent parts: Phase I and Phase II.

Phase I. Phase I of the load test was performed prior to installing the slotted well pipe and represented the case of marsh clay in the bottom of the canal A near watertight seal was maintained in the bottom of the cofferdam due to the marsh clay soils overlying the barrier beach sand. Based on the exploration data and the Technical Review Team's analysis results, the team was confident that the cofferdam bottom was covered with a relatively low permeability soil and would provide a very tight seal, limiting the hydraulic connection between the surface water in the cofferdam and the underlying barrier beach sands. At the stakeholders recommendation, the Corps placed bentonite pellets along the inside face of the sheetpile for the entire length of the 150-foot cofferdam wall to ensure that there would not be a seepage entrance along the sheet pile walls. The bentonite pellets were intended to seal the shallow, debris-removal excavation that had been made during the sheetpile installation.

Phase I was completed raising the cofferdam water surface in 6-inch increments to a maximum elevation of +7.0-feet (NAVD88). The Technical Review Team reviewed all survey prism, inclinometer, and piezometric data obtained from the ADAS during the Phase I test. The data obtained from the Phase I load test are described in subsequent chapters.

Phase II. Phase II of the load test was performed in the cofferdam once the "Blue Pipes" were removed. Recall that this phase of the test was intended to represent the condition of barrier beach sand in the bottom of the canal and assuming full hydraulic contact between the canal and the barrier beach sand. After completion of the Phase I test, the contractor installed 29 slotted PVC pipes along the inside edge of the 150-foot long cofferdam wall. The slotted pipe installation is described in Chapter 3 of this report. The analysis supporting the configuration of the slotted pipes may be found in Appendix J. Phase II was completed raising the cofferdam water surface in 6-inch increments to a maximum elevation of +7.0-feet (NAVD88). The Technical Review Team reviewed all survey prism, inclinometer, and piezometric data obtained from the ADAS during the Phase II test. All data are described in subsequent chapters.

Overnight Hold Elevation.

Phase I: During discussions with its stakeholders, the Corps agreed that if the cofferdam water surface was higher than +4.0-ft (NAVD88) at the end of the day, that the contractor would lower the cofferdam water surface to +4.0-ft and maintain that level overnight. Prior to and during the load test, +4.0-feet was the recognized SWE in the Canal. To maintain the water surface at the +4.0-feet elevation overnight, a URS employee monitored the ADAS and reported the cofferdam water surface elevation to the contractor with a recommendation to add water to the cofferdam, continue pumping, or to cease pumping. This was accomplished per plan with no reportable incidents or concerns during the overnight holds of the Phase I load test. The cofferdam water surface was held at +3-ft (NAVD88) the night of 18 August and at +4-feet the nights of 19, 20, and 21 August. The end of the Phase I load test occurred mid-afternoon on 22 August. Figure 4.1 shows a time plot of cofferdam and Canal water surface elevations for Phase I.

Phase II: The Phase II test was performed after the "shut-off" pipes were removed, thus allowing water from the cofferdam to enter the slotted pipes. Based on experience gained during the Phase I test and the desire to be conservative, the Technical Review Team recommended that the cofferdam be allowed to completely drain to the prevailing London Canal water surface elevation at the end of each days testing during Phase II. This was achieved by simply turning off the pumps and letting sheetpile interlock leakage empty the cofferdam. The Corp was comfortable doing this because the experience gained during Phase I showed that the contractor could completely re-water the cofferdam in 30 to 40 minutes. A URS employee remained at the ADAS overnight to monitor the landside piezometric levels during the Phase II test. This was accomplished per plan with no reportable incidents or concerns. Figure 4.2 shows a time plot of cofferdam and London Avenue Canal water surface elevations for Phase II. Note that the Phase II overnight hold elevations of the cofferdam water surface are the same as the open canal water surface elevations. Also note the increase in canal water surface elevation on the morning 27 August. This was due to a test of the S&WB pumps.

Load Test Decision Template. The Corps, the Peer Review Team, and the Technical Review Team developed a decision template that established "explicit responsibilities" for all individuals involved in the load test. This template is shown on Figure 4.3. As shown by the decision template, all recommendations from the contractor, the Technical Review Team, the stakeholders, and all other teams identified on the template would be funneled through the "Officer-In-Charge" who would make the final decision on whether or not the next load increment should be added to the wall.

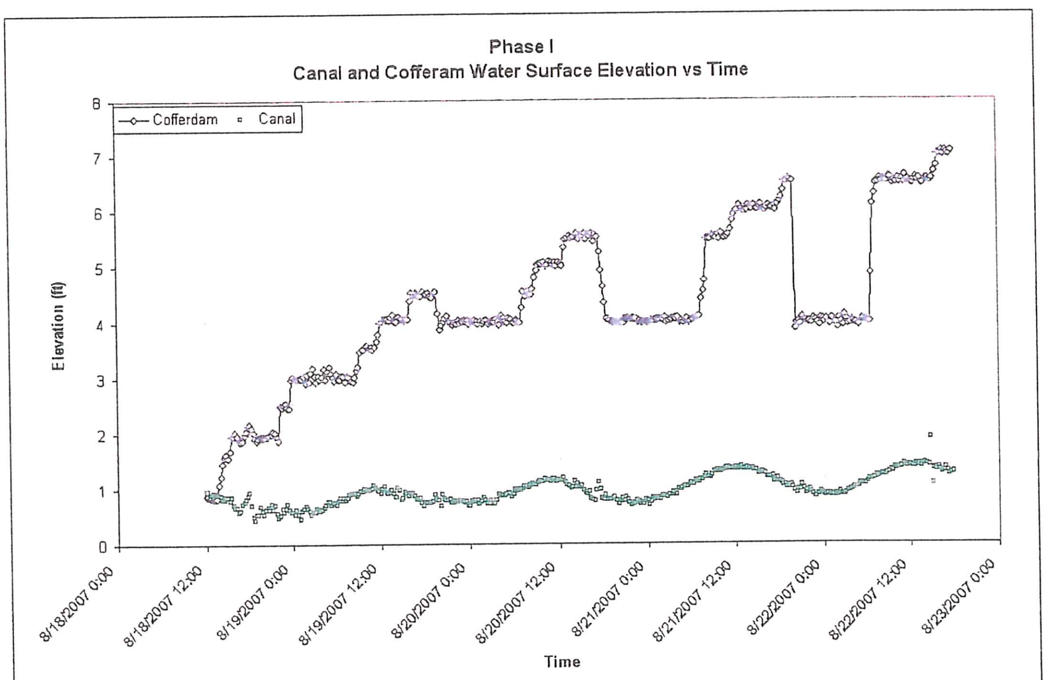


Figure 4.1 – Phase I Cofferdam and Canal WSE vs Time

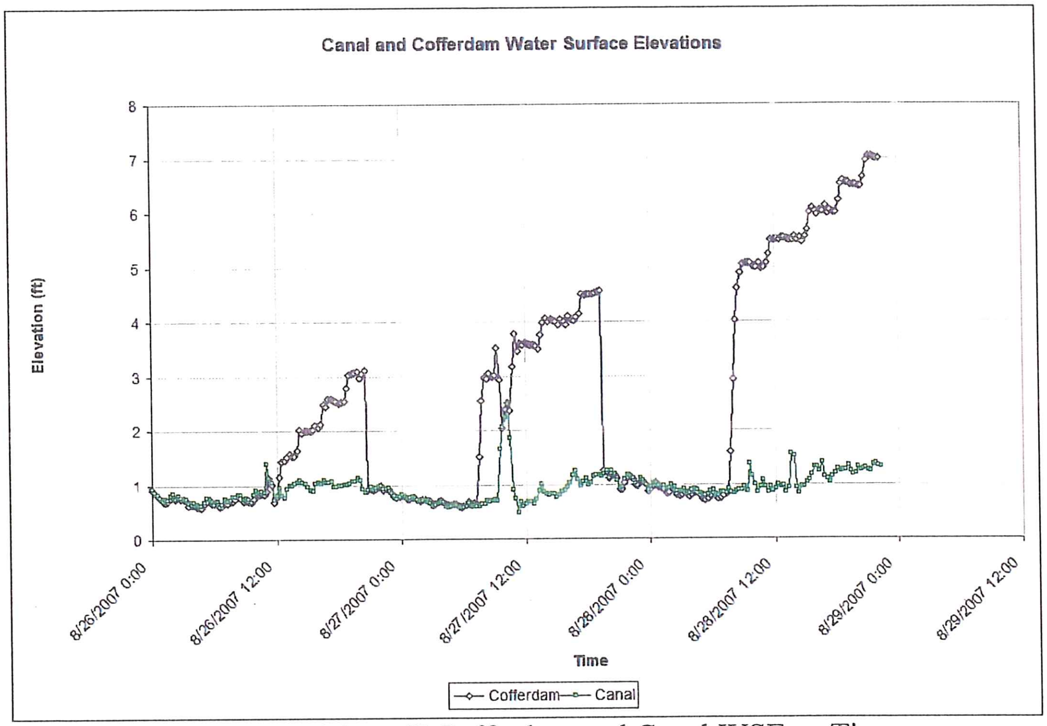


Figure 4.2 – Phase II Cofferdam and Canal WSE vs Time

Cofferdam Water Surface Control.

The contractor mobilized a bank of 4-inch, 6-inch, and 12-inch diesel powered pumps to fill the cofferdam. These pumps were arranged on the edge of the contractor's work barge along the 150-foot long cofferdam wall. The contractor ran the pumps as necessary, pumping water from the Canal into the cofferdam to raise or maintain the cofferdam water surface elevation as instructed by the Corps' Officer in Charge.

The Corps established a direct-communication radio link between the landside URS instrumentation control trailer and the contractor's superintendent located on the work barge. Also, the ADAS web cameras provided streaming video of the cofferdam to a monitor in the instrumentation control trailer. While the contractor was pumping water into the cofferdam, the Technical Review Team and the officer-in-charge (both located in the URS instrumentation trailer) could observe the progress of the cofferdam filling via the ADAS console, advise the contractor of his progress, and when the contractor should stop/continue filling. These procedures allowed the team to control the water surface elevation in the cofferdam to within +/- 0.10-foot of the desired level.

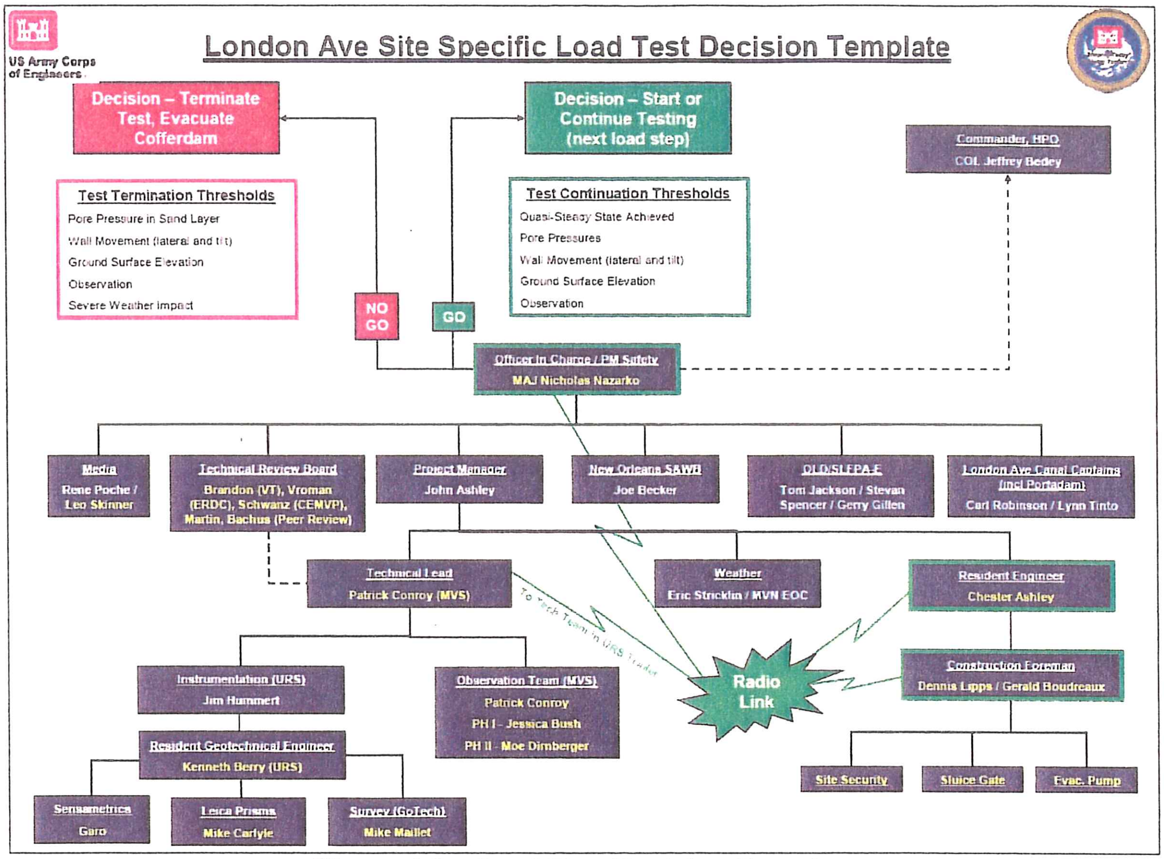


Figure 4.3 – Load Test Decision Template

CHAPTER 5 - PIEZOMETRIC MEASUREMENTS

General. This chapter describes the piezometric measurements obtained during the test, developed piezometric relationships, and discussion.

Piezometer Measurements. Open system piezometers were installed on the canal side of the wall in the cofferdam and on the protected side of the wall on the crest, mid-slope and at the toe of the embankment, 50-ft from the wall and at the street curb. A Geocon transducer was installed in each open system piezometer and wired into the ADAS system. The piezometers in each group are:

Canal Side: PZ-13, PZ-14, and PZ-15.
Protected Side Crest: PZ-1, PZ-2, PZ-3, PZ-3A, PZ-4, and PZ-5.
Protected Side Mid-Slope: PZ-16, PZ-17, and PZ-18.
Protected Side Toe: PZ-6 and PZ-6A;
Protected Side 50-feet from wall: PZ-7 and PZ-7A
Protected Side Street: PZ-10, PZ-11, and PZ-12.

A falling head test was completed on every open system piezometer once the piezometer was installed and confirmed operational by URS. The tip elevations of each piezometer and the recovery data obtained from the falling head tests are summarized in Table 5.1 below and more details may be found in table 5.3 at the end of this chapter.

Table 5.1 – Summary of Piezometer Falling Head Tests

Piezometer Number	Tip Elev	Recovery Time		Piezometer Number	Tip Elev	Recovery Time
PZ-1	-23	4-min		PZ-2	-23	2-min
PZ-3	-23	120-min		PZ-3A	-23	60-min
PZ-4	-23	4-min		PZ-5	-23	4-min
PZ-6	-19	15-min		PZ-6A	-19	60-min
PZ-7	-19	30-min		PZ-7A	-19	10-min
PZ-16	-20	4-min		PZ-17	-20	4-min
PZ-10	-29	4-min		PZ-11	-24.5	4-min
PZ-12	-27	4-min		PZ-13	-22.5	30-min
PZ-14	-22.5	15-min		PZ-15	-22.5	55-min
* No falling head test performed on PZ-18. PZ-8 & PZ-9 not installed.						

A study of this information reveals that without exception, any tip installed at or deeper than elevation -24.5 (PZ-10, -11, -12) returned to its original condition in 4 minutes or less. Six of the piezometer tips installed above elevation -24.5 responded in this fashion while the remaining 9 responded much slower. The slow response of the 9 piezometers indicates that they were 'tipped' in less permeable silty sands located at the top of the barrier beach sand as shown by Figure 2.6 "Test Site Centerline Stratigraphy".

Phase I Piezometric Measurements. All of the piezometers described above were used during the Phase I load test to obtain piezometric elevations immediately canal-side of the wall, immediately adjacent to the wall on the protected side, mid-slope on the protected side embankment, at the toe and 50-ft from the wall, and at the street. Detailed plots of measured piezometric and cofferdam water surface elevations versus time for the Phase I portion of the load test may be found on Figures 5.11 through 5.14 located at the end of this chapter.

Discussion of Phase I Piezometric Data.

The time plots show that during the Phase I test, the groundwater regime, as monitored by the canal-side and landside piezometers, did respond to the changes in the cofferdam water surface. Figure 5.1 shows a time plot of the piezometric data obtained from the piezometers located closest to the cofferdam centerline. Beginning with PZ-14 on the canal side of the wall and extending to PZ-11 on the protected side at Warrington Avenue, the piezometric response decreases with increasing distance from the cofferdam. This reflects the head loss in the groundwater as it flows through the levee embankment, marsh clay and barrier beach sand.

Figures 5.2, 5.3, and 5.4 are correlation plots between the cofferdam water surface elevation (CWE) and the piezometric elevations obtained from the canal side and protected side piezometers. Figure 5.2 is the correlation plot for the three canal side piezometers (PZ-13, -14, and -15). Figure 5.3 is the plot for the five piezometers located on the protected side levee crest, adjacent to the flood wall (PZ-1, -2, -3, -3A, -4, and -5). And Figure 5.4 is the plot for the four piezometers located at and near the protected side levee toe (PZ-6, -6A, -7, and -7A). These three plots show all of the piezometric data obtained from these instruments during the Phase I load test. These data represent when the cofferdam was being filled and when it was being emptied. These data also represent when the cofferdam water surface was being held constant after a load increment or when it was held constant during the overnight holds.

As expected, the piezometric response is positive with respect to the CWE, although the response is muted until the CWE reaches elevation +1.5 (NAVD88). From there until CWE of +4.0, the response of each piezometer forms a relatively linear relationship. At CWE equal to or greater than +4.0, the linear relationship becomes less apparent, due to the piezometric measurements made while the cofferdam water surface is lowered back to the overnight hold elevation of +4.0-ft. The data indicates that the piezometric regime responds more slowly when lowering the cofferdam water surface then it does when the cofferdam is being filled.

Figures 5.2a, 5.3a, and 5.4a are correlation plots between the CWE and the piezometric elevations measured only during times of cofferdam filling and cofferdam water surface holds after each load increment. These plots contain no data for when the CWE is being lowered nor do they contain data from overnight holds.

Figure 5.2a is the correlation plot for the three canal side piezometers. Figure 5.3a shows data only from piezometers PZ-3 and PZ-3A. These two piezometers are located closest to the cofferdam centerline on the protected side levee crest, adjacent to the flood wall. This plot is set up this way to reduce any scatter caused by 3-D effects that could occur near the ends of the cofferdam. Figure 5.4a is the plot for the four piezometers located at and near the protected side levee toe.

The linear trends described earlier become more apparent in figures 5.2a, 5.3a, and 5.4a. The straight lines scribed in these figures are drawn through the steady-state piezometric pressure, or maximum piezometric level, measured at each CWE. Note that this results in a "broken" relationship for each piezometer. With the exception of PZ-14, the linear relationship "breaks" at CWE of +6.50-feet and greater. The PZ-14 relationship breaks at CWE = +5.50-feet.

The higher piezometric measurements beyond the "break" are indicative of more aggressive underseepage conditions. A possible explanation for this "break" in the piezometric trend could be a gap on the riverside of the I-wall (between the I-wall concrete/sheetpile and the adjacent soil) that had propagated down to the top of the Beach Sand aquifer. Such a gap would cause a shorter effective entrance distance and thus a more aggressive underseepage condition.

The slope of each line defines the piezometer response of the barrier beach sand at the point of measurement to a successive increase in the cofferdam water level. The slopes of the initial (before the "break") correlations for the Phase I data are:

- The average slope of the three canal side piezometers is 0.252, meaning that the piezometric level rose about 0.25-foot for every foot of increase in the cofferdam water surface elevation.
- The average slope of the piezometers at the cofferdam centerline at the protected side crest is 0.218.
- The average slope of the piezometers at the cofferdam centerline at the protected side toe is 0.146.

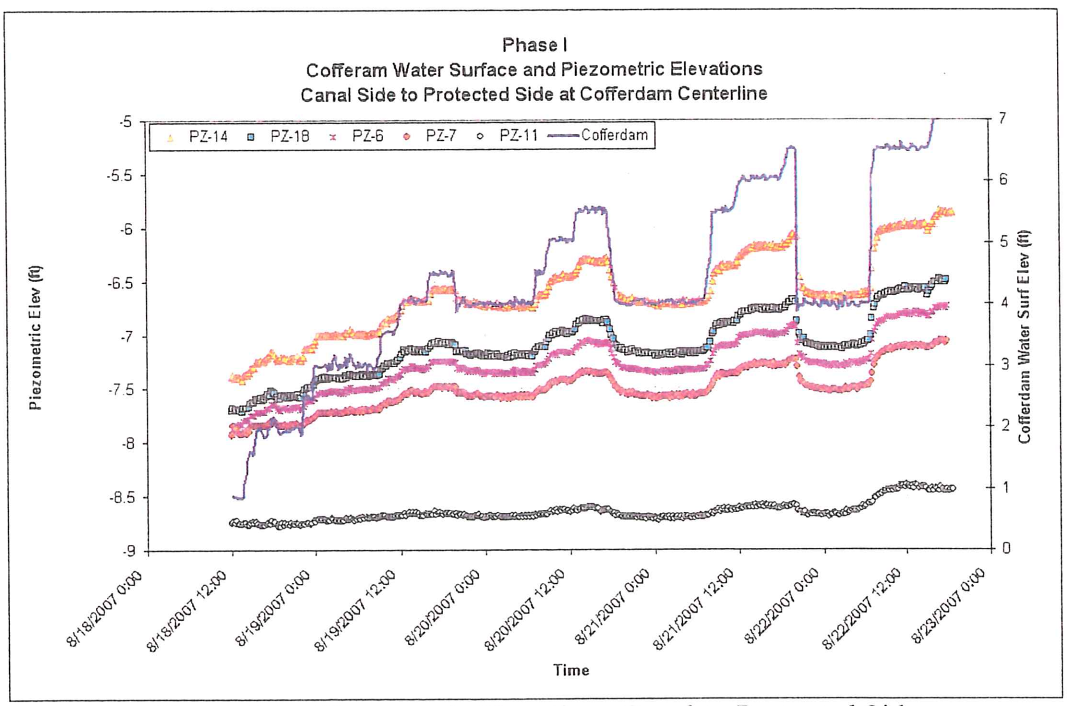


Figure 5.1 – Piezometric Elevations Canal to Protected Side

Phase II Piezometric Measurements. The same piezometers sets used during the Phase I portion of the load test were used to obtain piezometric elevations during the Phase II portion. Figures 5.15 through 5.18 (located at the end of this chapter) show the detailed piezometric, canal, and cofferdam water surface elevations plotted versus time for the Phase II portion test.

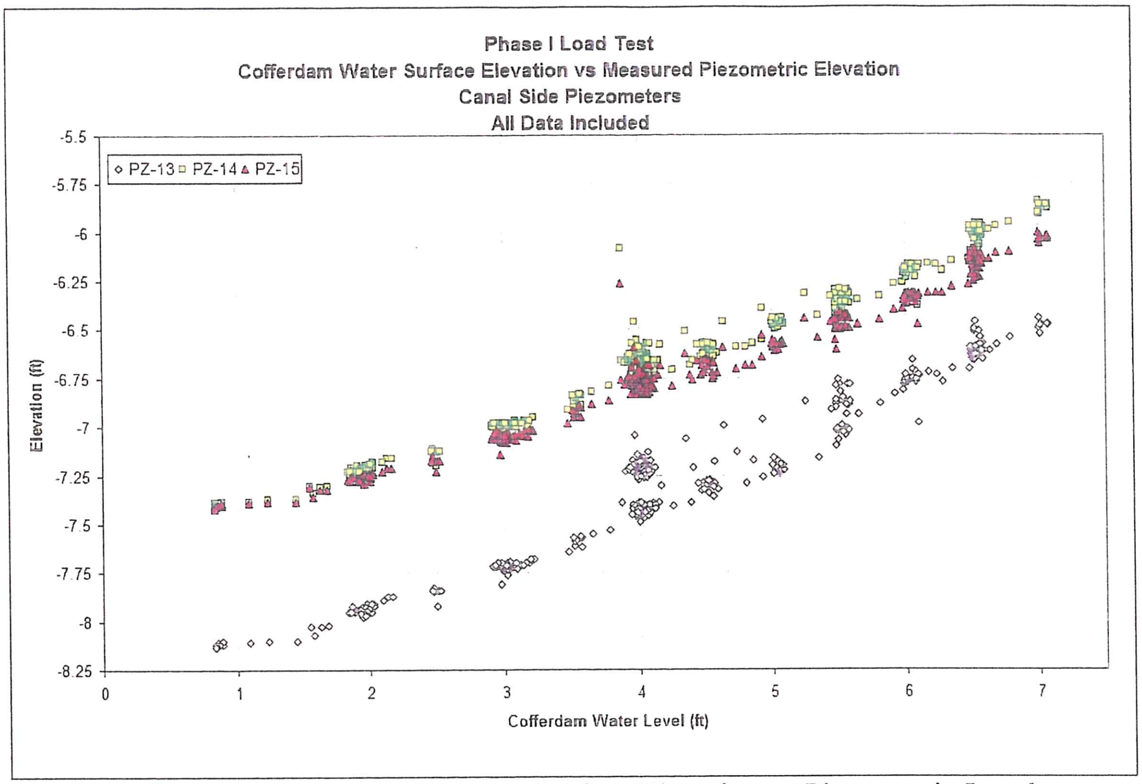


Figure 5.2 – Cofferdam Water Surface Elevation vs Piezometric Level
Canal Side Cofferdam, All Phase I Data

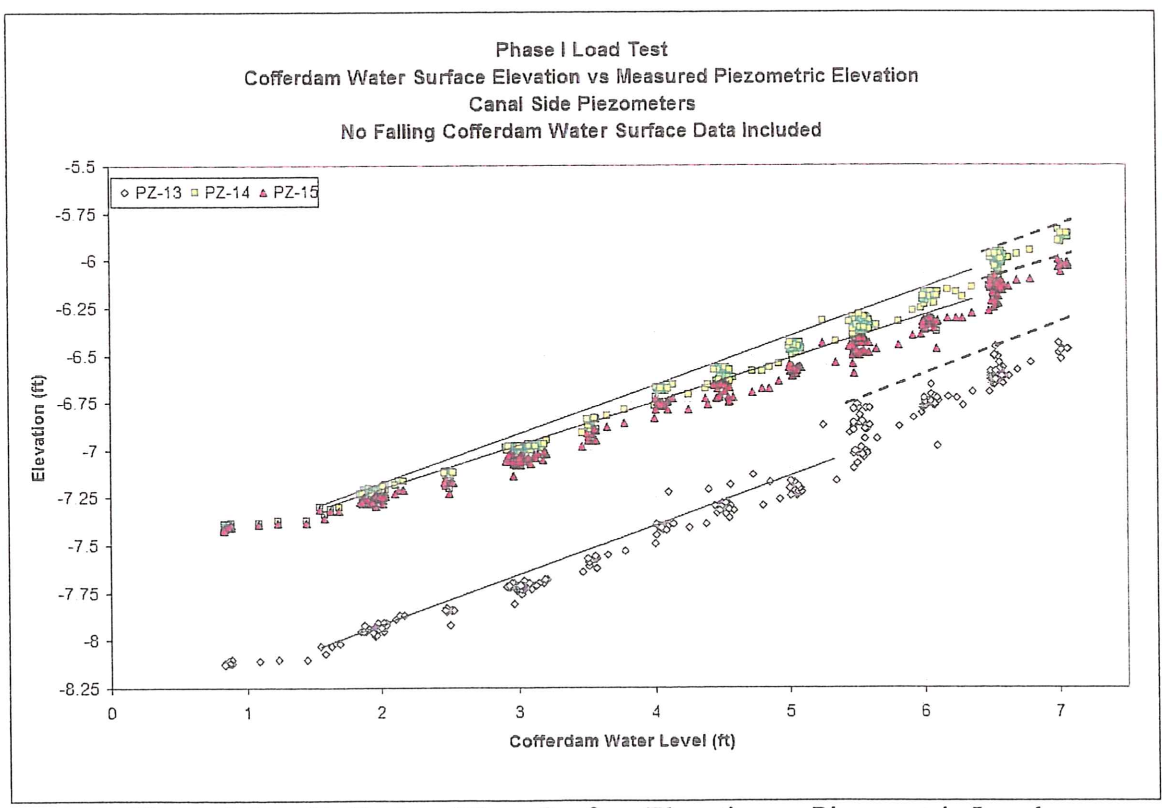


Figure 5.2a – Cofferdam Water Surface Elevation vs Piezometric Level
Canal Side Cofferdam, Phase I Data,
(No Data While Cofferdam Being Drained nor During Overnight Holds)

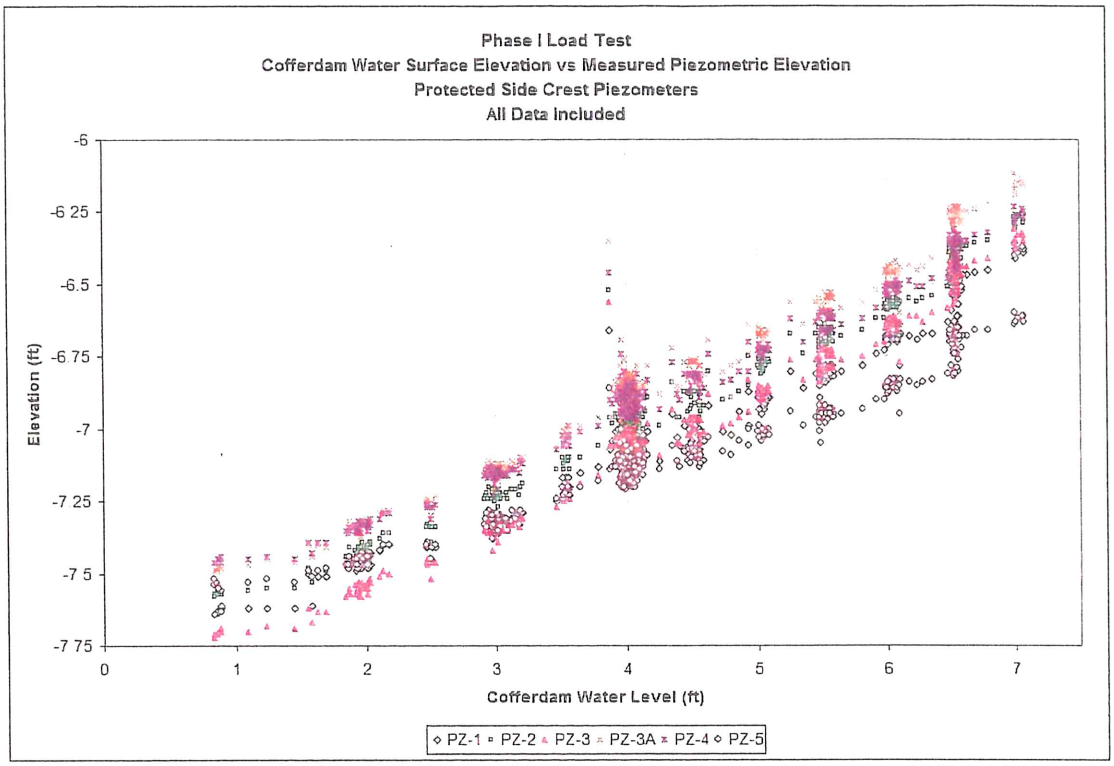


Figure 5.3 – Cofferdam Water Surface Elevation vs Piezometric Level
Protected Side Crest, All Phase I Data.

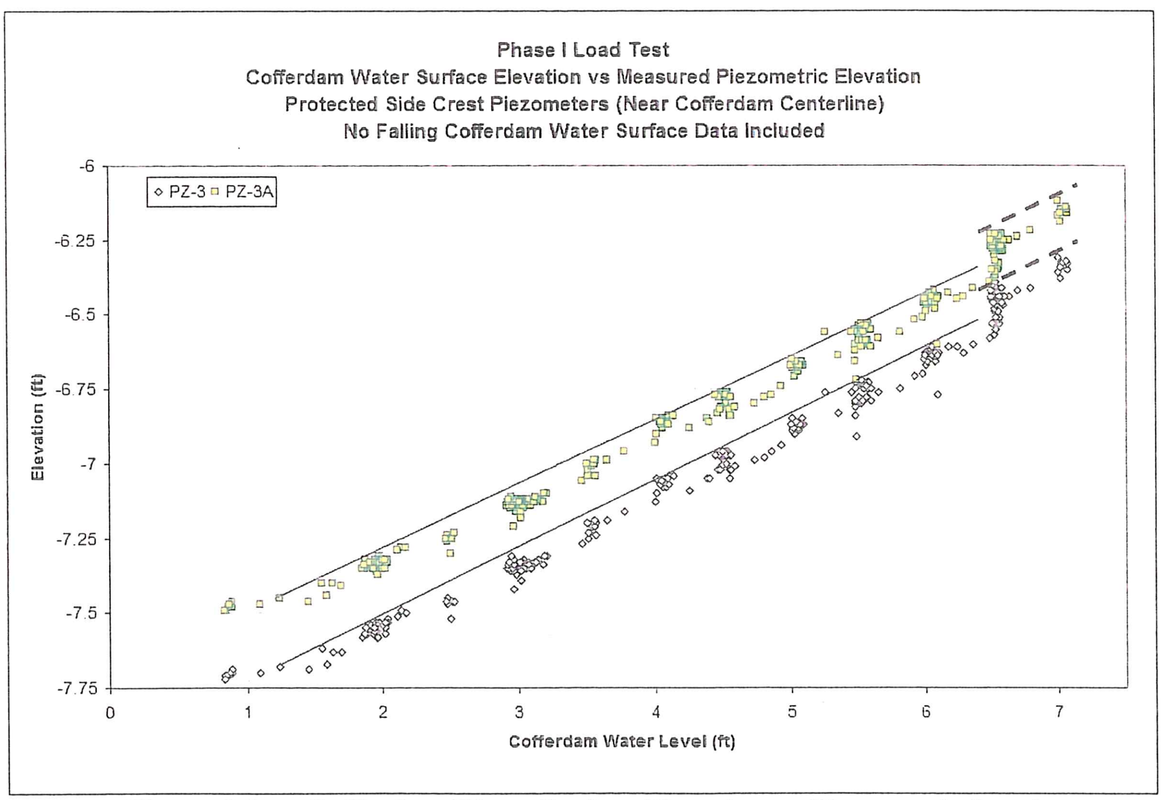


Figure 5.3a – Cofferdam Water Surface Elevation vs Piezometric Level
Protected Side Crest, Phase I Data,
(No Data While Cofferdam Being Drained nor During Overnight Holds)

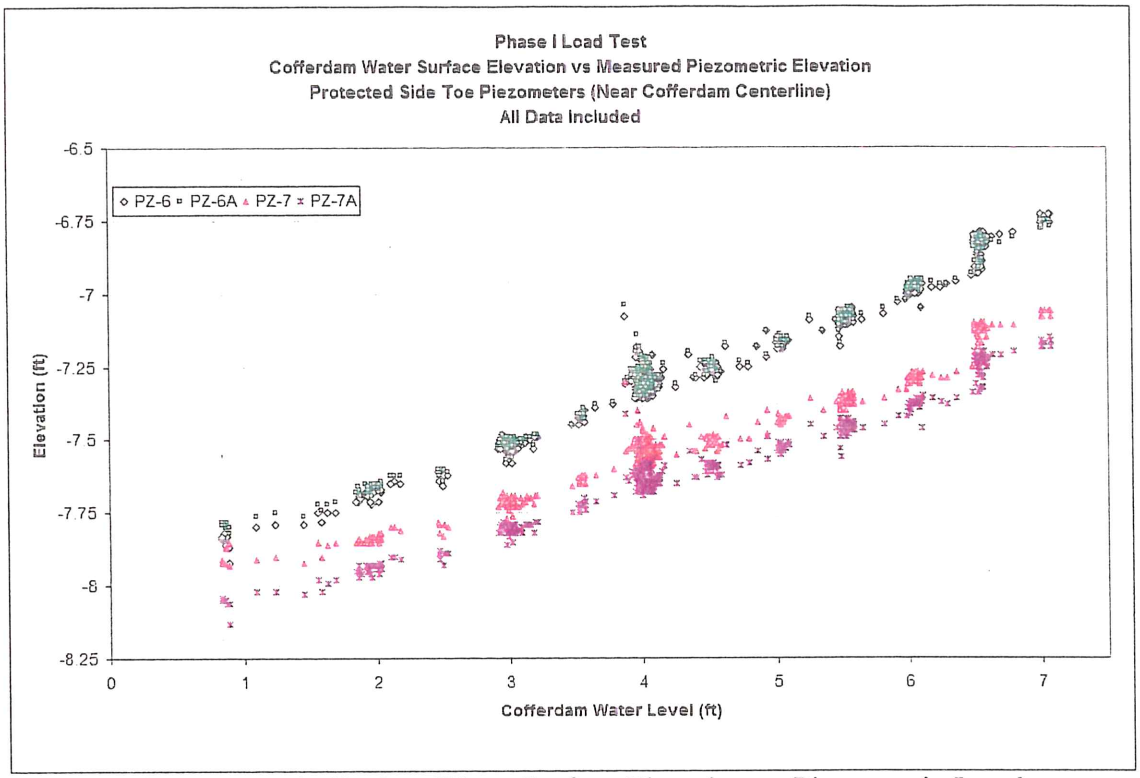


Figure 5.4 – Cofferdam Water Surface Elevation vs Piezometric Level
Protected Side Toe, All Phase I Data

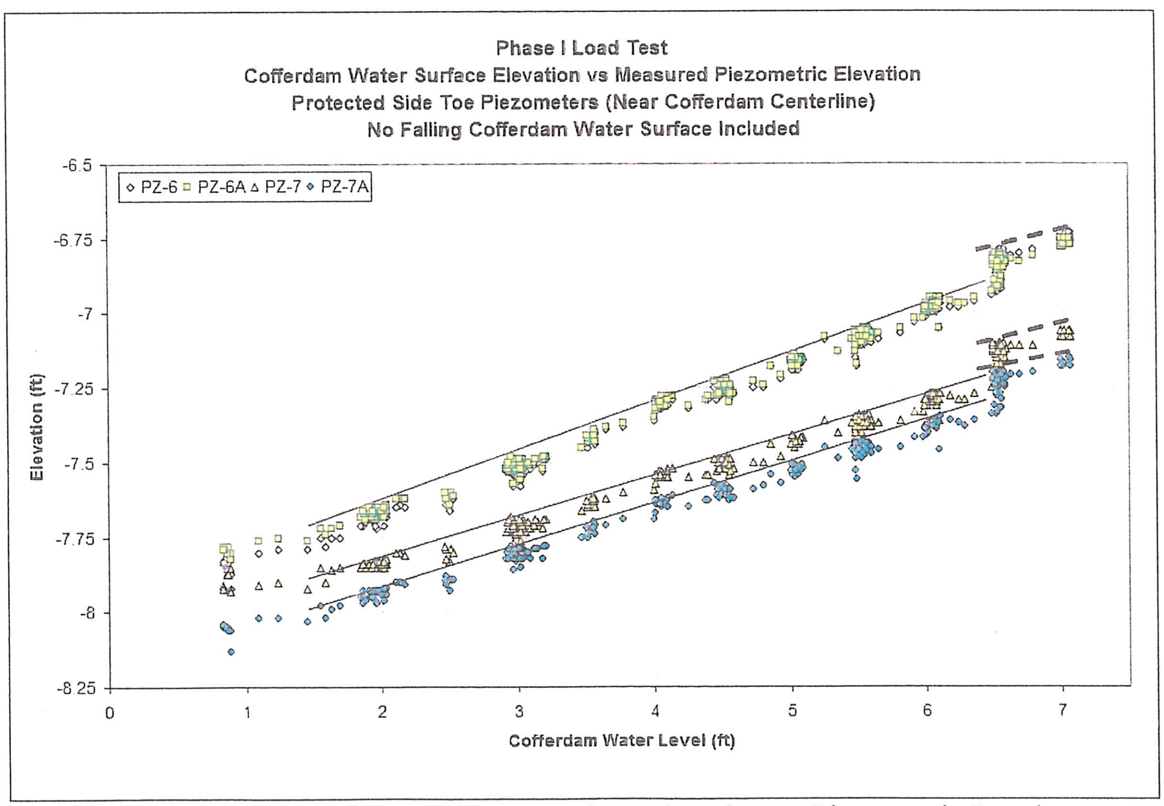


Figure 5.4a – Cofferdam Water Surface Elevation vs Piezometric Level
Protected Side Toe, Phase I Data,
(No Data While Cofferdam Being Drained nor During Overnight Holds)

The Phase II load test piezometric measurements reflect increased piezometric response in the underlying aquifer due to the slotted, PVC pipes installed in the cofferdam.

Discussion of Phase II Piezometric Data. The time plots show that during the Phase II test, the groundwater responds to the changes in the cofferdam water surface was increased compared to the Phase I results Figure 5.5 shows a time plot of the piezometric data obtained from the piezometers located closest to the cofferdam centerline. Beginning with PZ-14 on the canal side of the wall and extending to PZ-11 on the protected side at Warrington Avenue, the piezometric response decreases with increasing distance from the cofferdam. This reflects the head loss in the groundwater regime as it flows through the barrier beach sand. The piezometric measurements obtained during the Phase II load test, reflect increased piezometric response in the underlying barrier beach sand due to the functioning of the installed slotted pipes.

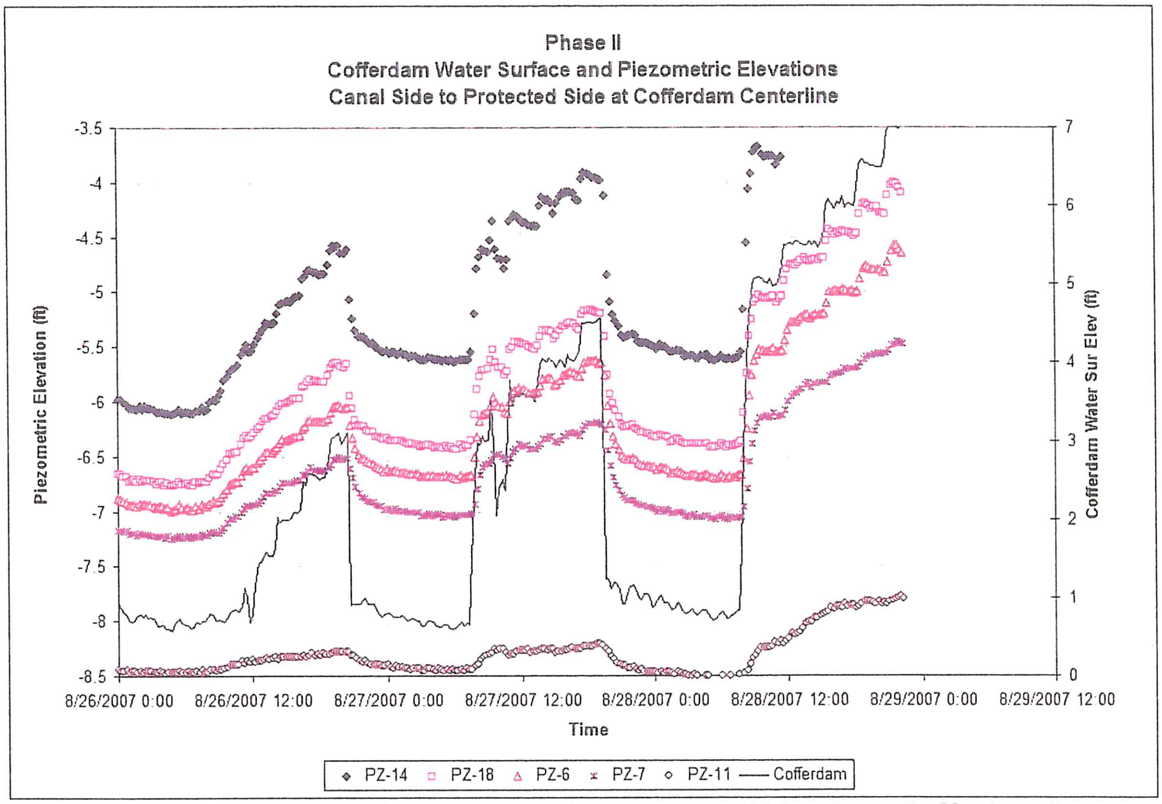


Figure 5.5 – Piezometric Elevations Canal to Protected Side Along Cofferdam Center.

Figures 5.6, 5.7, and 5.8 present the relationship between the cofferdam water surface elevation (CWE) and the piezometric elevations obtained from the canal side and protected side piezometers. The same piezometer sets were used for the Phase II portion as previously described for Phase I. Figures 5.6, 5.7, and 5.8 show piezometric data obtained when the cofferdam was being filled and when it was being emptied, when the cofferdam water surface was being held constant after a load increment, and at night when it was allowed to completely drain down to the prevailing canal water surface elevation. Figures 5.6a, 5.7a, and 5.8a are correlation plots using only data from cofferdam filling and water surface holds after each load increment. Similar to the Phase I results, the Phase II piezometric data indicate an initial linear trend, followed by an increasing upward trend beginning at CWE = +5.5-ft.

