FOUNDATION CONSIDERATIONS AT TAR ISLAND DYKE

By:

Ed McRoberts

Angela Küpper

and

Andy Small

AGRA Earth & Environmental Limited

For:

Vancouver Geotechnical Society
Construction on Peat and Soft Soils
9th Annual Symposium

June 2, 1995

Vancouver, BC

FOUNDATION CONSIDERATIONS AT TAR ISLAND DYKE

Ed McRoberts[1], Angela Küpper[2], and Andy Small[3]

A key element of the construction and operation of Tar Island Dyke (TID) on the Suncor Oilsands Lease, north of Fort McMurray, Alberta is its foundation. The foundation is a Holocene age fluvial deposit of the Athabasca River consisting of a complex succession of interbedded silts, clayey silts and silty clays. These fine grained deposits are underlain by river sands and gravels and in some locations overlain by muskeg. Construction of the now 320 foot[4] high structure began in 1965 when it was anticipated that the structure would be about 40 feet high and continued to 1984 when it reached full elevation. The dyke still functions as a component of the overall water and waste handling system at the Suncor Mine. The approximately 3H:1V downstream slope largely consists of tailings sand. This paper describes the essential elements of the foundation, the performance data obtained over almost 30 years and discusses aspects of stability and on-going movements experienced in the foundation of TID.

INTRODUCTION

Tar Island Dyke (TID) is located on the west bank of the Athabasca River in Northern Alberta as shown on Figures 1 and 2. The dyke was built mostly between 1967 and 1984 and is now about two miles long with a maximum height of 320 feet. The dyke is used to retain tailings sand and fine tails (sludge) produced by mining oil sands and is an integral part of Suncor's overall water handling system. The tailings storage facility is referred to as Pond 1. The reader interested in geotechnical aspects of the oilsands industry is referred to an overview paper by Morgenstern, Fair and McRoberts (1988).

The purpose of this paper is to review the geotechnical and related issues of the foundation elements of TID, in particular at the critical section of this structure.

[1] Chief Technical Officer, [2] Senior Geotechnical Engineer, AGRA Earth & Environmental Limited, 4810 - 93 Street, Edmonton Alta., T6E 5M4, [3] Manager, AGRA Earth & Environmental Limited, 3017 Faithfull Avenue, Saskatoon Sask., S7N 2S3

[4] The Imperial System of units is the standard adopted for the Suncor Lease and the design of TID for site layout and geometry of structures.

CONSTRUCTION HISTORY AND DESIGN BACKGROUND

Construction History

Figure 3 is an aerial photograph of the Athabasca River and Tar Island before construction of the dyke began. In 1965, the initial design for a tailings storage facility required the construction of a three year life 40 foot high retention dyke built over Tar Island. Tailings were to be discharged from the west escarpment of the Athabasca River valley and the dyke was to control the toe of the tailings deposit. It was to be built out of overburden that was removed as part of the mining operation. The original site investigation was configured to meet this requirement. By 1966, however, pilot plant operations indicated that the plan for depositing tailings off the valley escarpment would not work due to much flatter tailings slopes and the segregating nature of the material. The design was then changed to a 76 foot high structure also to be built out of overburden.

In 1967, the plant went into operation and it became clear that tailings, fines and water were accumulating behind the dyke at a faster rate than the dyke could be raised and consequently there was insufficient storage. In addition, it was determined that construction of the dyke with tailings sand instead of overburden would be more economic. As a result, a 200 foot high dyke made of tailings sand was designed. This design configuration adopted a centreline construction method with a shell of compacted sand retaining beached sands. The 200 foot height was reached in 1973.

During the winter of 1974, a variety of planning and schedule considerations led to the design to a final height of about 320 feet which was achieved in 1984, except in the north end of the dyke where sand was stored to a higher elevation. Construction activities continued at the north end of the dyke until 1986.

Design Background

The original designs for the several and clearly significant changes were carried out by the late Dean R. M. Hardy. Reference can be made to Hardy (1974) and Mittal and Hardy (1977) for detailed information on the overall design and especially the hydraulic fill procedures that were an integral part of the TID construction.

Figure 4 delineates the current essential elements of the dyke cross-section, providing a schematic representation of the critical section of TID. The key design features of the critical section include an overall 3H:1V downstream slope, the overburden starter dyke, compacted tailings sand shell, three internal drains which control the phreatic surface in the compacted tailings sand, and beached tailings sand. The foundation consists of alluvial clays underlain by alluvial sands and gravels and limestone. Berms (benches) are located at elevations 823, 866, 910, 955, 1000, and 1045 feet. The berms are about 25 feet wide. The critical section of TID was defined by the greatest depth and areal extent of fine grained soils beneath the downstream section. While thick layers of clay are also found to the south, their lateral extent is more limited. They were considered to be less important in terms of stability effects and this has been confirmed by monitoring. This paper focusses on the critical section. The remainder of the dyke has less clay and/or flatter downstream slopes that result in higher factors of safety.

Table 1. Overview of Performance Data to 1984

Factor	Data	Comments
Factor of Safety	As low as 1.42 in 1976 rising to 1.65 due to pore pressure dissipation.	Based on effective stress peak strength parameters .
Settlement	Up to 9 -13 feet.	Based on changes in foundation thickness and water content.
Horizontal Movement	Measured deformations in foundation clays distributed with depth up to 0.25 m at the top of clay.	Data only from 1975 on.
Pore-pressures	Excess pore pressures generated in foundation clay by dyke loading exist beneath the crest of dyke but not in clay beneath toe.	Pore pressures dissipating to tailings and underlying sands and gravel.

The safe operation and stability of TID has always been an essential element of Suncor's overall mine plan. There has been considerable owner and regulatory effort in ensuring that mining requirements could be met while also considering the safe operations of this vital structure. As part of this process, there has been a substantial on-going program of performance review, including an independent Design Review Panel (DRP) that reported on TID to the Minister of Environment, Province of Alberta, in 1977.

The essential elements of the performance review up to the early 1980's can be found in DRP (1977), Mittal and Hardy (1977), Chan, Martschuk and McRoberts (1983) and Chan and McRoberts (1989) and will be touched on as appropriate in subsequent sections of this paper. The early performance data up to the early 1980's are summarized in Table 1.