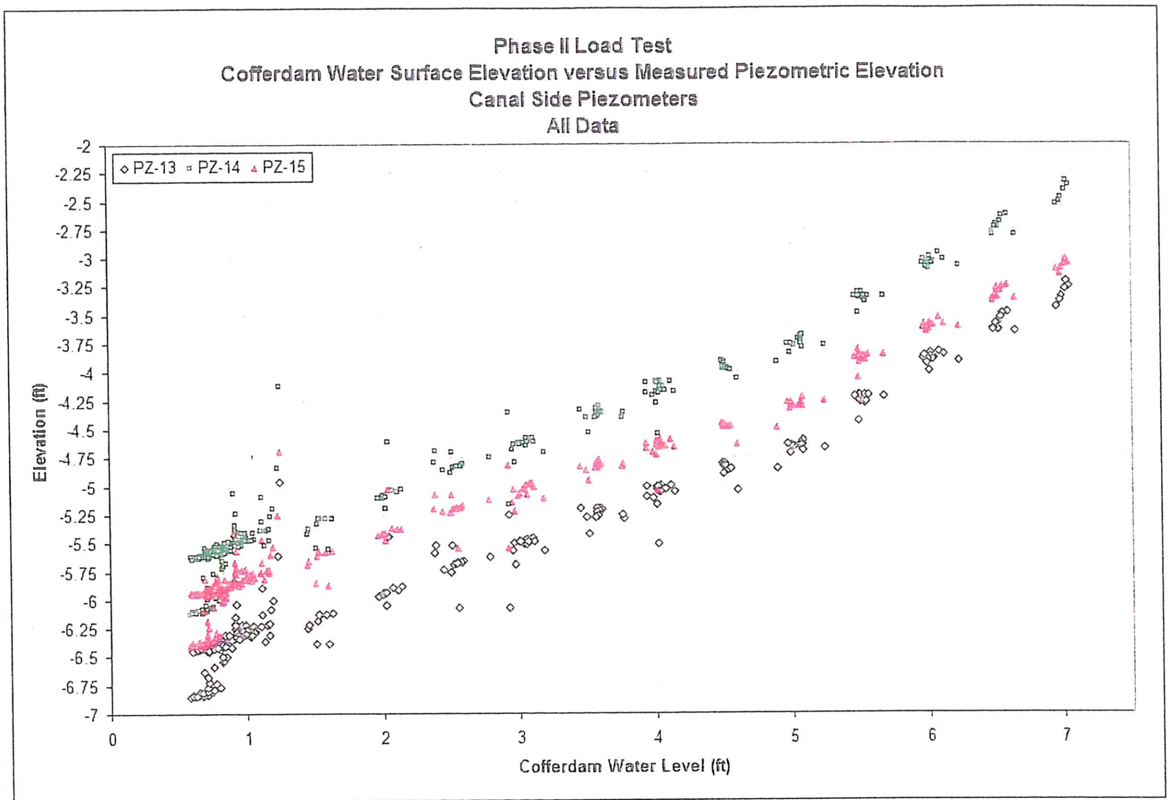


Figure 5.6 – Cofferdam Water Surface Elevation vs Piezometric Elevation Canal Side Cofferdam, All Phase II Data.

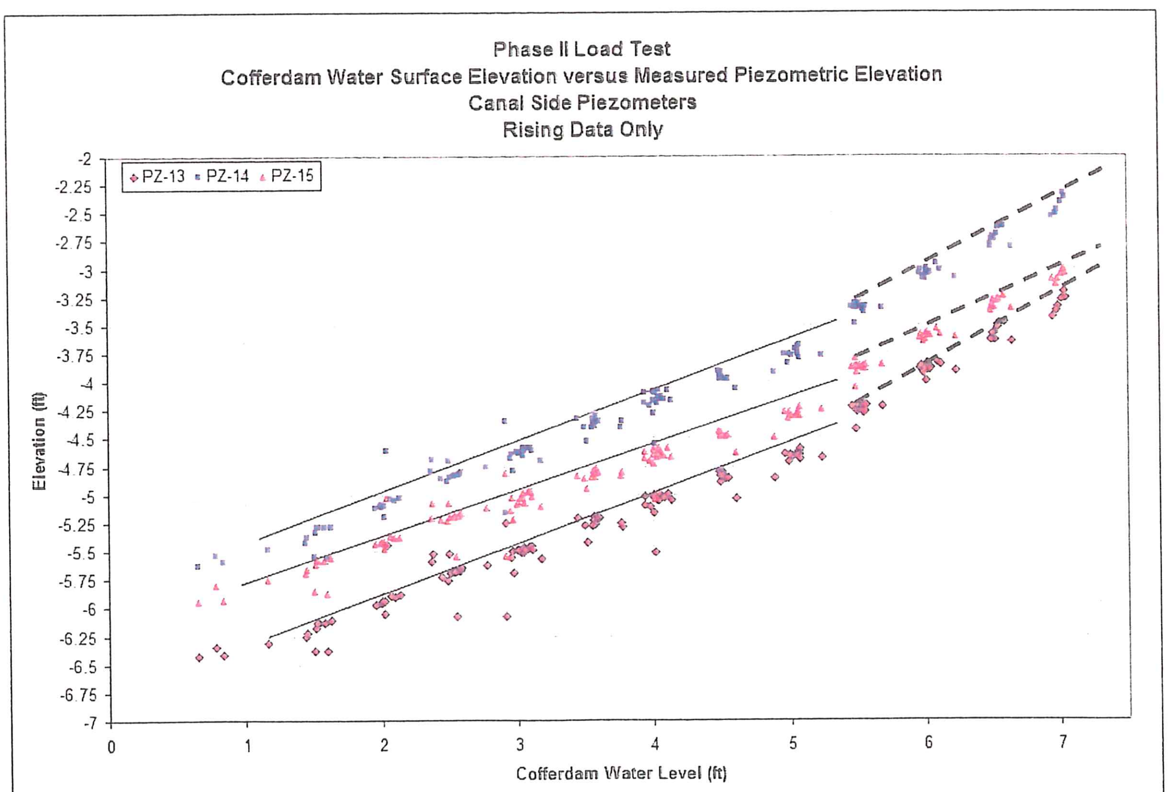


Figure 5.6a – Cofferdam Water Surface Elevation vs Piezometric Elevation Canal Side Cofferdam, Phase II Data, (No Data While Cofferdam Being Drained nor During Overnight Holds)

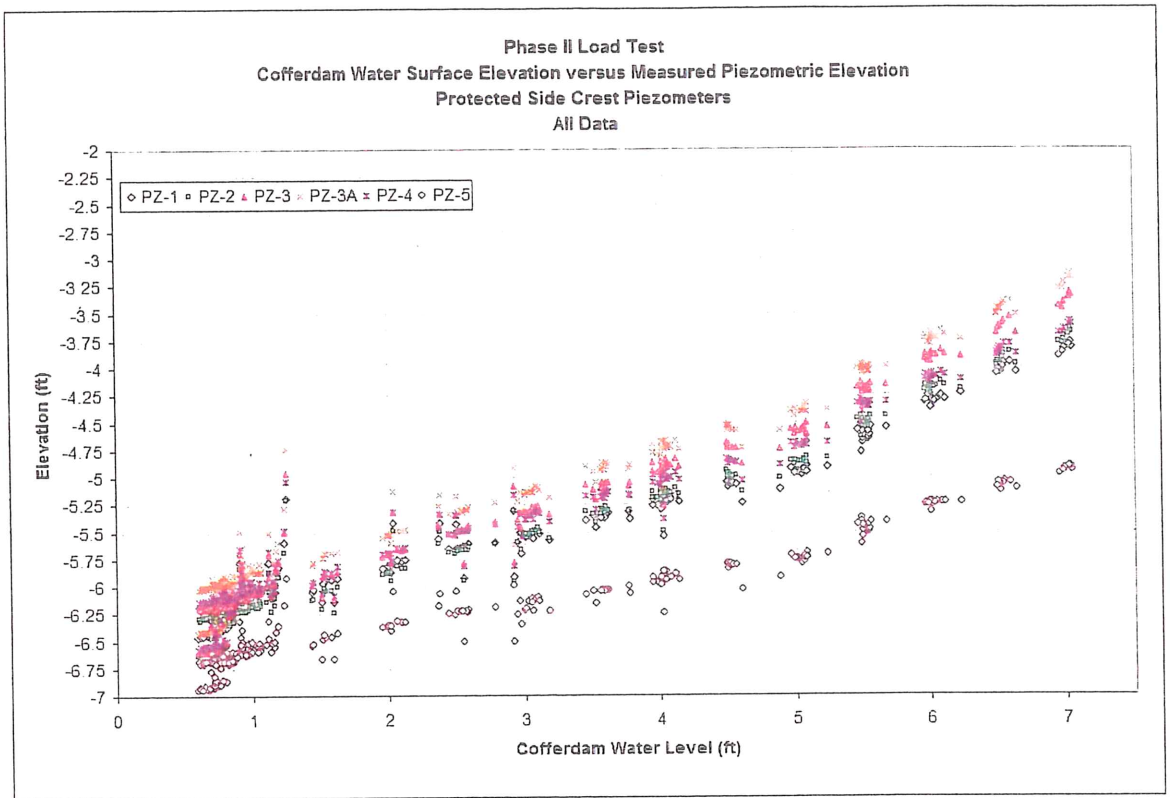


Figure 5.7 – Cofferdam Water Surface Elevation vs Piezometric Elevation
Protected Side Crest, All Phase II Data.

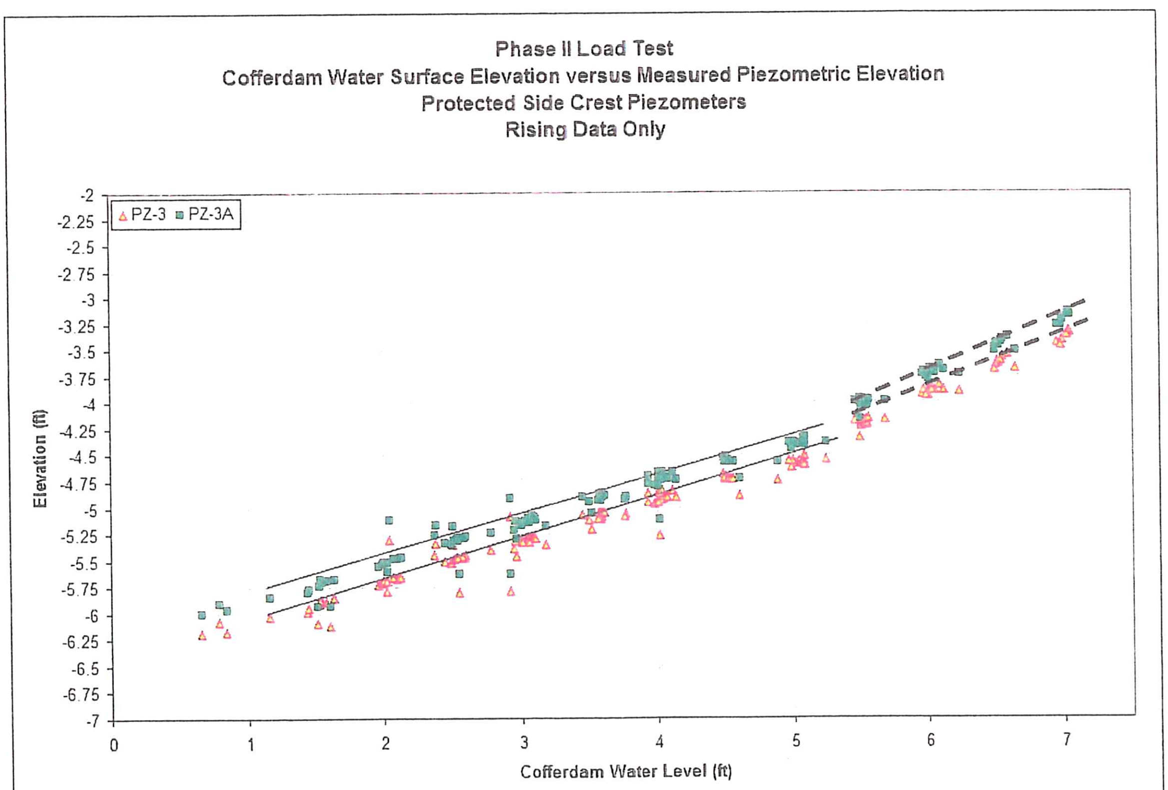


Figure 5.7a – Cofferdam Water Surface Elevation vs Piezometric Elevation
Protected Side Crest, Phase II Data,
(No Data While Cofferdam Being Drained nor During Overnight Holds)

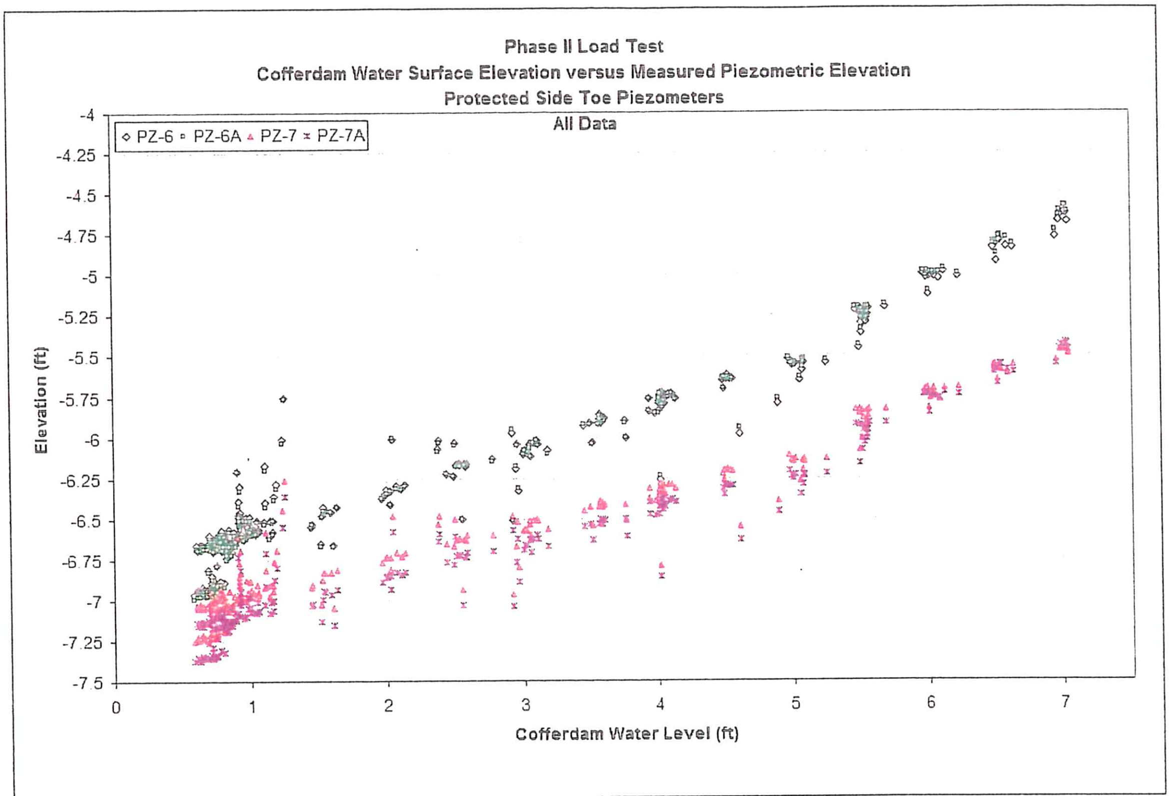


Figure 5.8 – Cofferdam Water Surface Elevation vs Piezometric Elevation
Protected Side Toe, All Phase II Data.

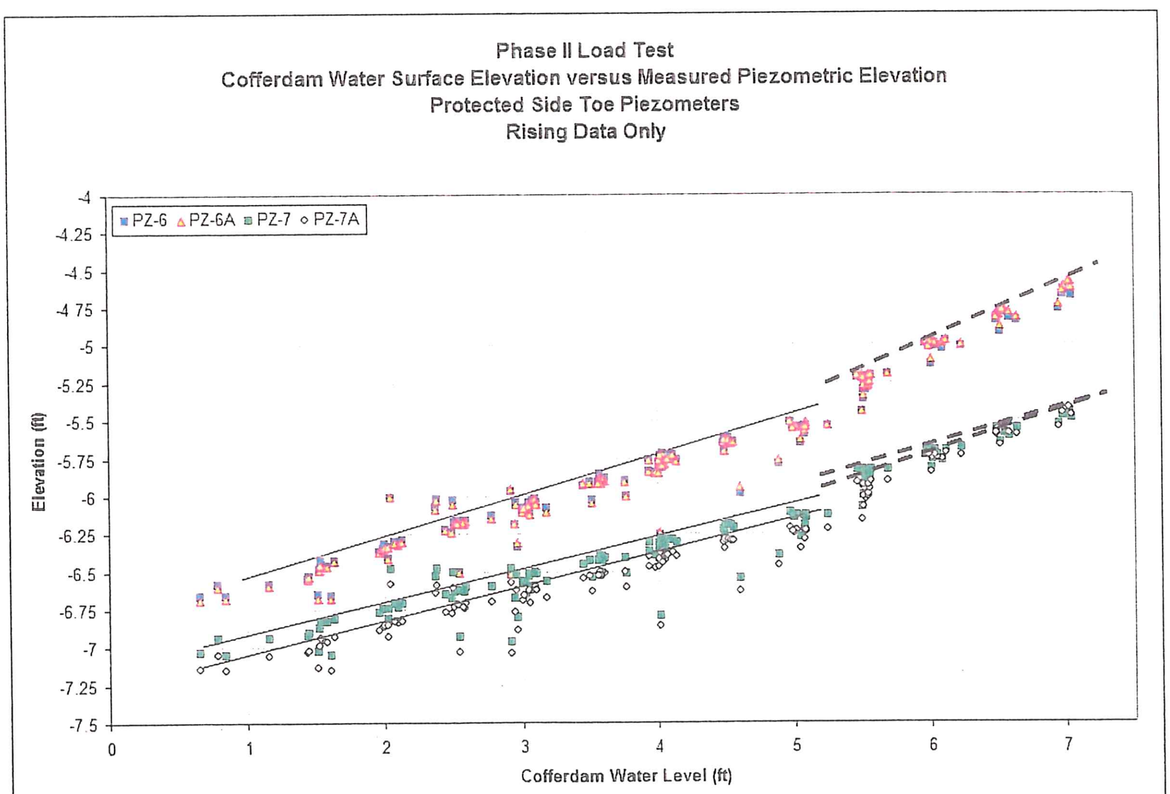


Figure 5.8a – Cofferdam Water Surface Elevation vs Piezometric Elevation
Protected Side Toe, Phase II Data,
(No Data While Cofferdam Being Drained nor During Overnight Holds

The slopes of the initial correlations for the Phase II data and their comparison to results from the Phase I test are:

- The average slope of the three canal side piezometers in Phase II is 0.440. The corresponding Phase I value is 0.252.
- The average slope of the piezometers at the cofferdam centerline at the protected side crest for Phase II is 0.373. The Phase I value is 0.218.
- The average slope of the piezometers at the cofferdam centerline at the protected side toe for Phase II is 0.258. The Phase I value is 0.146.

The average slope of each Phase II correlation line is about 1.73 times the average slope of the corresponding Phase I slope. The increased piezometric response is due to the slotted pipes installed inside of the cofferdam. The piezometric measurements of the groundwater flow regime with the slotted pipes installed are compared to the predicted values later in this chapter.The Phase II correlation data shows that the piezometric data departs from the established linear trend between cofferdam water surfaces elevations of +5.0 and +5.5-ft. This behavior is similar to that measured in Phase I, but the departure from the trend occurs at a lower cofferdam water surface elevation in the Phase II test.

The measured piezometric heads during the Phase II test were higher than those measured during the Phase I test, indicating that the slotted pipes were indeed charging the aquifer. Table 5.2 compares the measured piezometric elevations from Phase I and Phase II for the CWE equal to 5.0-ft.

		Ph I @ +5.0-ft	Ph II @ +5.0-ft	Delta H
Canal Side	PZ-13	-7.125	-4.5	+2.6
	PZ-14	-6.4	-3.6	+2.8
	PZ-15	-6.5	-4.125	+2.4
Protected Side Crest	PZ-3	-6.85	-4.5	+2.35
	PZ-3A	-6.61	-4.3	+2.31
Protected Side Toe	PZ-6	-7.125	-5.45	+1.68
	PZ-6A	-7.125	-5.45	+1.68
	PZ-7	-7.4	-6.05	+1.35
	PZ-7A	-7.5	-6.12	+1.38

The data shows that the slotted pipes caused an additional 2.6-feet of head on the canal side; and an additional 2.3-feet of head at the protected side crest; and an additional 1.3 to 1.7 feet of additional head at the protected side toe.

Comparison of Piezometric Heads: Measured and Predicted. The intent of the slotted PVC pipe installation was that they would create groundwater flow and uplift conditions that mimicked the worst case condition that the bottom of the canal was founded in barrier beach sand. Figures 3, 5, and 7 in Appendix J "*Analyses of Injection Wells*" presents results of the predicted uplift pressures at the marsh clay /barrier beach-sand interface directly under the levee toe. The following table summarizes the analysis results presented in Appendix J. The predicted piezometric elevation is based on a unit weight of water of 62.4 pcf and the marsh clay /barrier beach-sand interface located at elevation -11.4 (from Figure 2.6, Planning and Design chapter). The actual measured piezometric elevation heads are read from Figure 5.8a in this chapter) (the correlation between piezometric elevation and the CWE for piezometers PZ-6 and PZ-6A).

Table 5.2 – Predicted vs Measured Uplifts

Fig from App J	Canal WSE	Predicted Pressure & Head	Predicted Head Elevation	Measured Elevation
Figure 3	+4.0	530 psf, 8.49-ft	-2.91	-5.75
Figure 5	+6.0	608 psf, 9.72-ft	-1.68	-5.1
Figure 7	+8.0	682 psf, 10.9-ft	-0.48	Not Tested

This seepage analysis (detailed in Appendix J) was completed using the best available information on the foundation stratigraphy. Subsequent exploration efforts (after completion of this seepage analysis) identified the 10-foot thick layer of barrier beach silty-sand existent immediately below the marsh clay. This layer was not included in the seepage analyses presented in Appendix J and its presence substantially changes the response of the groundwater regime. This change in stratigraphy renders the seepage analyses predictions invalid.

When, during the Phase II load test, the Technical Review Team noticed that the measured piezometric elevations at the protected side toe were not matching the predicted values, the team completed a 2-dimensional, numeric seepage model using the finite element seepage program Groundwater Modeling System (GMS). This model was run for the cofferdam water surface at +4.0-ft and accounted for all aspects of the load test system including the fully penetrating cofferdam sheetpile cutoff, the slotted pipes, and a 10-foot thick layer of silty sands located at the top of the Beach Sand Aquifer. This model predicted piezometric pressures of slightly less than -5.50-feet at the protected side toe, a value that closely matches the measured value of -5.75-ft. Plots of the total heads obtained from this model are shown in Figures 5.9 and 5.10. This numeric model shows that substantial head losses occur in the silty sand layer and subsequently less pressure head remaining at the protected side toe.

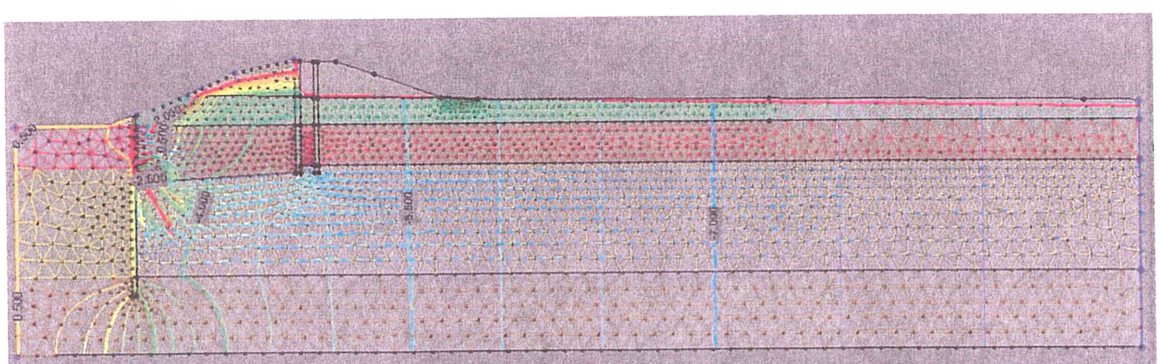


Figure 5.9 – GMS Flownet for As-Built Load Test Conditions

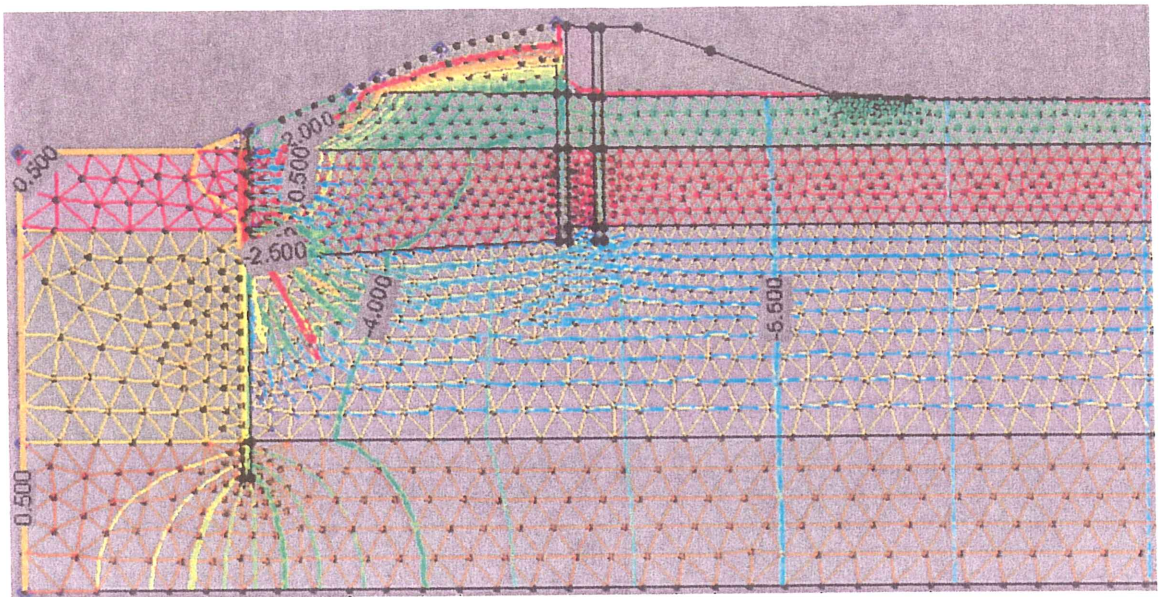


Figure 5.10 – GMS Flownet, Details at Protected Side Toe

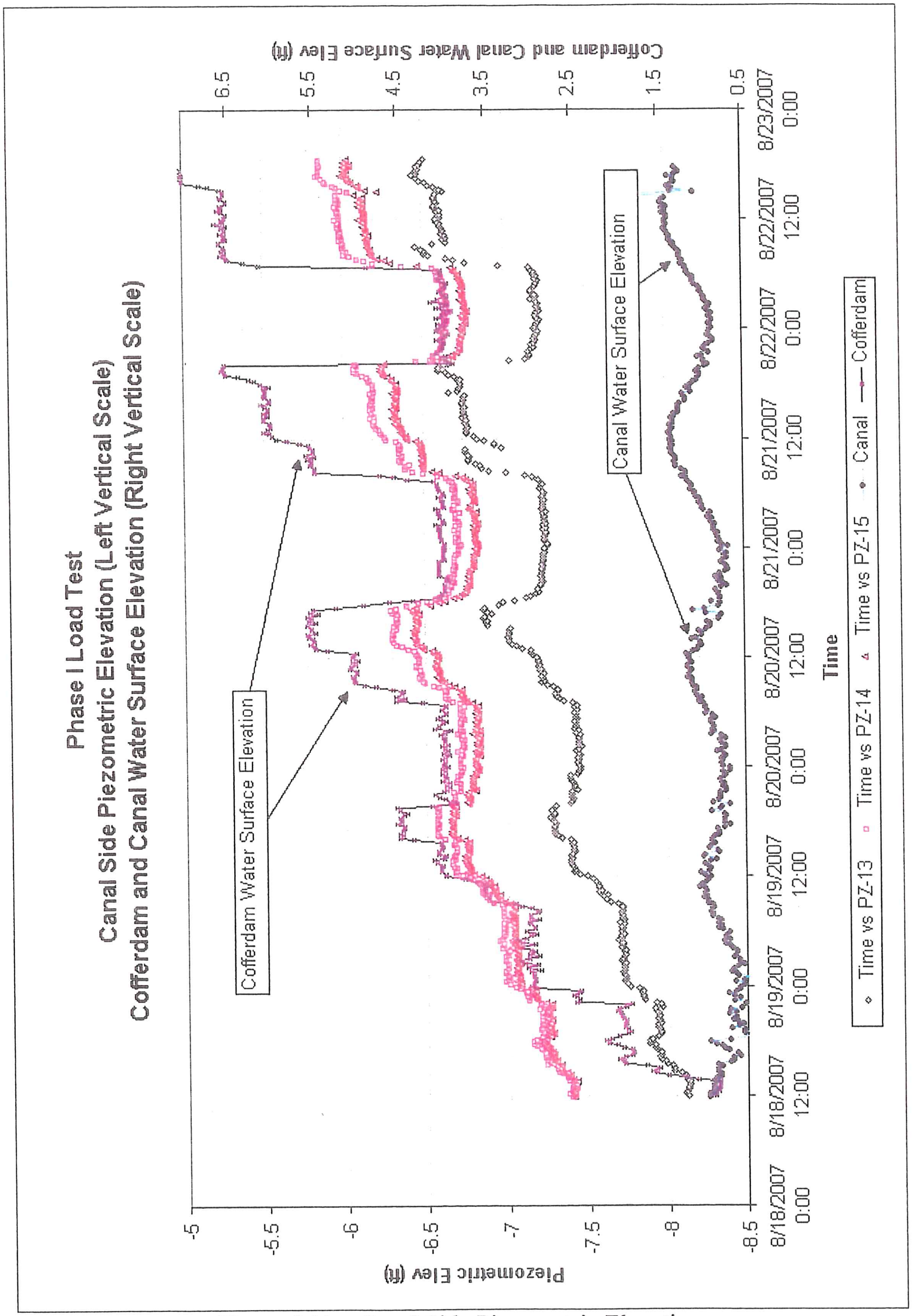


Figure 5.11 – Canal Side Piezometric Elevations

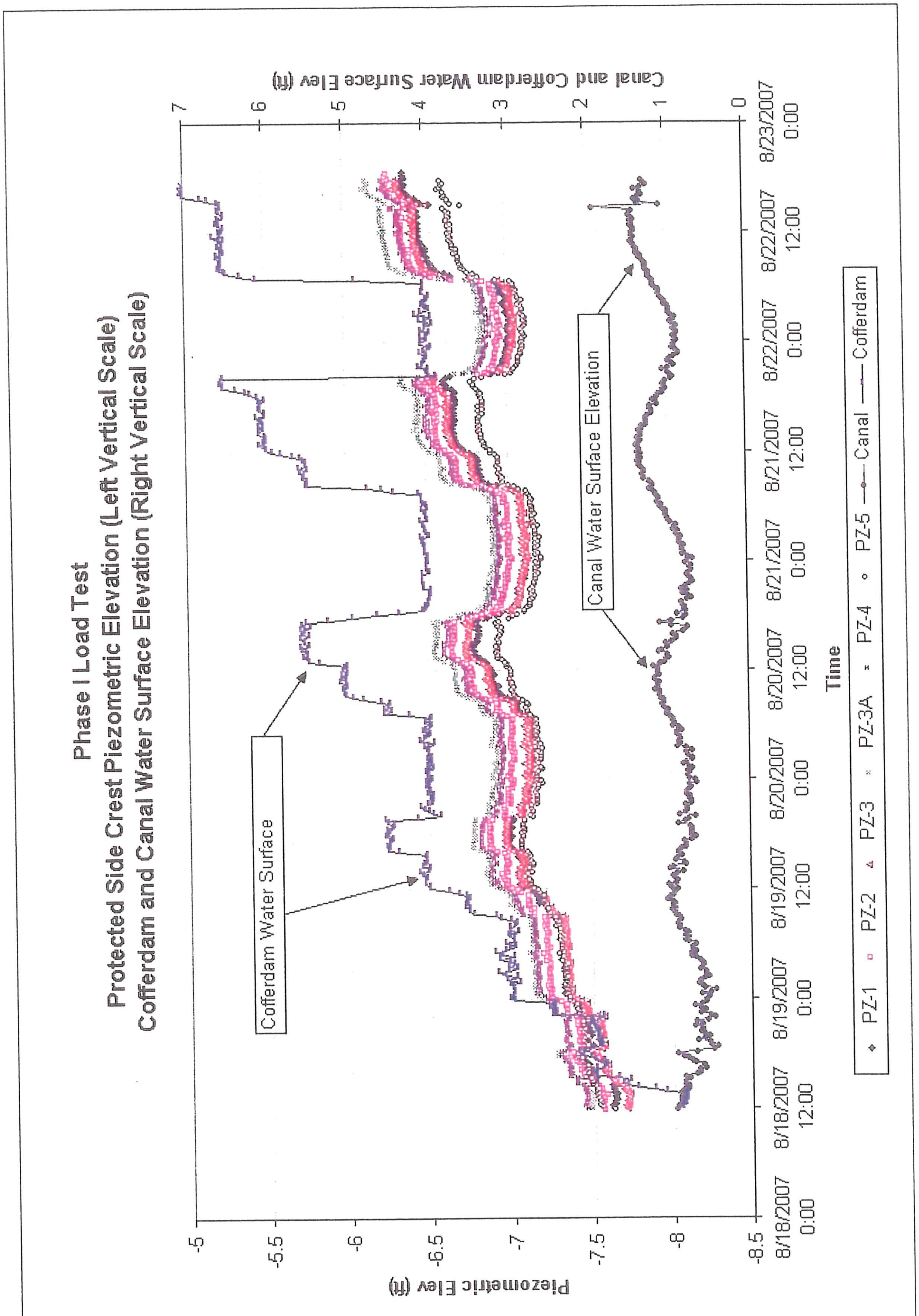


Figure 5.12 – Protected Side Crest Piezometric Elevations

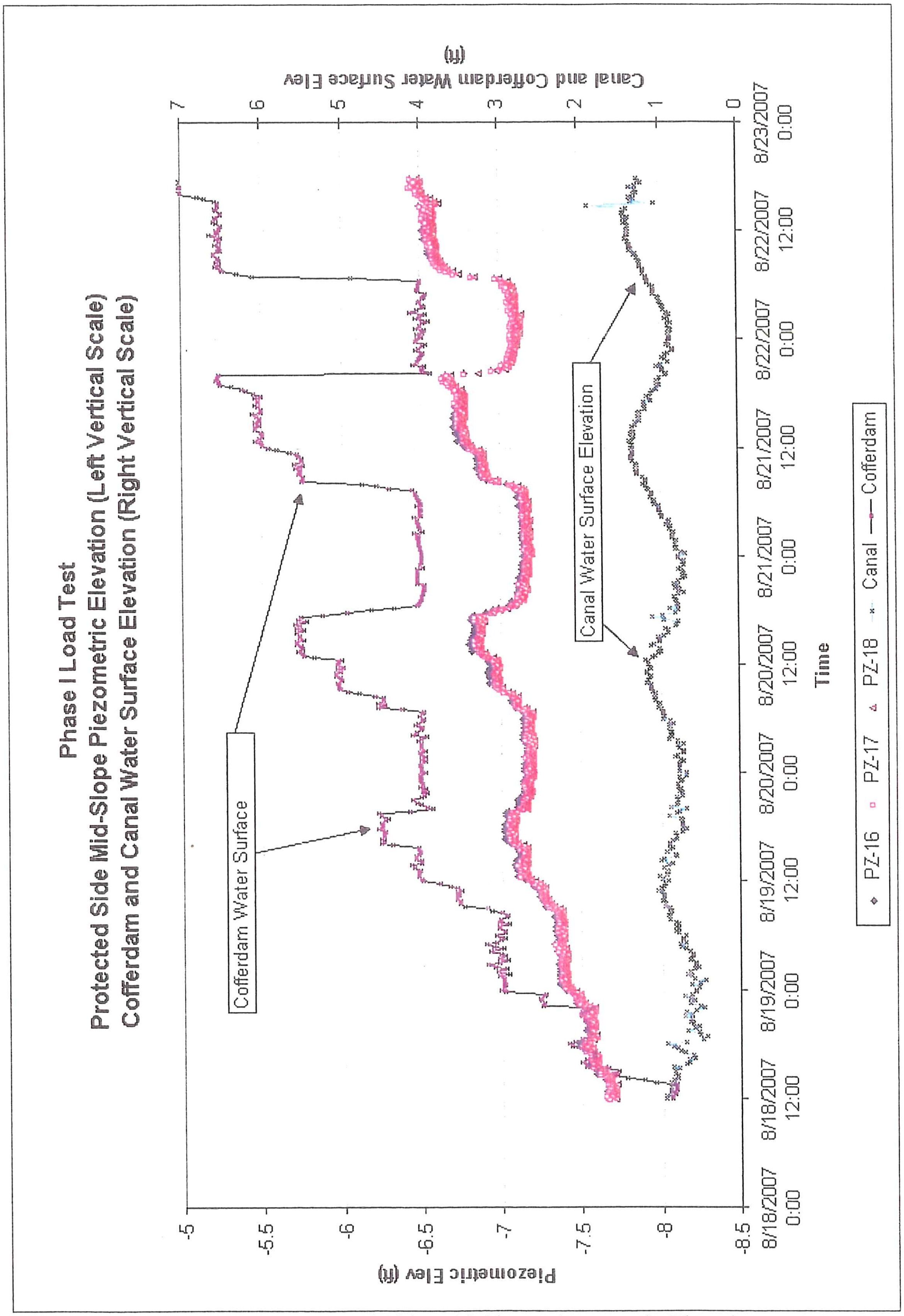


Figure 5.13 – Protected Side Mid-Slope Piezometric Elevations

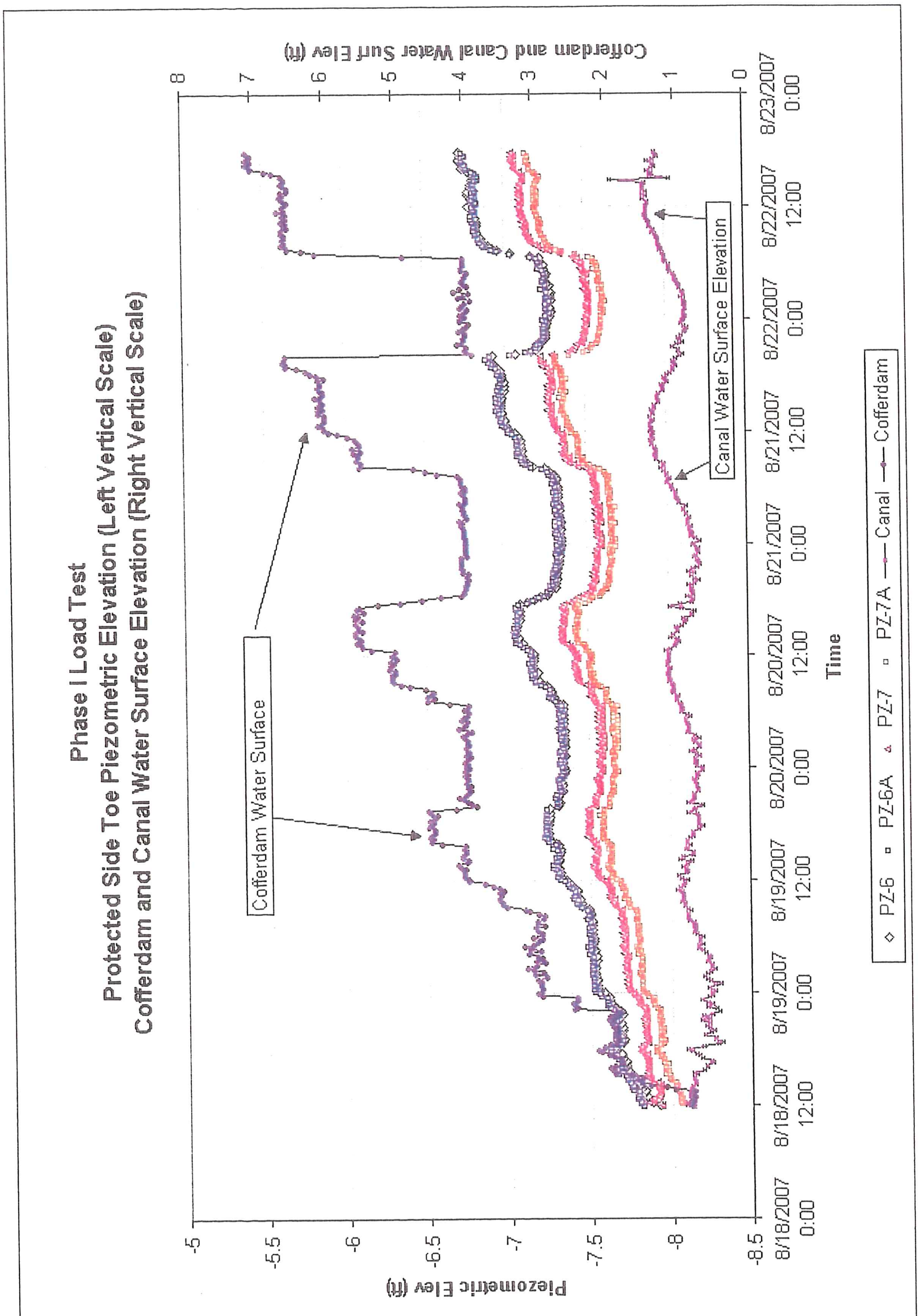


Figure 5.14 – Protected Side Toe Piezometric Elevations

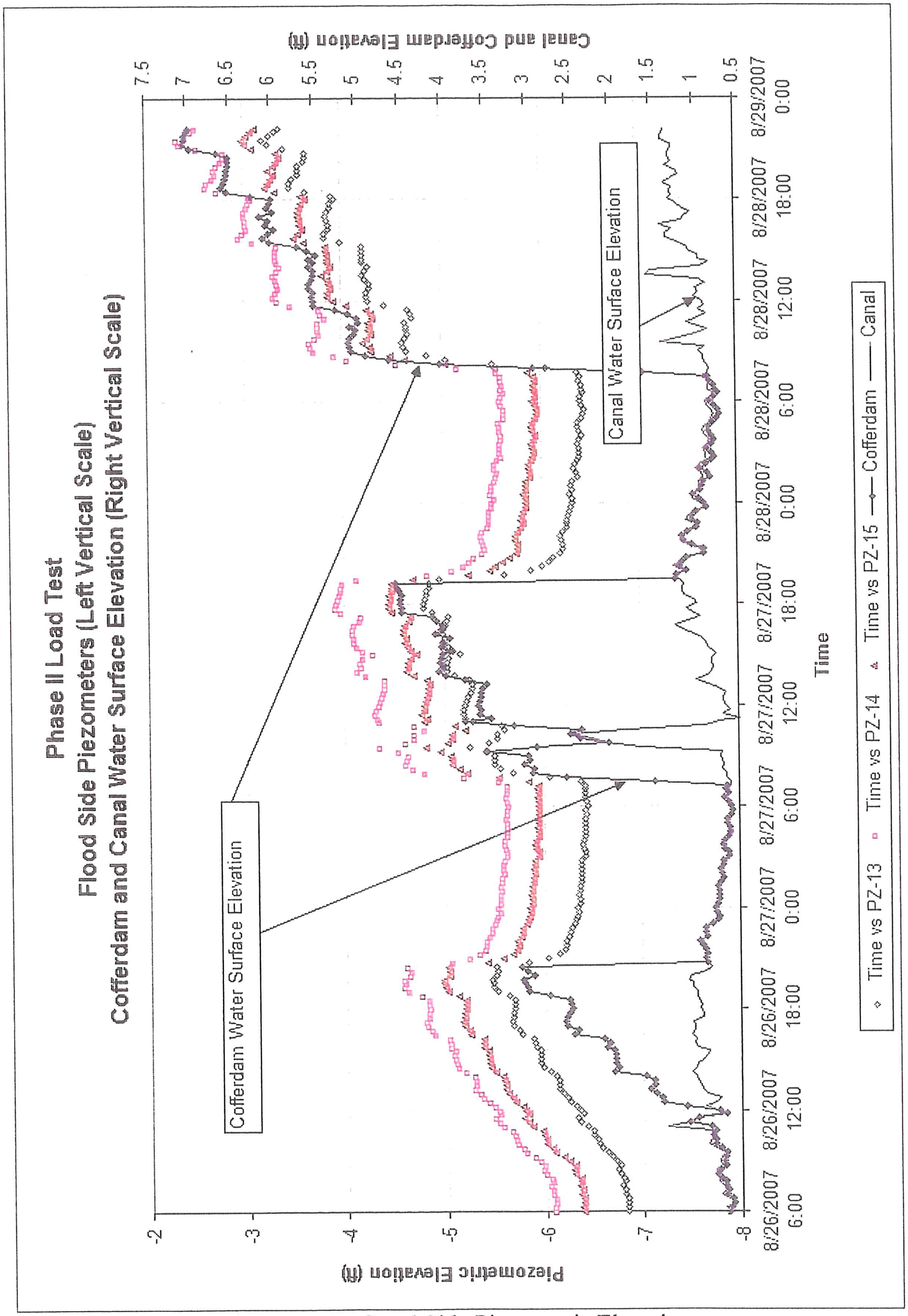


Figure 5.15 – Canal Side Piezometric Elevations

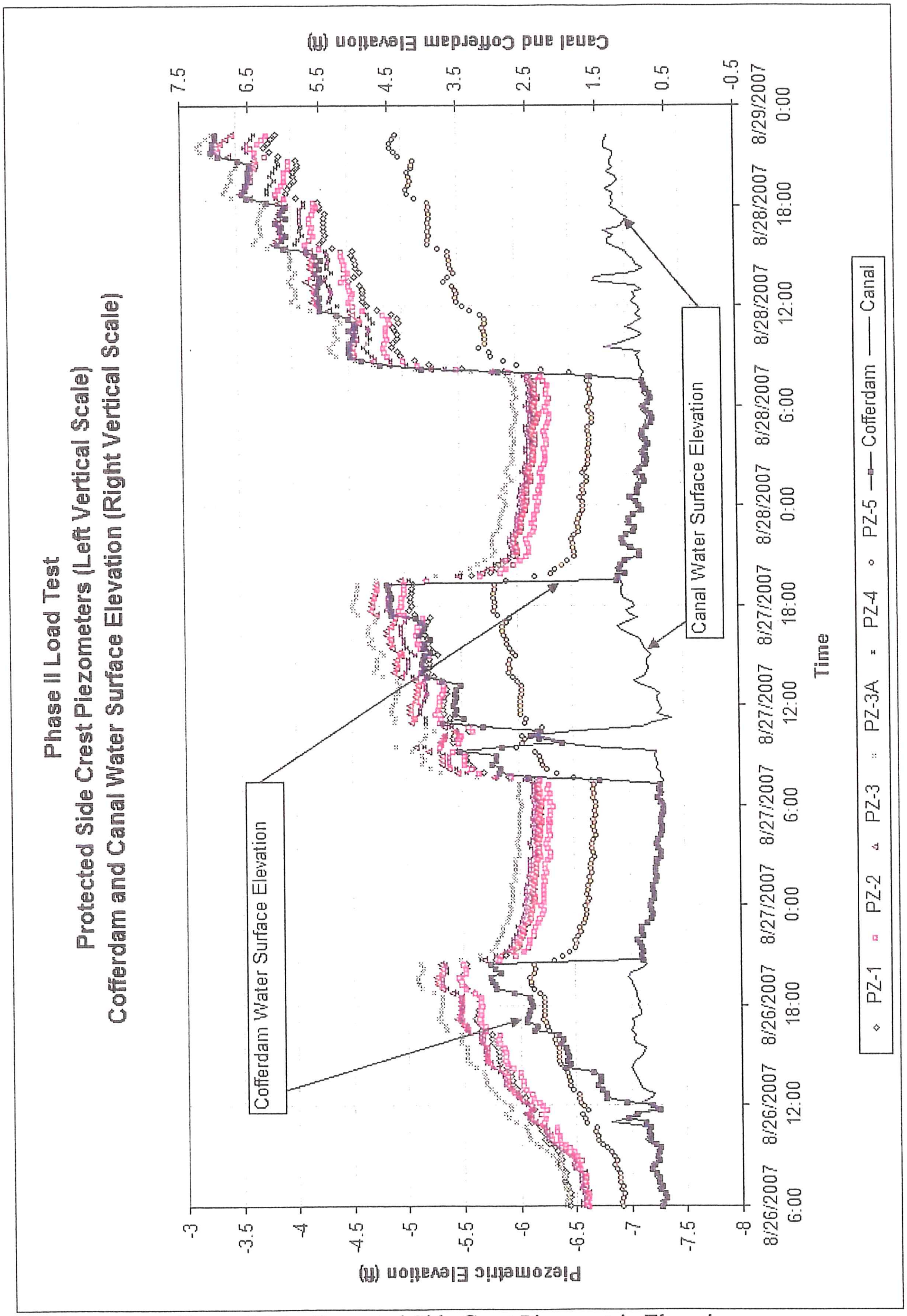


Figure 5.16 – Protected Side Crest Piezometric Elevations

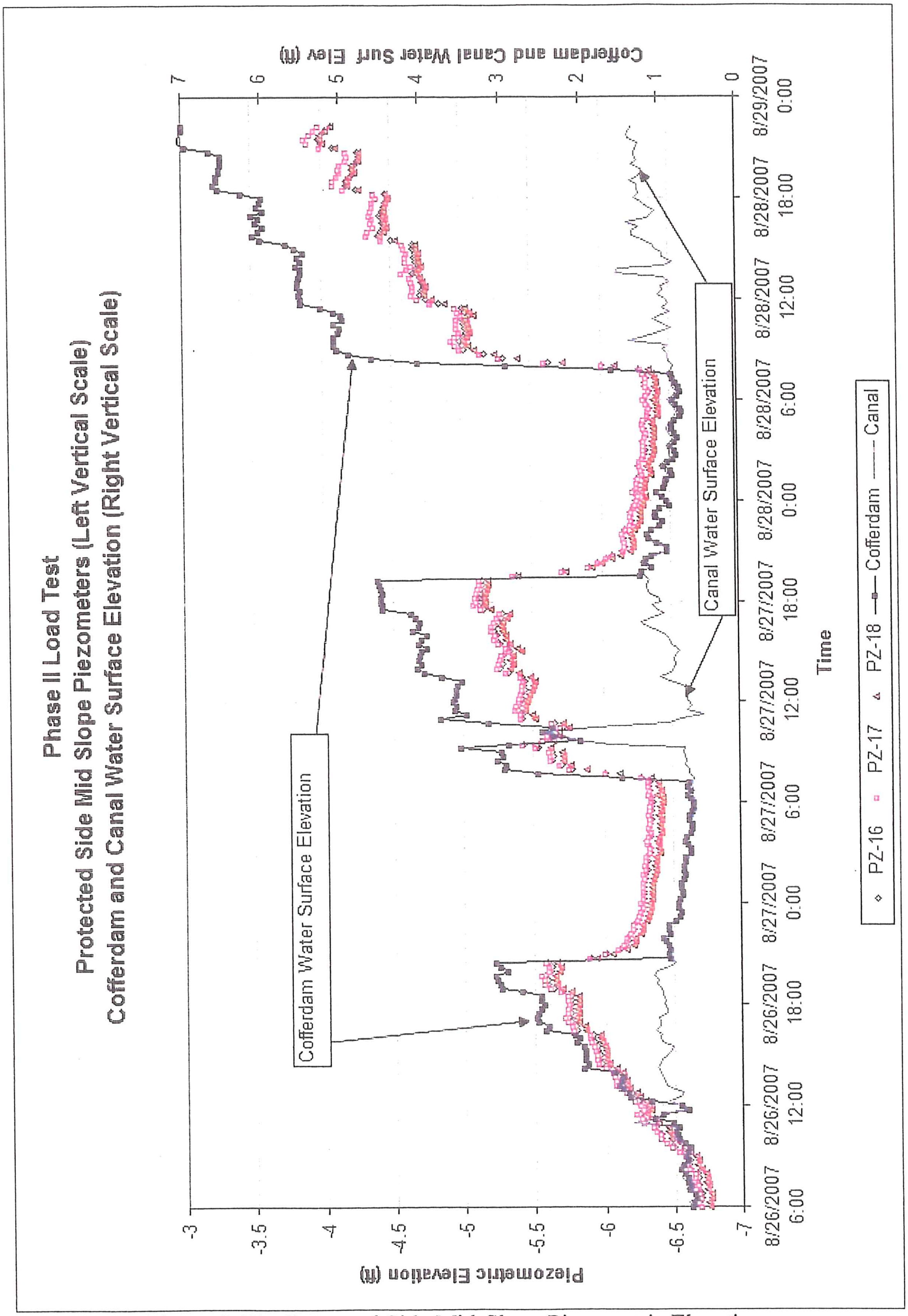


Figure 5.17 – Protected Side Mid-Slope Piezometric Elevations

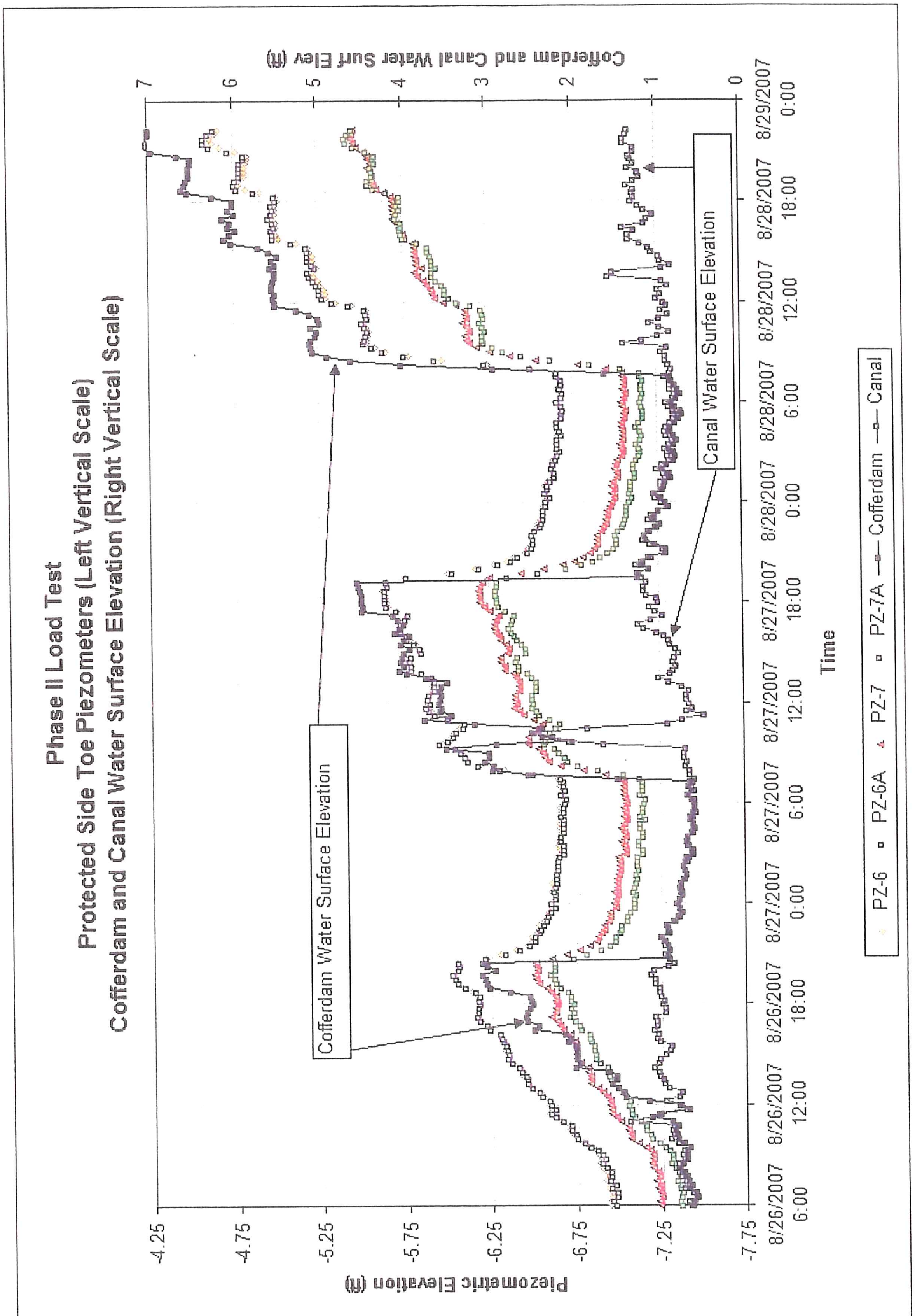


Figure 5.18 – Protected Side Embankment Toe Piezometric Elevations

Table 5.3 – Piezometer Falling Head Test Details

London Avenue Canal Falling Head Test Data
Tests on PZ-13, PZ-14, and PZ-15 by URS. All other tests performed by others.
All data in Feet and Minutes

PZ-1
7/29/2007
Initial Reading = 9.9

Time	Reading
0	0
1	6.8
2	9.5
4	9.9

PZ-2
7/28/2007
Initial Reading = 10.0

Time	Reading
0	0
1	8
2	10

PZ-3
7/28/2007
Initial Reading = 10.3

Time	Reading
0	0
1	1.1
2	1.7
4	2.7
8	4.65
15	5.65
30	7.95
60	9.8
120	10.2

PZ-3A
7/27/2007
Initial Reading = 10.4

Time	Reading
0	0
1	3.1
2	4.2
4	5.7
8	7.7
15	9.25
30	9.8
60	10.4

PZ-4
7/29/2007
Initial Reading = 10.0

Time	Reading
0	0
1	8
2	9.85
4	10

PZ-5
7/30/2007
Initial Reading = 9.85

Time	Reading
0	0
1	7.5
2	9.2
4	9.85

PZ-6
7/11/2007
Initial Reading = 1.4

Time	Reading
0	0.3
1	0.5
2	0.7
4	1
8	1.2
15	1.4

PZ-6A
7/11/2007
Initial Reading = 1.7

Time	Reading
0	0
1	0.25
2	0.35
4	0.6
8	1
15	1.2
30	1.3
60	1.7

PZ-7
7/11/2007
Initial Reading = 1.7

Time	Reading
0	0
1	0.3
2	0.5
4	0.7
8	1.55
15	1.65
30	1.7

Table 5.3 – Piezometer Falling Head Test Details (con't)

PZ-7A		PZ-16		PZ-17	
7/11/2007		8/1/2007		8/1/2007	
Init Reading = 1.8		Initial Reading = 5.25		Initial Reading = 5.4	
Time	Reading	Time	Reading	Time	Reading
0	0	0	0	0	0
1	0.4	1	4.75	1	4.8
2	0.7	2	5	2	5.1
4	1.1	4	5.25	4	5.25
8	1.7				
9.45	1.75				
10	1.8				
PZ-10		PZ-11		PZ-12	
7/10/2007		7/10/2007		7/10/2007	
Init Reading = 1.4		Initial Reading = 1.5		Initial Reading = 1.7	
Time	Reading	Time	Reading	Time	Reading
0	0	0	0	0	0
1	1.3	1	1	1	1.5
2	1.35	2	1.4	2	1.6
4	1.4	4	1.5	4	1.7
PZ-13		PZ-14		PZ-15	
8/13/2007		8/13/2007		8/13/2007	
Init Reading = 10.7		Initial Reading = 13.59		Initial Reading = 10.80	
Time	Reading	Time	Reading	Time	Reading
0	0	0	0	0	0
1	3.48	1	4.9	1	6.85
2	5	2	7.81	2	7.37
4	7.75	4	10.68	4	7.79
8	9.79	8	12.91	8	8.21
15	10.59	15	13.5	15	8.68
30	10.64			30	9.61
				55	10.25

CHAPTER 6 – I-WALL AND LEVEE EMBANKMENT MOVEMENTS

GENERAL. This section describes the instruments used and the detailed measurements of wall and embankment movements obtained during Phase I and Phase II portions of the load test. Table 6.1 summarizes the survey prisms, survey monuments and titlmeters installaed on the wall monoliths. Data measurement plots with respect to time and correlations with cofferdam water surface are plotted on Figures 6.14 through 6.33 located at the end of this chapter.

Table 6.1 – Summary of Survey Prisms, Monuments and Tiltmeters

	South	Monoliths Captured within 150-foot Cofferdam					North
Mono #	1	2	3	4	5	6	7
SP Top	SP-1	SP-2	SP-3	SP-4	SP-5	SP-6	SP-7
SP Bottom	SP-1A	SP-2A	SP-3A	SP-4A	SP-5A	SP-6A	SP-7A
SM Crest	SM-1	SM-2	SM-3	SM-4	SM-5	SM-6	SM-7
SM Mid	SM-8	SM-9	SM-10	SM-11	SM-12	SM-13	SM-14
SM Toe	SM-15	SM-16	SM-17	SM-18	SM-19	SM-20	SM-21
SM 50-feet			SM-22, -23, -24 and -25				
SM Street			SM-26	SM-27	SM-28		
Tiltmeters		TM-1	TM-2	TM-3	TM-4	TM-5	

Wall Monuments. Two survey prisms were mounted on each of the five I-wall monoliths located within the 150-foot long cofferdam. Two additional monoliths, one just beyond the north cofferdam tie-in, and the second just beyond the south cofferdam tie-in, were also instrumented with survey prisms for a total of seven instrumented monoliths. Figure 2.12, Instrumentation Layout in Chapter – 2, Planning and Design, shows the location of these survey prisms. The prisms were mounted on the vertical centerline of each monolith, one near the top of the monolith on the protected side vertical face and the second, 12-inches above the ground line on the same face. The survey prisms mounted on the wall tops were numbered sequentially from SP-1 on the south end to SP-7 on the north and the prisms at the base were assigned the same number but with the suffix "A", SP-1A to SP-7A. Table 6.1 summarizes the wall survey prism naming convention. These monoliths were surveyed by two robotic, Leica electronic total stations.

Embankment Monuments. The embankment was monitored using the same type of instruments as installed on the I-wall monoliths, but these prisms were installed on vertical lengths of pipe placed in concrete within the levee embankment on the protected side of the wall. These survey monuments were installed on the embankment crest, at mid-slope, at the embankment toe, 50 feet from the wall, and at the street. The latter location was thought to represent a control measurement that was not anticipated to be impacted by the load test. The crest, mid-slope, and toe monuments were installed in line with the vertical centerline of each I-wall monolith. In other words, the two prisms mounted on the concrete monoliths and the three survey monuments installed on the embankment form straight line ranges. Table 6.1 summarizes the embankment survey prism naming convention. These monoliths were also surveyed by the robotic, Leica total stations.

Wall Tiltmeters. A vibrating-wire tiltmeter (manufactured by Geocon) was attached to each of the five I-wall monoliths located within the 150-foot long cofferdam. Each tiltmeter was attached near the center of the wall monolith on it's protected-side face. These instruments will measure the tilt/rotation of the wall monoliths during the load test and were read automatically by the ADAS. Table 6.1 summarizes the tiltmeter installation and naming convention.

Phase I and II Base Wall Movements. Inspection of the Phase I survey prism base deflection data (Figures 6.14 to 6.20) shows that monoliths 1 and 7 (beyond the extent of the cofferdam) moved the least during the load test. These monoliths were not directly loaded by the test so it is likely that they are being "dragged along" by the adjacent monolith that was under a direct load. For the five monoliths within the cofferdam, the three internal monoliths, (i.e., Numbers 3, 4, and 5), moved more than monoliths 2 and 6 at the cofferdam tie-ins. Except for slightly larger movements, the pattern of the base wall deflections measured in Phase II (Figures 6.24 to 6.30) is the same as measured in Phase I. Figure 6.1 illustrates the actual movements and the generalized pattern of base movement (i.e., the A-series monuments) for the instrumented monoliths.

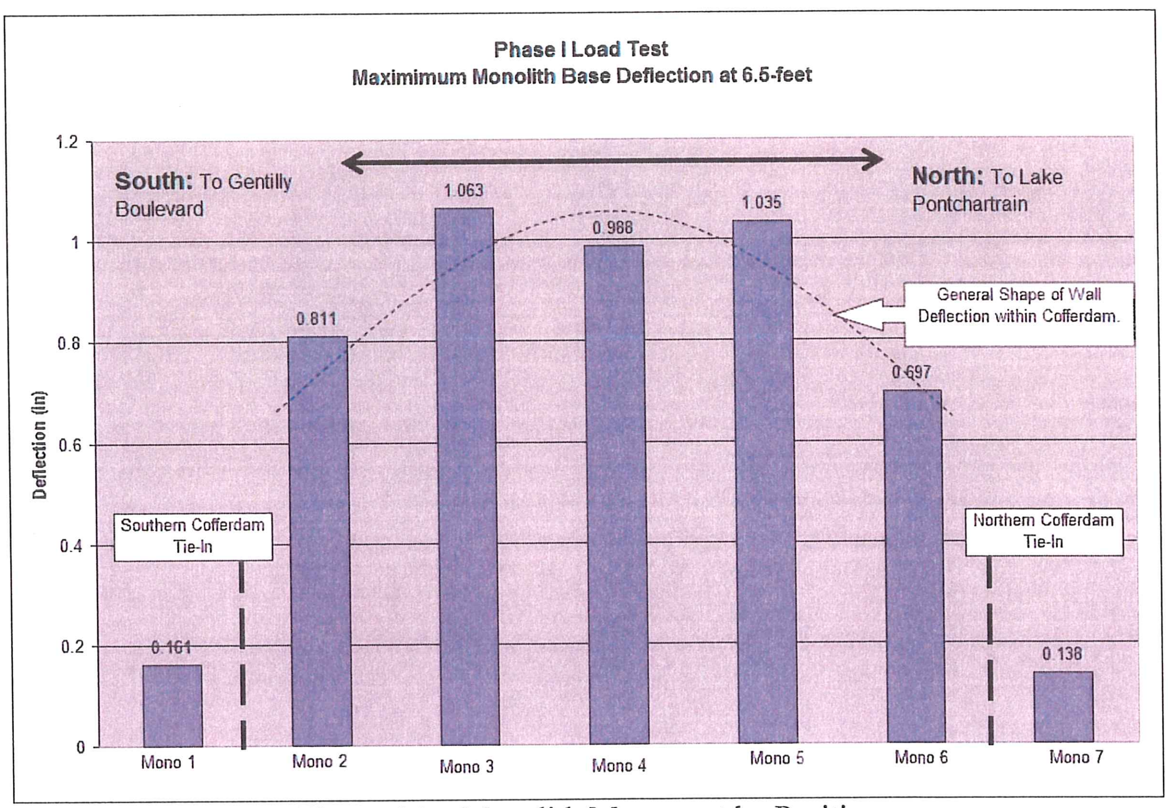


Figure 6.1 – Monolith Movement by Position

Figures 6.2 and 6.3 present a plot of the measured horizontal movement at the base of the three internal monoliths against the cofferdam water surface elevation. The maximum horizontal movement at the base of the monoliths ranged from 1.18 to 1.2 inches in Phase I. During the Phase II portion of the load test, the maximum deflection of the same three monoliths was almost identical, 1.16 to 1.22 inches.

Although the maximum horizontal movement of the base of the monoliths is very similar between Phase I and Phase II, the wall behavior is very different. Figures 6.4, 6.5, and 6.6 show the horizontal deflection measured at the base of the monolith plotted against the cofferdam water surface elevation for all Phase I and Phase II loadings. Inspection of these figures shows that the horizontal deflection at the base of each monolith (at the maximum water surface elevation of +7.0-feet) was similar at the end of the Phase I and Phase II loadings. These figures also show that the initial load-deflection curve is steeper in the Phase II loading than in the Phase I loading. The load test caused a horizontal permanent set of about 0.60-inch at the base of monoliths 3, 4, and 5.

Phase I and II Embankment Movements. The embankment movements were measured by the survey monuments installed on that embankment as described previously. Table 6.1 describes the nomenclature and positioning of these instruments.

Embankment crest horizontal deflections were measured by SM-1 through SM-7 and these are plotted on Figure 6.21. This figure shows that the embankment crest sustained its maximum horizontal movement of 0.80-inches in front of the three center I-wall monoliths (#s 3, 4, and 5). This maximum movement is measured by instruments SM-3, SM-4, and SM-5. The horizontal movement to either side of this area is less.

Mid-slope horizontal deflections were measured by SM-8 through SM-14 and the measured lateral movements are plotted on Figure 6.22. This figure shows that the embankment mid-slope sustained its maximum horizontal movement of 0.25 to 0.40-inches in front of the three center I-wall monoliths. This maximum movement is measured by instruments SM-10, -11, -12, and SM-13. Similar to the crest movements, the horizontal movement to either side of this area is less.

Figure 6.23 shows the heave of +0.025 to -0.025-inches measured by SM-15 through SM-21 at the embankment toe. This value of heave is negligible.

Figures 6.7 and 6.8 are reproduced here from the URS report titled "*Structural and Foundation Response Measured During the Site Specific Load Test on the London Avenue Outfall Canal I-Wall/Levee*" (full report found in Appendix I). These figures plot the measured wall and embankment deflections at canal water surface elevations of +2, +4, +6, and +7 feet for the Phase I and Phase II load tests. Figure 6.7 shows that the Phase I wall and embankment crest lateral movement is symmetrical around the center of the cofferdam. The mid-slope movements are much less symmetrical, with the maximum movement centered around monolith 5 in the cofferdam. There is little to no movement at the embankment toe. Figure 6.8 shows the permanent set (about 0.60-inch) toward the protected side and a much less symmetrical pattern of deflections. In Phase II, the maximum wall and embankment crest deflection is centered about monolith 3 in the cofferdam. On the other hand, the maximum mid-slope deflection in Phase II remains centered around monolith 5. In Phase II there is slightly more toe movement and this movement occurs more randomly across the test site.

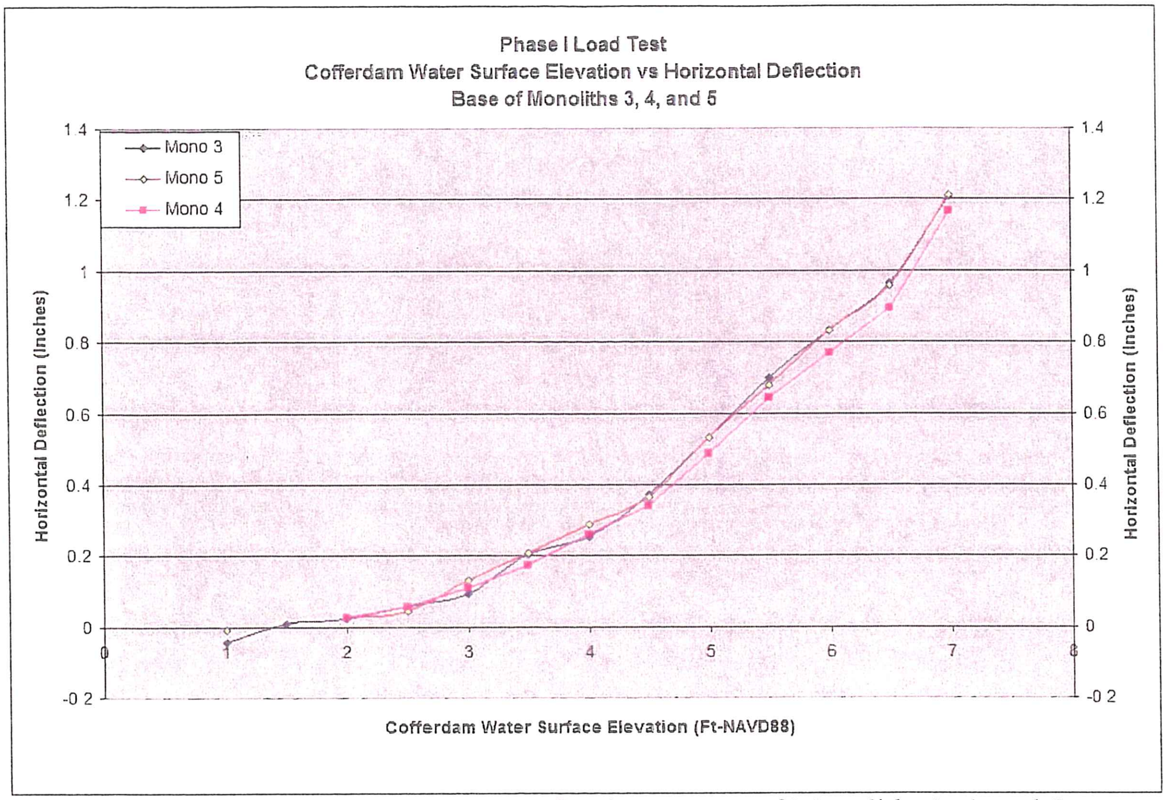


Figure 6.2 – Phase I Horizontal Deflection at Base of Monoliths 3, 4, and 5.

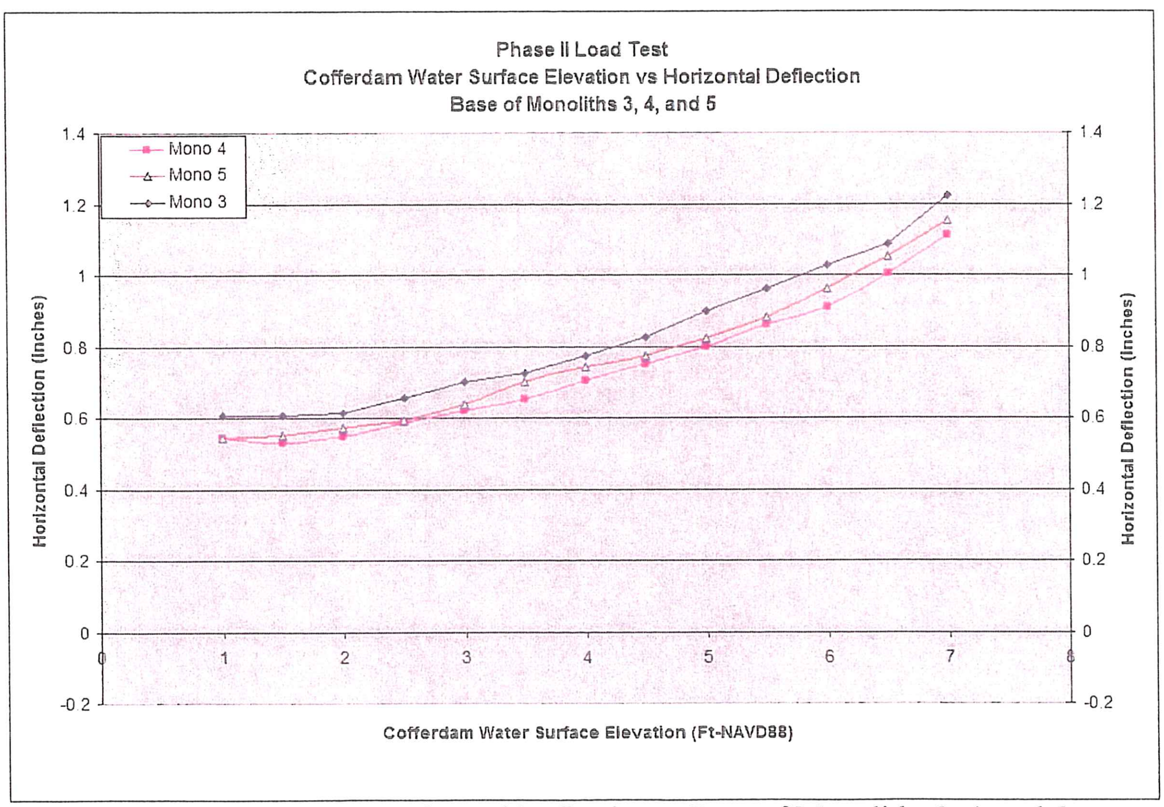


Figure 6.3 – Phase II Horizontal Deflection at Base of Monoliths 3, 4, and 5.

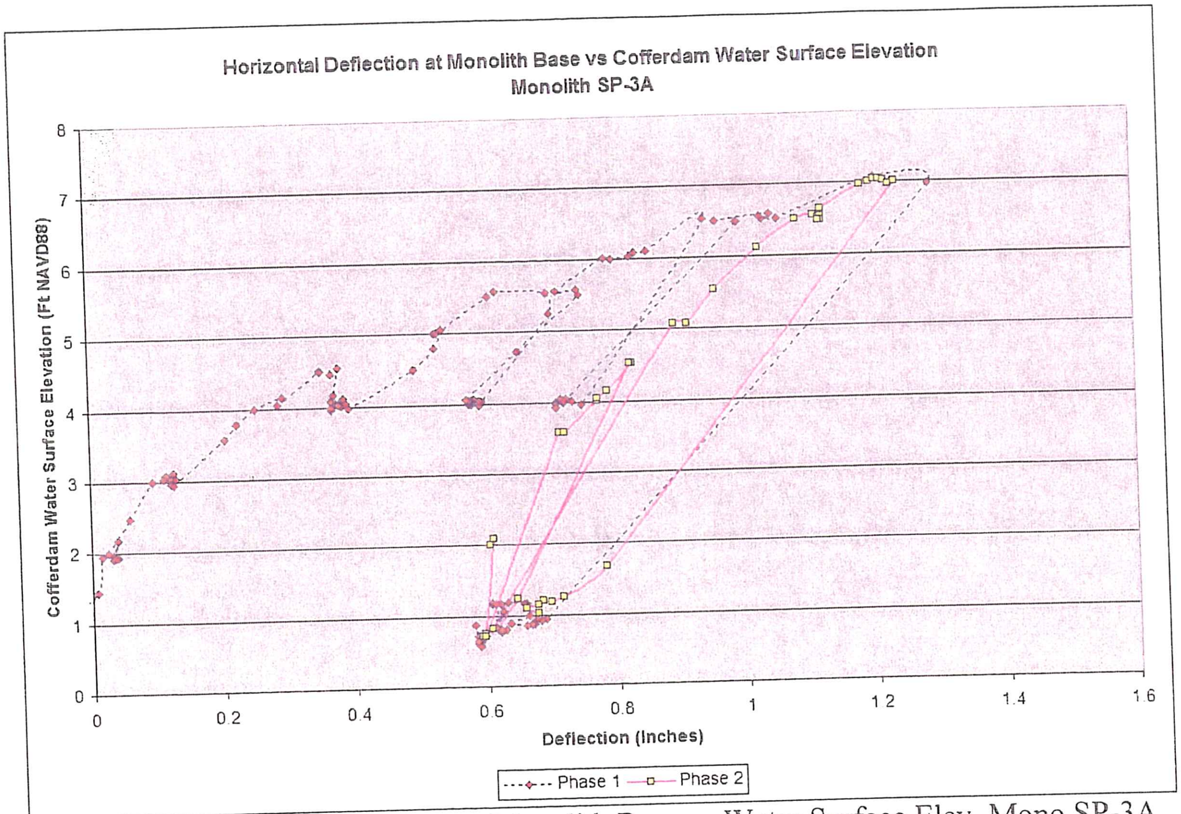


Fig 6.4 – Horizontal Deflection at Monolith Base vs Water Surface Elev. Mono SP-3A

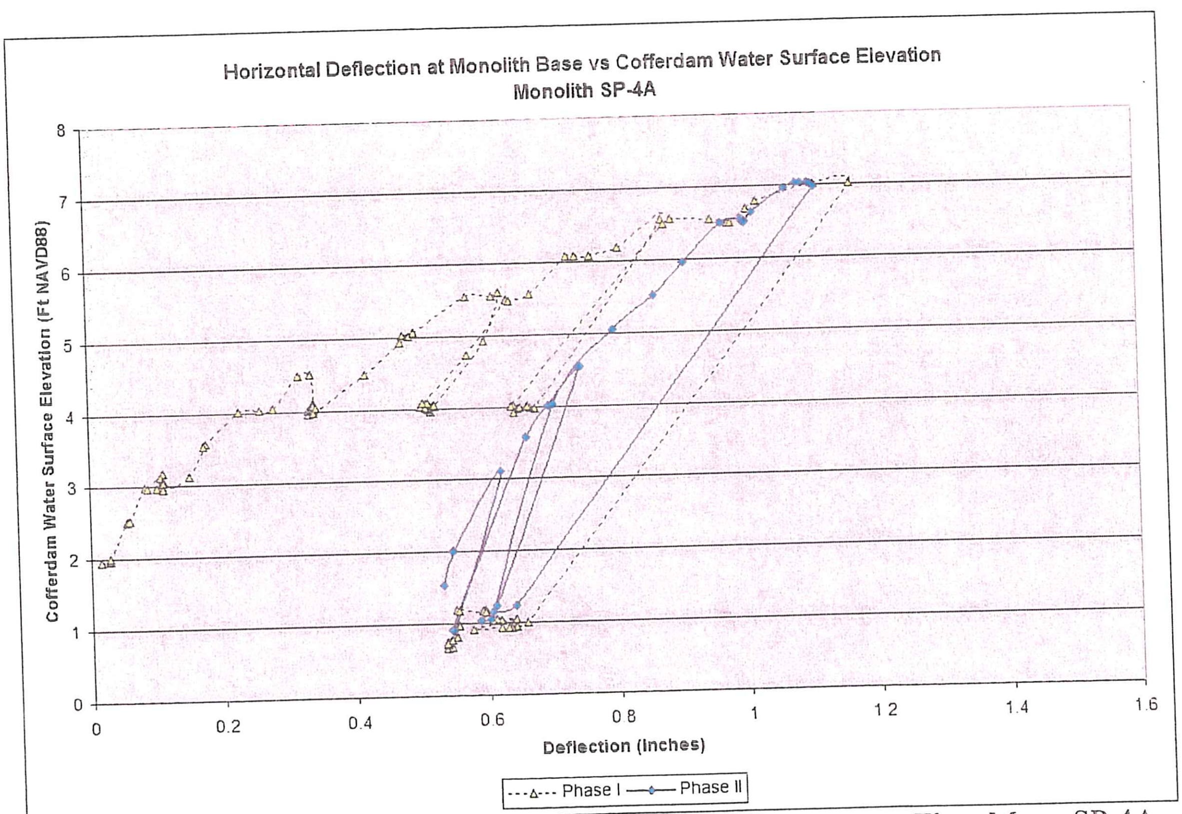


Fig 6.5 – Horizontal Deflection at Monolith Base vs Water Surface Elev. Mono SP-4A

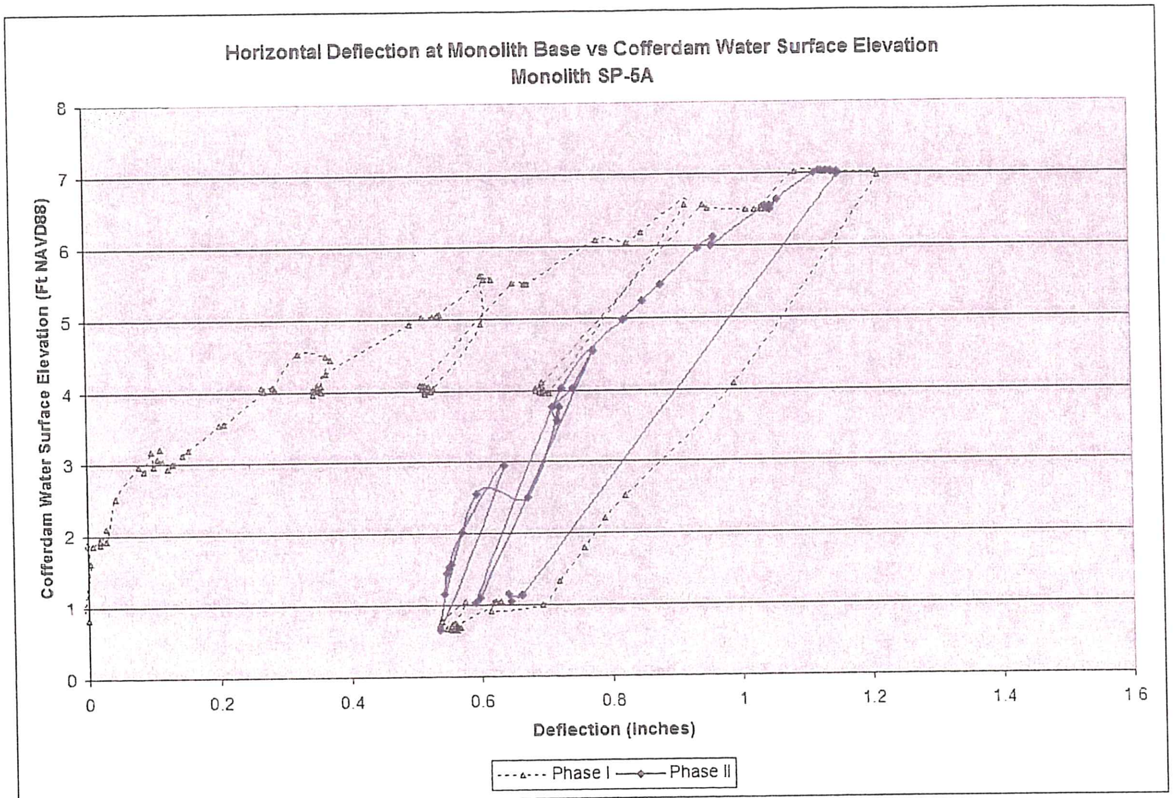


Fig 6.6 – Horizontal Deflection at Monolith Base vs Water Surface Elev. Mono SP-5A

Phase I and Phase II Tiltmeter Data. The measurements of wall rotation of the three center monoliths as recorded by tiltmeters TM-2, TM-3, and TM-4 are plotted with respect to the Cofferdam water surface elevation in Figures 6.9 and 6.10 (Phases I and II). The monolith's rotation clearly responds to the water loads placed on the I-wall. As the water levels increase, so does the measured rotation angle on each monolith. Similar to the measured horizontal deflection data, the internal three monoliths registered the greatest rotation. During Phase I, the maximum rotation angle measured on the three internal monoliths varied from -0.32° to -0.36° and corresponded with the application of the maximum cofferdam water surface elevations of +7.0-feet. During Phase II, the maximum measured rotation on the same monoliths varied from -0.40° to -0.44°. The tiltmeters also register a slight permanent rotation of -.25 degrees remaining at the end of the Phase I test.

The tiltmeters have captured thermal changes in the wall monoliths. The URS ADAS obtained tiltmeter data during the days between the Phase I and Phase II load tests, when no water loads were placed on the I-wall (i.e., between 23 August to mid-morning of 26 August). As shown on Figure 6.11, during the first three days of this period, each of the five tiltmeters registered a small, but measureable response beginning at about 6:00am and lasting until the late afternoon hours of each day. No such response can be seen on the day of 26 August. URS checked the video record obtained during the load test and reported that August 23rd, 24th, and 25th were bright, sunny days but August 26th was overcast.

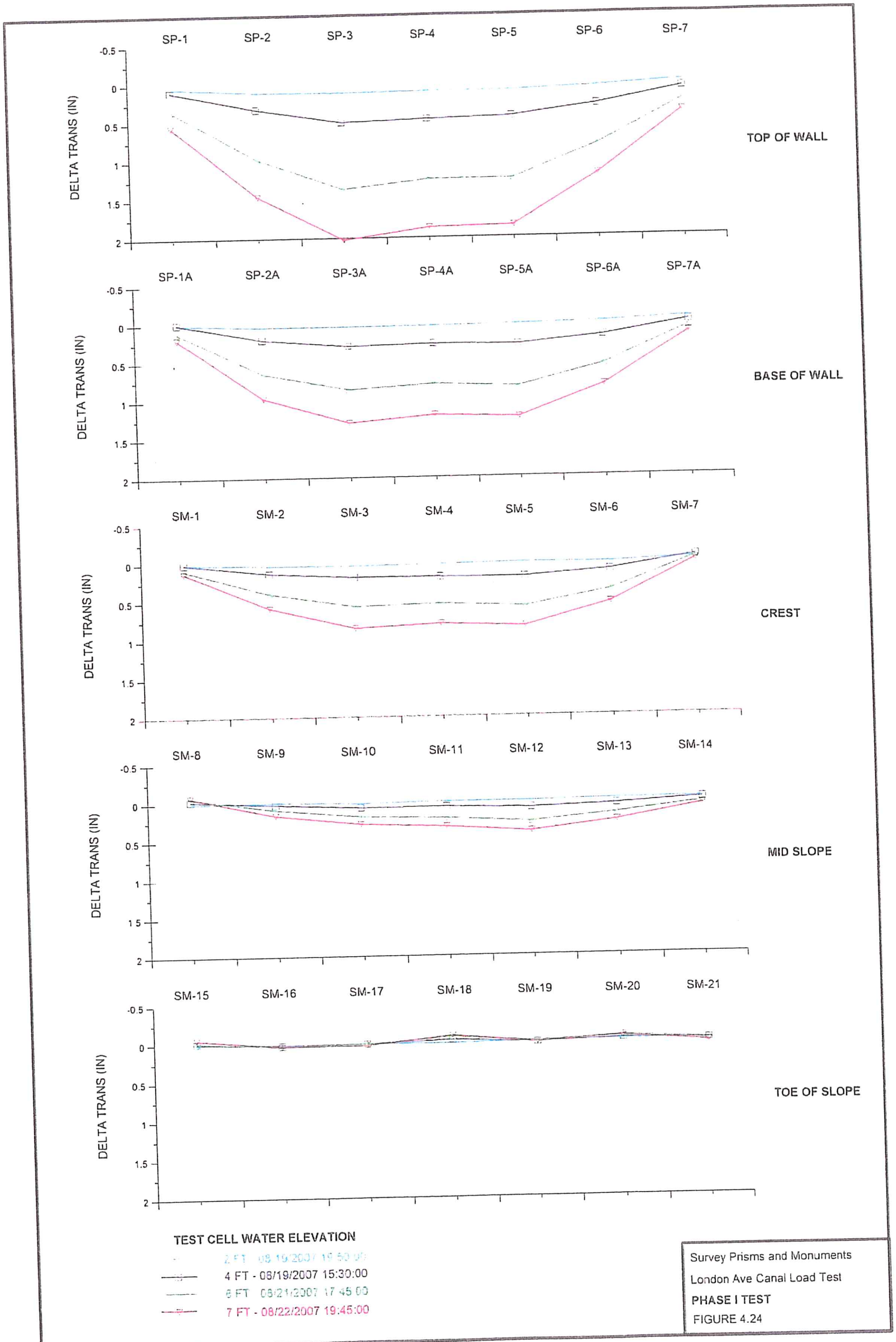


Fig 6.7 – Phase I Embankment Movements

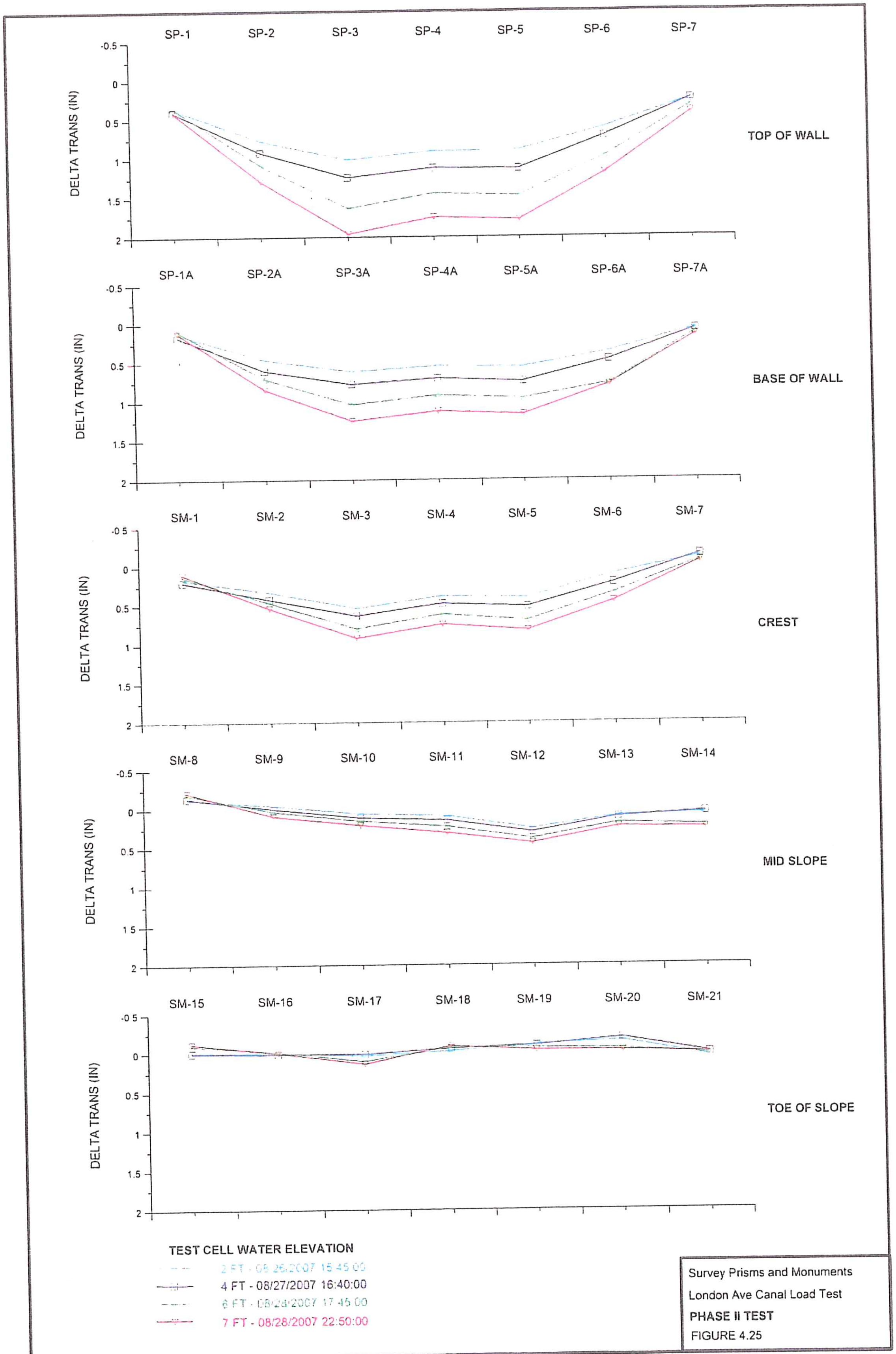


Figure 6.8 - Phase II Embankment Movements

In this application, the ADAS sign convention is that wall rotation due to load test loading causes a (-) value of rotation (i.e. the top of the wall rotates toward the protected side). On August 23rd, 24th, and 25th, from 0600 to about Noon, the sun shone on the landside, vertical face of the wall and, at this time, the ADAS registers a rotation opposite to that seen during load test loading. The morning sun will heat up the landside of the wall faster than the canal side of the wall. There will be more expansion on the warmer side of the wall causing a small, but measureable, bow in the wall. Near noontime, the sun passes directly over the top of the wall and then begins to shine on the canal side of the wall. The phenomena repeats itself but in the opposite direction. URS and CEMVS structural engineers confirmed that thermal changes caused by daily sun cycles will cause concrete I-walls to expand and contract. The tiltmeters have captured these changes.