In December 1984, it was discovered that inclinometer S81-101C located on the critical cross section area on the 1000 berm had become inoperative near the contact between the foundation clay and the underlying sands and gravels. In September, just prior to this event, the crest of TID had been raised from Elevation 1072 to 1088 feet and the pond level had risen about 10 feet since the last set of inclinometer readings in August 1984. While the critical potential slip surface was not at depth, prudence dictated an evaluation of all other data and a detailed investigation program. While movements after this event remained stable (i.e. not accelerating) and could be attributed to several mechanisms including settlement and creep, a conservative assessment of all conditions was undertaken and in 1987 a decision was made to improve the factor of safety and seek a reduction in the movement rate. This decision was partly motivated by criteria related to reclamation and final abandonment of the structure.

After considering several alternatives, it was decided to place a modest berm along the toe of the critical section and cut back the crest slightly. The material excavated from the crest was placed at the toe. The berm and crest excavation were designed in two phases with a re-assessment of conditions and performance at the end of the first phase. The response of the first phase indicated acceptable performance and the second phase was eliminated.

SITE DATA

Foundation Characteristics

The Athabasca River shown on Figure 3 flows from south to north and an old arm of the river, referred to as a snye, passed on the west side of Tar Island and now underlies the dyke and tailings pond. This snye was cut-off by placing fill at either end. Tree cover on Tar Island and the river escarpment was stripped. Areas of organics with typically up to 6 to 8 feet of muskeg and locally reported to be up to 12 feet were identified in the original site investigation and in initial construction monitoring. These areas of muskeg were underlain by high moisture content soft silts and clays. These muskegs were left in place in the areas shown on Figure 5. The northern muskeg section is now under the downstream section of TID. As reported by Mittal and Hardy (1977), the original design intent was to strip this material from the foundation. At the recommendation of Dean Hardy, these organics were left in place and the TID downstream slope was flattened to 4H:1V in this section. The muskegs were stage loaded with careful attention to ensuring that only thin 3-4 feet lifts were placed until the muskegs were well consolidated. Based on muskeg research at the time, the design premise was that, when well-consolidated, muskegs would exhibit better friction angles than the clay soils, and as long as the design could accommodate deformations then an acceptable configuration would result. Satisfactory performance has been obtained, and this section has not been the subject of further design concern. An inclinometer in the area measures on-going deformation rates of shear strains in the order of 0.04 %/year. As it will be seen later in this paper, this rate of deformation is at the lower end of movement rates in the critical section.

The surface elevation of the island in the vicinity of the downstream toe was in the order of Elevation 783-787 feet prior to construction. The river level has typically been in the vicinity of 770 feet over the life of TID. The sand and gravel layers underlying the clay are hydraulically connected to the river.

Sedimentary and Stratigraphical Record

The sedimentary record at the TID site is best represented by a model of a meandering stream sequence with features similar to those found along the modern Athabasca River. In order to better understand depositional environments and to provide a basis for generalizing laboratory testing data, a facies study of the foundation unit was undertaken. Catto (1988) defined seven facies as described in Table 2.

The stratigraphical record suggests that the site was dominated by several depositional events. On deglaciation, the defining event was about 9,900 years before present when a paleo-flood originating from Glacial Lake Agassiz abruptly swept down the Clearwater River from Saskatchewan and flowed north down the Athabasca River. This flood originated from a breach of an ice dam and resulted in much of the evident stripping of Quaternary sediments, channel scars, and incised gravel channel fills along the lowlands adjacent to the Athabasca River. One suspects that the gravels overlying bedrock may date from this event, but they could also relate to infilling at the end of a period of subsequent Holocene downcutting. Beginning about 6,600 years before present, there is suggestion that the river entered an equilibrium or even an aggradational stage and sediments began to build up. It is therefore likely there are several old sub-aerial surfaces represented in the vertical stratigraphy. Subsequently, the river appears to have undergone a channel shifting to the west with the channel apparent in Figure 3 representing the western limits. During this time there was a series of channel deposits formed and in turn buried by overbank deposits.

The facies work lead to several major conclusions that assisted in interpretations of older geotechnical data and in assessing more recent information. Firstly, it was clear there was a complex origin of water deposited sediments with the potential for many old sub-aerial surfaces during deposition. At the same time, channel infill representing possible cut-off oxbow type environments allowed for the local sub-aqueous deposition of thicker units that finally emerged sub-aerially near the pre-construction land surface. Secondly, the massive clay Facies 1 unit (which possessed the lowest shear strength) has limited lateral extent and thickness. The geological model developed by Catto (1988) assisted in understanding the deposits and provided a valuable basis within which to rationalize the geotechnical data. If such a model had not been developed, a much greater and more extensive drilling and testing program would likely have been required to support a statistical approach to handling extreme data variations, especially the low residual strength data in Facies 1.

Table 2. Facies Types in TID Foundation

Facies Number	Facies Description	Origin & Comments
1-clay	1 Massive clay 1b Massive clay with silt balls	Very rare in core successions deposited in very low energy overbank environments in swale areas between meander scrolls thin and of limited lateral extent (seldom more than 10 m in modern environment). Cannot correlate between cores and extent therefore less than 30 m.
2-clayey silt	2a- Clayey silt to silty clay with detrital organic horizons	These clayey silt - silty clay facies occur in most of the core and represent deposits under very low energy flow or stagnant conditions in either overbank or abandoned channel situations. Most common form is 2a in association with interbedded thin coarse silt,very fine sand or fine sand layers described as 2a/4, 2a/5 and 2b/5 sequences.
	2b- Clayey silt with irregular detrital horizons	
	2c- Clayey silt with clay balls	
3-silt	3a- Massive silt with clay balls	Massive silts usually found in fining up sequences as end member of 5/4/3 deposition in lake or river ending in abandonment of site by flowing water. Repetitive sequences indicate cyclic abandonment, sub-aerial exposure and desiccation followed by reoccupation of site by running water.
4-	Very fine sand and occasional silt inclusions	These facies are of geotechnical importance for foundation considerations in that they appear as parting to layers within the finer grained facies and additionally underlie them.
5	Fine -medium sand	
6	Medium - coarse sand	
7	Gravel	Initial (post glacial) deposition.

Pre-Construction Geotechnical Data

The original site investigation was carried out in 1964 (see Figure 5). The investigation established that up to 52 feet of locally soft to stiff silty clays or clayey silts (referred to generically as clay in this paper) were present. The clay was underlain by granular deposits varying from fine sand to gravel, which rest unconformably on limestone bedrock. A range of index parameters for the clay deposit obtained primarily from the 1964 investigation is given in Table 3.

A program of undrained compression tests (unconfined and confined) and a limited series of triaxial drained tests were undertaken on samples from the 1964 investigation. No brittleness was observed in any of these tests and no slickensides were reported in any of the original boreholes.

Table 3. Representative Index Properties of Foundation Clay Before Construction

Index	Range	Average
Water Content	32 - 75%	45-50%
Liquid Limit	40 - 65%	50%
Plastic Limit	22 - 28%	24%
Compression Index C_c (1964 and more recent tests)	Facies 2: $\bar{x}$=0.034, s =0.1 (n =5)	
	Facies 3: $\bar{x}$= 0.030, s = 0.05 (n=15)	
Coefficient of Consolidation c_v	1×10^{-2} - 1×10^{-4} cm^2/s	$1\times10^{-3}cm^2/s$
Overconsolidation Ratio	5.0 near surface to 1.2 at depth	
Secondary Compression C_α (1964 and more recent tests)	Facies 2: $\bar{x}$= 0.012, s = 0.003	
	Facies 3: $\bar{x}$= 0.011, s =0.006	

Of particular interest to this discussion is the likelihood that the clay was somewhat overconsolidated prior to construction. Such overconsolidation could occur from some combination of the following mechanisms:

- Desiccation of exposed sediments.
- Repeated freezing and thawing that would have occurred in those sediments above or close to the water level.
- Changes in the river level that would have resulted in variations in the effective stresses within the clay layer.

The following discussion reviews the data from 1964 and comments on the estimated degree of overconsolidation as this has a bearing on the interpretation of recent geotechnical data obtained from the foundation clay.

Several oedometer tests were performed in 1964 and suggested the clay possessed an overconsolidation ratio (OCR) of 3 to 5 near the surface. There was a trend towards normally consolidated behaviour with depth and an OCR as low as 1.2. It is noted that the oedometer tests were likely carried out on samples that were relatively softer than the other sediments. It is therefore possible that the oedometer results might have underestimated the OCR.

The pre-construction liquidity index (LI) data with depth below original ground is shown on Figure 6. The data has been grouped according to the unified soil classification codes CH, CI-CL and ML-MH. It can be seen that the data varies from an LI of about 0.5 to over 1.5. Five data points for CI-CL clay are close to unity at 30 to 40 foot depth, however a CH clay sample in this depth range had an LI of 0.3. Based on the data shown in Figure 6, it was concluded that the stress conditions in the foundation clay are variable and certainly not uniformly normally consolidated with depth. These LI data suggest OCR of 2 to 4.

In situ vane shear tests were conducted on the clay to a depth of about 45 feet. The results are summarized in Figure 7. The vane shear test data are subdivided into clay (CI and CH) and silt (ML to MH). The data indicate that the upper 6 to 10 feet of the clay is heavily overconsolidated. Overlain on Figure 7 are the highest and lowest estimates of vertical effective stress ($\sigma'_{v,max}$ and $\sigma'_{v,min}$) that was likely in effect at the time of the investigation. It was assumed that the water table at the time of investigation could have been between the ground surface and as much as 15 feet below ground level. The lower water level corresponds to the average river level and assumes that there were sufficient sand seams in the clay to allow for drainage. Assuming that the ratio between the undrained shear strength measured with the field vane test (S_{ufv}) and the vertical effective stress (σ'_v) can be considered to be 0.45, based on considerations discussed later, a range of S_{ufv}/σ'_v was calculated and it is shown in Figure 7. Only a few reported shear strength values are within such a low range. The results shown in Figure 7 indicate that the clay is likely overconsolidated, at least to a depth of about 45 feet. It is possible that the vane shear data may also reflect varying horizontal stresses due to pre-construction differential consolidation of the complex foundation.

RECENT STRENGTH AND COMPRESSION DATA

Investigation Programs

Investigation programs proceeded on a limited scale after construction started in 1967, and information was obtained in respect to the review process documented in DRP (1977). However, after inclinometer S81-101C became inoperative at the end of 1984, a considerable effort was directed towards the acquisition of additional information on the properties and strength of the foundation materials. This information is reviewed here and selected data up to the 1976 period are also discussed as appropriate.

The 1985 campaign began with a series of boreholes which continuously sampled or cored the foundation clay. All samples were inspected by senior engineers and by September, 1987, 41 holes had been drilled mostly related to the foundation clay. The foundation clay beneath all of the berms was examined for evidence of slickensiding. Detailed logging was undertaken for facies determinations and strength testing undertaken. This work led to the recommendation to construct the mitigative measures which were undertaken in the Summer of 1988.

Additional holes were drilled from the crest of the toe berm at elevation 823 feet in 1989, 1990, and 1992. Clay samples were obtained for further examination and lab testing after the toe berm had been constructed.

Slickensided Clays

Slickensides were detected in several of the holes drilled in the 1985 to 1990 investigation programs and significant effort was devoted to delineating their extent. The slickensides were of infrequent nature and never at a common elevation between adjacent boreholes. Figure 8 shows the areal extent of the foundation clay where slickensides were observed. Upstream of the 823 berm, if present, the slickensides were generally continuous across the clay samples. Some slickensides were detected beneath the 823 berm but they were not continuous and were only found in three holes of a total of 15 holes. No slickensides were found in the toe region, see Figure 8.

In general all slickensides were poorly developed and had a surface tending to be inclined or undulating. At the time of the 1987 investigation, slickensides in 13 boreholes upstream of the 866 Berm showed uni- or multi-directional striations indicating movement along the surface. Horizontal striations with uni-directional movements were apparent in only 5 of these 13 boreholes. Subsequent drilling has confirmed these relative statistics.