As part of the on-going quality-control program for measured data, the Technical Review Team asked the Leica technician to calculate wall rotation using the known distance between the survey prisms located at the top and bottom of each monolith and the horizontal deflections obtained from those instruments. A comparison between the calculated rotation and the tiltmeter measured rotation is plotted on the URS Figure 4.26 (titled "*Structural and Foundation Response Measured During the Site Specific Load Test on the London Avenue Outfall Canal I-Wall/Levee*" - full report found in Appendix I) and that plot is included here as Figure 6.12. Initially, the measured rotation values from the tiltmeters did not match the computed values from the Leica system. Ensuing discussion between URS and Leica revealed that environmental corrections applied to the Leica survey data was unnecessary due to the close proximity of the survey prisms to the robotic total station units.

Leica installed backsights 200-feet north and 400-feet south of the load test site. A third backsight was installed at the rear of the test side at the edge of the street. The Leica total station systems automatically compute corrections based on ambient temperature and humidity and these environmental corrections were enabled when surveying the backsights. But the same environmental correction was applied during the initial parts of the Phase I load test when surveying the prisms and monuments, some of which were only ten feet away from the total station unit. When the environmental corrections were removed, there began a very close correlation between the rotation measurements from the tiltmeters and the calculated rotation values from the survey prisms. This correlation is plotted on URS plot 4.26. That plot is reproduced here as Figure 6.12.

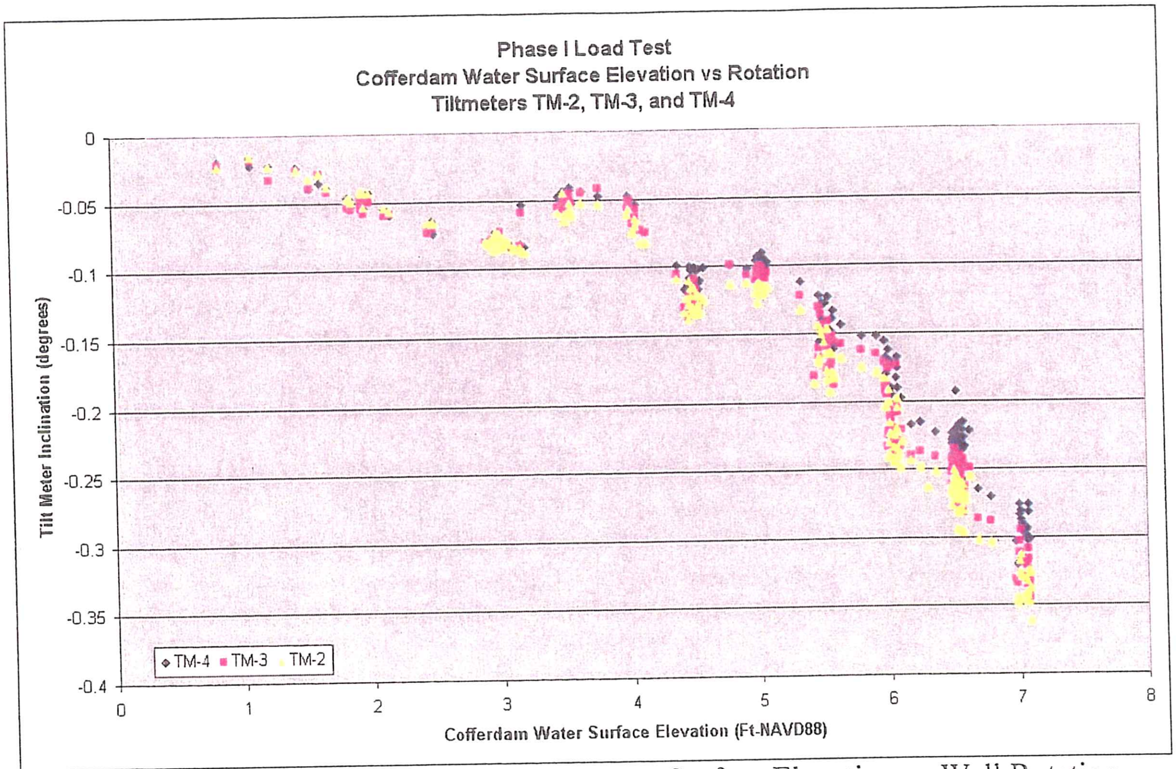


Figure 6.9 – Phase I Cofferdam Water Surface Elevation vs Wall Rotation

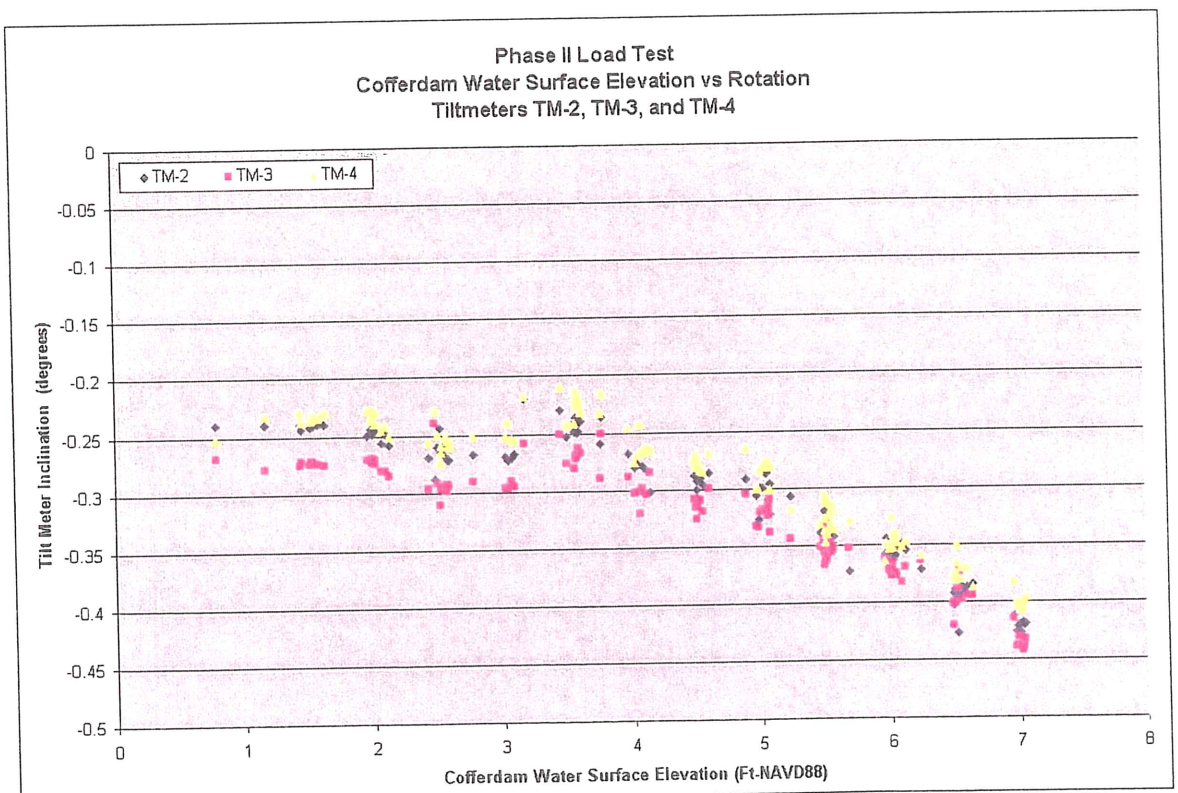


Figure 6.10 – Phase I Cofferdam Water Surface Elevation vs Wall Rotation

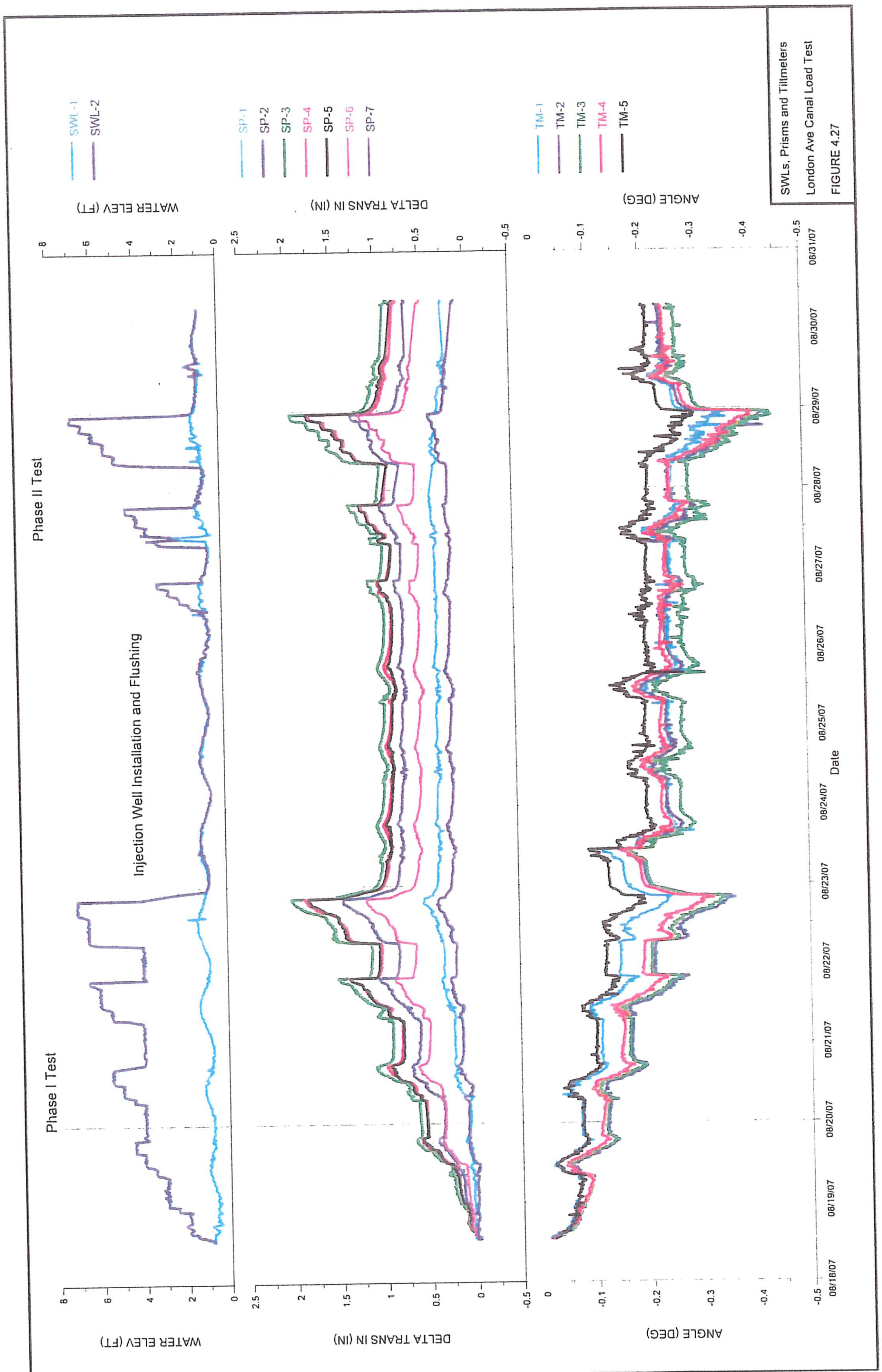


Figure 6.11 – Surface Water Levels, Survey Prisms, and Tiltmeter (From URS Fig 4.27)

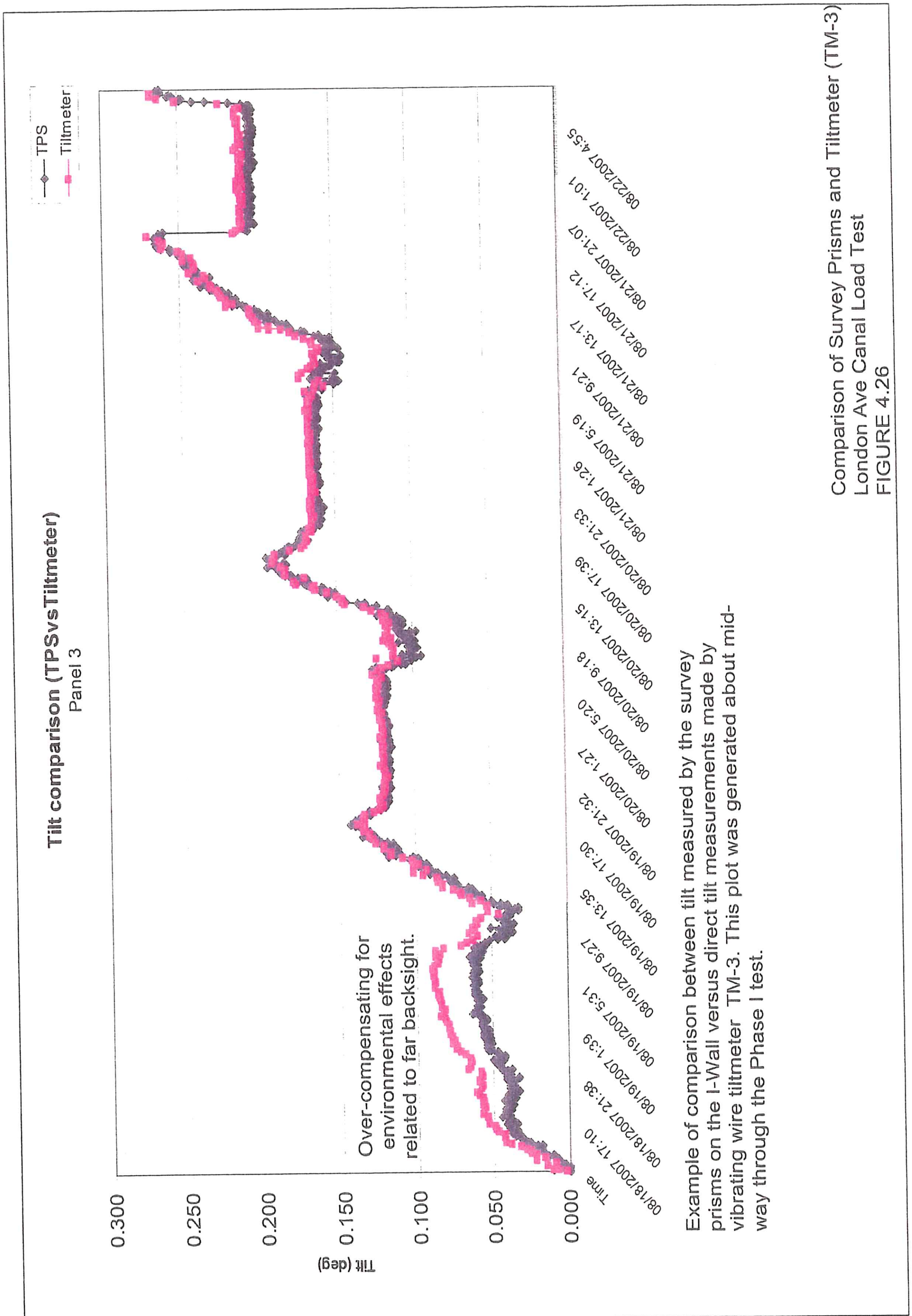


Figure 6.12 – Tiltmeter Data vs Computed Tilt from Survey Prisms

Observed Transverse Crack in Canal Side Embankment. After the Phase I test was completed and the cofferdam drained, the Technical Review Team inspected the canal side I-wall embankment and observed a crack in the top of the embankment that runs roughly parallel to the wall. Figure 6.13 shows the crack. This crack was not present at the beginning of the load test. This picture shows that the top of the soil mass between the crack and the wall is lower indicating that the soil has collapsed against the wall. There is no information that defines the orientation of neither the crack nor the shape and size of the soil mass. It is impossible to tell when the soil mass moved, but given the fact that vertical cuts in clay will stand for some time before collapsing, it is likely that the soil mass stayed intact for some time following the beginning of wall deflection. .

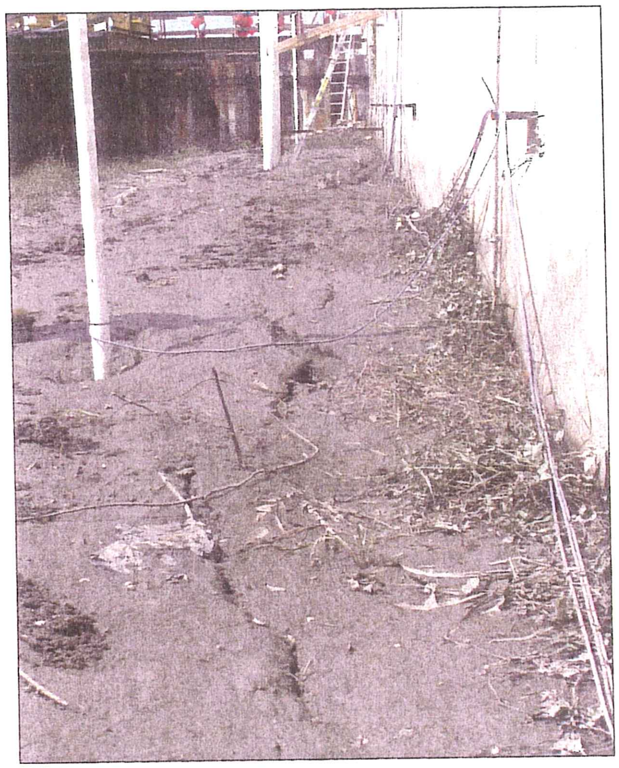


Figure 6.13 – Crack in Canal Side Embankment After Phase I Load Test.

The following table contains detailed measurements. A second crack, parallel to the first, was noted in some places. In all cases, the crack depth was measured to be 1-inch or less.

Table 6.2 – Transverse Crack Dimensions

Distance from South Tie-In (FT)	Distance from Wall to 1st Crack (FT)	Distance from Wall to 2nd Crack (FT)	Crack Depth (IN)
14.6	Begin Crack		
19.6	2.5	-	0
24.6	2.6	-	0
29.6	2.7	-	1
34.6	2.25	3.6	0
39.6	2.25	3.33	0
44.6	1.75	-	0
48.6	2.7	-	0

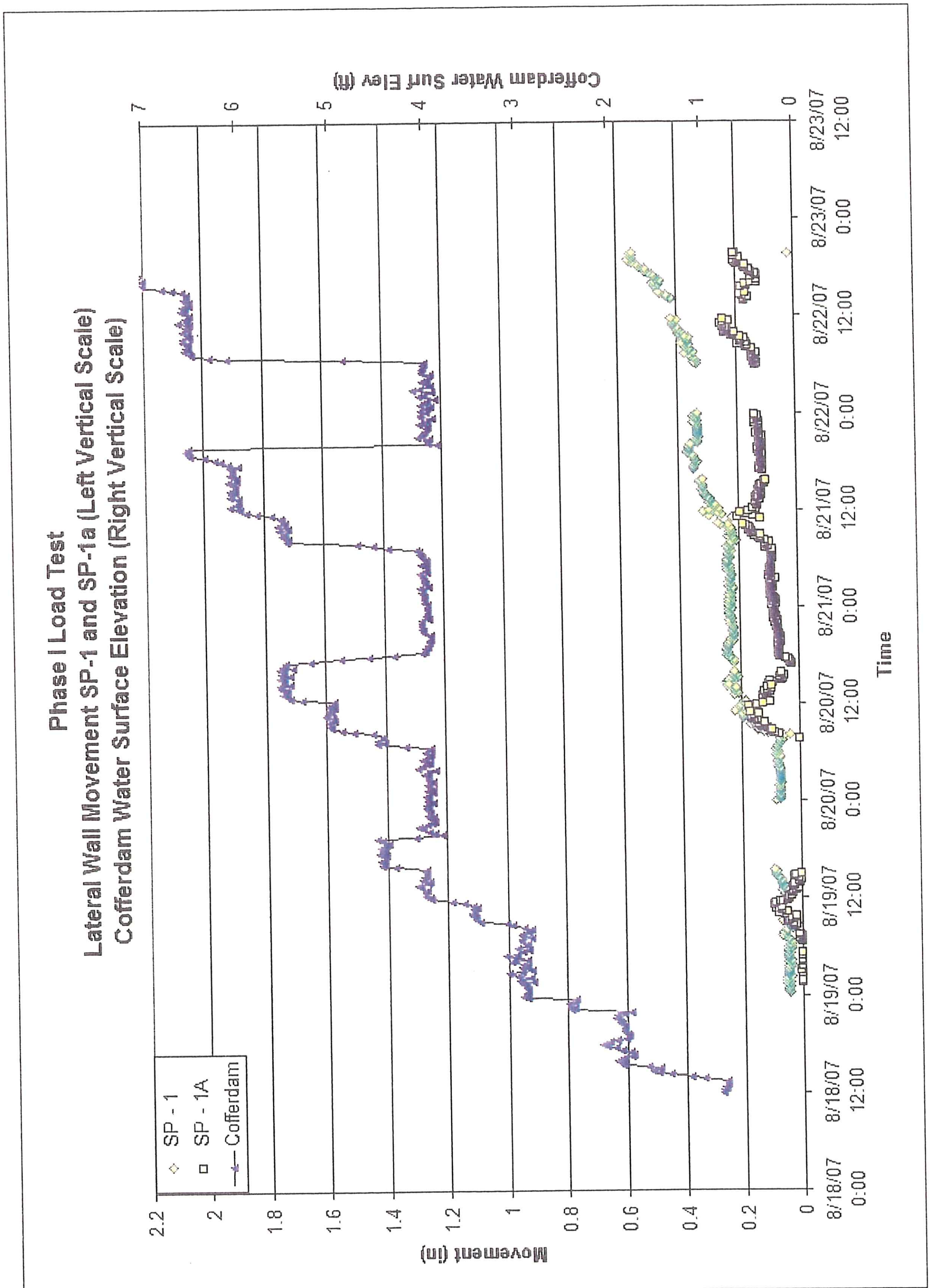


Figure 6.14 – SP-1 and SP-1A Movement During Phase I.

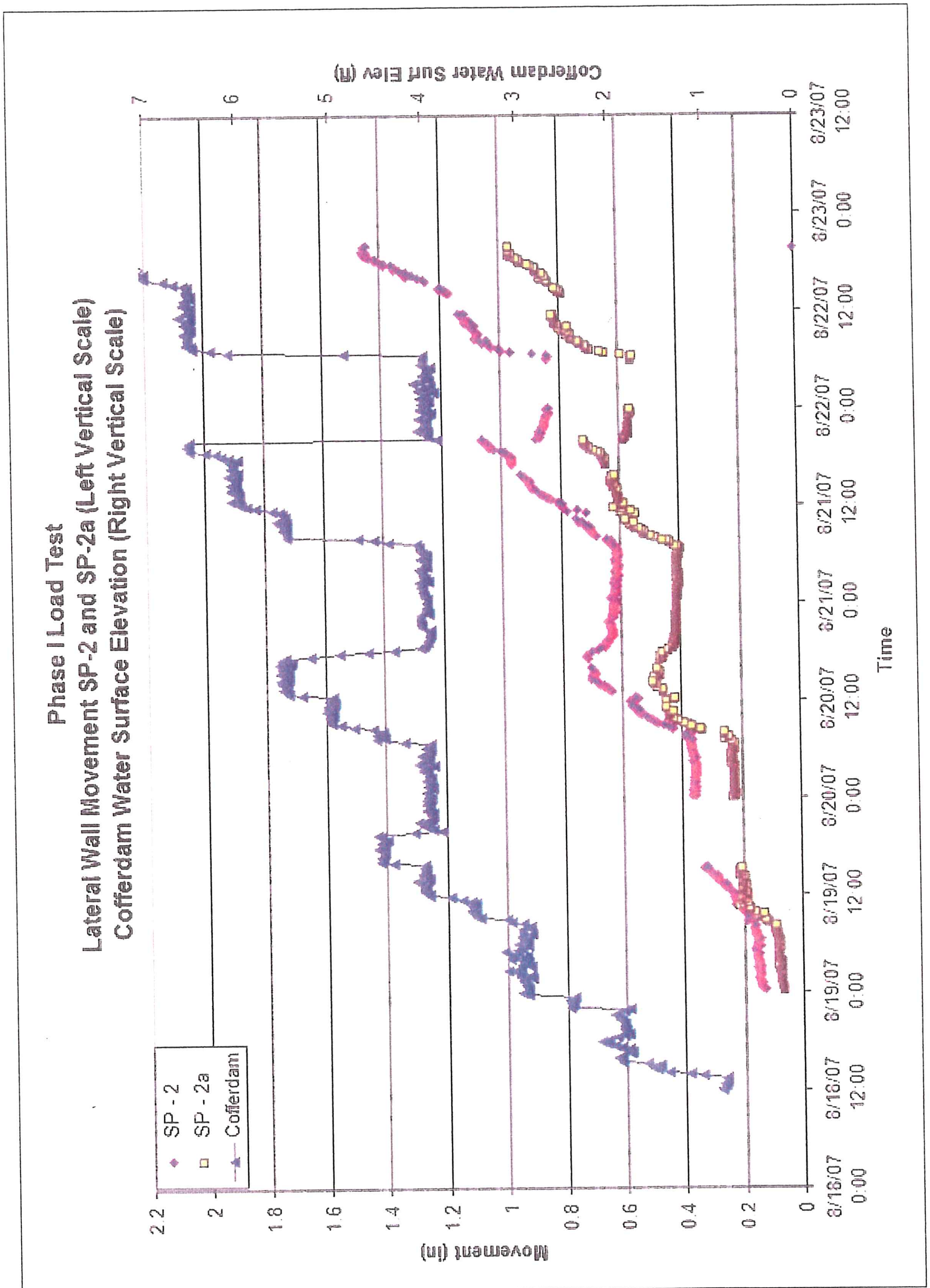


Figure 6.15 – SP-2 and SP-2A Movement During Phase I.

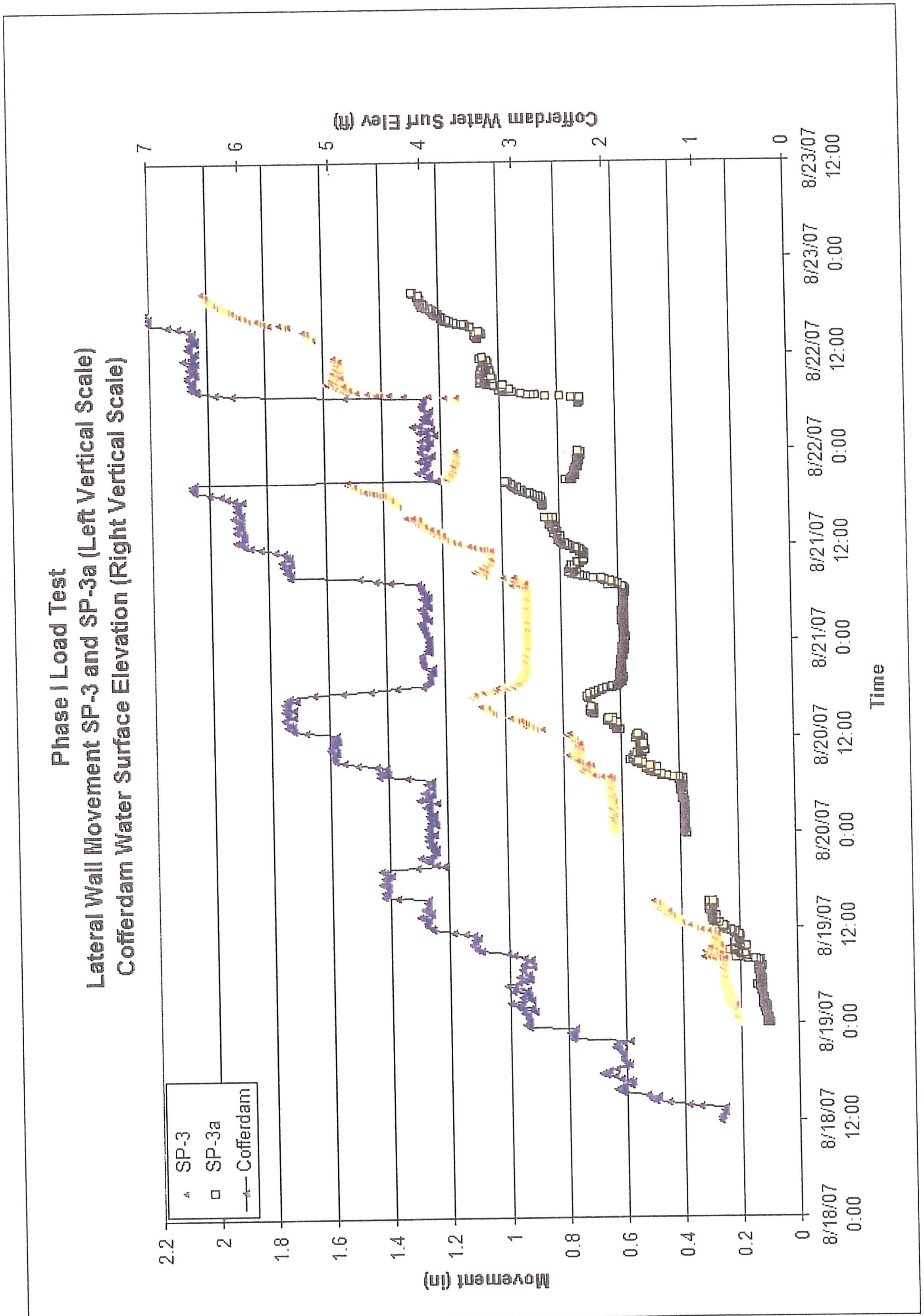


Figure 6.16 – SP-3 and SP-3A Movement During Phase I.

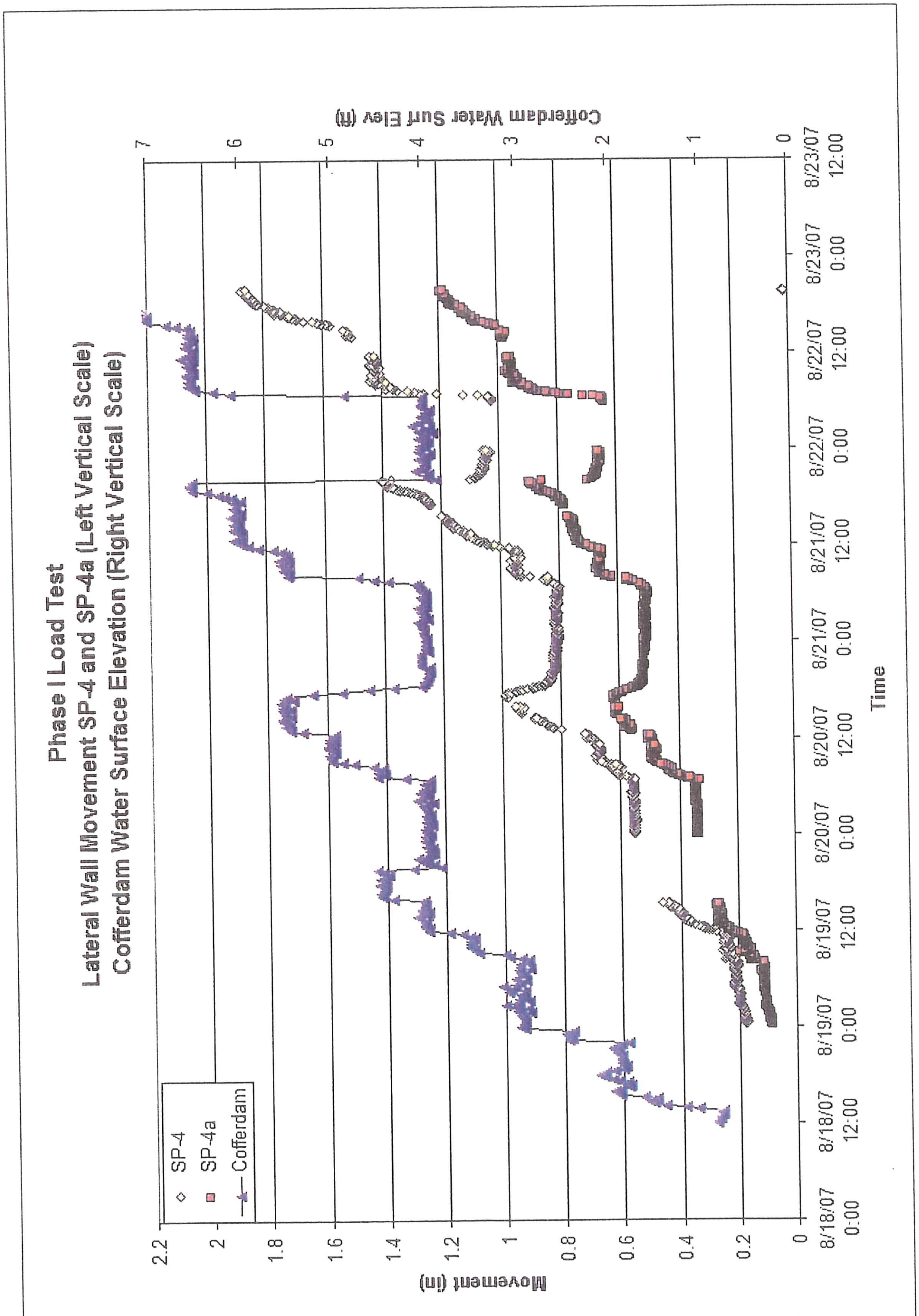


Figure 6.17 – SP-4 and SP-4A Movement During Phase I.

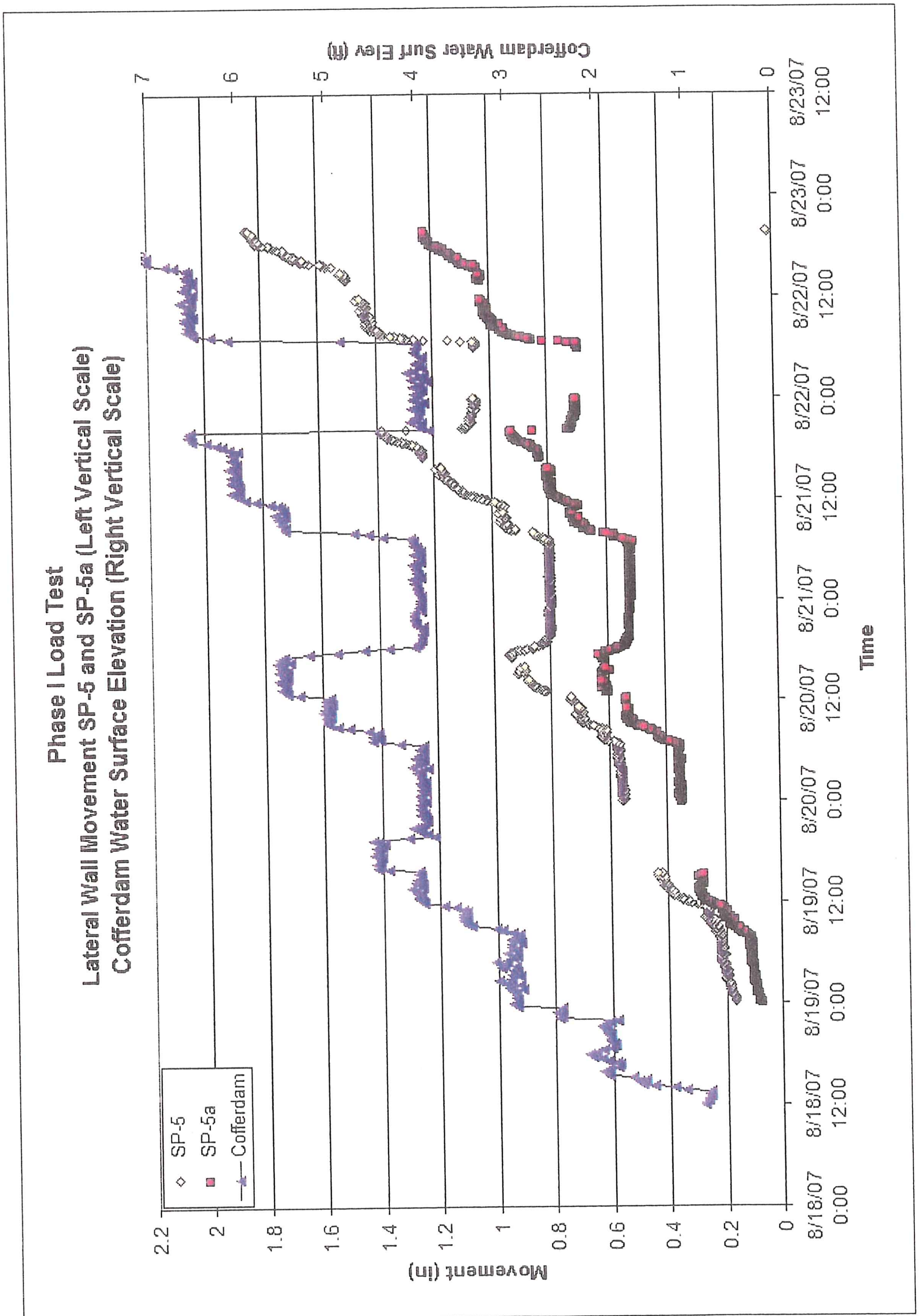


Figure 6.18 – SP-5 and SP-5A Movement During Phase I.

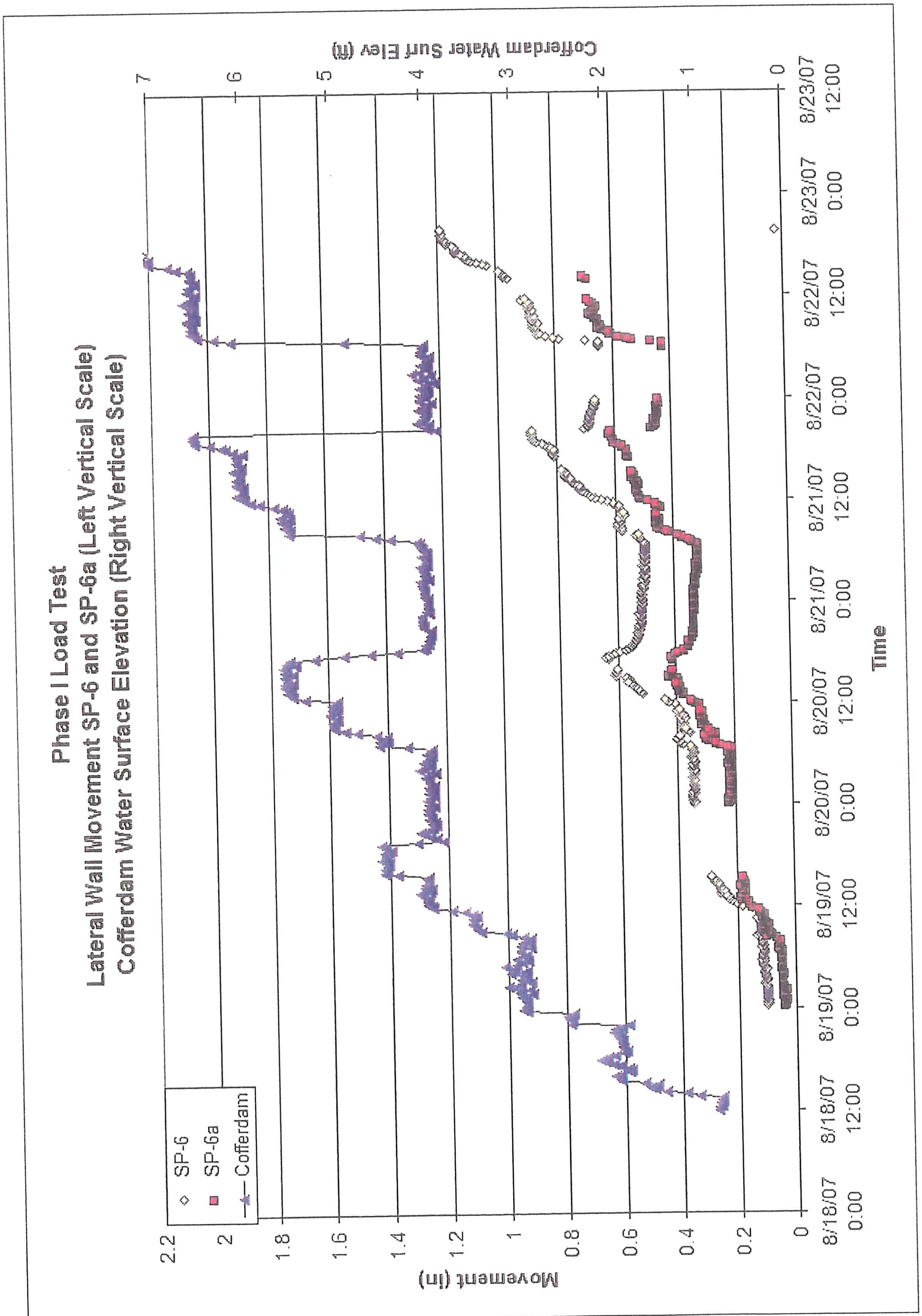


Figure 6.19 – SP-6 and SP-6A Movement During Phase I.

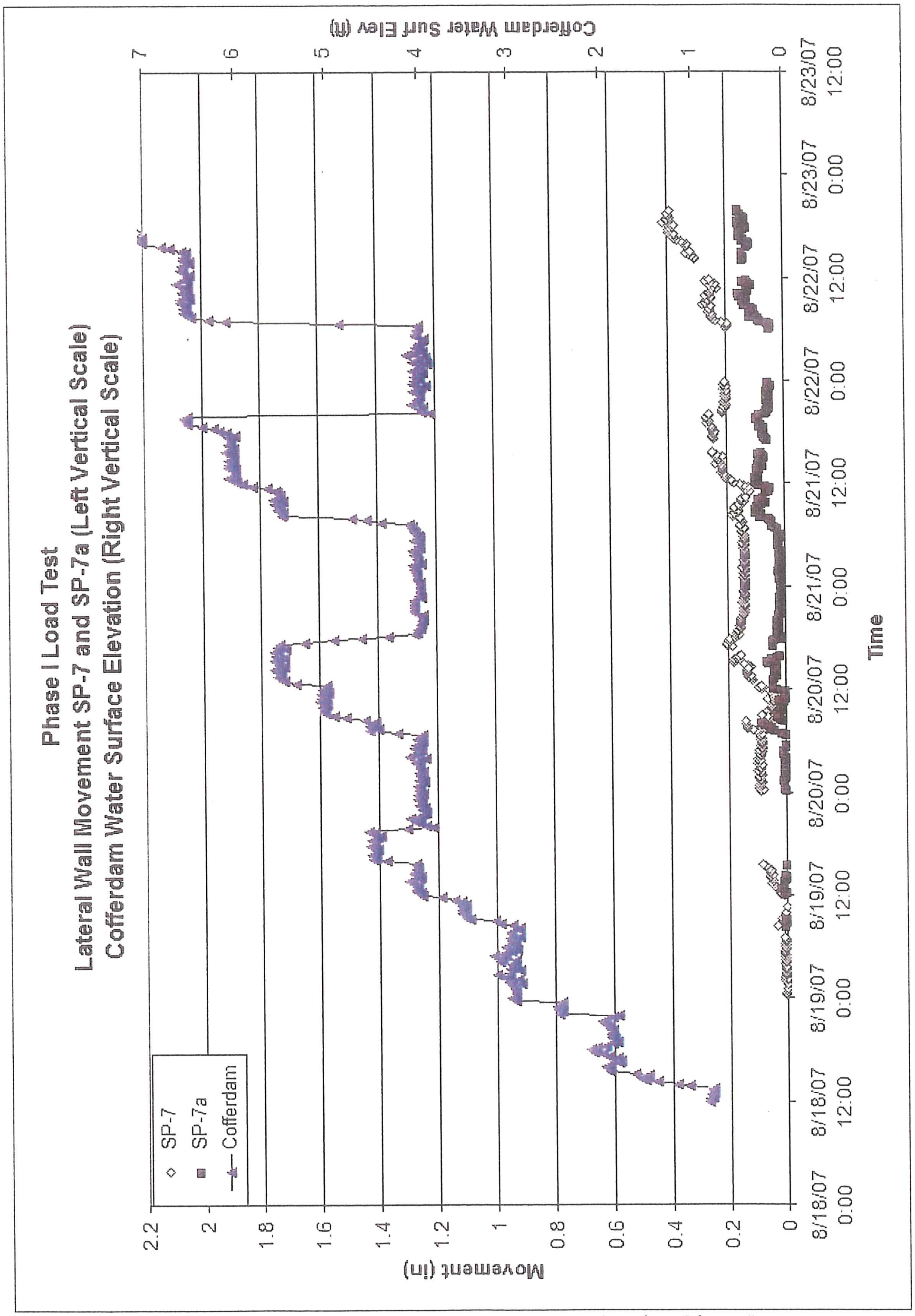


Figure 6.20 – SP-7 and SP-7A Movement During Phase I.