The location of slickensides at seemingly random elevations between boreholes and the occurrence of multiple slickensides at different elevations in several boreholes suggest that more than one process may be contributing to their formation. Four potential processes have been identified:

- High wheel loadings from construction traffic during site preparation and placement of the overburden dyke fill. These might extend to depths of up to four feet below original grade.
- Differential consolidation of the highly variable deposits introducing differential strains and resulting in slickensides at all depths.
- Ice jamming resulting in pushing surface sediments and locally inducing slickensides. This would have occurred both in recent times as well as historically when, now deeper, but then sub-aerial surfaces were exposed to river induced ice action.
- Shear stresses induced by the dyke construction itself resulted in localized shearing or exacerbating conditions from the other mechanisms.

Regardless of the mechanism of formation, it was conservatively concluded that the presence of slickensides could be embrittleing the foundation. While it was highly unlikely that a continuous band of slickensides induced by gross shear movements could be present, the available strength of the foundation clay was downrated.

Compression Characteristics of Foundation Clay

A series of 6 oedometer tests was carried out on samples from beneath the 823 Berm indicated vertical preconsolidation stresses of 180 to 250 kPa, consistent with existing vertical effective stresses below that berm. Previous testing by Watts (1981) on samples taken from beneath higher berms indicated preconsolidation stresses at or less than the calculated vertical effective stresses. Preconsolidation stresses less than the calculated vertical level may be due to the complications of excess pore pressures existing at the time or to distribution of insitu stresses due to creep mechanisms.

Therefore while the clay likely was preconsolidated prior to construction, it is now in a normally consolidated state upslope of the 823 Berm.

Base of Tailings Sludge Layer

In addition to the foundation clay, a second possible layer of low strength was detected during the investigation programs and it is referred to as the base of tailings sludge layer or basal layer. During the detailed sampling, field protocols required sampling to begin above the tailings sand/original ground contact. Detailed inspection of this contact revealed a thin (up to 200 mm thick) layer of highly consolidated fine tailings or sludge in 10 of 15 boreholes upstream of the 910 Berm. It is likely that this layer was formed during the early deposition of tailings as an accumulation of the sludge against the overburden starter dyke. The upstream toe of the starter dyke extended approximately midway between the present 910 and 955 berms. The large settlement of 10-15 feet of the foundation clay has masked the original ground relief, but it is possible this material collected in local depressions and was trapped when tailings sand was subsequently overboarded off the overburden dyke.

The basal sludge layer was consolidated to a clayey silt and found to be slickensided in 5 of the 15 samples obtained. This material, reflecting a process dominated genesis, had complex properties. Some bitumen was apparent and the range of Atterberg Limits were w_l = 58 to 118% and w_p = 29 to 48% on field samples and w_l = 30 to 50% and w_p = 30 to 34% after bitumen extraction.

Peak Strength

A series of drained and undrained triaxial tests, direct simple shear and direct shear tests have been conducted on the foundation clay. A summary of effective stress interpretations of peak strength results is given in Figure 9 for triaxial tests and in Figure 10 for direct shear tests. Assuming zero cohesion, the peak strength triaxial test data fall within a range of friction angles of about 24 to 33 degrees. The direct shear test data fit a similar range except for one test that was conducted on the massive clay (Facies 1). This test measured a relatively low peak shear strength.

Residual Strength

A series of direct shear residual strength determinations were made on samples selected by senior staff logging the core. These tests are reported in Table 4 correlated in terms of facies types and including a test done on the basal sludge. These tests were undertaken in 1987 and were used for initial re-assessment of stability, as will be discussed in a subsequent section.

A correlation of residual friction angle ϕ'_r and liquid limit was developed and used for design as shown in Figure 11.

Table 4. Summary of 1987 Residual Tests on Foundation Clay and Basal Sludge

Facies	Range of Atterberg Limits		Residual Angle (degrees) assumes c' = 0.0	
	w_l	w_p	no of tests	ϕ'_r
1	53-75	28-48	3	6.5,10[1],15
2	45-61	22-29	6	13[1],16,18[1],20[1],23,24[1]
3	43	24	1	26
4d	58	31	1	13[1]
Sludge	78	33	1	8.5[1]
[1] Denotes shear plane cut with piano wire prior to test				

Undrained Shear Strength

While effective stress methods have always been the primary basis of considering the stability of TID, effort has also been directed to understanding the potential undrained response. The undrained shear test results are summarized in Figure 12 where the data has been normalized by the vertical effective stress after consolidation σ'_{vc} and plotted against σ'_{vc}. Details of all test data shown in Figure 12 are provided in Table 5.

In Figure 12 the triaxial tests have been shown for beneath the 823 Berm as one data set and all others shown with a different symbol. The sample labelled OCR = 1.2 represents Tests 90-3 which was a pair to 90-2. Both these samples were isotropically loaded to 600 kPa and 90-3 was rebounded to 500 kPa before shearing. The three K_o = 0.56 tests are also labelled separately.

Previous undrained analyses (see DRP 1977) had considered normalized strength ratios from 0.45 to 0.3. The more recent data support this range and taken together the data are consistent with a preconsolidation stress of up to 250-300 kPa.

Direct Simple Shear Tests

A series of five direct simple shear (DSS) tests were run with shear direction in the downstream direction. The volume of the sample was kept constant providing results similar to an undrained test. The samples were consolidated to 360 kPa, in excess of the preconsolidation stress as predicted by oedometer tests. These tests have the drawback of non-uniform stress distribution within the sample making it difficult to interpret results at high strains. Moreover as a uniform deformation condition is applied, failure does not occur along discrete shear planes.

Inclined Triaxial Samples

Four triaxial tests, two drained and two undrained, were conducted on inclined specimens obtained from 6" diameter, 18" long thin walled sampling tubes. The downstream direction was constantly tracked and marked on the tubes and on the samples so the direction of shearing in the field could be continued in the lab. The samples were inclined at 45 + $\phi/2$ degrees to the horizontal so that failure could take place parallel to the bedding. The other option of failure across bedding was also available to the sample. Figure 13 provides photographs of one of the inclined samples after shearing.

The samples were obtained from beneath the toe berm placed as part of the 1988 mitigative measures. The objective of these tests was to consider the potential brittleness of the Facies 2 clays underlying the design sensitive toe area. Figure 14 is an example of the results from a drained test. The peak strengths determined from the drained inclined tests were ϕ' of 29° and 33° and from the undrained tests, the peak ϕ' was 33° and 34°. The high strain strengths from the drained tests were interpreted as 21° and 23° which are consistent with the high end of the residual data reported for Facies 2 in Table 4.

The two CUP test samples failed along bedding surfaces and in fact the shear planes protruded through the top of the samples indicating that they had developed along planes of weakness. The shear surfaces after testing were slickensided and wavy. Examination of trimmings as the sample was prepared did not reveal any slickensides before the test. Hence it can be inferred the slickensided developed during the tests. The vertical CUP tests from below the toe berm appear to follow a consistent trend line, see Figure 10.

Table 5. Normalized Undrained Strength Toe Berm

Sample Number	Depth (ft)	$\sigma'_{1,c}$ or σ'_{vc} (kPa)	K_o	$S_u / \sigma'_{1,c}$ Triaxial S_u / σ'_{vc} for DSS	Borehole
Triaxial Tests					
12U3-C	14	588	1.0	0.36	1964-12,13 In channel infill just downstream of 910 Berm
12U4-B	20.5	406		0.49	
13U2-A	10.5	207		0.49	
26U3A	50	117		0.74	823 Berm
26U3B	49	241		0.53	
26U3C	48.5	379		0.43	
1001-U2-T	235	552		0.53	1975-1001 at 1000 Berm
1001-U2-B	236	827		0.43	
1001-U3-T	241	552		0.53	
1001-U3-B	242	827		0.44	
C1	111	621		0.52	86-115 866 Berm
C2	111	1028	0.56	0.37	
T1	111	616		0.45	
T2	111	859		0.43	
1W	49'	206	1	0.81	87-151 866 Berm
2W		82		1.53	
89	45.3	700		0.33	H89-1, TB
90-2	50.0	600		0.34	H90-7A,TB
90-3	50.5	500 (ocr 1.2)		0.50	
90-1-I	49.2	360		0.62 inclined sample	H90-9B,TB
90-2-I	49.5	360		0.62 inclined sample	H90-9B, TB
Direct Simple Shear Tests					
DSS-1	50	320	n/a	0.39	H89-1 823 TB
DSS-2	46.5	360		0.32	
DSS-3	46.8	500		0.35	
DSS-4	49.3	402		0.37	H89-3A 823 TB
DSS-5	49.4	450		0.37	

However the inclined tests themselves and the DSS tests do have some odd behaviour which may in part be due to the structure of the soil which is in highly banded reflecting an overbank origin. What one sees are thin layers in the order of mm to cm thick of silty clay, clayey silts and silts with occasional fine sands layers from a few to several mm thick. It is inferred that shearing in the field and the lab takes place along bands with higher clay content and certainly the failure planes have a slick clayey appearance, see Figure 13. A comparison of the two inclined tests and the 5 DSS tests indicates undrained strength ratios of 0.62 and in the order of 0.32 - 0.39, respectively. If both tests are measuring strength along a horizontal plane, the difference is substantial. This may reflect different stress history and variability as the samples were obtained from different boreholes, but in close proximity. Following Wroth (1984) it is possible that the strengths being measured in the DSS tests are on a sub-vertical plane and this complicates a simplistic interpretation. The difference between the results has not been resolved and while this may be of research interest it is of lesser practical importance as the design has focused on conservative interpretations of the data.

PERFORMANCE DATA

The performance of TID has been continuously monitored since construction began. Until 1976, the emphasis was on measuring pore pressures in the tailings sand and foundation to support limit equilibrium stability calculations. In 1976, inclinometers were installed to monitor horizontal movements in support of the decision to raise the dyke to a height of 320 feet. This section provides a brief review of the performance data.

Pore Pressures

In 1993, there was a total of 250 piezometers in operation at TID. They were a combination of sealed standpipes, open standpipes, and pneumatic piezometers. Figure 15 shows cross sections through the critical section of TID from 1976, 1980, and 1993 that summarize the pore pressure information. The location of the phreatic surface is shown with the distribution of pore pressures beneath the phreatic surface plotted as a pressure head. The main features to note on this figure are:

- The pore pressures beneath the clay foundation are controlled by the Athabasca River level which is typically at about elevation 770 feet.

- The pore pressures in the clay resulted from dyke loading. Significant excess pore pressures were developed in the 1970's where the clay was thickest. Near the toe, where the clay was relatively thin and overlain by the initial portions of the original overburden dyke, there was little buildup of excess pore pressures. In the thicker clay deposits, the excess pore pressures remained until the late 1980's. The plot for 1993 shows that all excess pore pressures had dissipated in the clay foundation.

- The pore pressures in the tailings sand were less than hydrostatic because of the permeability anisotropy in the tailings sand and, after the late 1980's, because of downward seepage into the foundation clay.

- In general, the internal drains were effective in controlling the phreatic surface. Some seepage did exit on the surface of the dyke near the toe, however, the exit gradients were low.

- By 1980, it was evident that the phreatic surface beneath the crest of the dyke was much lower than the pond level. Investigation in the 1980's revealed that a significant unsaturated zone existed beneath the pond. This was due to the formation of low permeability layers at the base of the pond that acted as an upstream blanket. As shown in the 1993 section, the phreatic surface beneath the crest of the dyke was about 100 feet below the pond level. The phreatic surface beneath the crest has been dropping at a rate of 5 to 10 feet per year.

Although the excess pore pressures in the foundation clay have dissipated, the pore pressures are in a quasi-steady state condition because of the changing pore pressure conditions at the top of the clay. As the phreatic surface continues to decrease in elevation, the pore pressures at the top of the clay also reduce. This adjusts the steady state pore pressures in the foundation clay.

Settlement

No direct measurements of settlement of the dyke have been made. However, an estimate of the change in elevation of the top of the foundation clay was made using the three methods outlined in Table 6. This was for the channel infill sequences upstream of the 910 berm where the foundation clay was typically about 50 feet thick before construction.

Table 6. Summary of TID Settlement Determinations Upstream of 910 Berm

Method of Calculation	Settlement Range
Comparison of top elevation of boreholes from the 1964 original site investigation and recent data.	11 to 13 feet
Settlement based on measured water content changes.	9 to 12 feet
One dimensional consolidation theory based on C_c =0.33	9 to 11 feet

As seen in Table 6, the dyke settled about 9 to 13 feet.