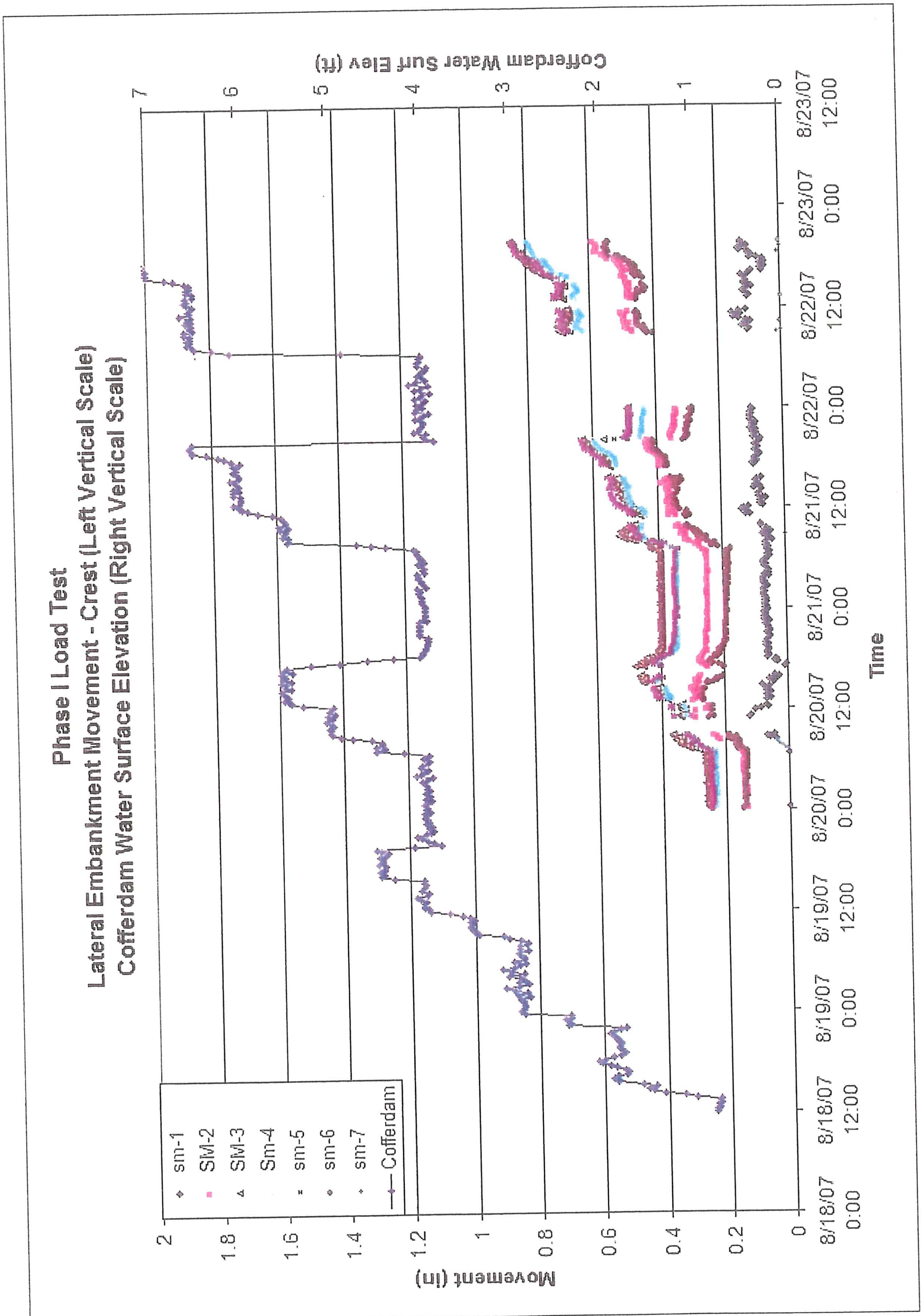


Figure 6.21 – Crest Embankment Movement During Phase I

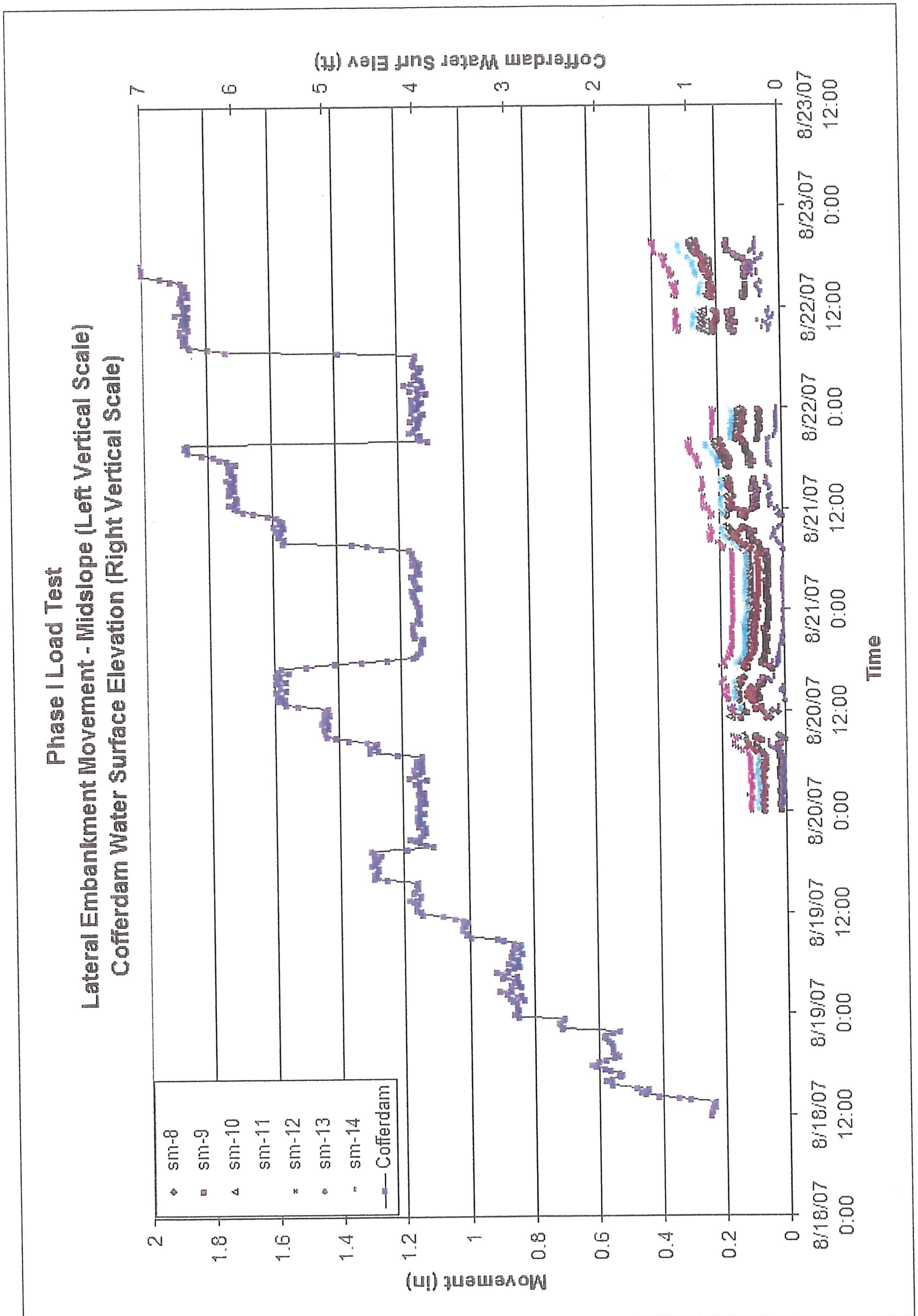


Figure 6.22 – Mid-Slope Embankment Movement During Phase I

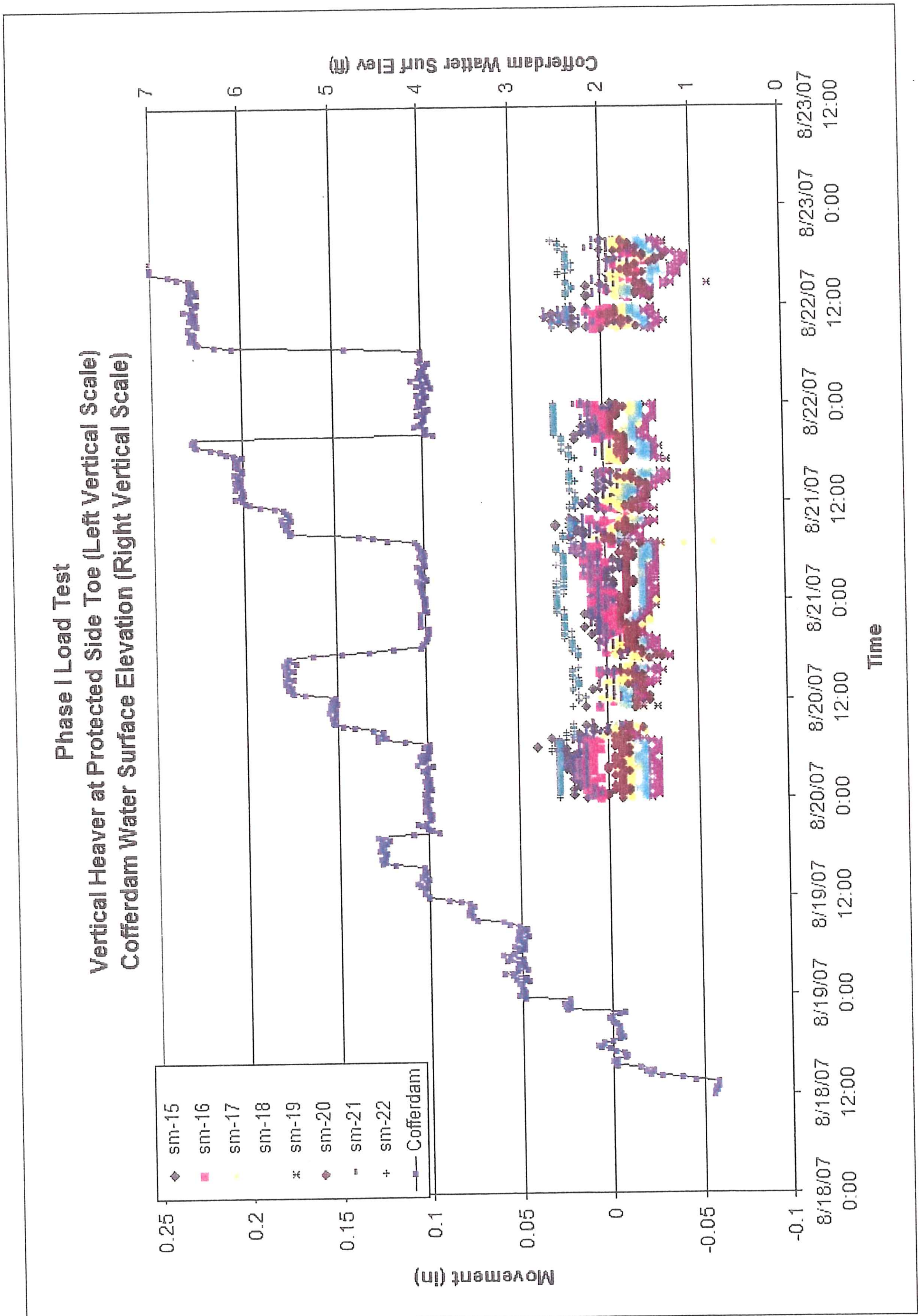


Figure 6.23 –Embankment Toe Vertical Heave During Phase I

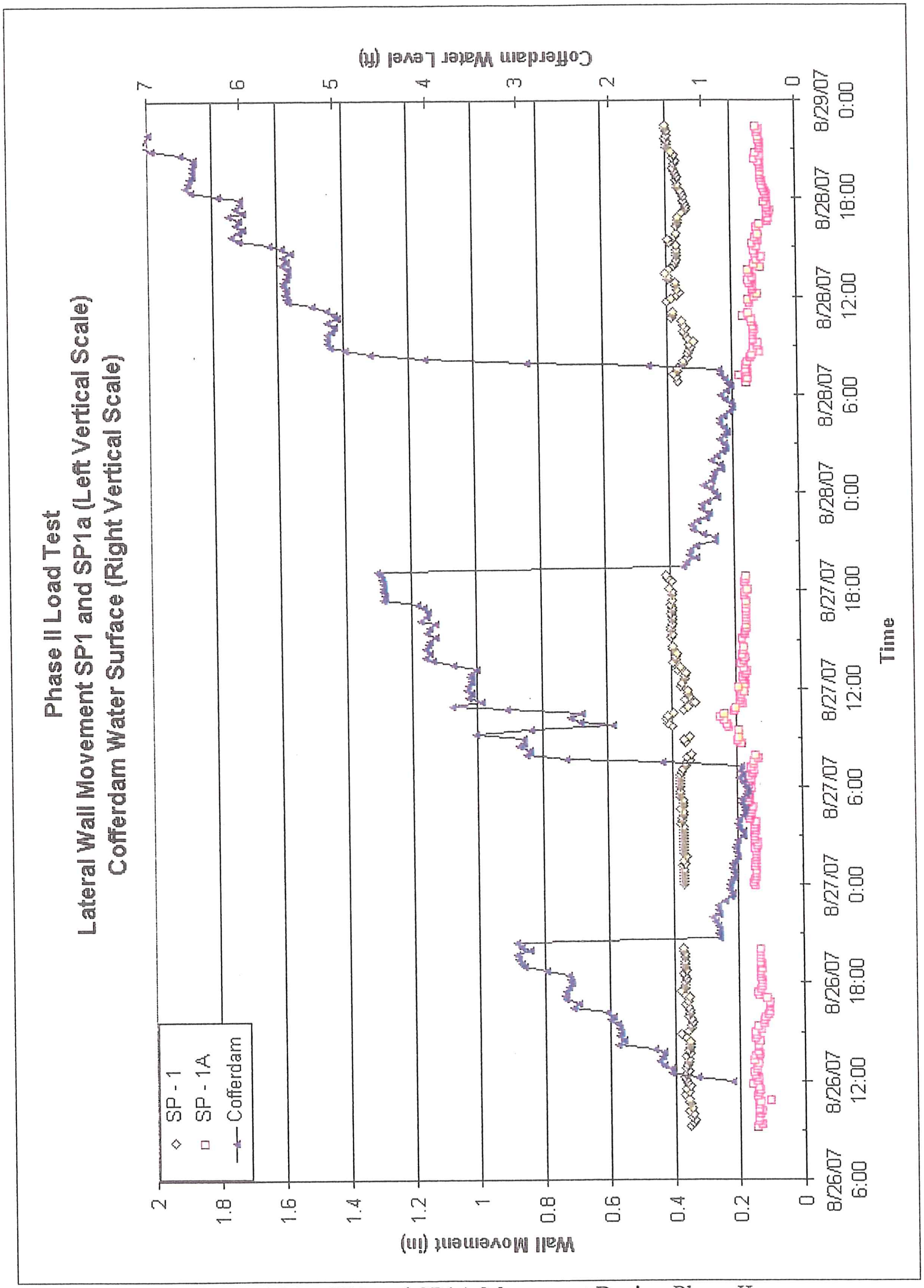


Figure 6.24 – SP1 and SP1A Movement During Phase II

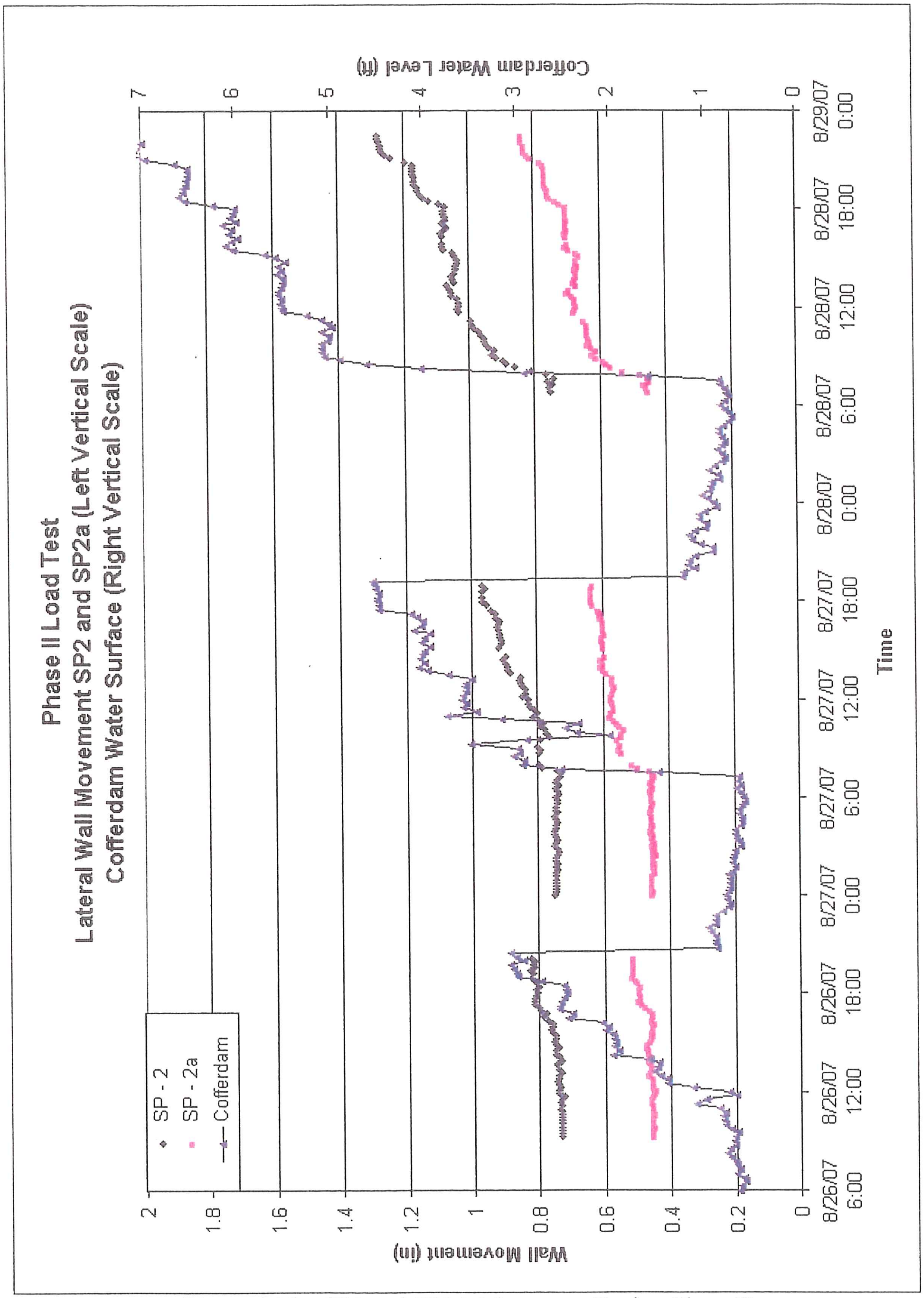


Figure 6.25 – SP2 and SP2A Movement During Phase II

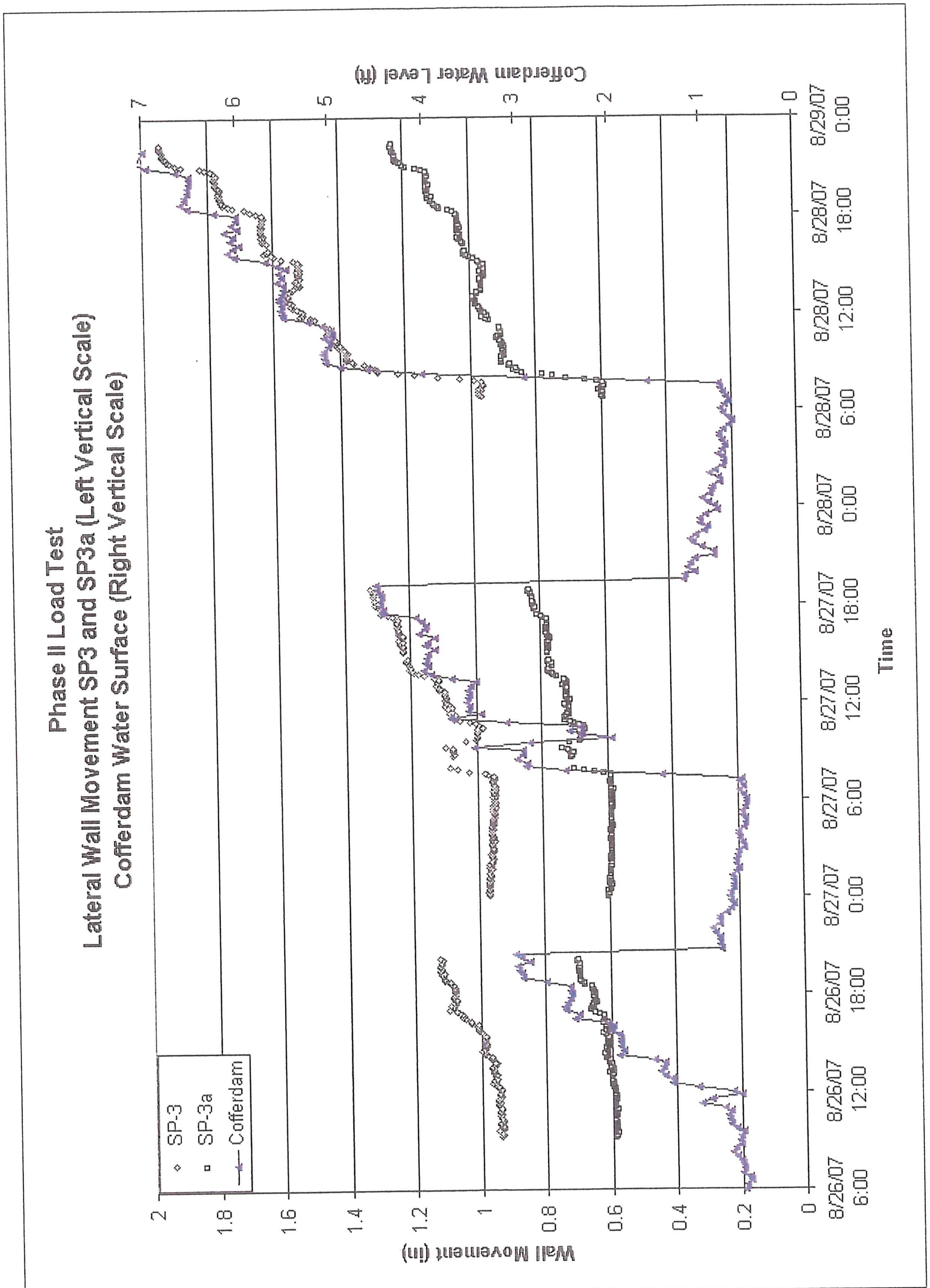


Figure 6.26 – SP3 and SP3A Movement During Phase II

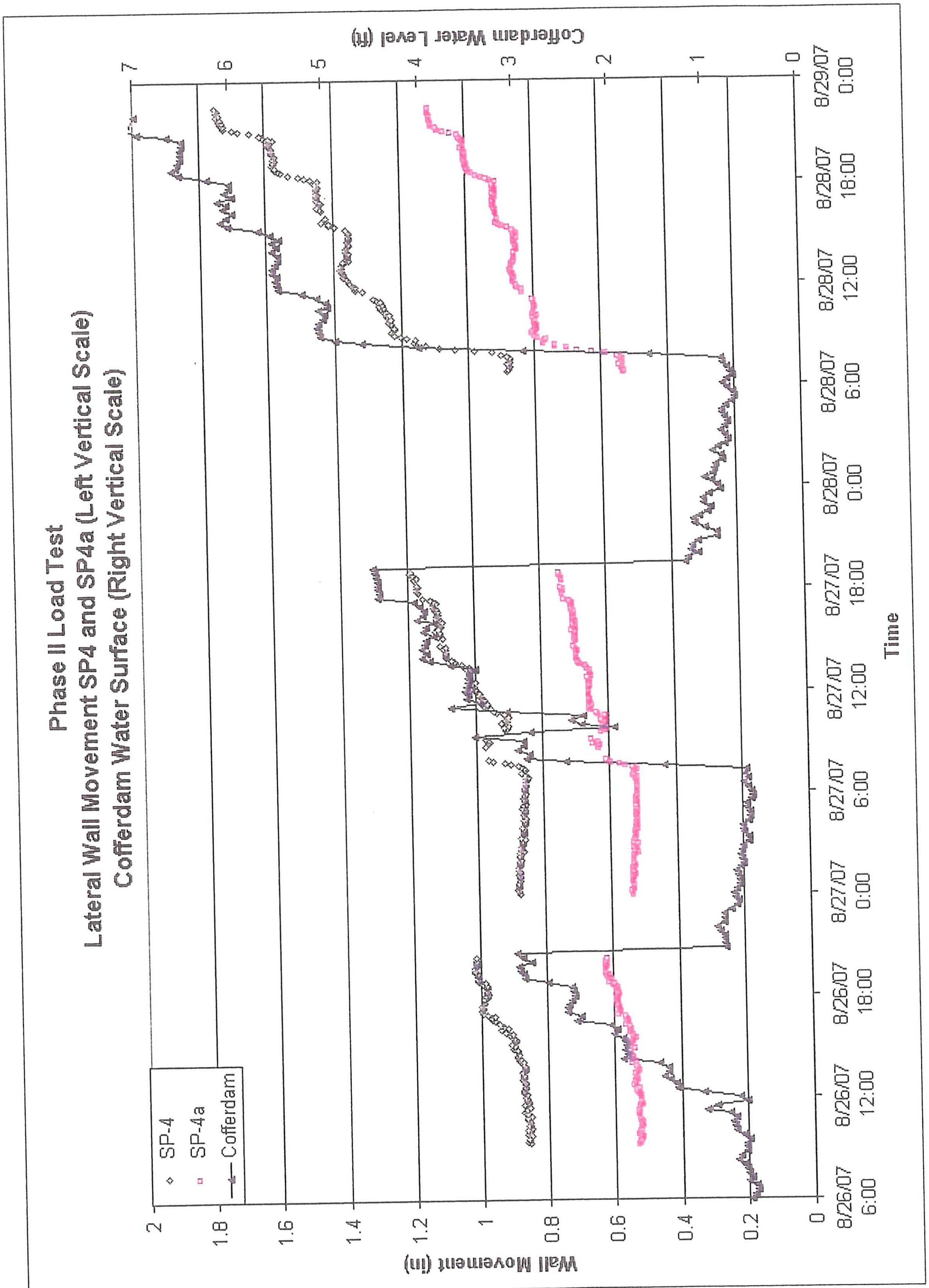


Figure 6.27 – SP4 and SP4A Movement During Phase II

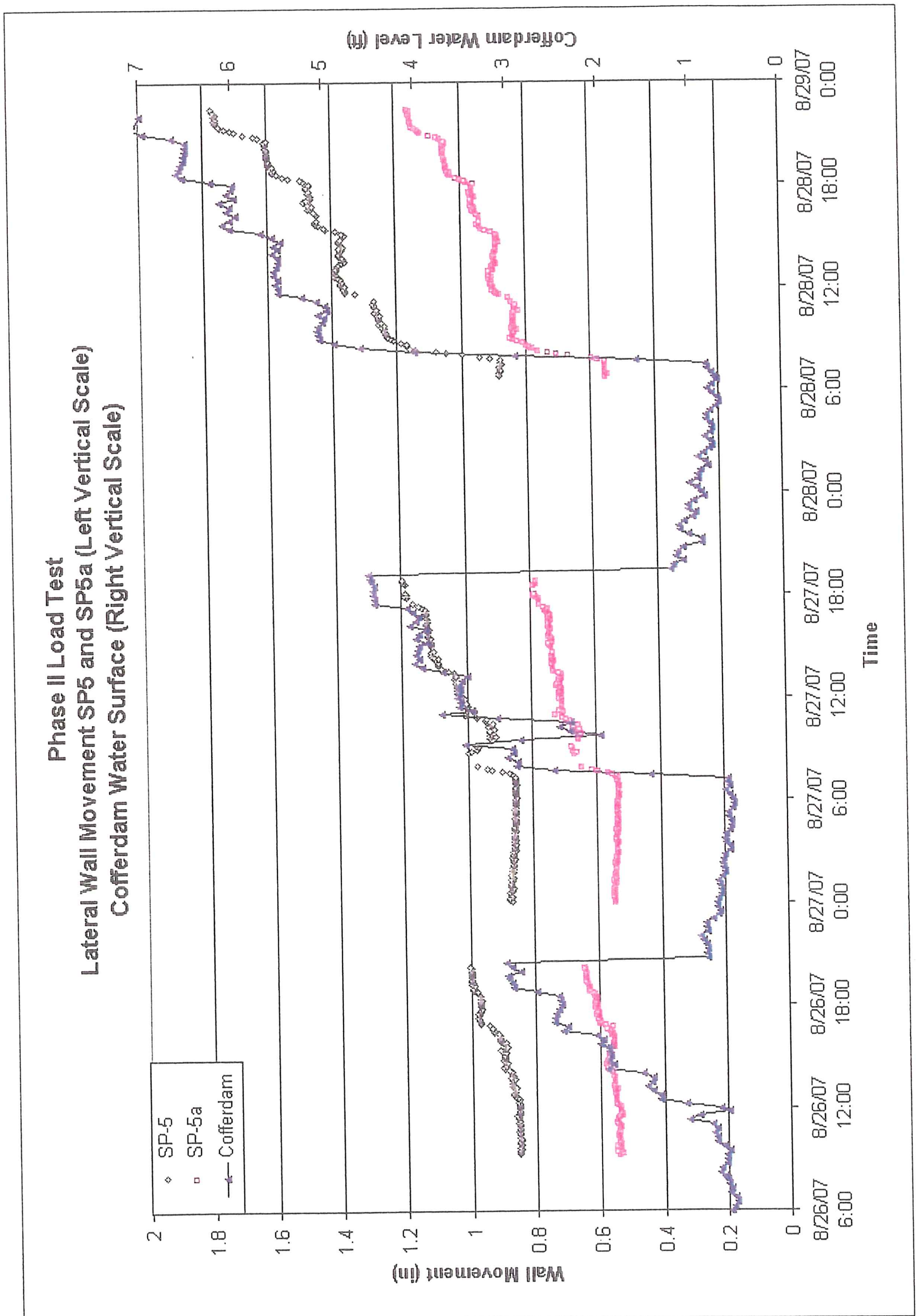


Figure 6.28 – SP5 and SP5A Movement During Phase II

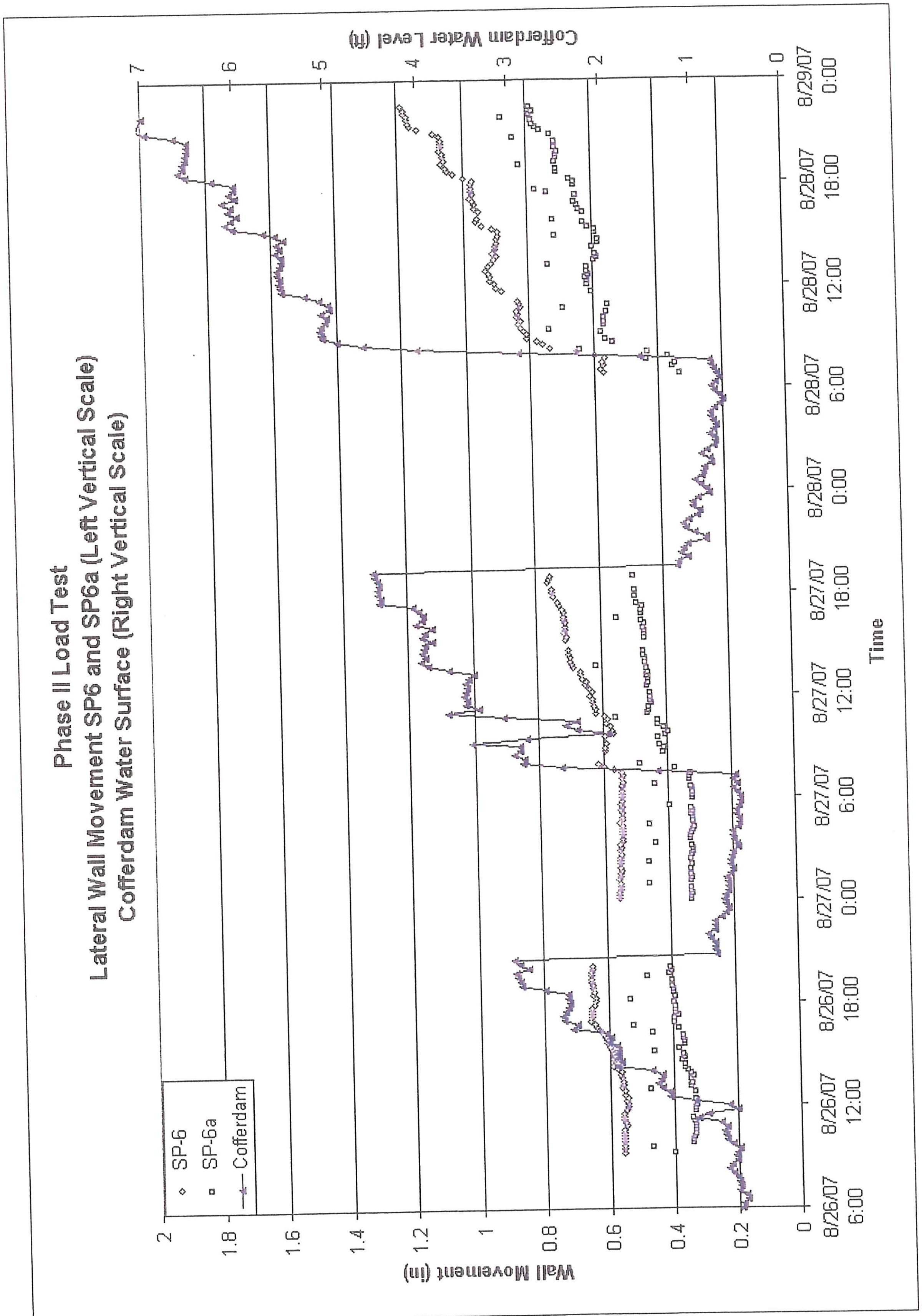


Figure 6.29 – SP6 and SP6A Movement During Phase II

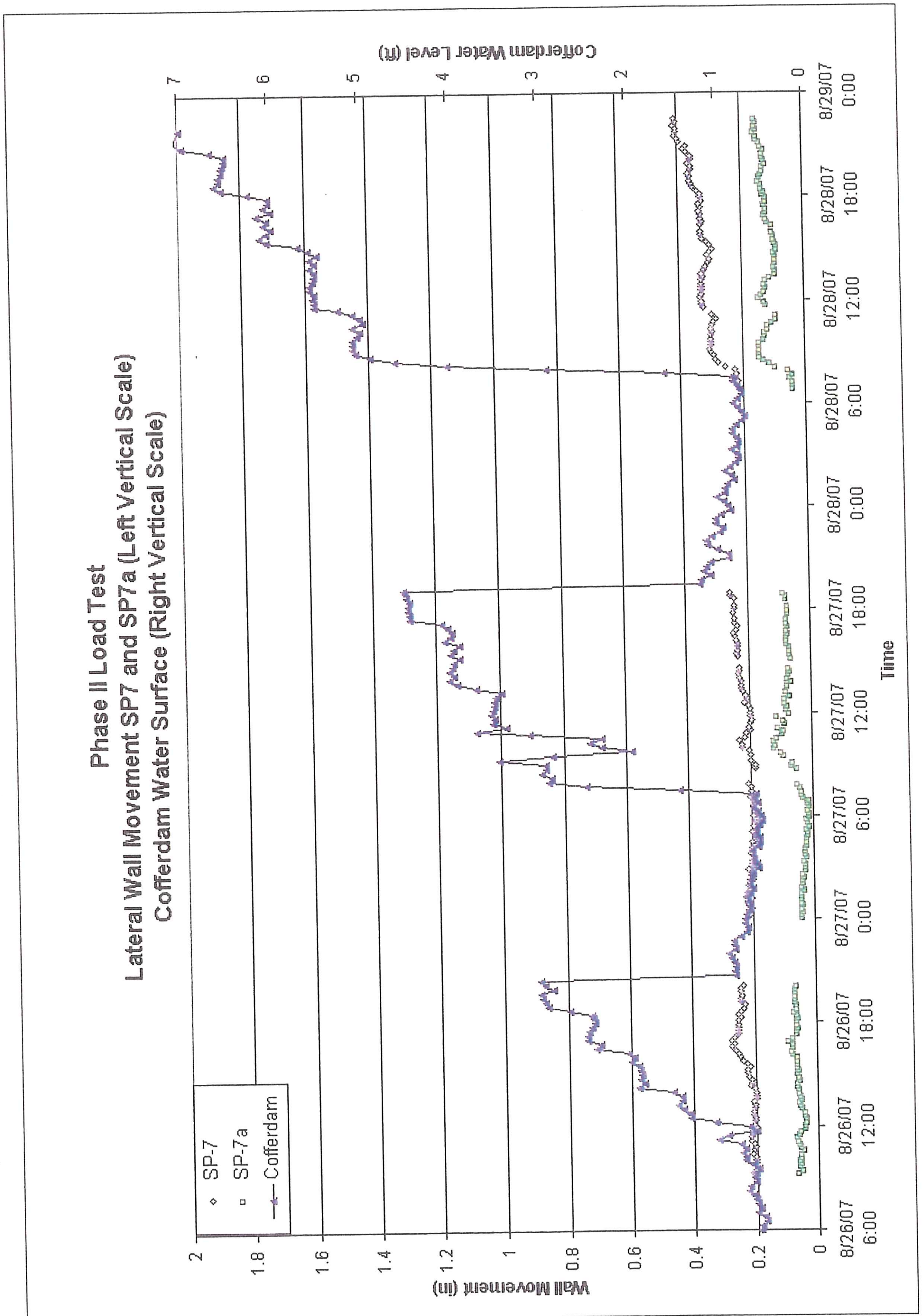


Figure 6.30 – SP7 and SP7A Movement During Phase II

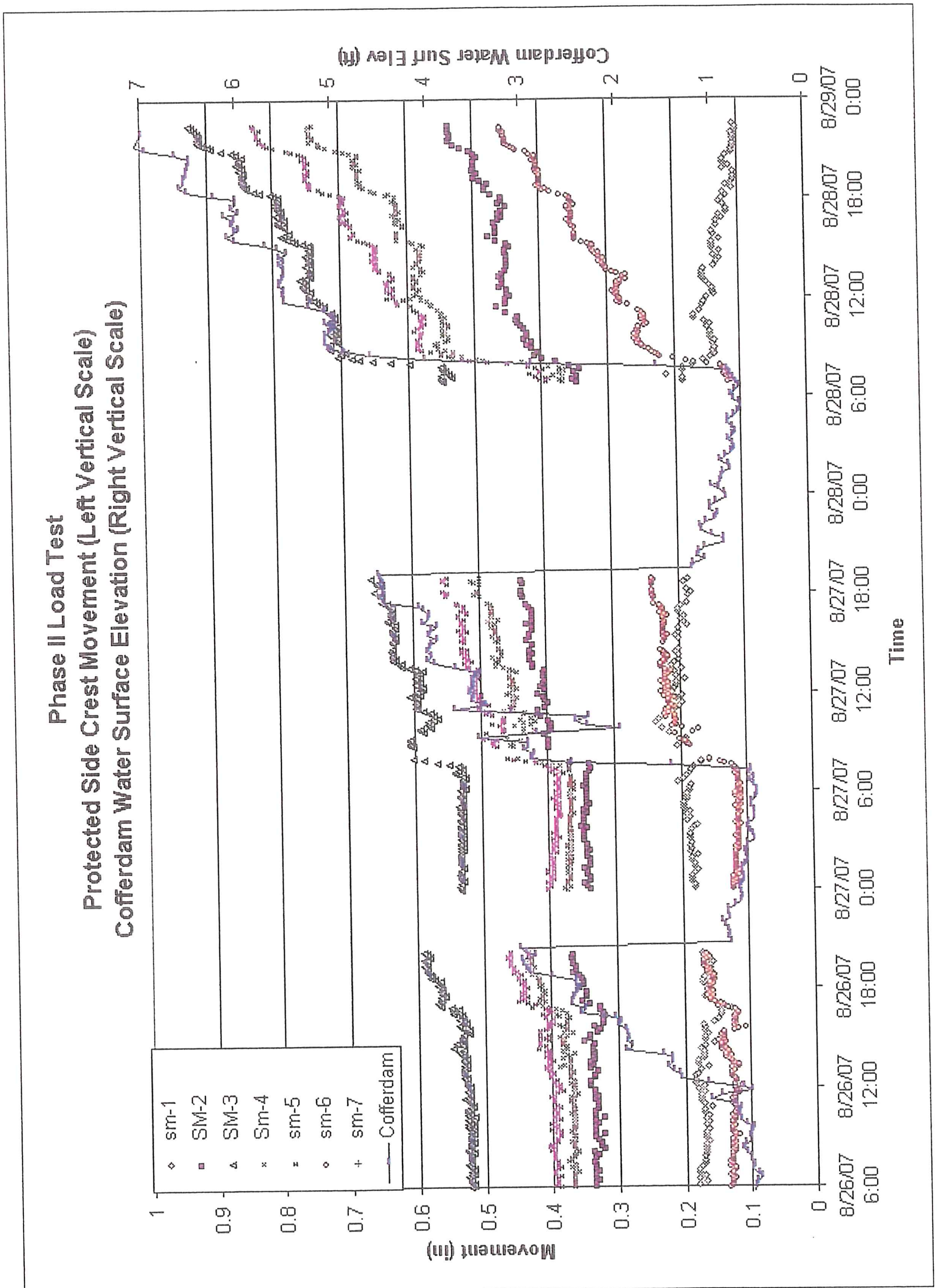


Figure 6.31 – Embankment Crest Survey Monument Movement During Phase II

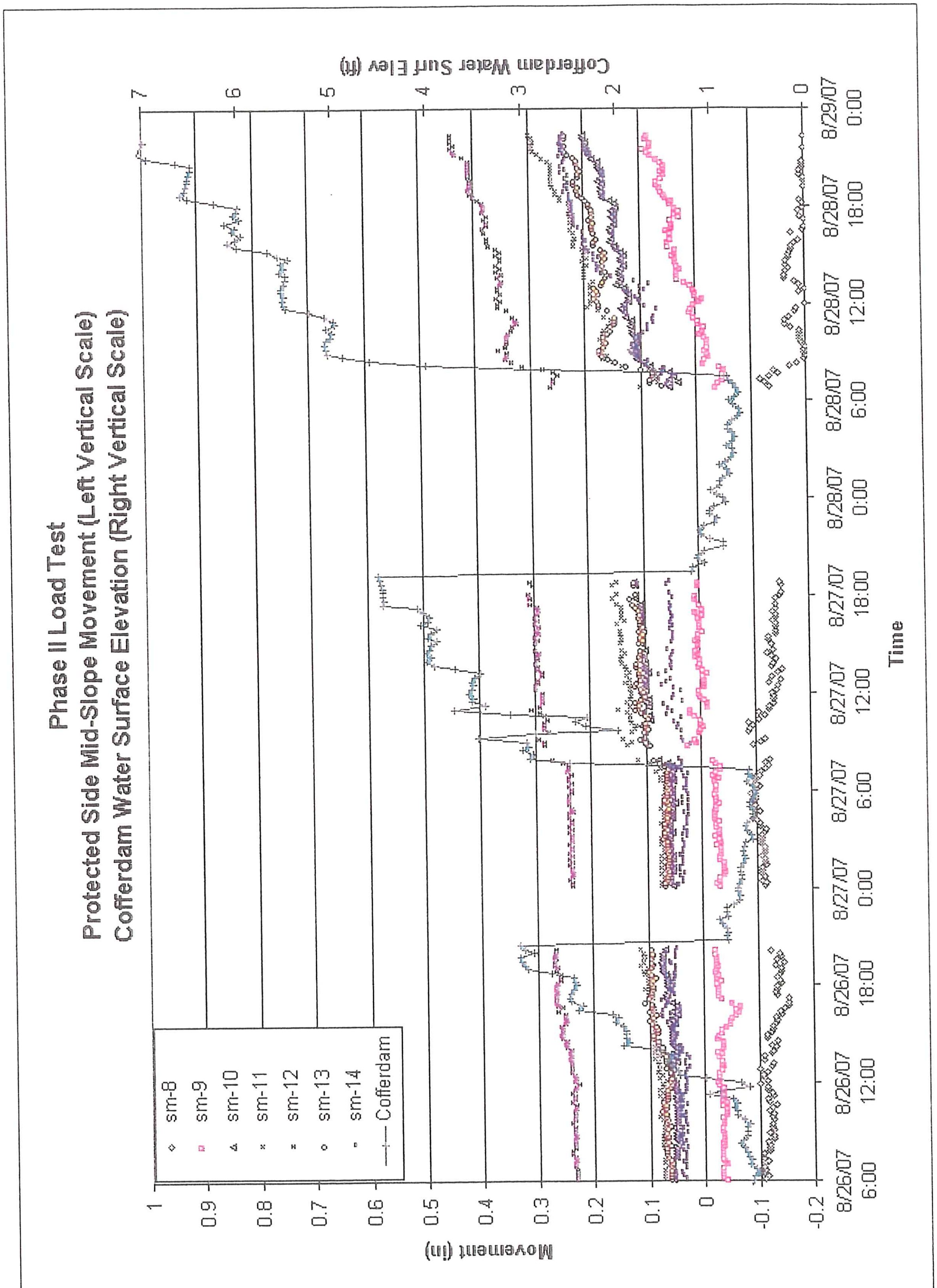


Figure 6.32 – Embankment Mid-Slope Survey Monument Movement During Phase II

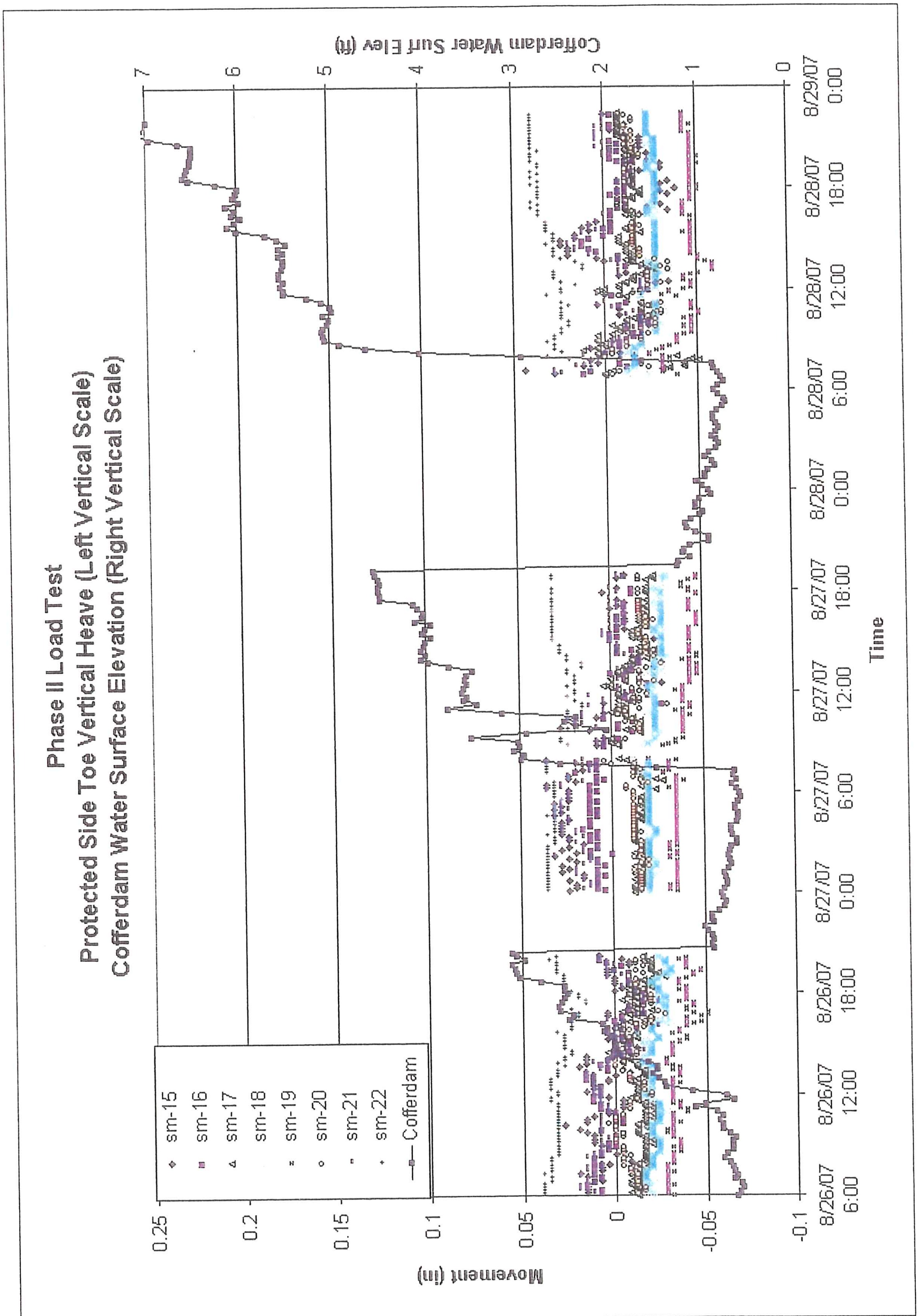


Figure 6.33 – Embankment Toe Vertical Heave During Phase II

CHAPTER 7 – INCLINOMETER MEASUREMENTS

GENERAL. This section describes the instruments used and the detailed measurements obtained from the three in-place inclinometers obtained during the Phase I and Phase II portions of the load test. An "in-place inclinometer" is an inclinometer in which multiple accelerometers are installed inside the inclinometer casing, wired into the ADAS, and are available for multiple readings for an extended period of time. Calculated deflections from the three in-place inclinometers for the Phase I portion of the load test are shown on Figures 7.8 to 7.16. Likewise, the Phase II data is plotted on Figures 7.17 to 7.25.