Horizontal Movements

A total of 40 locations have been instrumented with inclinometers since 1976. Several of these locations have seen more than one inclinometer as installations became inoperative or deformations lead to unreliable readings. Figure 16 is a cross section through the critical section that summarizes the accumulated movement measured for the oldest inclinometers. A preliminary deformation analysis suggested that the horizontal movement since 1976 constitutes about one quarter to a third of the total horizontal movement, and this is supported by deformation analyses reported by Morsy (1994), see Figure 21.

As can be seen on Figure 16, the majority of horizontal movement has been in the foundation clay. The movement pattern indicates shear straining in the clay and no localized shear surface. This is consistent with the absence of a continuous slickensided layer or zone in the foundation clay as determined by the field investigation programs.

While accumulated movement data has been of interest, more emphasis has been placed on the magnitude and rate of shear straining in the clay because of the movement patterns. The shear strain over each two foot increment of the inclinometer readings was calculated and expressed in radians as a percentage. The shear strain was determined between successive readings and the maximum rate calculated. Table 7 summarizes the maximum shear strain rate as percent radians per year. For comparison, the elevations of the crest of the dyke, the pond level, and the upstream beach are also shown. The inset to Figure 16 provides a summary of shear strains at the top of the clay layer.

There are several general conclusions.

- Total horizontal movements are greater in the thicker clay upstream of the 910 Berm.
- Maximum shear strain rates in the earlier years recorded in Table 7 tended to be in the middle of the clay consistent with the zones of highest pore pressure.
- Maximum shear strain rates now are highest at the top of the foundation clay unit.
- Shear strain rates are tending to reduce and in some cases have approached zero.
- Of some interest is the backtilting observed in some inclinometers upstream of 910 Berm, see Figures 16 and 17. This movement pattern is discussed subsequently.

Table 7. Summary Of Maximum Shear Strain Rate In Foundation Clay

	Inclinometer	Maximum Shear Strain Rate (%/Year)						
		1976-1977	**1979-1981**	**1981-1983**	**1983- 1984**	**1985-1987**	**1989-1990**	**Current**
Dyke Elev. Pond Elev. Beach Elev.		1020-1050 995-1020 990-1020	1065-1080 1025-1040 1030-1048	1080 1040-1055 1048	1080 1055-1061 1048-1072	1088 1066-1069 1072	1082-1055 1072 1074	1082-1055 1072 1074
1080 Berm	S81-110	--	--	--	0.10	0.10	--	--
1000 Berm	S81-101C/S85-101D S85-111	1.2 --	0.7 --	0.35 --	0.28 --	0.15 0.13	0.07 0.04	≈0.0 0.04
955 Berm	S79-109 S76-103,S81-103	-- 1.3	-- 1.2	0.14 0.36	0.25 0.40	0.11 0.20	0.07 0.17	0.03 0.09
910 Berm	S79-106, S90-106A S79-102/S85-102A S85-112	-- 0.6 --	-- 0.4 --	0.21 0.28 --	0.28 0.38 --	0.15 0.15 0.11	-- 0.07 0.12	0.05 0.04 0.05
866 Berm	S85-113 S85-115	-- --	-- --	-- --	-- --	0.14 0.15	0.14 0.13	* 0.05
823 Berm	S76-104 S76-105/S85-105A S85-114	0.5 0.75 --	0.29 0.55 --	0.25 -- --	0.33 -- --	0.17 0.16 0.12	0.12 0.10 0.10	<0.04* 0.05 0.05
TOE	S79-107 S79-108	-- --	-- --	0.29 0.16	0.25 0.16	0.30 0.0	toe berm construction	0.08 0.07

* TREND NOT CLEAR -- Slope Indicator not installed or being replaced

STABILITY

Since the beginning of the design process the factor of safety of TID has been predicted using effective stress methods, but with total stress methods being an additional consideration. In the early years, the total stress approach considered a c_u, ϕ_u method, but this approach has been discontinued. A summary of the factors of safety at different stages in the design of TID is given in Table 8.

Table 8. Summary of Factor of Safety Calculations

Case & Time Period	Foundation Parameters Operational & (Full Residual)	FOS Operational (Full Residual)	Comments
Up to 1984	U/S 910 Berm $c'=0, \phi' = 24°$ D/S 910 Berm $c'=0, \phi'=26.5°$	1.42 to 1.62	Factor of safety varied as pore pressure in foundation dissipated
Post 1984 pre Mitigation	U/S 910 Berm $c'=0, \phi'_r = 11°$ 910-866 berm $c'=0, \phi'_r = 15°$ D/S 866 Berm $c'=0, \phi' = 24°$, ($\phi'_r = 15°$)	1.21 (1.13)	Pre mitigative measures with observed pore pressures
Post Mitigative Action 1990		1.32 (1.22)	Due to toe berm construction & pore pressure dissipation
1994		1.40 (1.29) { 1.97 }[1]	Phreatic surface dropping 1 Full peak from 1984
1994	$S_u / \sigma'_{vc} = 0.33$ [use $\phi' = 18.3°$]	1.62	Undrained comparison, but not used as design basis
	U/S 910 $S_u / \sigma'_{vc} = 0.25$ [use $\phi' = 14.0°$] D/S 910 $S_u / \sigma'_{vc} = 0.33$ [use $\phi' = 18.3°$]	1.48	

Any stability analysis requires a model of the stratigraphy within which to generalize foundation elements and assign values to strength parameters. The facies analysis provides a framework within which to discuss this process. Figure 18 provides a schematic representation of the foundation stratigraphy in two locations (Stations 56+00 and 65+00) close to the critical section taken from Catto (1988). The original ground in the toe area is an overbank sequence of Facies 2 from near the 866 berm and downstream. This sequence is associated with flood stage deposition from the modern Athabasca River immediately to the east of TID. The lower overbank Facies 2 from between 866-823 to the toe were deposited by overbank sedimentation marginal to a former river channel located to the west of the 866 berm at a lower elevation. This channel was then infilled during a series of phases leading to its eventual abandonment. At Station 65+00 the boundary between the overbank and channel infill sequences is between the

965 and 910 Berms and for station 56+00 somewhere between the 910 and 866 berms. This complex stratigraphy has been modelled by the cross-section geometry shown on Figure 19. This model is in general agreement with the subsequent facies work and with a faster rate of pore pressure dissipation in the toe area.

Up to 1984

The initial stability analysis assigned effective stress parameters to the foundation clay as given in Table 8. The parameters were selected differentiating between slightly more favourable conditions downstream of the 910 berm based on index parameters and soils description and the material upstream of the 910 berm. The peak friction angles adopted can be compared to the data presented earlier (Table 4, Figure 9 to 11) and it can be seen that a conservative bound was used.

Post 1984 and Prior to Implementing Mitigative Measures

As previously discussed, it was concluded that the design and operation had to assume that the presence of slickensides could embrittle the foundation, and while it was highly unlikely that a continuous band of slickensides could be present, the strength available had to be downrated. The process adopted for this decision was as follows.

Firstly the foundation clay was differentiated into one channel infill zone upstream of the 910 berm and a lower and upper clay of overbank sequences downstream of the 910 berm. Based on the slickenside mapping it was decided to downrate the clay strength to a residual value for locations upstream of the 866 berm. Because of the occasional presence of partially developed slickensides beneath the 823 berm, stability analyses were carried out for both peak and residual operational strength between the 866 berm and the toe in order to define the possible range of values of factor of safety as an aid to the overall geotechnical decision making process.

The operational peak friction angle between the toe and the 866 berm was downrated to 24° from the 26.5° used in previous analyses.

The residual friction angles were selected based on the correlation with liquid limit shown earlier (Figure 11). The design liquid limit for the two depositional environments was obtained from a zonation of some 169 liquid limit determinations, and was taken as the maximum liquid limit continuous through a horizontal plane in the depositional sequence. These limits are reported in Table 9. The design liquid limit w_L of 65% for infill sequence was exceeded in only 5 of 115 tests and the w_L of 54% in the toe was an upper bound to 54 determinations. The full residual angles of 9° and 13° were selected on the basis of the previously discussed correlation, see Figure 11.

Table 9. Selection of Residual Parameters for Foundation Clay

BERM	FACIES	DESIGN LIQUID LIMIT	FULL RESIDUAL ANGLE	DESIGN RESIDUAL ANGLE
Crest - 910	Channel Infill	65%	9°	11°
910 - Toe	Overbank	58%	13°	15°

Design residual angles were then selected 2° higher in consideration of the immature development of the observed slickensides and the roughness of any possible plane connecting elevations of slickensides in the foundation units. It also accounted for the local presence of the silty sludge unit previously discussed. This was considered to be a highly conservative but prudent zonation.

On this basis and as reported in Table 8 the design operational factor of safety was now assessed to be as low as 1.21, and 1.13 for the design case using full residual angles. The actual factor of safety was considered to be somewhere between the original peak strength basis of over 1.6 (as pore pressures were continually dropping) and the operational basis of 1.2 (Table 8). It was considered that the cost of determining the exact nature of the foundation to refine the factor of safety in more definitive terms would be excessive and even for such a program some uncertainty might still remain. As heavy equipment was available within the mine, the cost of mitigative action was reasonable and the latter course was recommended and adopted.

Post Mitigative Action 1989

The mitigative method considered to be most effective required a combination of crest regrading to reduce driving stresses and a modest toe berm. The first phase required a reduction in driving stresses by regrading the crest from elevation 1082' down to 1055' and extending out the 823 toe berm as shown on the cross-section of Figure 19. As will be discussed a second phase requiring additional crest regrading to 1045 and extending the toe berm to 866 was not undertaken. The first phase mitigative operations resulted in an increase in the operational factor of safety to 1.32 by the end of 1989.

It was anticipated that one consequence of berm construction would be an increase in shear strain rates in the toe area and this was the case. In view of the potential brittleness of the design critical region between the toe and the 823 berm, it was decided to investigate the state of the overbank sequence in this region. As previously discussed, samples of the clay were obtained and tested after the berm was constructed. Several observations were made.

1. No slickensides were observed in any samples or in the trimmings from samples being prepared for strength testing confirming earlier observations.

2. Both the inclined drained and undrained triaxial tests preferentially failed along bedding and exhibited post peak brittleness particularly in the drained tests.

3. There was some anomalous response between the different types of tests which could not be resolved.

4. Thin bands of higher clay concentrations are present in the clay unit and these strongly influence its behaviour. These layers have likely been the focus of most of the gross shear strains as measured by the inclinometers and therefore have been previously subjected to an even higher level of localized shear straining than measured by the gross inclinometer measurements.

The second phase was potentially attractive as it offered the possibility of obtaining a higher factor of safety immediately. However, based on the implied brittleness of the toe clay it was recommended not to proceed with additional berm construction as there was concern that the potential additional straining could be counter productive. This possibility had been considered in the initial mitigative planning and was a major reason for the two stage approach.

1994 Conditions

It was recognized in 1989, when the geotechnical recommendation was made not to proceed with the second phase of the mitigative measures, that performance projections of the hydrogeological regime would predict more favourable conditions with time. By 1989, excess pore pressures in the foundation had dissipated and were in a steady state condition as previously discussed. As shown in Figure 19 a drop of 40 feet in the phreatic surface was predicted to result in an increase in the factor of safety to 1.42. By 1994 this condition had been essentially reached.

As previously discussed, the operational case selected was a conservative assessment of the effective stress parameters. It is considered that the actual factor of safety is somewhere between the operational and the peak case for which the factor of safety, as noted in Table 8, is now in the order of 1.97.

Undrained Conditions

An alternative model for the foundation strength is to assume that the observed slickensides are a localized phenomenon and that peak undrained conditions apply in what is now a largely normally consolidated soil. Ignoring, within this undrained model an overconsolidated zone near the toe a conservative lower bound of the data suggests a $S_u / \sigma'_{vc} = 0.33$ from the 910 berm to the toe. There is less data upstream of this point but a ratio of 0.25 is empirically selected to conservatively account for more clayey material. These are equivalent to friction angles of 18.3° and 14° respectively. The predicted factor of safety on this basis is 1.48. A more realistic lower bound estimate for the data of 0.33 below all berms predicts an undrained factor of safety of 1.62.