INSTRUMENT LOCATIONS. In accordance with the URS instrumentation plan, only three in-place inclinometers were installed on the cofferdam centerline: IPI-1 was installed on the canal side of the wall, as close as possible to the I-wall; IPI-2 was installed on the protected side of the wall, as close as possible to the I-wall; and IPI-3 was installed on the mid-slope of the protected side embankment. These locations are shown on Figure 2.12 in Chapter 2. The top elevation of each landside in-place inclinometer was set at the prevailing ground surface elevation. The top elevation of the flood-side inclinometer was set near the top of the existing I-wall. The bottom of the three inclinometers was set approximately 5-feet into the Pleistocene clay below the barrier beach sand to ensure that that the instrument was fixed. URS manually probed each in-place inclinometer casing prior to installation of the automated devices, and prior to the beginning of the load test, to determine the initial position of each casing. Table 7.1 shows the elevations of the top of the casing and the installed elevations of the accelerometers in the three in-place inclinometers.

Table 7.1 – Elevations of Inclinometer Accelerometers (Ft – NAVD88)

	IPI-1 (Canal)	IPI-2 (LS Crest)	IPI-3 (Mid Slope)
Top Riser	+12.23	+2.04	-2.59
Accel 6	-7.7	-7.96	-7.41
Accel 5	-17.7	-17.96	-12.41
Accel 4	-22.8	-22.96	-17.41
Accel 3	-27.7	-27.97	-22.41
Accel 2	-42.8	-42.96	-37.41
Accel 1	-57.7	-57.96	-53.4

The shorter, 5 foot distance between accelerometers was used in those areas where more accuracy was desired, such as above and below the tip of the existing sheetpile and in the protected side embankment. The longer 15 foot distance was used between accelerometers where little or no motion was anticipated, such as the lower portion of the Beach Sand aquifer. An intermediate 10-foot length was used in the upper portion of IPI-

1 and IPI-2 where movements were expected to be uniformly distributed throughout the soil layer. The data will show that, with one exception, these setup lengths accurately describe the movements in the foundations soils. The only shortcoming in the setup is apparent in IPI-2 between elevations -27.97 and -42.96. The 15-foot long inclinometer rod is too coarse to accurately assess the location of the horizontal movement in the Beach Sands below the sheet pile tip.

INSTRUMENT READING SCHEDULE and DATA QUALITY. The ADAS polled each in-place inclinometer every 15-minutes and URS provided these data to the Technical Review Team in comma-separated-variable format. The team reduced and plotted the data, reviewed it for quality, and then critically examined the reduced data to determine the foundation response. When the technical review team noticed that IPI-1 was yielding unexpected results, the team began to plot the inclination values versus time from each of the six accelerometers in each of the in-place inclinometers. Figure 7.1, 7.3, and 7.4 plot the inclination values versus time for IPI-1, -2, and -3. A data review yields the following conclusions.

IPI-1 Quality. The variability of the data from this instrument indicates that it is fraught with "noise", readings that vary dramatically and with randomly spiking throughout the record. A detailed examination of the raw data for IPI-1 (Figure 7.1) shows that four of the six accelerometers were subject to this noise. URS checked the ADAS to ensure that the 'noise' was not being generated from within the system. Their check of the physical wiring, electronics, temperature thermistors, and database revealed nothing amiss.

The in-place inclinometers consist of steel rods of varying length that are linked together end to end and hung from the top of the inclinometer casing. Any disturbance (shaking or vibration) of any part of the in-place inclinometer will be translated to other parts of the instrument. Inclinometer IPI-1 was located on the canal side of the I-wall in the cofferdam pool and would be subject to surface disturbances and vibrations created when the contractor was filling the cofferdam with water, running diesel motors, or moving equipment. In anticipation of this, the AE placed a larger diameter PVC pipe (white pipe in Figure 7.2) over the inclinometer casing (blue pipe) above the canal-side grade to shield it from surface disturbances. The data indicates that this external casing was ineffective in preventing construction related vibrations from affecting the canal side in-place inclinometer IPI-1.

The Corps considered if the noise could be caused by movements of the canal side soil mass related to the crack shown in Figure 6.13. But comparing the crack location to the IPI-1 location indicates that the instrument is located beyond the soil mass involved with the crack.

The AE, the Corps, and the Technical Review Team believe that it is the location of IPI-1 in the cofferdam pool and its exposure to the aforementioned surface disturbances that has caused the noise in the IPI-1 record, and not any physical or electronic failure of the ADAS. In-place inclinometers IPI-2 and IPI-3, both landside of the wall, are built of the same components and show little if any of the noise that shows so clearly in the IPI-1 record.

These unanticipated disturbances negatively affect the quality of this record and render suspect any computations subsequently made from it.

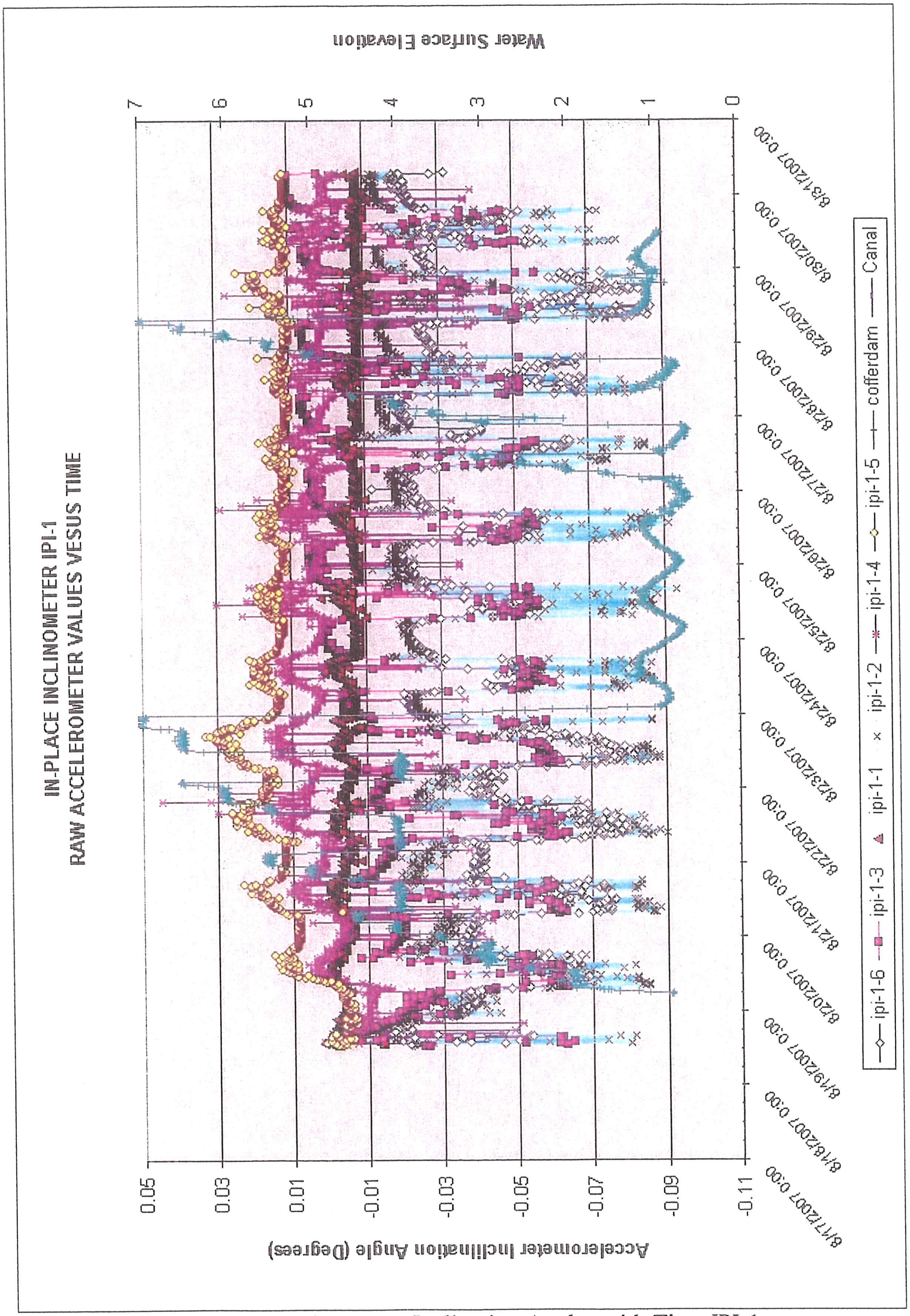


Fig 7.1 – Accelerometer Inclination Angles with Time IPI-1

Fig 7.2 – IPI-1 Installation
Note: The timber supports were removed prior to beginning the test.
(Note: The adjacent small white PVC casing is piezometer PZ-14.)

IPI-2 and IPI-3 Quality. These two inclinometers, installed on the protected side of the I-wall in a flush-mount system (Figure 7.5) exhibit no sustained "noise". Although the data in Figures 7.3 and 7.4 shows a few, intermittent spikes, there are no sustained periods of disturbance in the record. These records are clear enough to show a daily, sinusoidal shaped, pattern most notable in the near surface accelerometers, -5, and -6 of IPI-2 and IPI-3. This fluctuation is clearest during the period between Phase I and Phase II of the load test (24 through 27 August) and indicates regular movements of (+) and (-) 0.04 to 0.05 inches. The frequency of the fluctuation is very similar to that of the daily canal water surface.

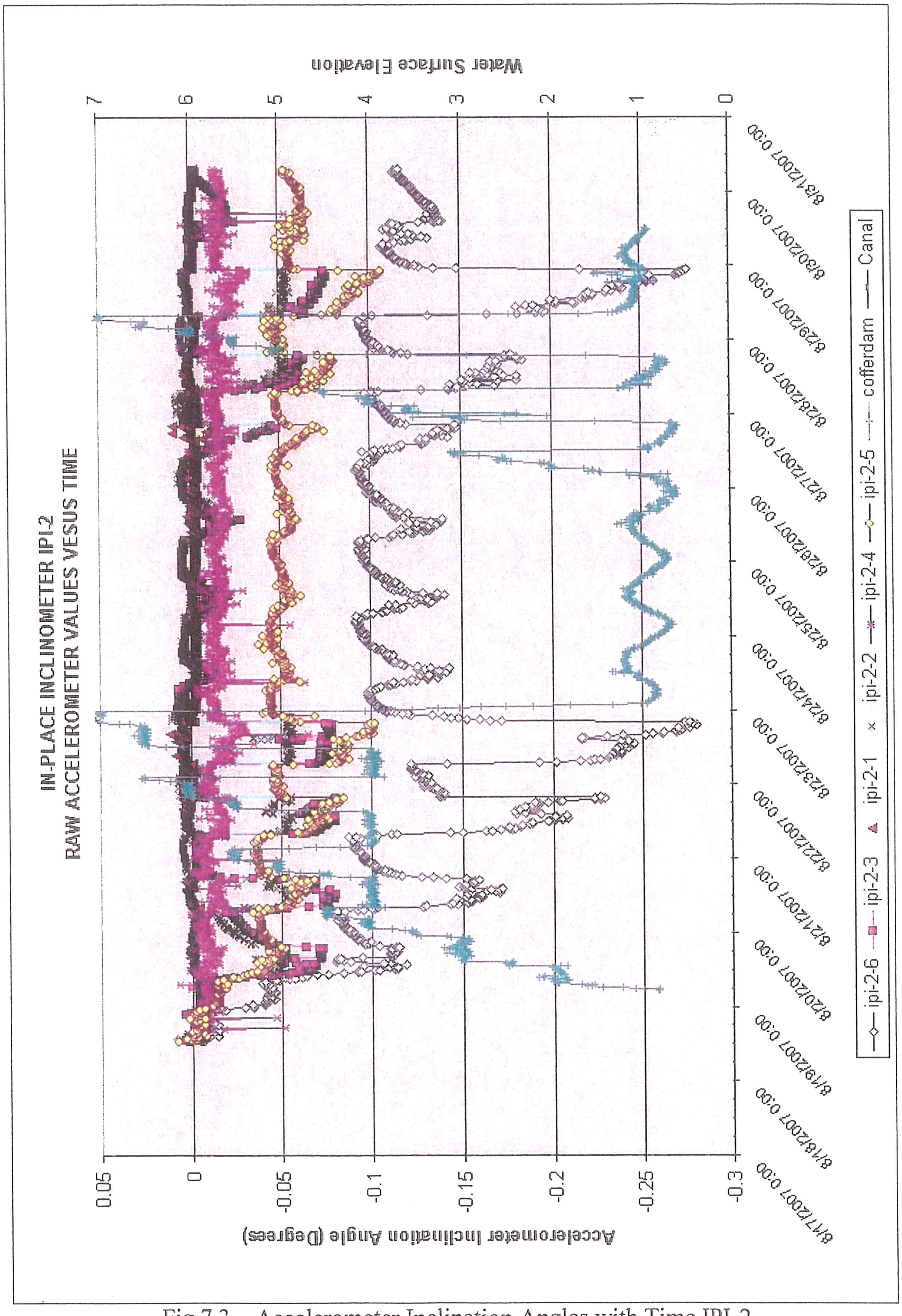


Fig 7.3 – Accelerometer Inclination Angles with Time IPI-2

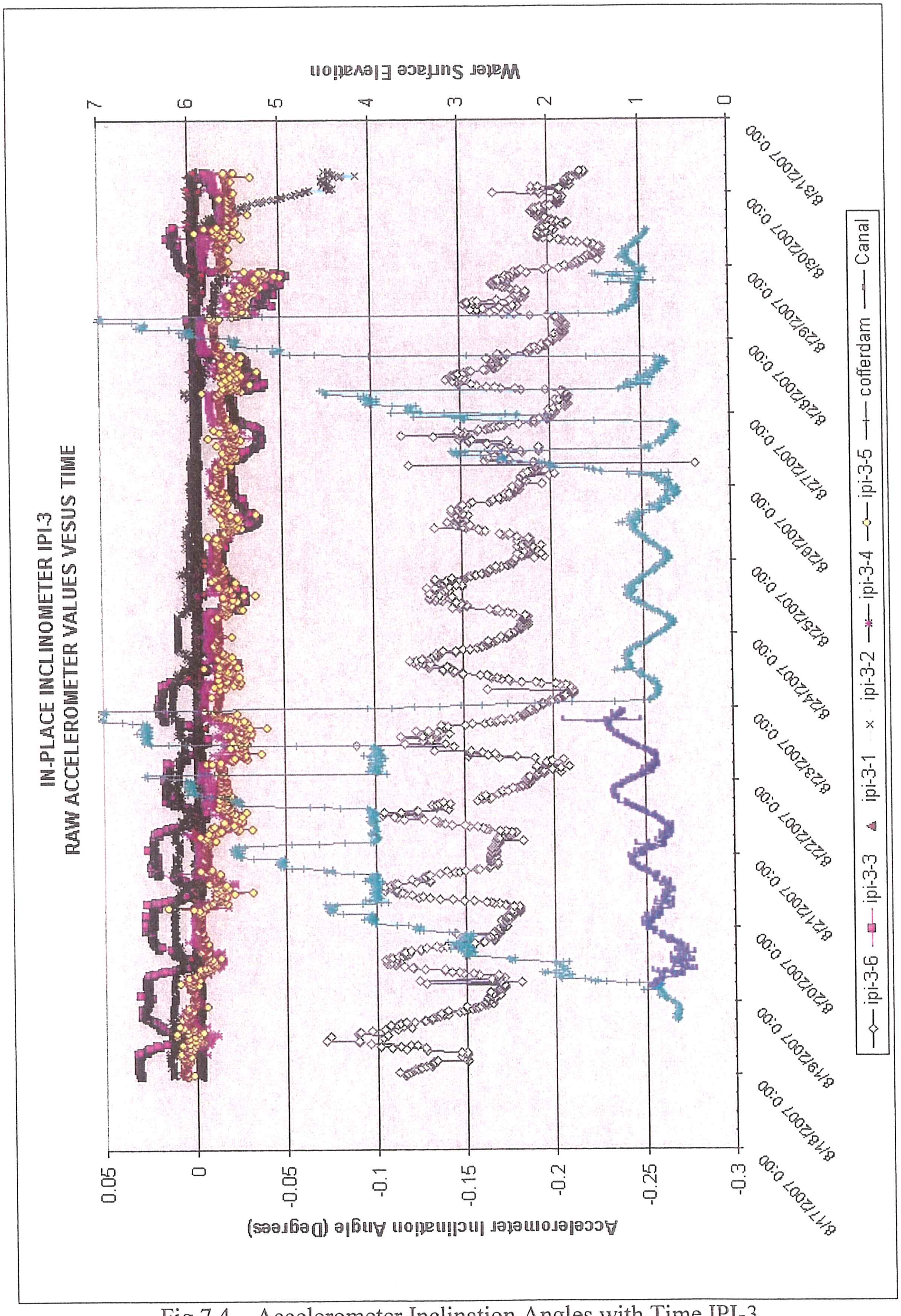


Fig 7.4 – Accelerometer Inclination Angles with Time IPI-3

Figure 7.5 - Flush Mount Installation Typical of IPI-2 and IPI-3

Calculated Deflections from In-Place Inclinometers. The calculated IPI-1 Phase I deflections are shown on Figures 7.8, 7.9, and 7.10 (at the end of this chapter) and the Phase II deflections are shown on Figures 7.17, 7.18, and 7.19. The deflections for IPI-2 are shown on Figures 7.11 through 7.13 (Phase I) and Figures 7.20 through 7.22 (Phase II). Those for IPI-3 are shown on Figures 7.14 through 7.16 (Phase I) and Figures 7.23 through 7.25 (Phase II). Each page shows one day's duration for each instrument. For example, Figure 7.11 shows the hourly inclinometer deflections for IPI-2 from 0700 to Noontime on August 20, 2007. Figure 7.11a (same page) shows the readings for the same instrument from Noontime to 1700 hours (5:00pm) on the same day. All of the Phase I data plots are shown for each instrument and are immediately followed by the Phase II data.

Summary Plots of Phase I and Phase II Inclinometer Deflection.

The data plots for in-place inclinometers IPI-2 and IPI-3 have been summarized on Figures 7.6 and 7.7. Due to poor quality data described earlier, IPI-1 is not summarized. The summary plots present the calculated inclinometer deflection obtained at specific times and dates that correspond to a given cofferdam water surface elevation. The times and dates of the cofferdam water surface elevations were obtained from the data presented in Figures 4.1 and 4.2 and are summarized in Table 7.2.

Table 7.2 – Times and Dates of Cofferdam Water Surface Elevations

CWE		Phase I			Phase II		
		Time	Date		Time	Date	
2.0		21:30	8/18/2007		16:00	8/26/2007	
2.5		23:30	8/18/2007		18:30	8/26/2007	
3.0		8:30	8/19/2007		20:30	8/26/2007	
3.5		11:15	8/19/2007		13:15	8/27/2007	
4.0		15:45	8/19/2007		16:45	8/27/2007	
4.5		19:30	8/19/2007		19:15	8/27/2007	
5.0		12:30	8/20/2007		11:15	8/28/2007	
5.5		17:15	8/20/2007		15:00	8/28/2007	
6.0		17:45	8/21/2007		18:00	8/28/2007	
6.5		20:00	8/21/2007		20:30	8/28/2007	
7.0		17:30	8/22/2007		22:15	8/28/2007	

DISCUSSION OF SUMMARY INCLINOMETER DEFLECTION.

IPI-1 Deflections. As previously noted the "noise" negatively affects the quality of the IPI-1 data. The calculated deflections are implausible, showing no sustainable trends and unexpected "reversing" movements. This section contains neither summary plots nor discussion of IPI-1 deflections.

Total Deflection.

Phase I Deflections. The calculated deflections look reasonable, with the ground surface deflection increasing as the cofferdam water surface elevation increases. The deflections of IPI-2 begin with little or no deflection at cofferdam water surface equal to +2.0 and increase as the cofferdam water surface elevation is raised. The deflections of IPI-3 are limited to about 0.13-inch.

Phase II Deflections. The Phase II plots show that some of the horizontal deflections caused during the Phase I loading were still locked-in at the beginning of the Phase II loading. The calculated deflection of IPI-2 begins with 0.48-inches of deflection with only a CWE of only 2.0-feet. The Phase II IPI-3 deflection starts at 0.20-inches at his same low cofferdam level. IPI-3 showed increased deflections during Phase II, with maximum deflection of about 0.38-inches at CWE of +7.0-ft.

Depth of Deflection.

Phase I Depth. IPI-2 indicates that the entire soil mass from the ground surface to the base of the beach sand contributes to the total deflection. At the maximum cofferdam water surface elevation of +7.0-ft NAVD88, the soil mass above the sheetpile tip (elevation -22 NAVD88) deflects 0.75-inches, while about 0.20-inches of additional deflection is contributed by the soil mass below the sheetpile. Per the previous discussion due to the excessive inclinometer rod length at this depth (see the previous discussion), the horizontal deflection may end somewhere higher than the bottom of the Beach Sand at elevation -42 NAVD88.

IPI-3 Phase I deflections extend down to elevation -22, the sheetpile tip elevation. No deflections are indicated deeper than this.

Phase II Depth. IPI-2 depth of deflection is the same, but the Phase II IPI-3 deflections indicate a greater depth of soil mobilized during the Phase II loading. The IPI-3 curves indicate that deflections extend down to elevation -37.4 NAVD88.

Comparison with Previous FLAC Analyses. The previously completed soil-structure-interaction analyses (Located in Appendix D) indicated that a water surface elevation of +7.0-ft in the cofferdam would create 0.20 to 0.50-inches of horizontal deflection near the sheet pile tip at elevaton -22 NAVD88. The range of deflection in the FLAC study is due to assumptions made on shear modulus, either inferred from triaxial strength testing or measured from in-situ pressuremeter tests. The calculated deflections in IPI-2 at this elevation are 0.20 to 0.25-inches and are within the range of the numerical estimates.

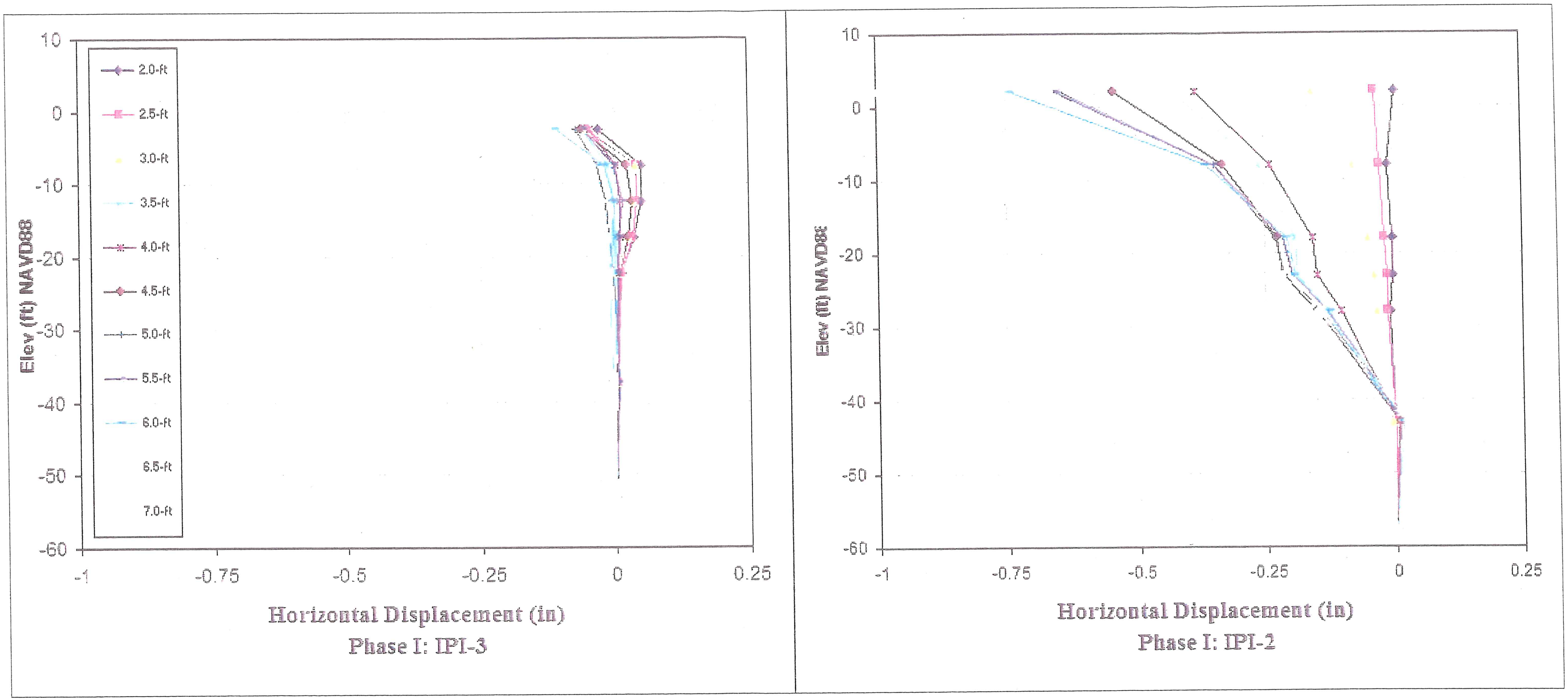


Figure 7.6 – Summary Phase I Inclimometer Plots IPI-2 and IPI-3

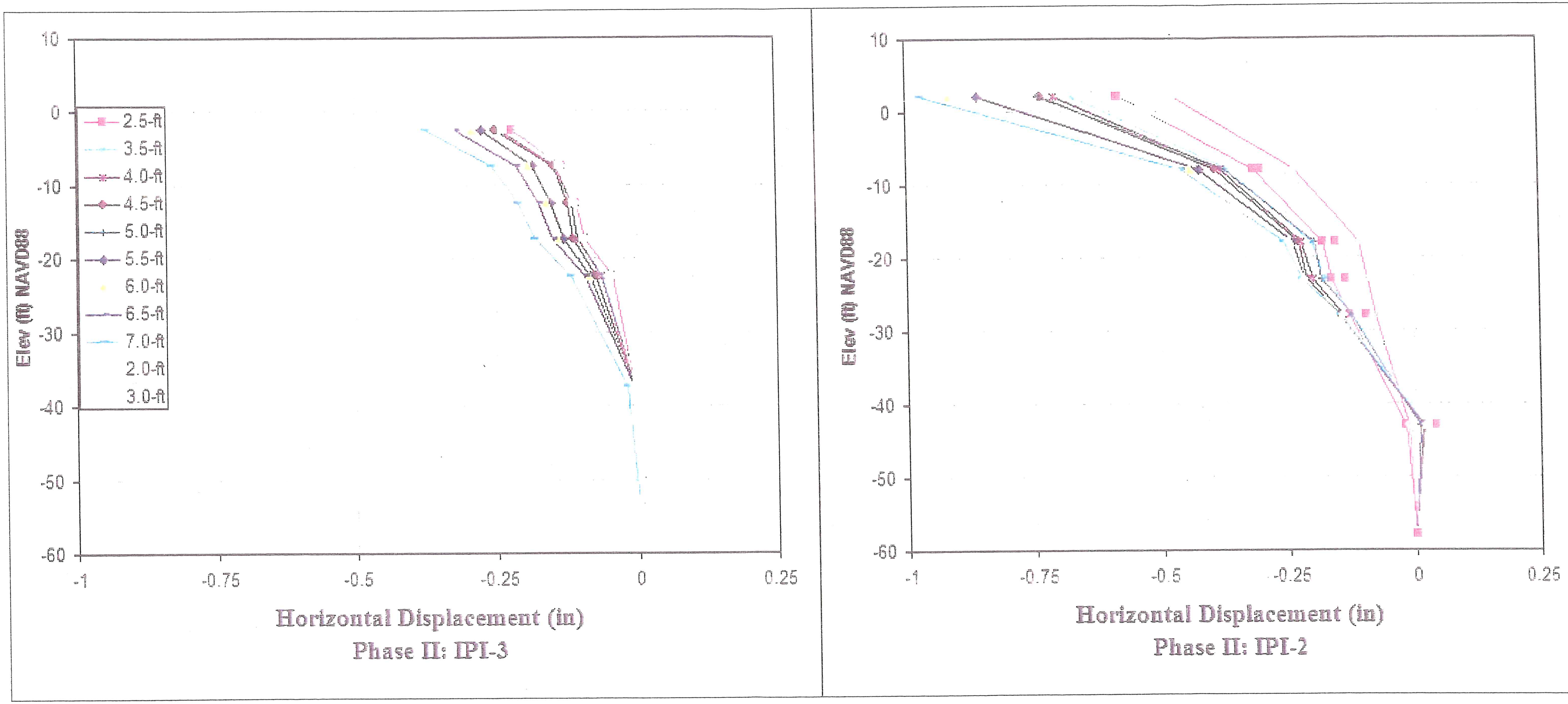
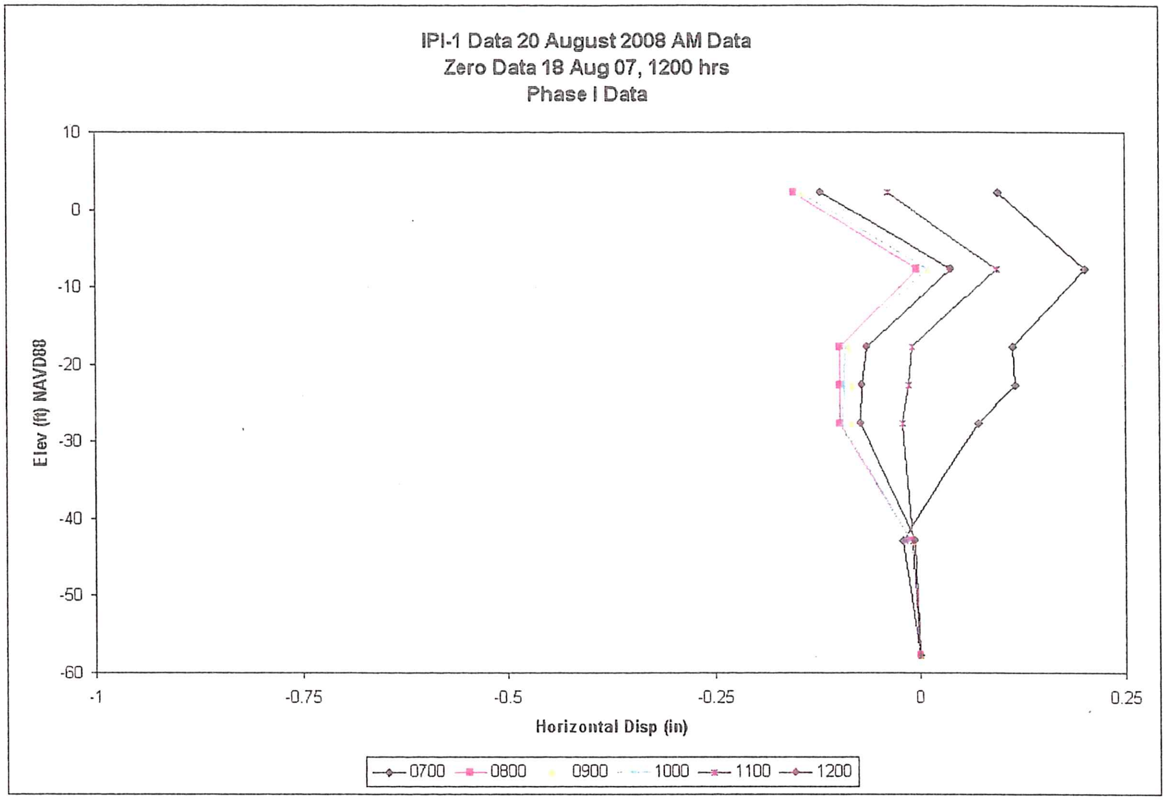
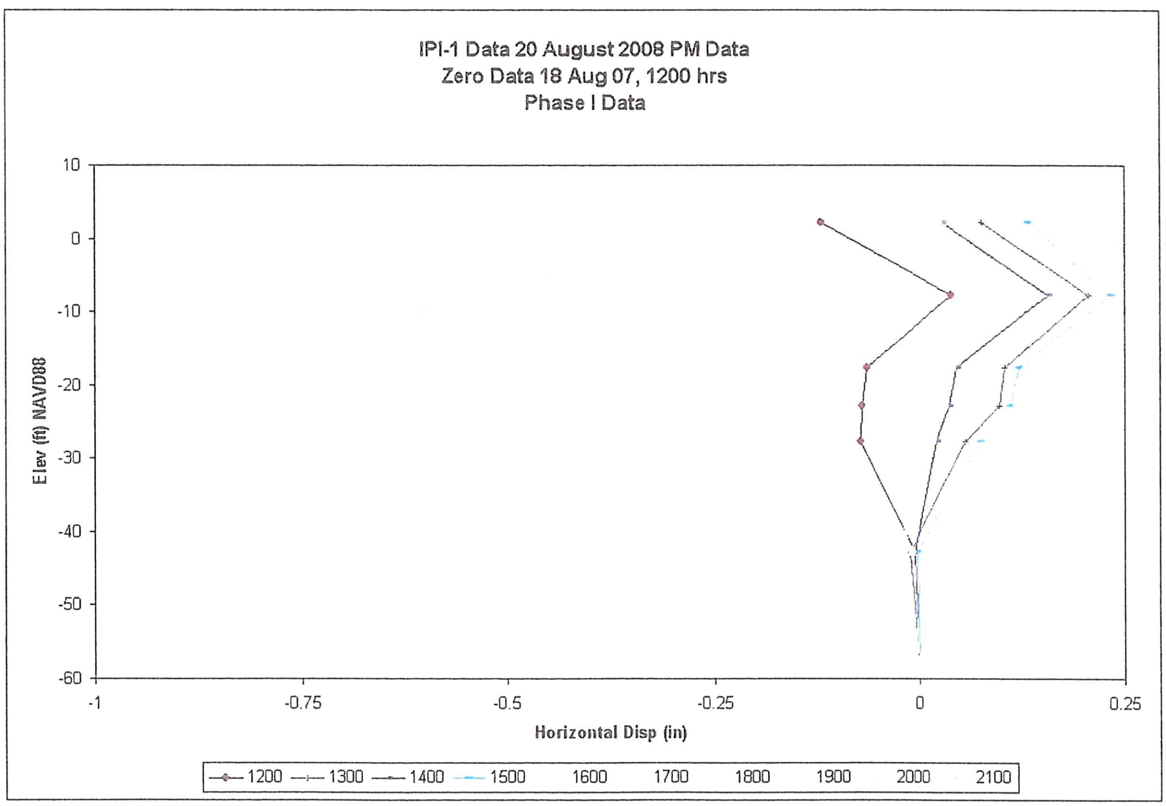
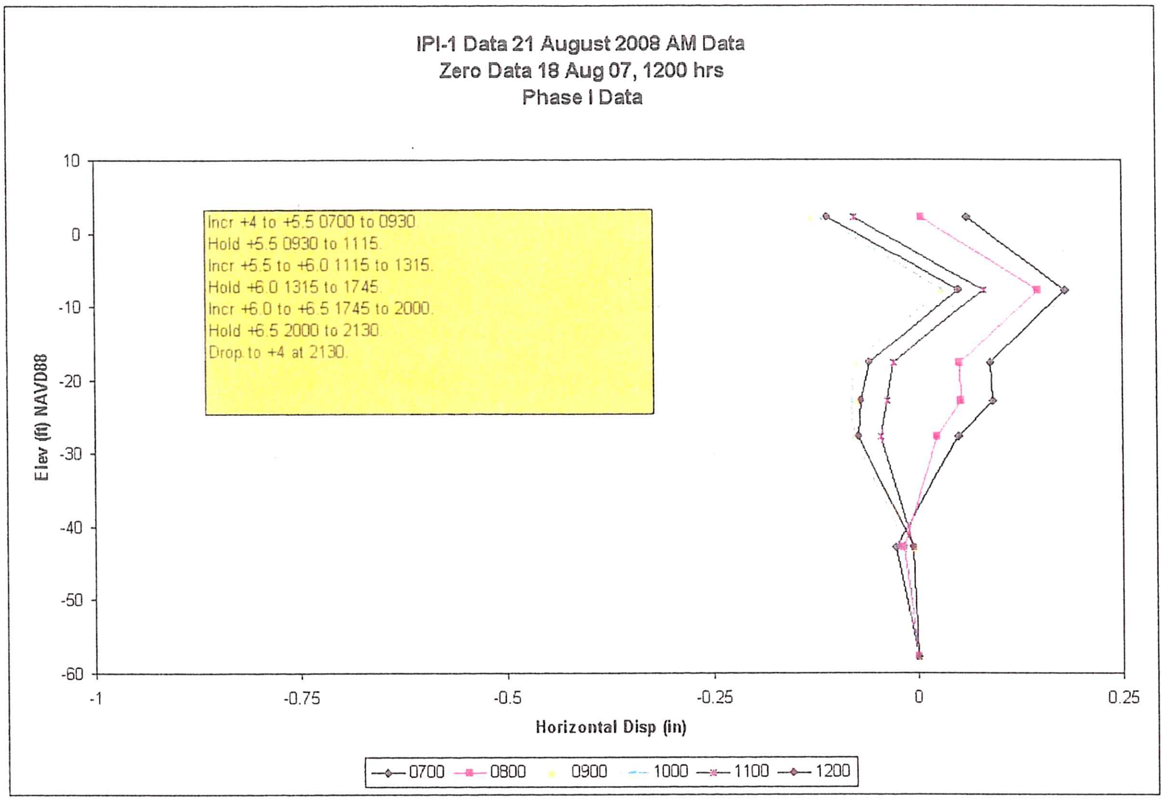


Figure 7.7 – Summary Phase II Inclimometer Plots IPI-2 and IPI-3





Figs 7.8 and 7.8a.



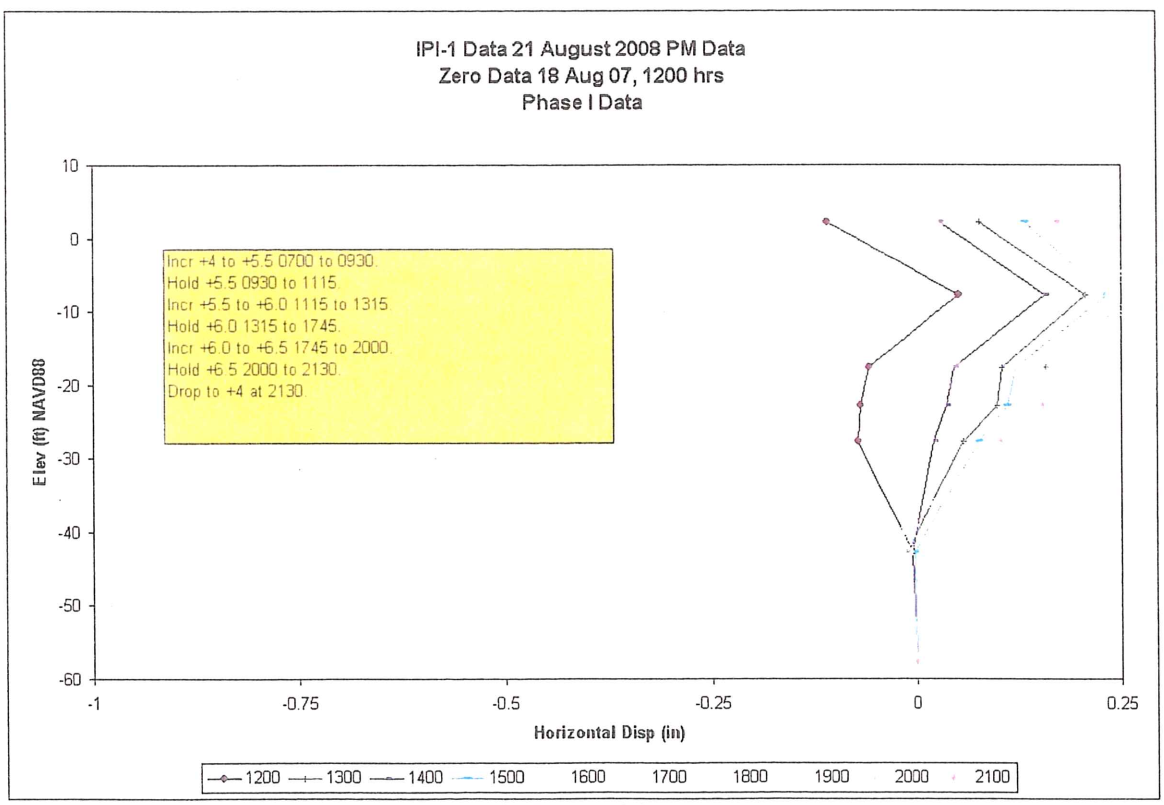
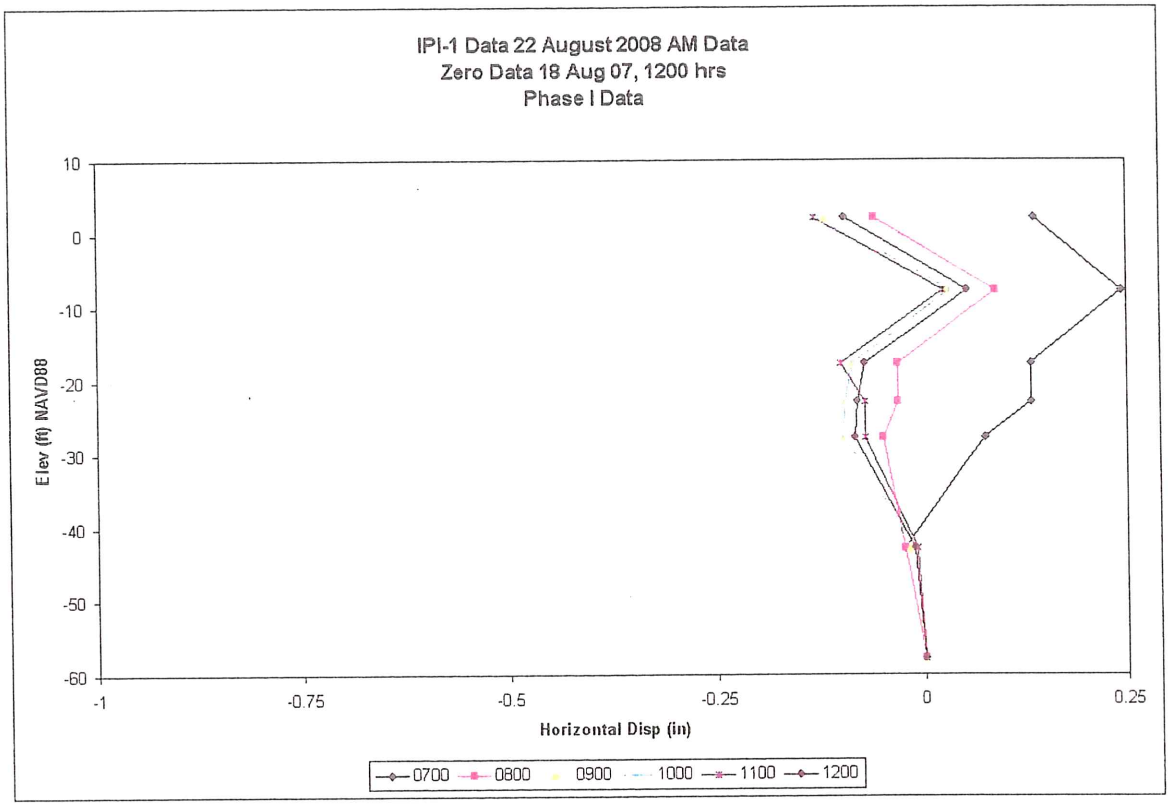


Fig 7.9 and 7.9a



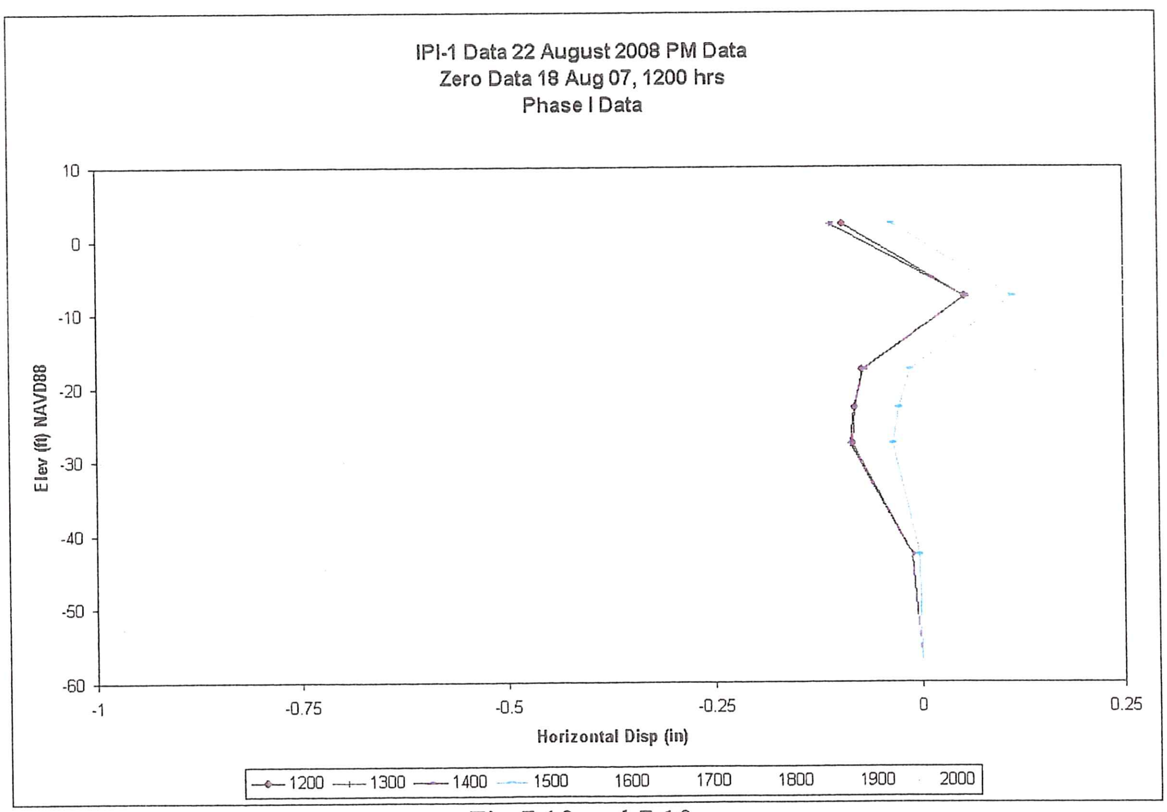
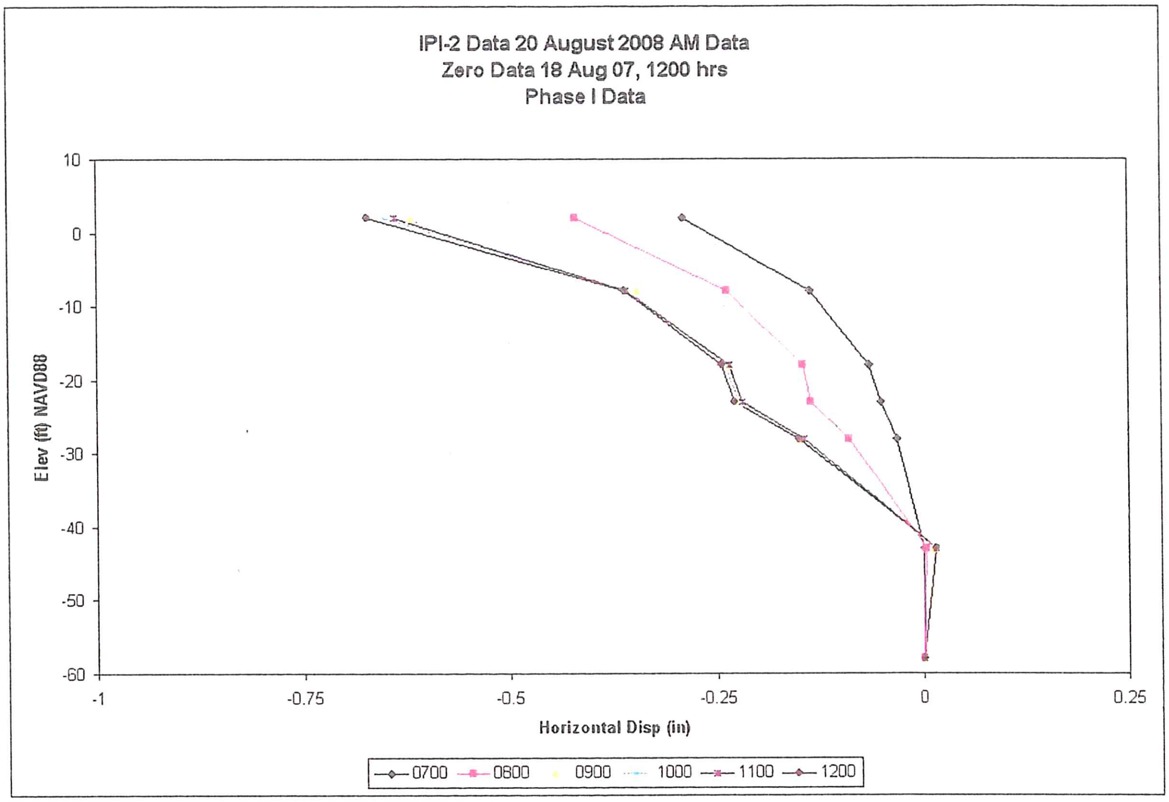


Fig 7.10 and 7.10a



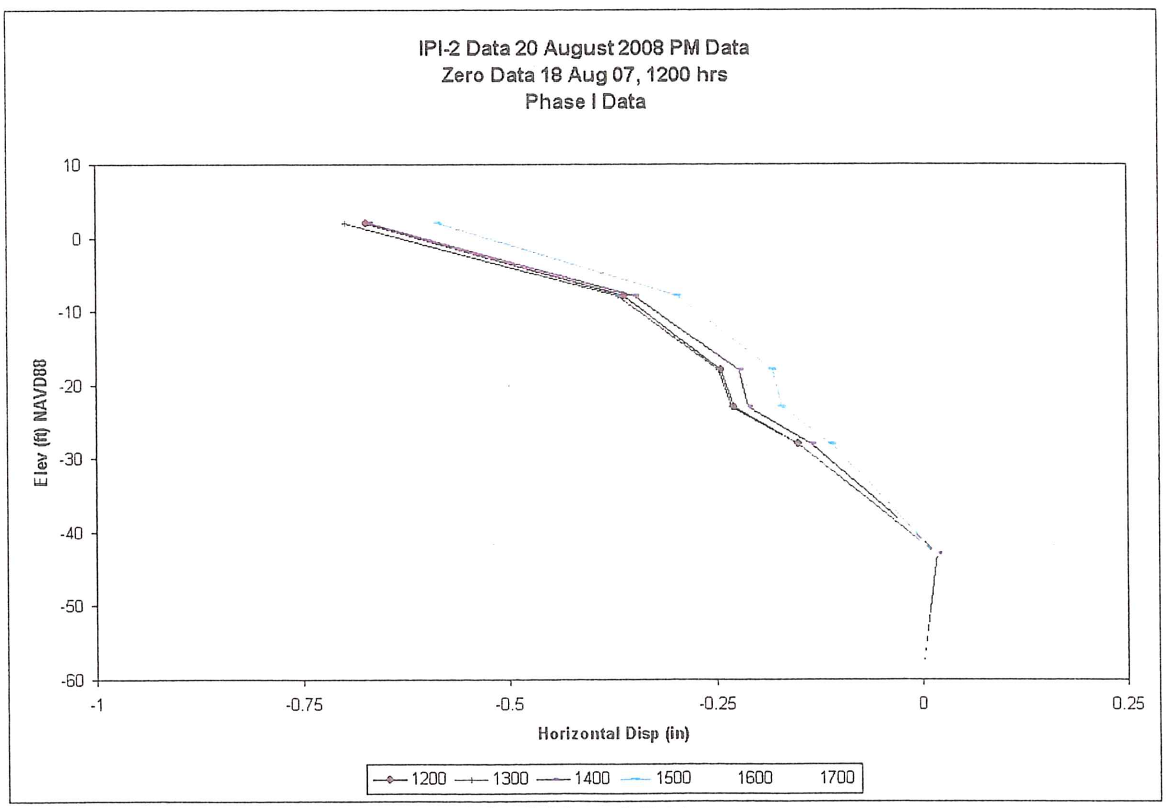
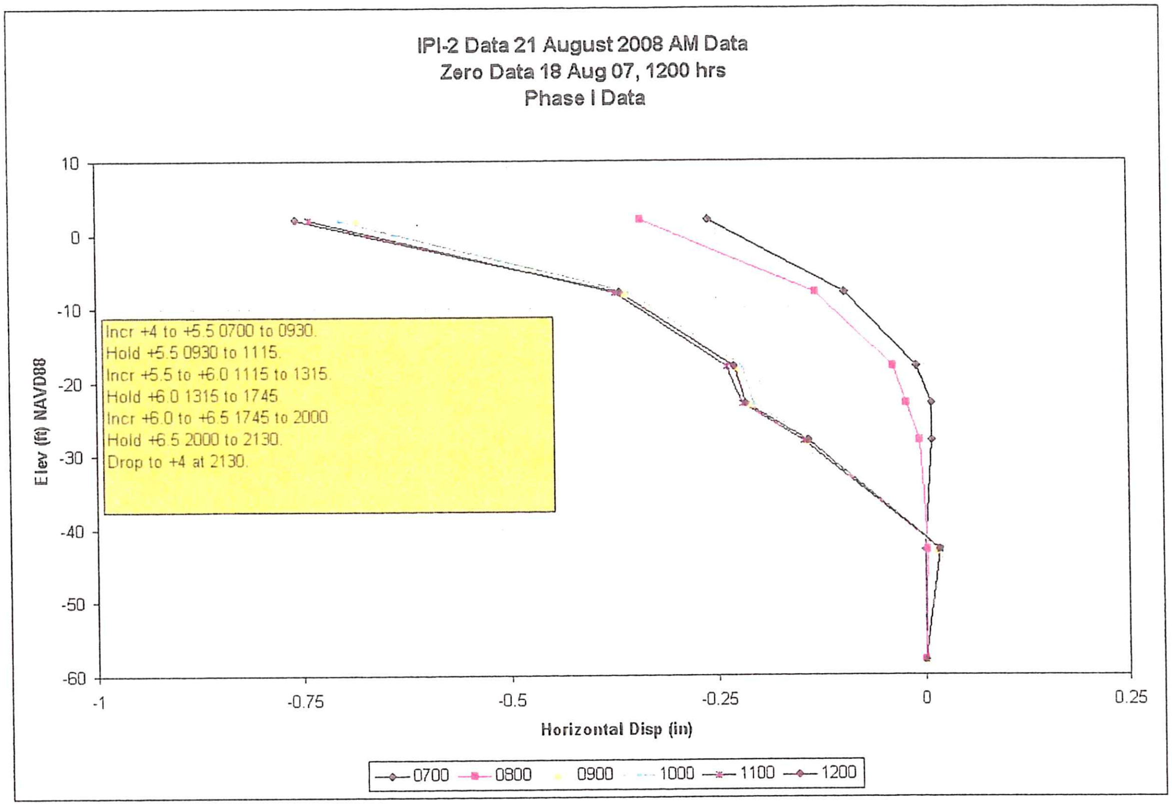


Fig 7.11 and 7.11a



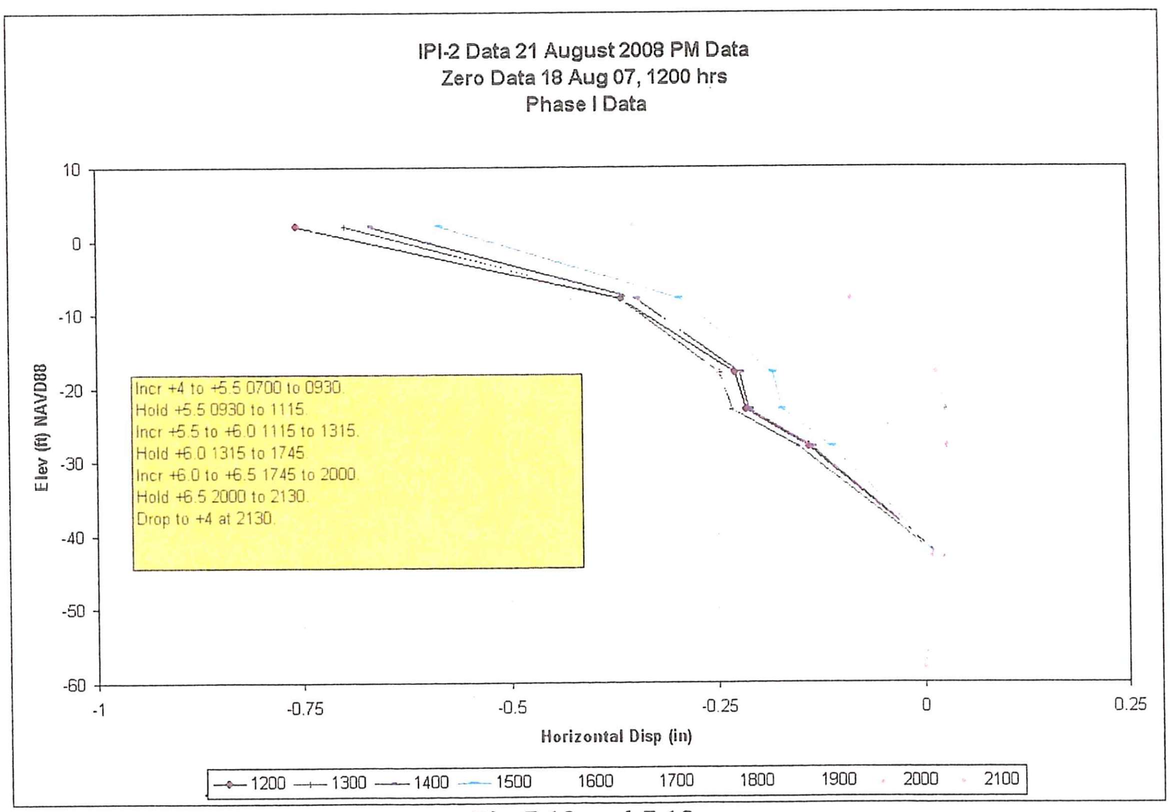
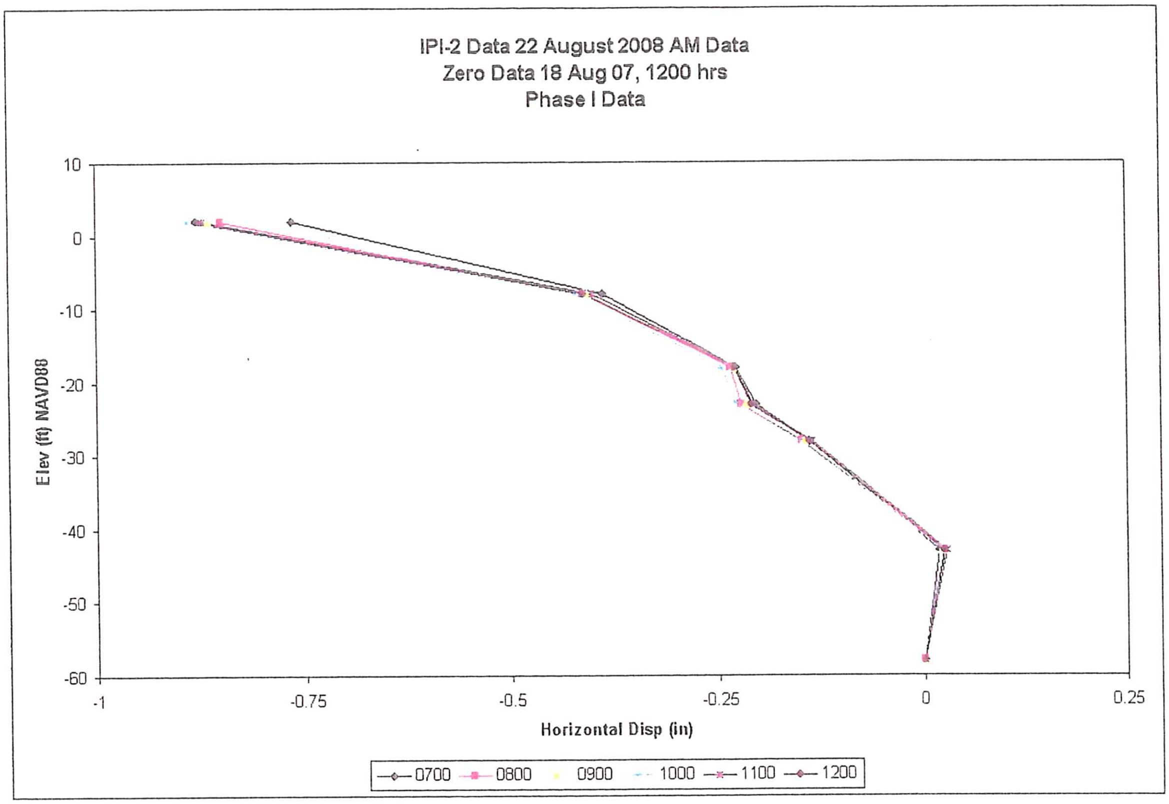


Fig 7.12 and 7.12a



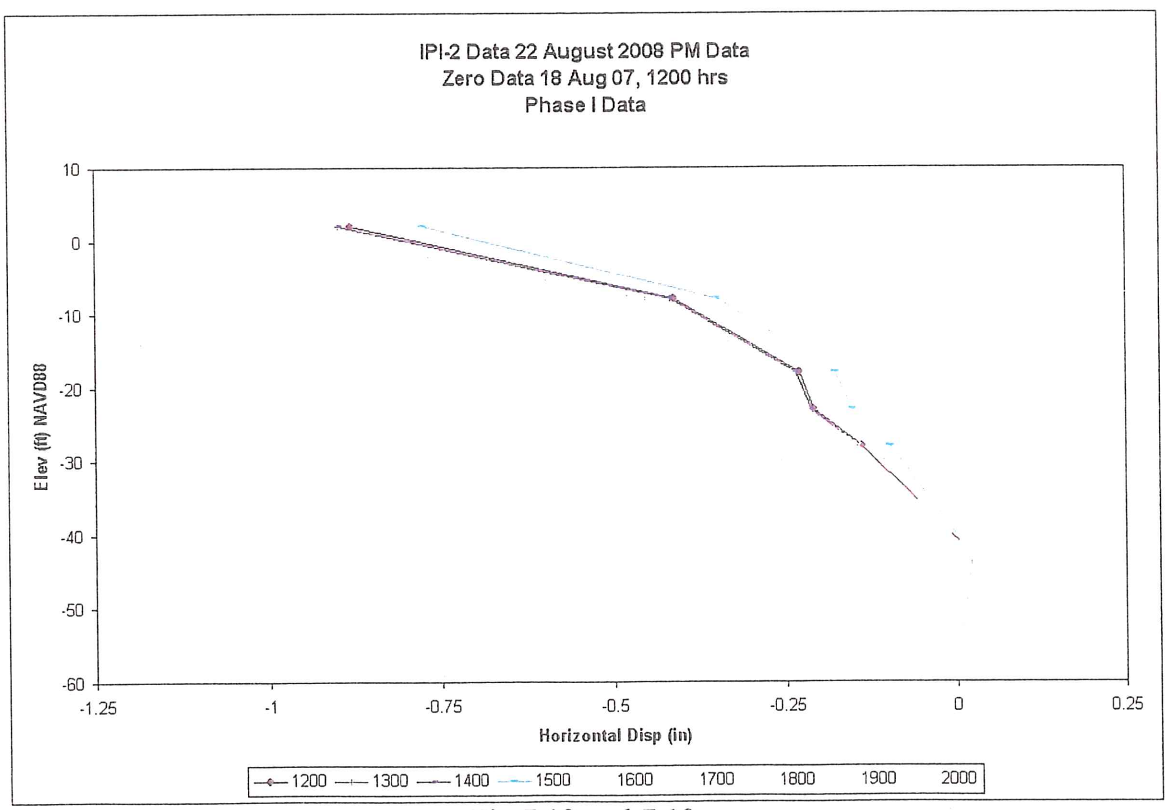
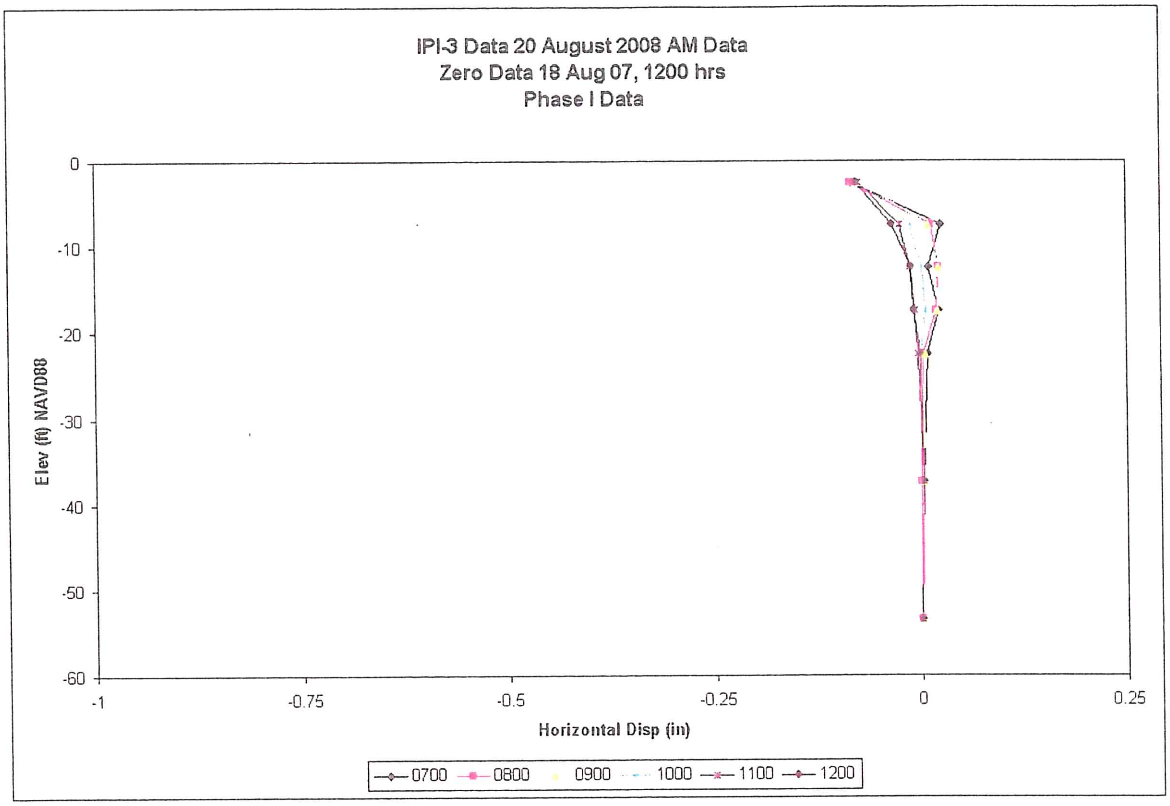


Fig 7.13 and 7.13a



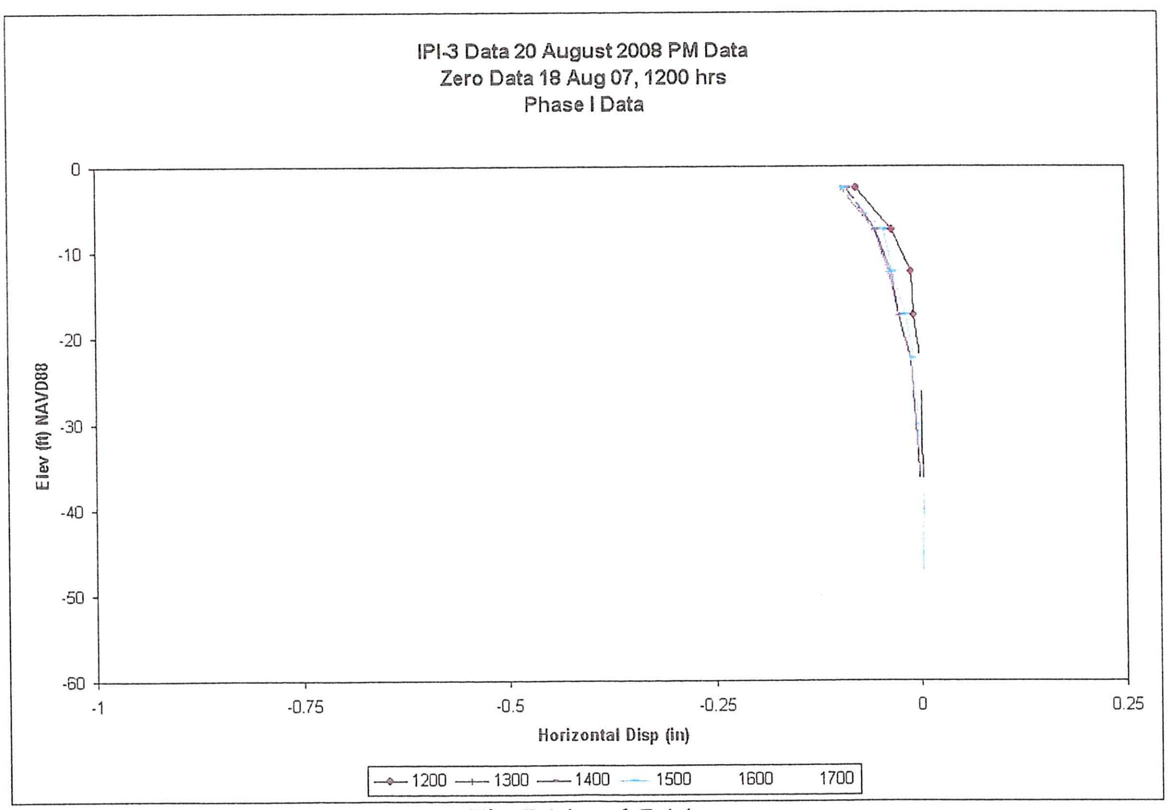
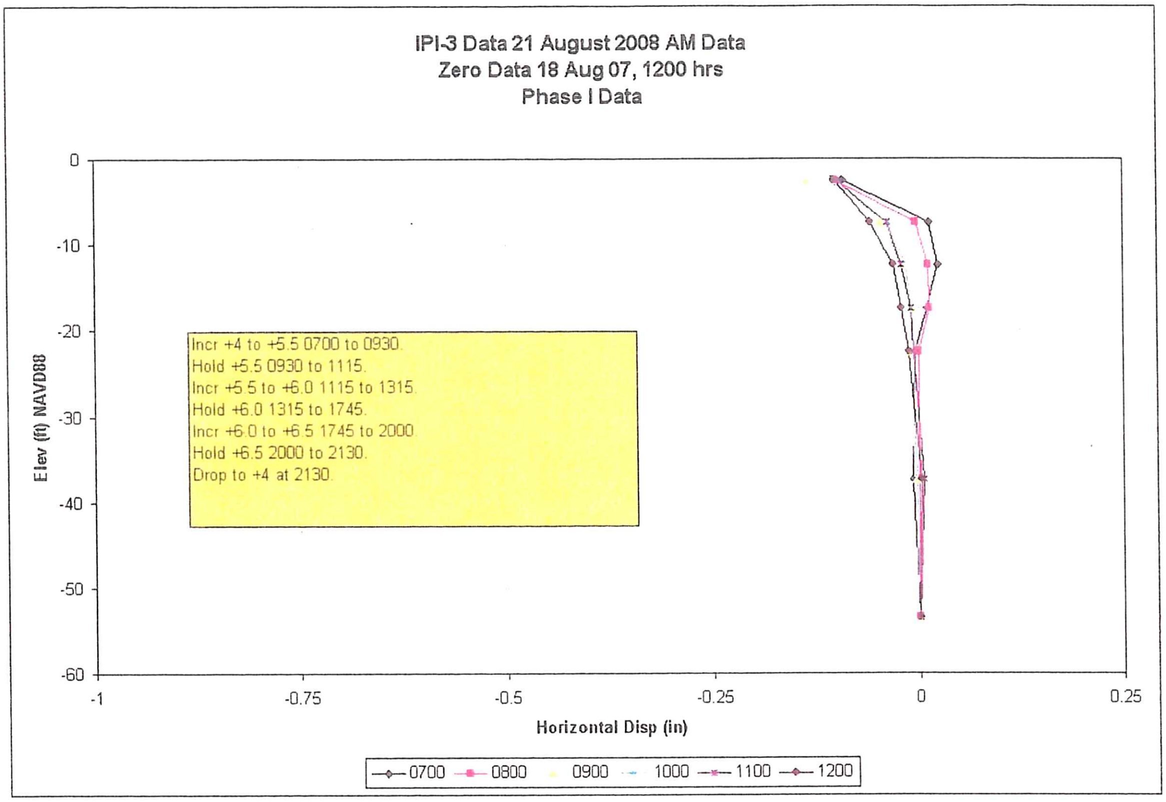


Fig 7.14 and 7.14a



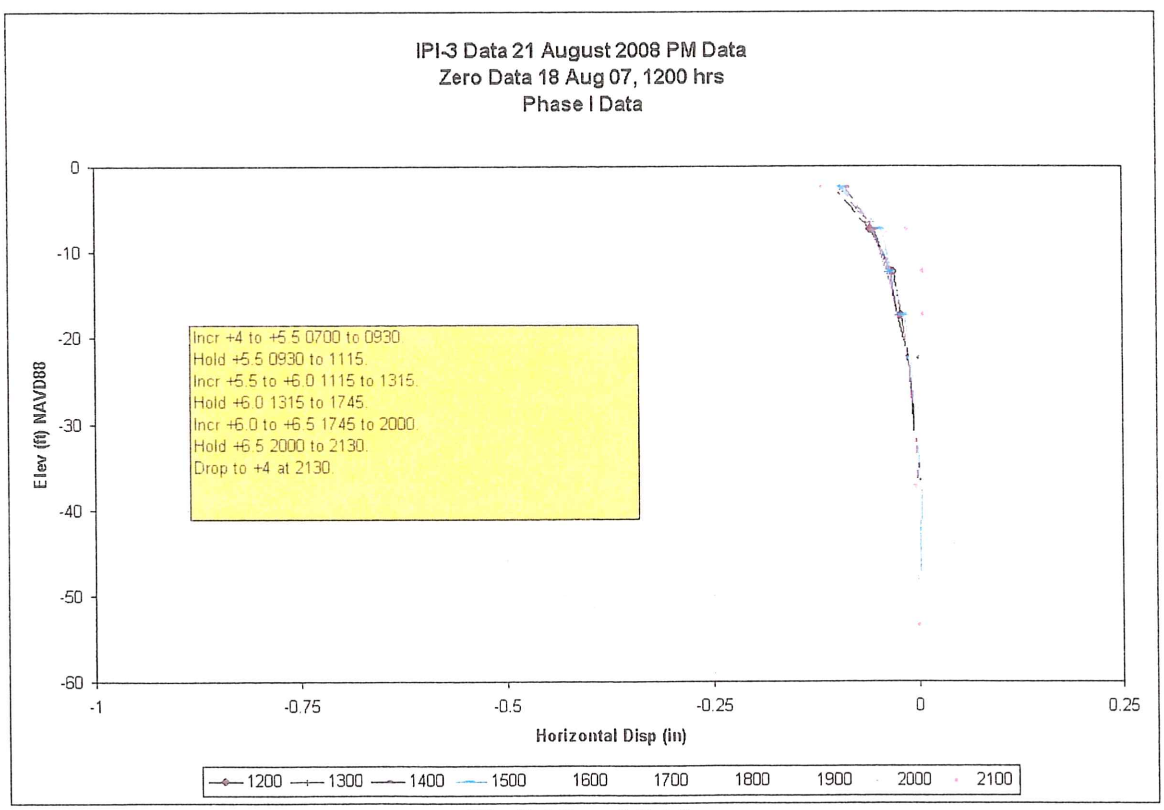
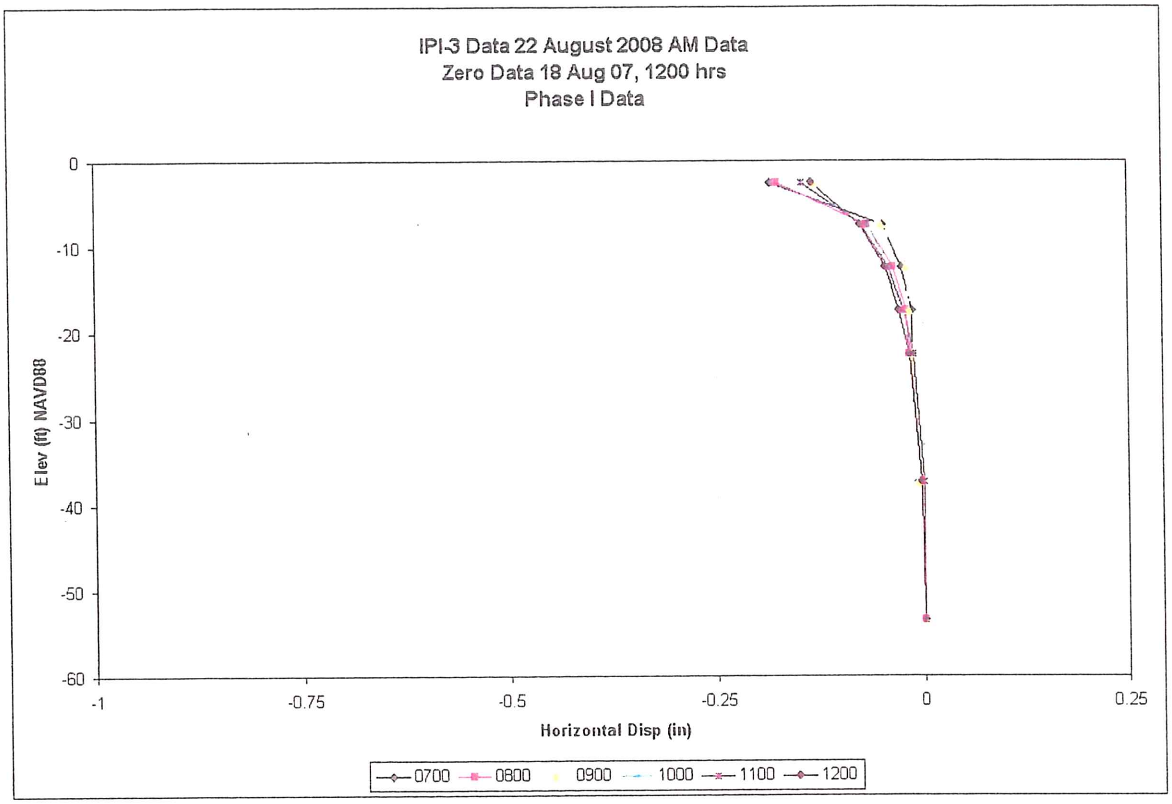


Fig 7.15 and 7.15a



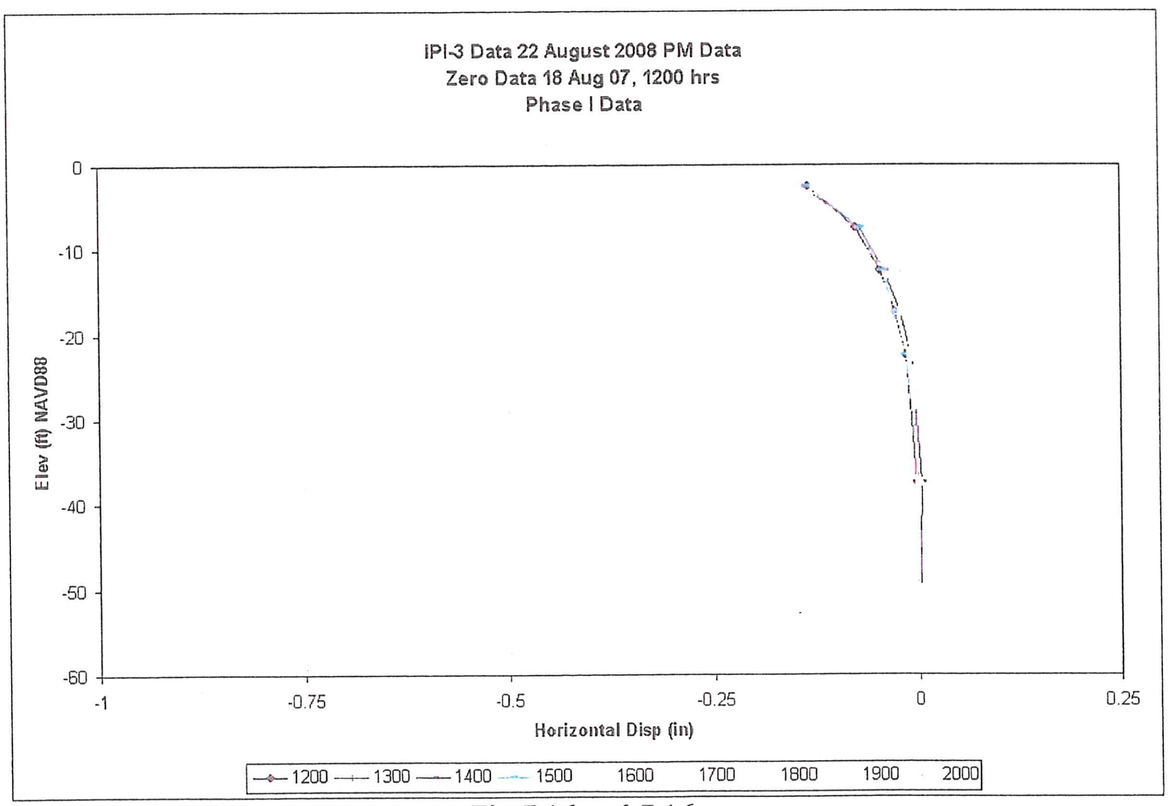
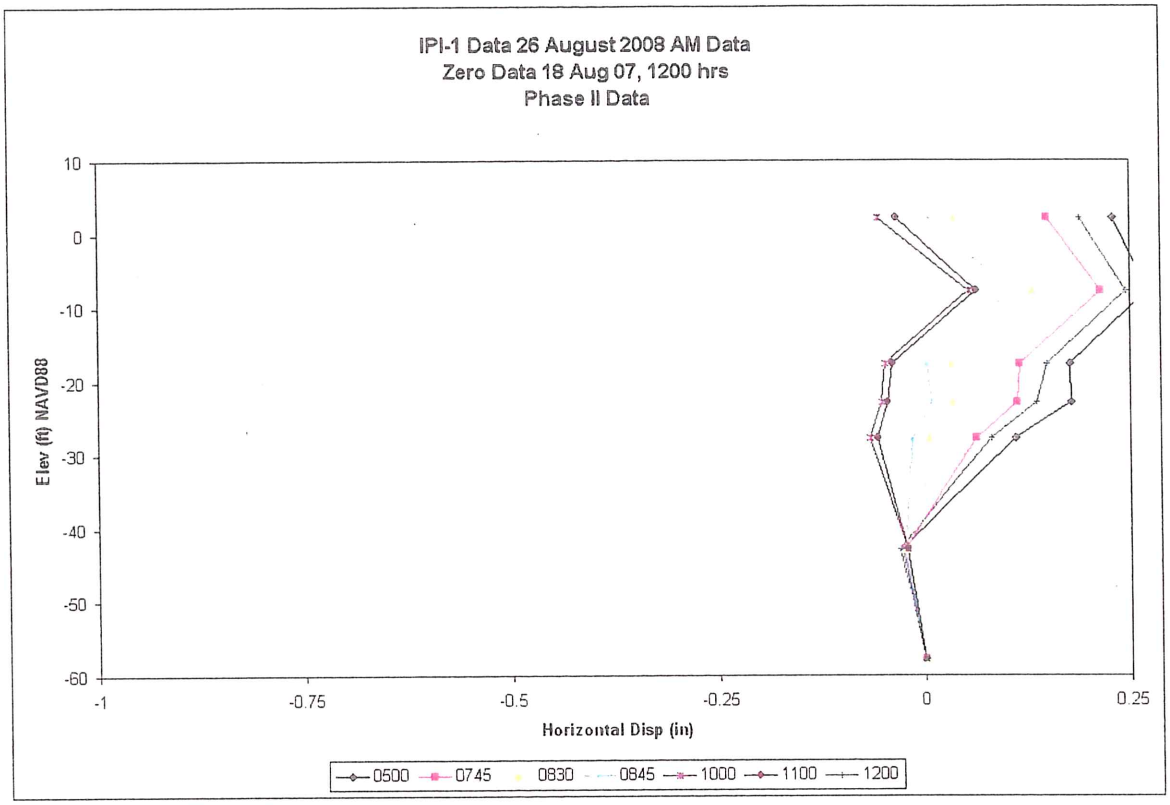
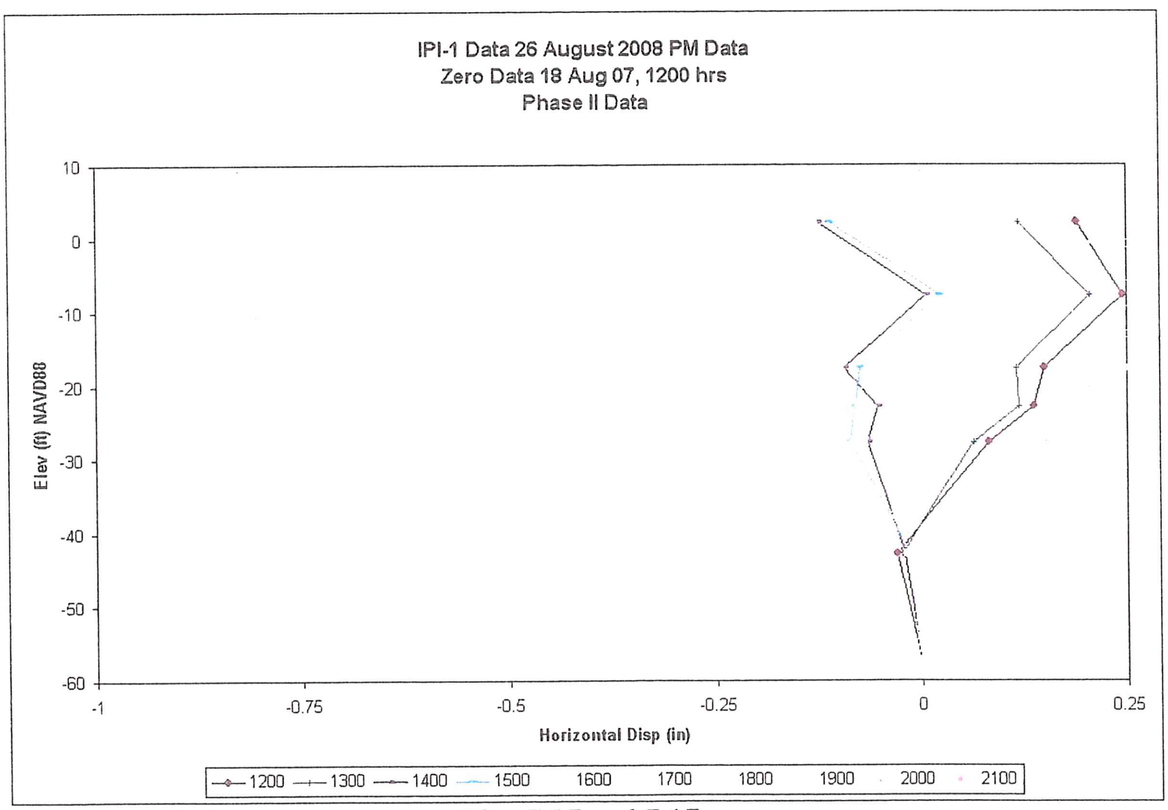
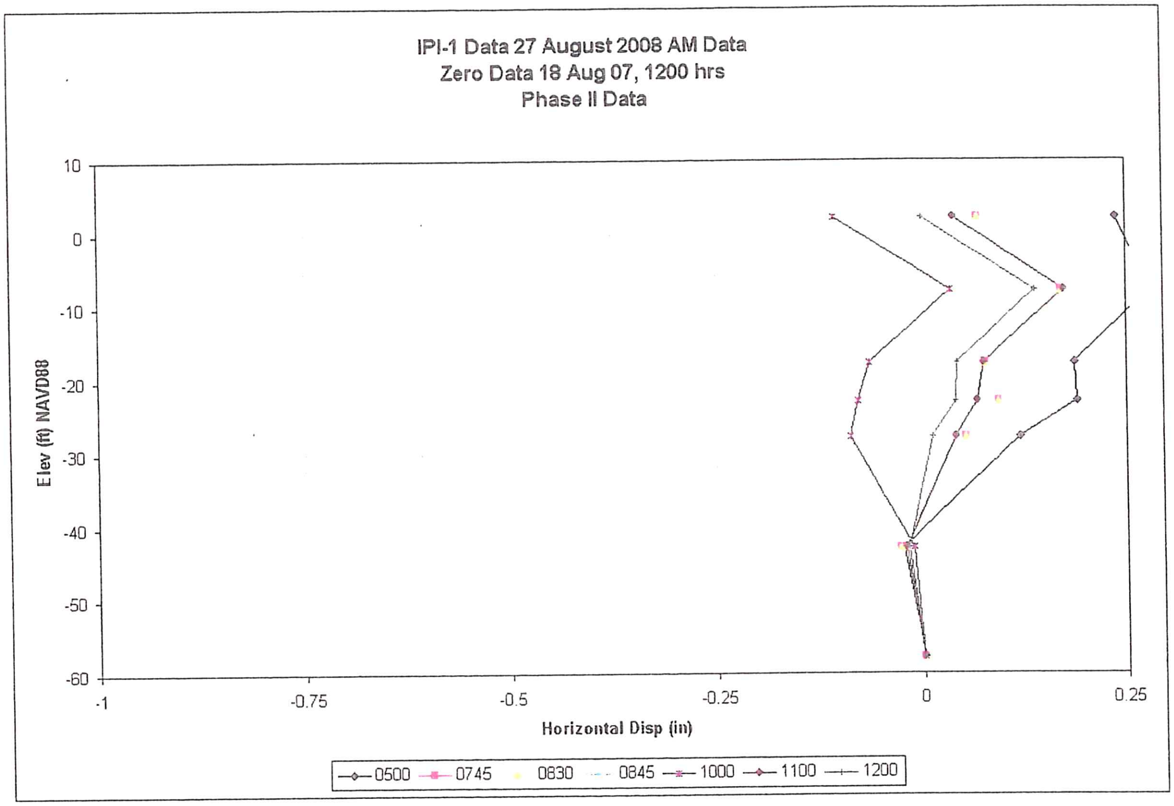


Fig 7.16 and 7.16a





Figs 7.17 and 7.17a



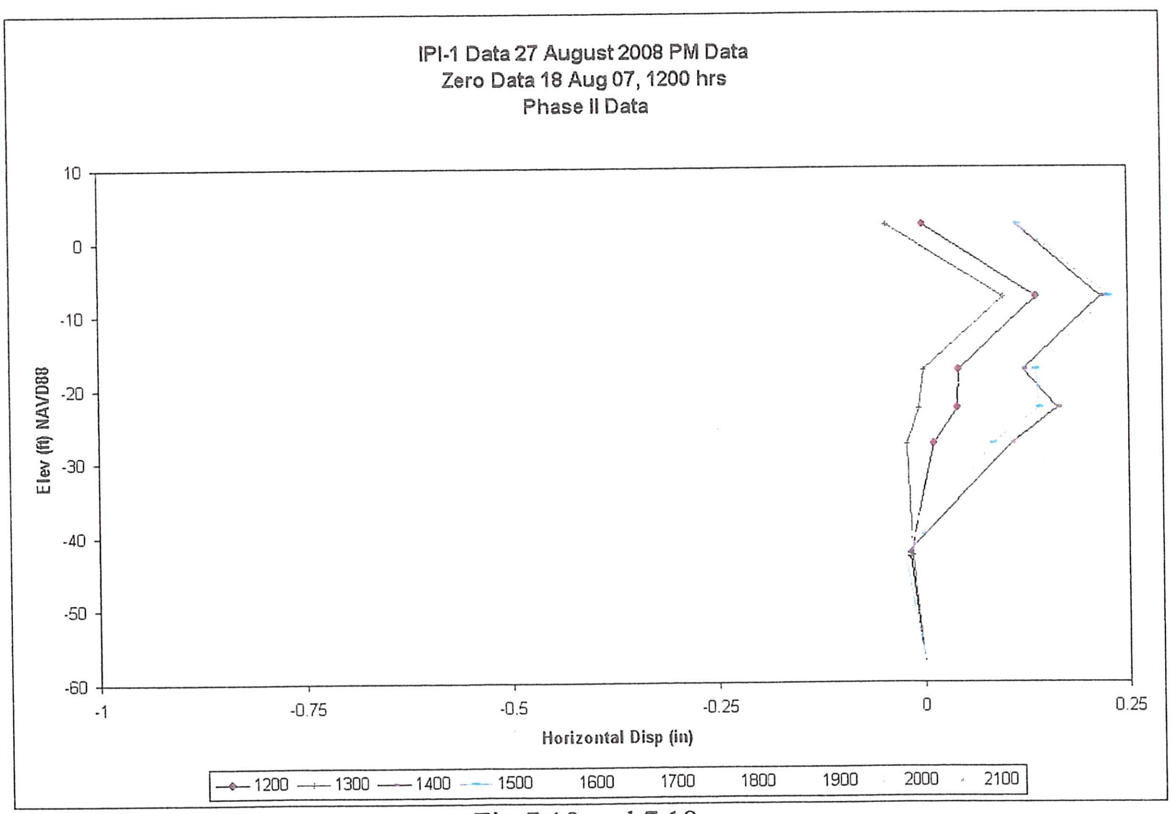
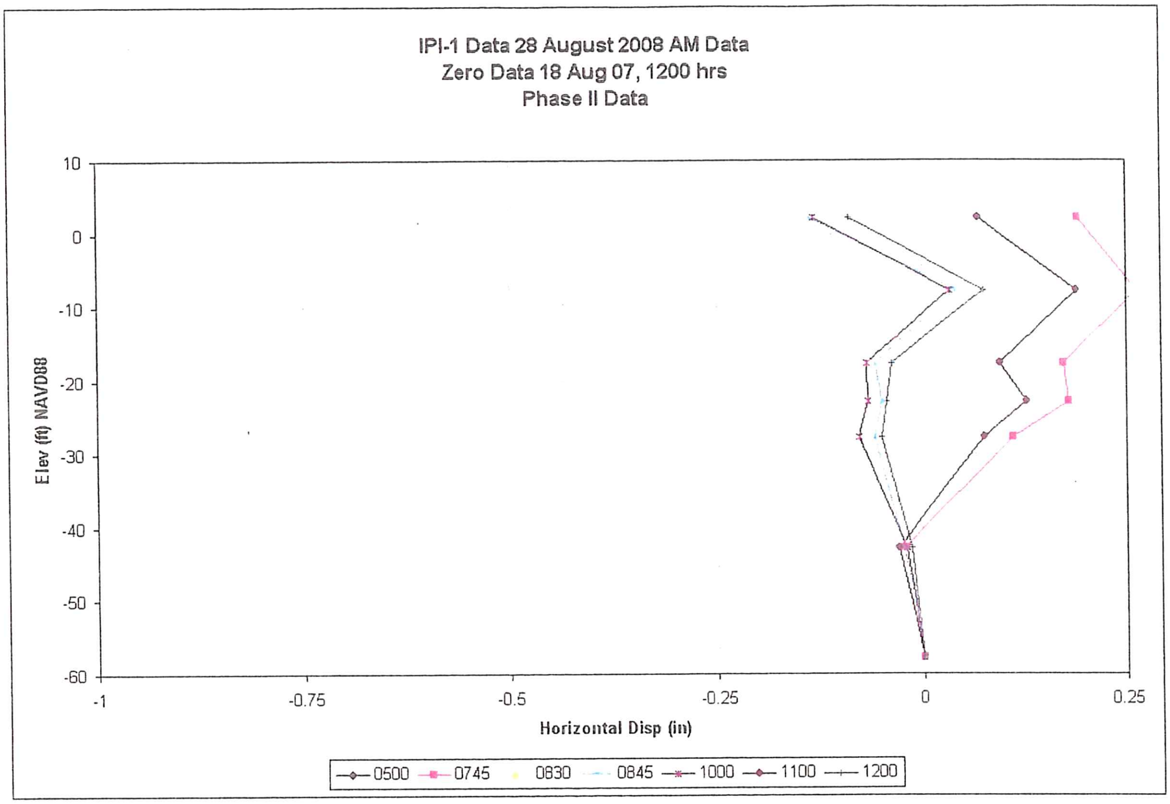


Fig 7.18 and 7.18a



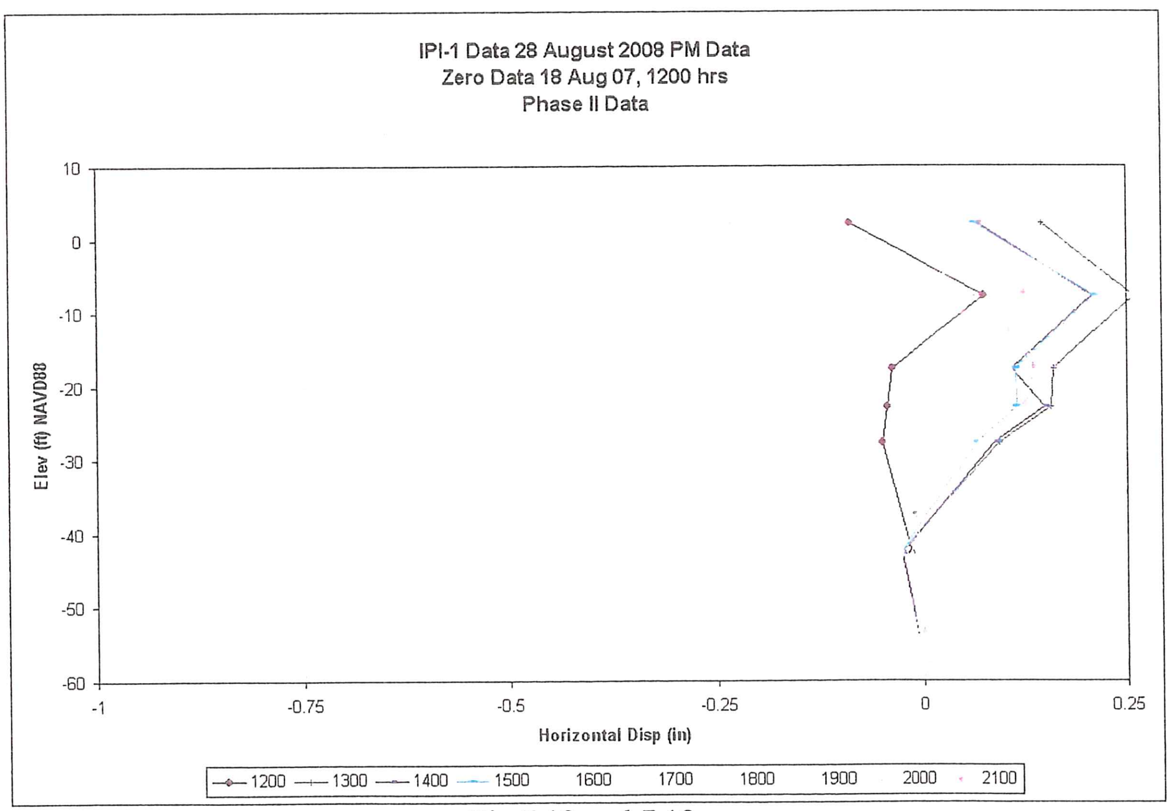
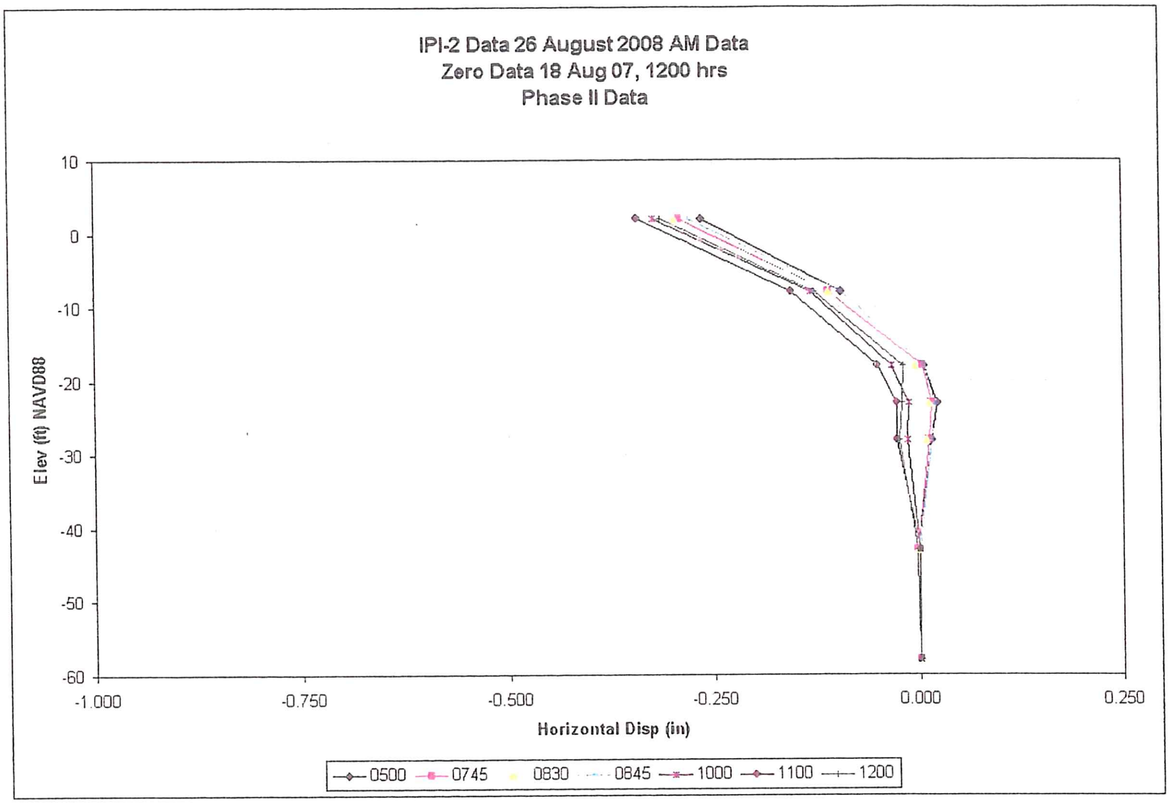
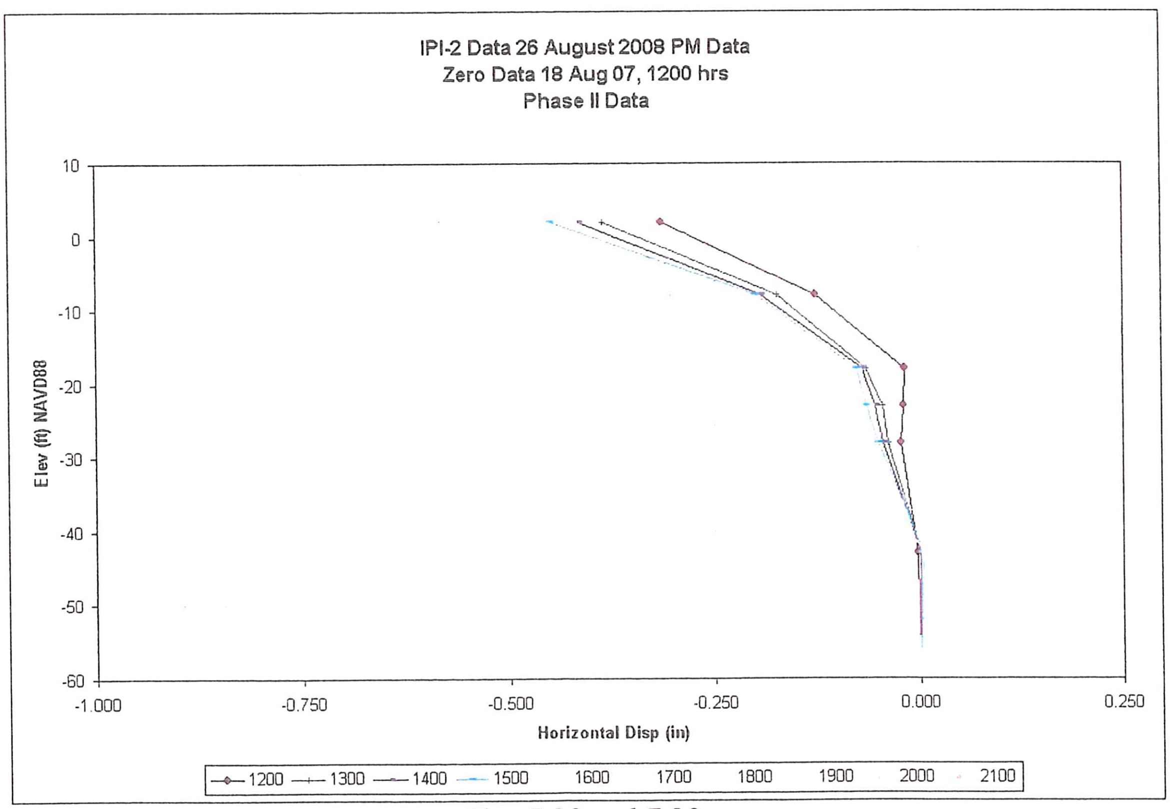
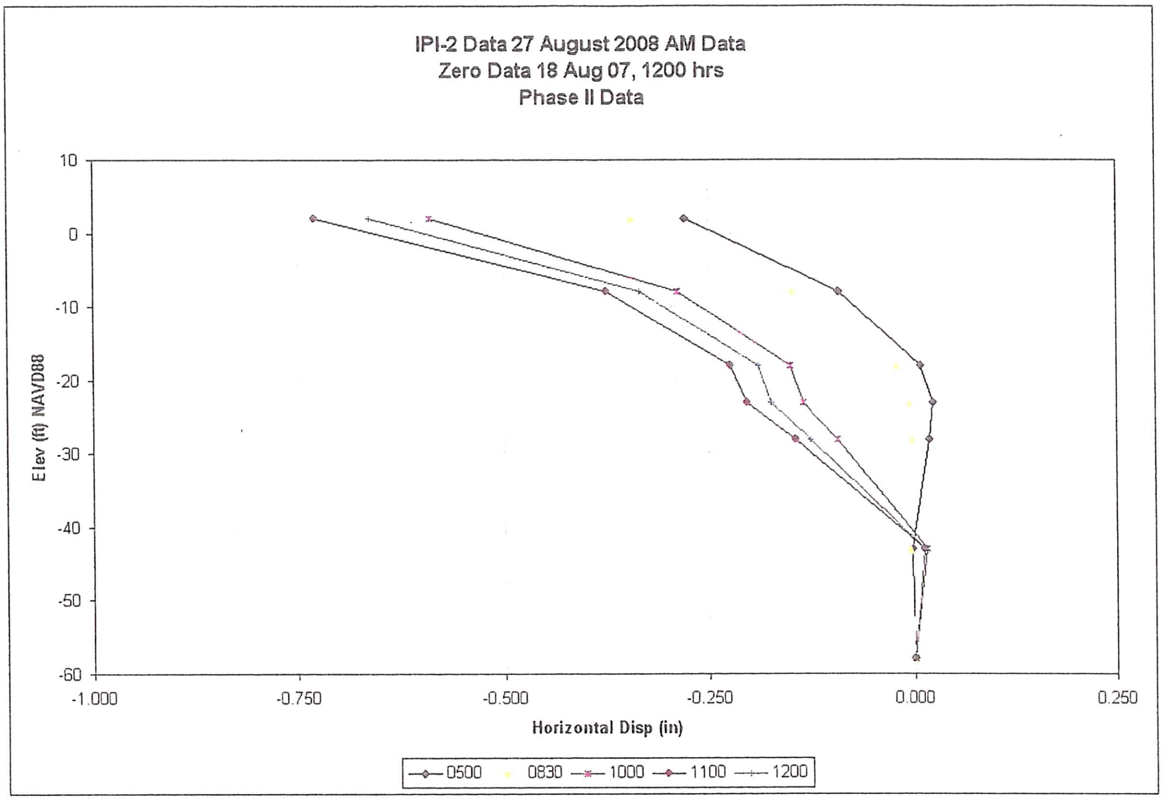


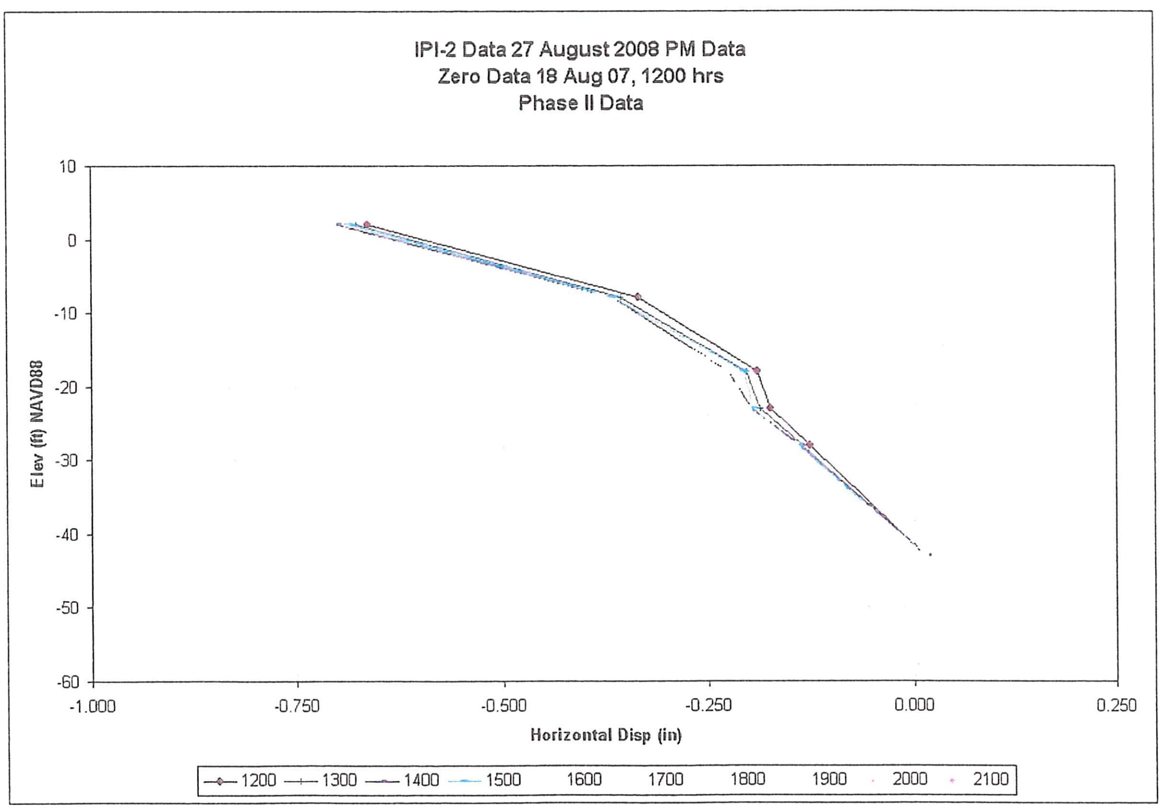
Fig 7.19 and 7.19a



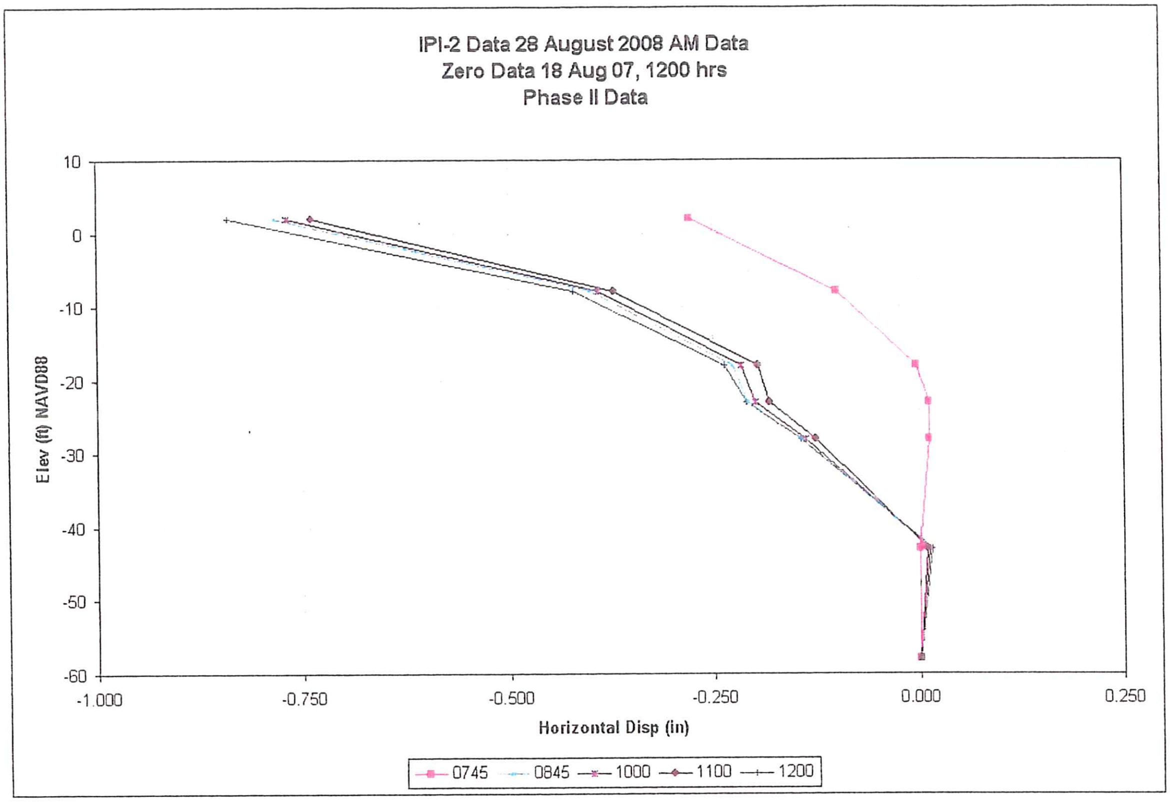


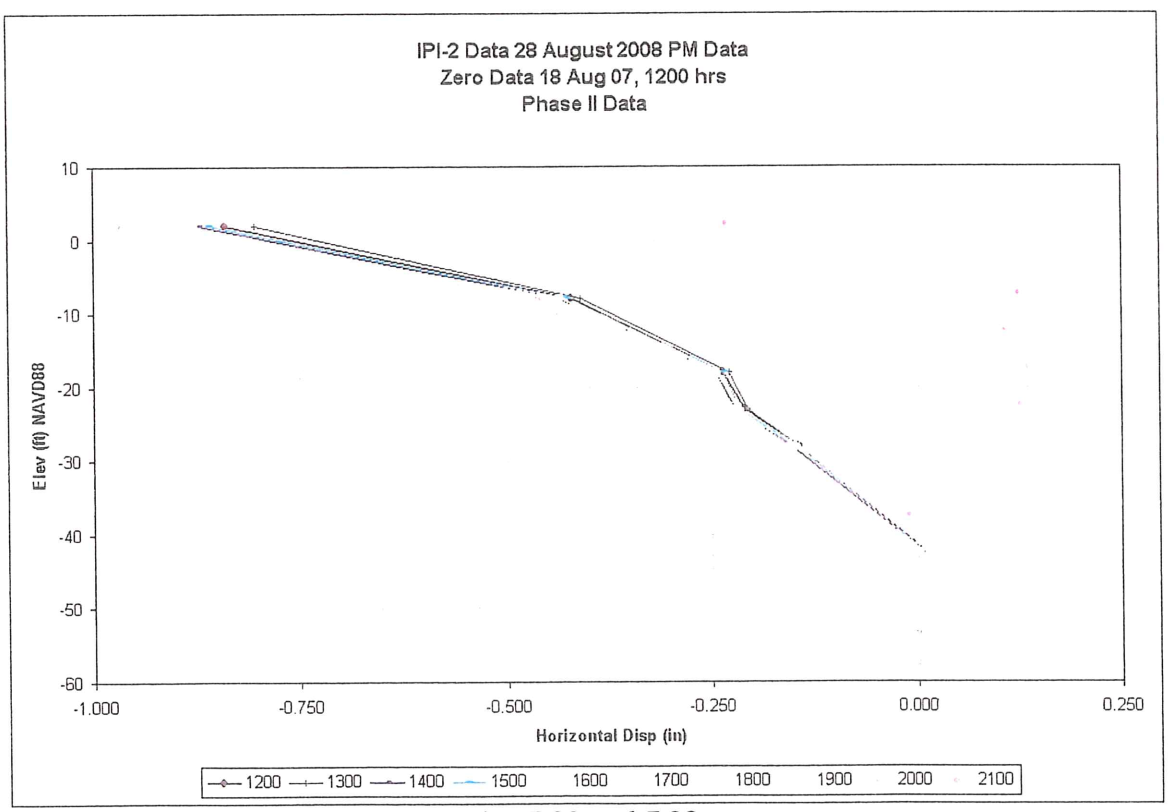
Figs 7.20 and 7.20a



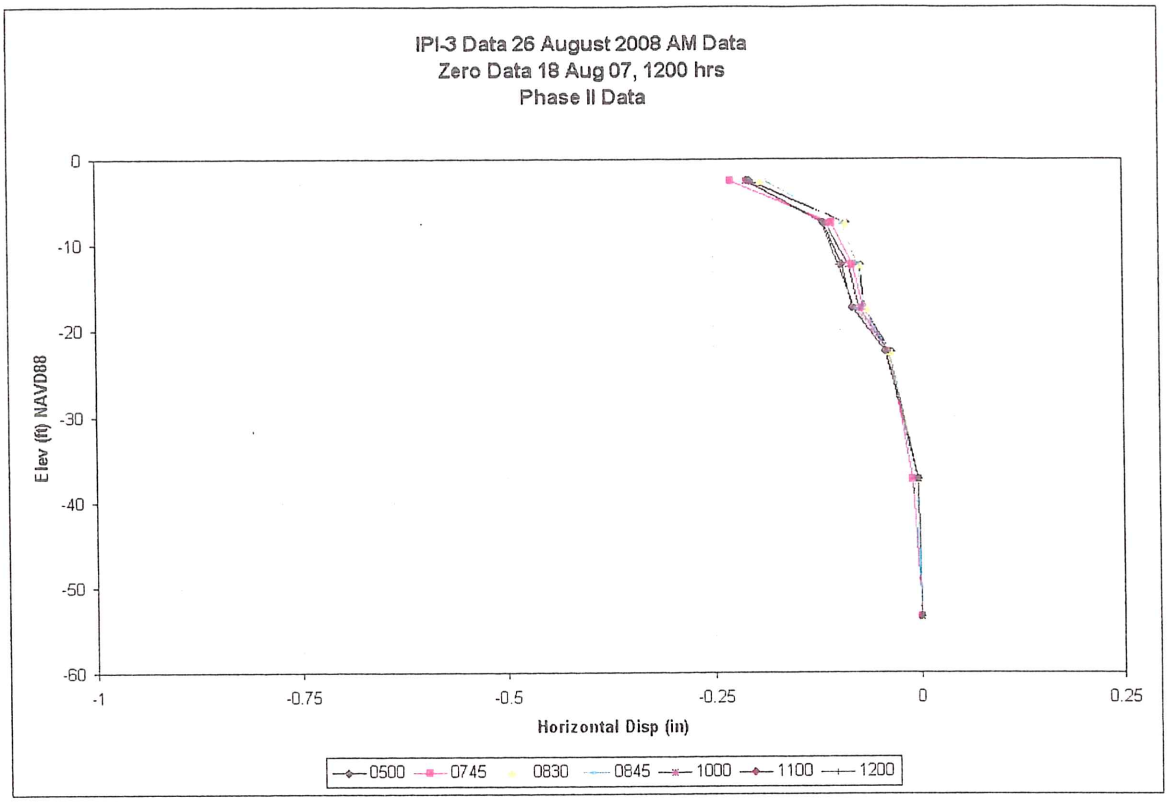


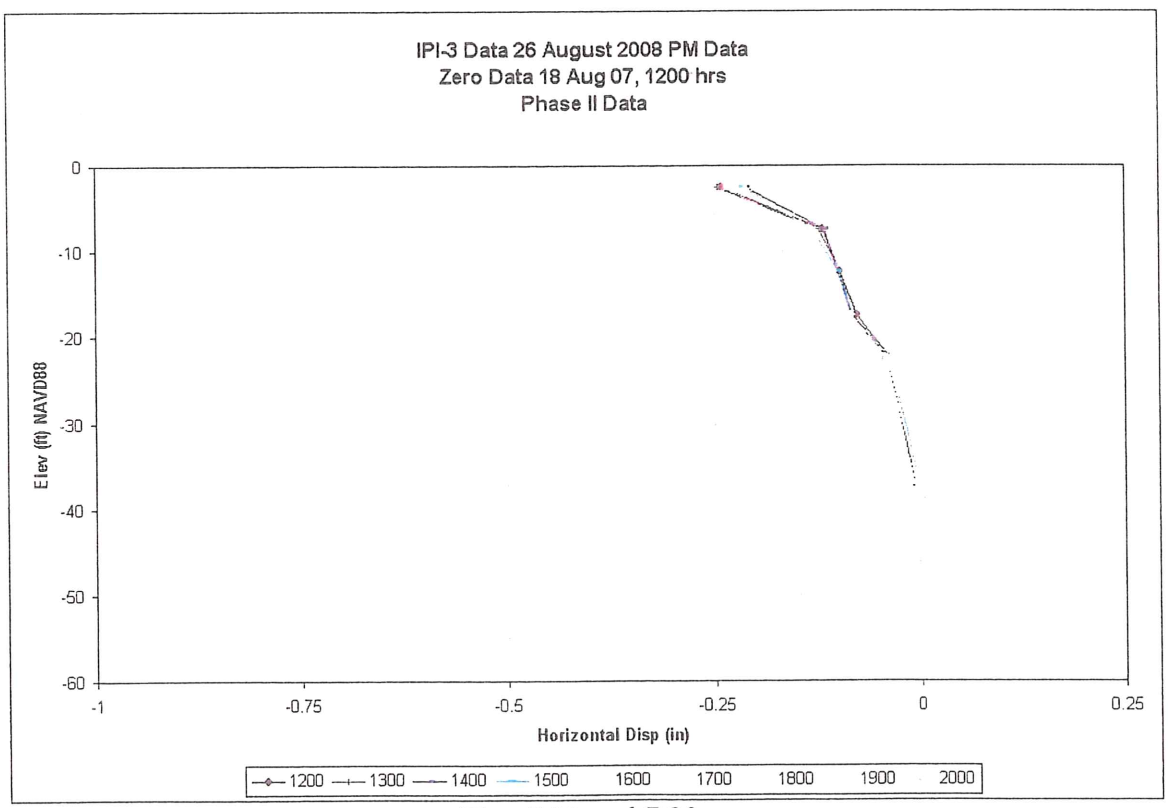
Figs 7.21 and 7.21a



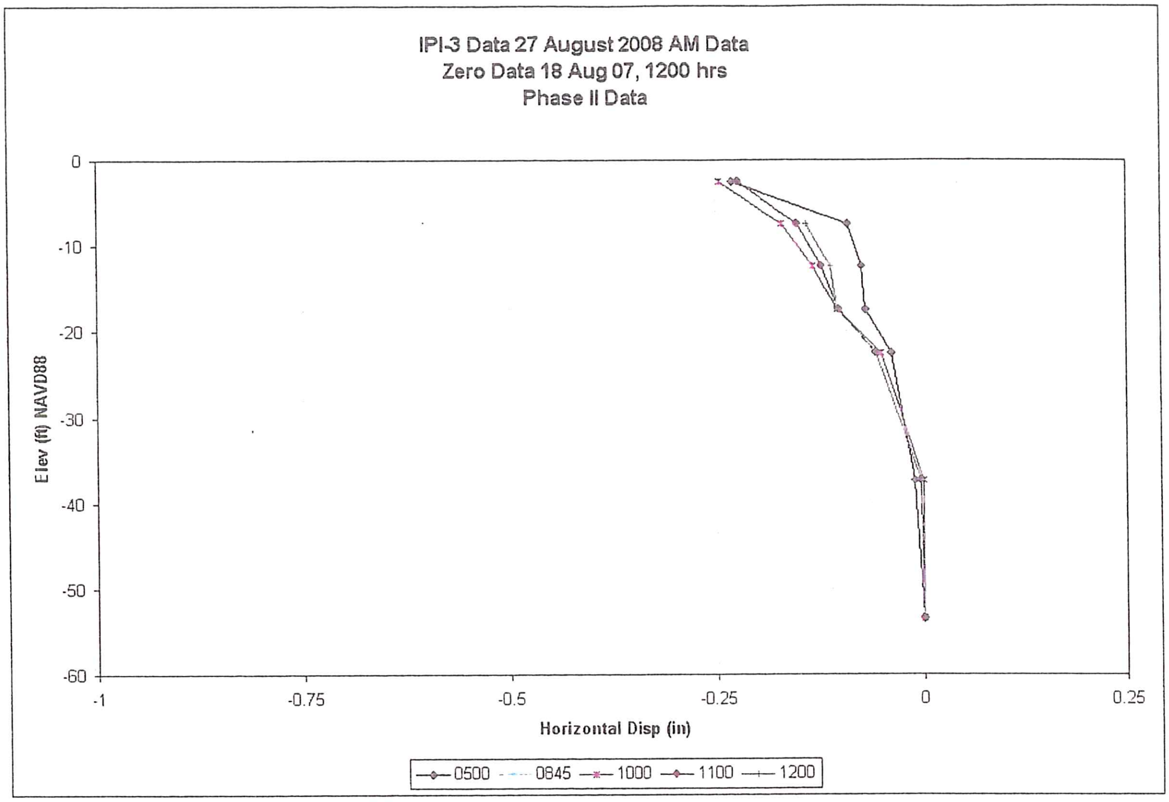


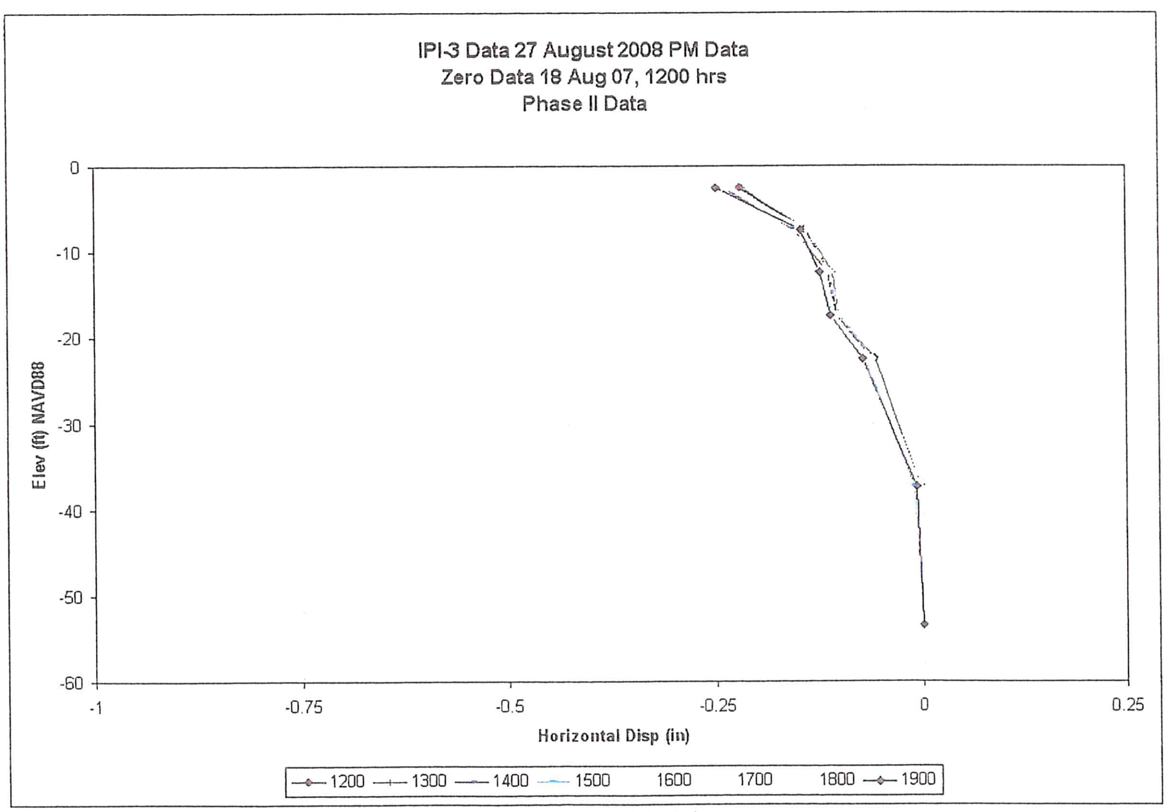
Figs 7.22 and 7.22a



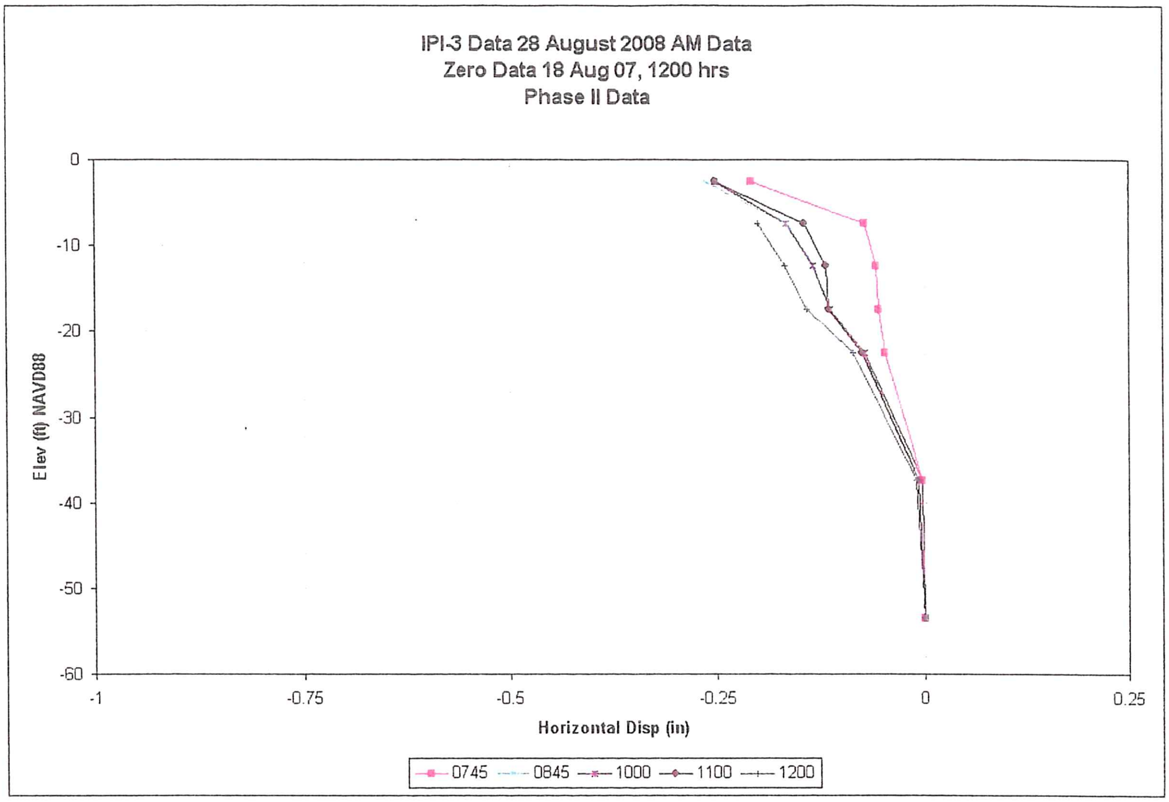


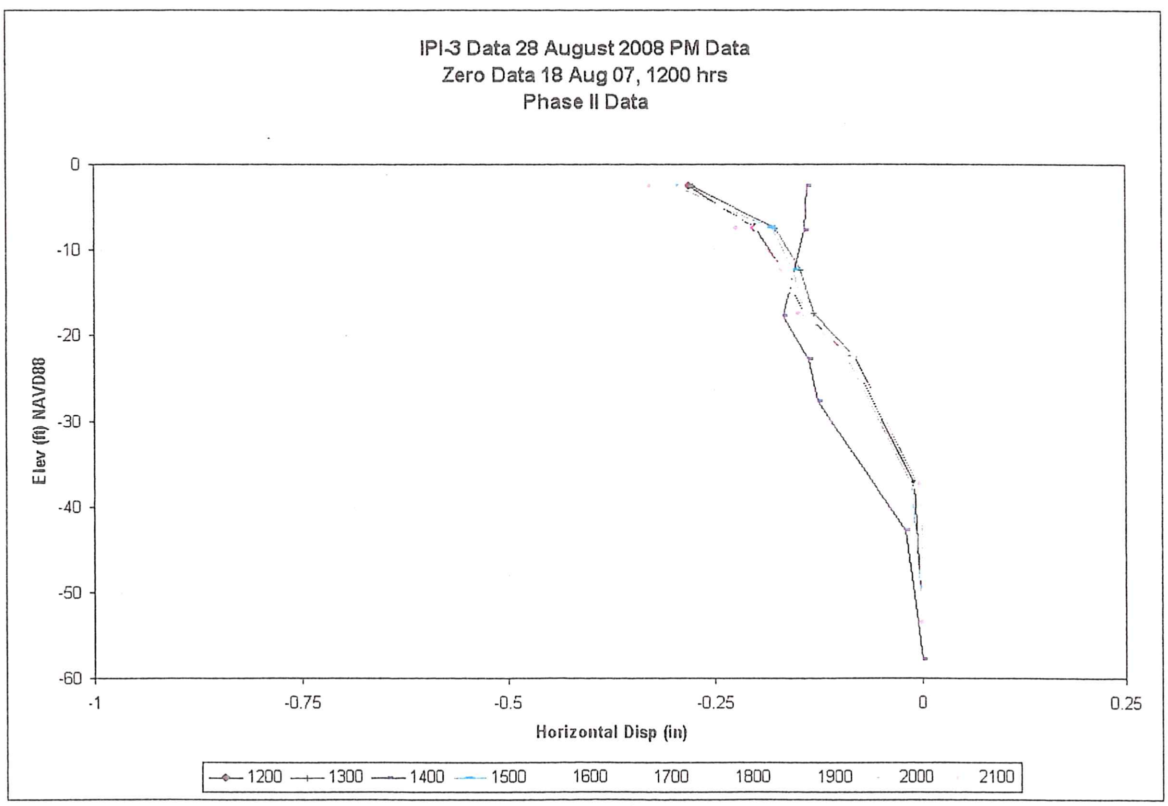
Figs 7.23 and 7.23a





Figs 7.24 and 7.24a





Figs 7.25 and 7.25a