CREEP AND MOVEMENTS

A key feature of this case record is the deformations exhibited by the foundation elements co-existing with the stability of the structure. It has been considered that these movements are in part due to creep.

This subject has been considered in two research projects at the University of Alberta: a M.Sc. thesis by Watts (1981) who undertook a series of drained creep tests and more recently a Ph.D. thesis by Morsy (1994) who coupled Singh-Mitchell type creep function into an effective stress FEM code developed at the University. This work is reported by Morsy, Morgenstern and Chan (1995). Undertaking a deformation analysis of this structure is a challenging task in terms of the analytical framework. This is made more complex by the variable soil properties and geometry of the foundation elements both in plan and section which make it difficult to generalize a representative cross section.

Within the bounds of these generalizations this study has found that it is possible to make realistic predictions of the movement patterns. An example of the shear strain prediction made is given in Figure 20 which provides a comparison of the observed and predicted shear strain for 3 inclinometers at the toe, the 823 and 955 berms, adapted from Morsy (1994). Figure 21 provides movement predictions for the 823, 910, and 955 Berms, adapted from Morsy et al (1995).

The horizontal movement measured by the inclinometers in the foundation clay can be attributed to a combination of shear deformation and settlement. Shear stresses that develop as a result of dyke construction would induce shear straining in the foundation clay with the largest component in the horizontal direction.

Superimposed on this effect is settlement. In the thicker deposits upstream of the 910 Berm, there will have been considerable settlement both of a primary and secondary (i.e., K_o drained creep) nature. These settlements will not be solely vertical because they are caused by principal stresses which were both increasing and rotating as construction proceeded. For a vertically installed inclinometer, these movements will appear as downslope deformations and this mechanism is considered to be a dominant influence in the deeper channel infills. This mechanism also explains to the backtilting observed in Figure 17, and the relative proportions of creep to total predicted strains in Figure 20. Figure 20 illustrates the relative components of the deformation, as measured by shear strain at specific depths, between total and creep related deformation. It can be seen that the relative amount of deformation due to creep versus settlement varies as one proceeds upstream from the toe. These patterns are reasonable when one considers the relative proportion of clay upstream and downstream of the 910 berm which, as was previously discussed, is at about the interface between the thinner overbank soils and the thicker channel infills.

Downstream of the 910 berm the relative amount of deformation due to creep in terms of shear strain is greater as predicted in Figure 20.

The movement predictions in Figure 21 suggests that total horizontal movements since construction began are in the order of 0.25 m below the 823 Berm, 0.7 m at the 910 Berm and in the order of 1.0 m below the 915 Berm. These predictions confirm the differences in response upstream and downstream of the 910 Berm due to the different thickness of the deposits.

CONCLUSIONS

The construction and operation of Tar Island Dyke demonstrate the successful application of the observational method for a high dam in which the complex clay and peat deposits in the foundation were the main design consideration. The observational method provided enough flexibility to respond to the evolving nature of planning and construction requirements, while maintaining an optimized and secure design.

Another important aspect of the approach adopted for the analysis of the foundation materials was the detailed evaluation of the stratigraphy and geomorphology of the deposits. Understanding the nature of the various facies allowed for a meaningful interpretation of fewer test results instead of statistical interpretation of a significantly larger number of boreholes and tests. This also illustrates the complexities that can be associated with natural clay deposits and their importance to the geotechnical behaviour of these deposits.

A key feature of Tar Island Dyke has been the coexistence of movement and stable conditions. There are still slow ongoing movements today but they are considered to be due to creep. Drained creep movements imply a slowly reducing rate and it is judged that such a process is underway insitu. From a practical perspective, and given the sensitivity of inclinometer installations and the expected reduction of rate on a log time basis, considerable time must elapse until significant further decreases in rates are observed.

Tar Island Dyke performance has continuously improved since most construction activities have been completed. The structure will soon be decommissioned and the tailings storage facility will be abandoned as a reclaimed sand deposit.

ACKNOWLEDGEMENTS

The authors would like to express gratitude to Suncor Inc. Oil Sands Group, and in particular to Dr. Don Sheeran, P.Eng., for permission to publish this paper.

REFERENCES

Catto, N.R. 1988. Sedimentary Analysis, Tar Island Dyke Site. Report Prepared for Hardy BBT Limited by the Department of Geology, University of Alberta, Edmonton, Alberta.

Chan, W.K., Martschuk, J., and McRoberts, E.C. 1983. Performance review of Tar Island Dyke. Seventh Panamerican Conference of Soil Mechanics and Foundation Engineering. Vancouver, BC., pp. 553-566.

Chan, W.K., and McRoberts, E.C. 1989. Geotechnical aspect of waste disposal at Suncor Oil Sands Mine. Geotechnical Aspects of Tailings Disposal and Acid Mine Drainage, Vancouver Geotechnical Society, Vancouver, BC., May 26, 1989.

Design Review Panel. 1977. Great Canadian Oil Sands Tar Island Tailings Dyke. Report to Department of Environment, Province of Alberta.

Hardy, R.M. 1974. Tailings dams. E.I.C. Western Regional Tar Sands Conference, Edmonton, AB., April 17-19, 1974.

Mittal, H.K., and Hardy, R.M. 1977. Geotechnical aspects of a tar sand tailings dyke. Proceedings of the Conference on Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan, June 13-15, 1977.

Morgenstern, N.R., Fair, A.E., and McRoberts, E.C. 1988. Geotechnical engineering beyond soil mechanics - a case study. Canadian Geotechnical Journal, **29**: 637-661.

Morsy, M.M. 1994. Effective Stress Modelling of Creep Behaviour of Clay. Ph.D. Thesis, University of Alberta, Edmonton, Alberta.

Morsy, M.M., Morgenstern, N.R., and Chan, D.H. 1995. Simulation of creep deformation in the foundation of Tar Island Dyke. Accepted For Publication to Canadian Geotechnical Journal.

Watts, B.D. 1981. Lateral creep deformations in the foundation of a high dam. M.Sc. Thesis, University of Alberta, Edmonton, Alberta.

Wroth, C.P. 1984. The interpretation of in situ soil tests. Geotechnique **34**, No. 4: 449-489.

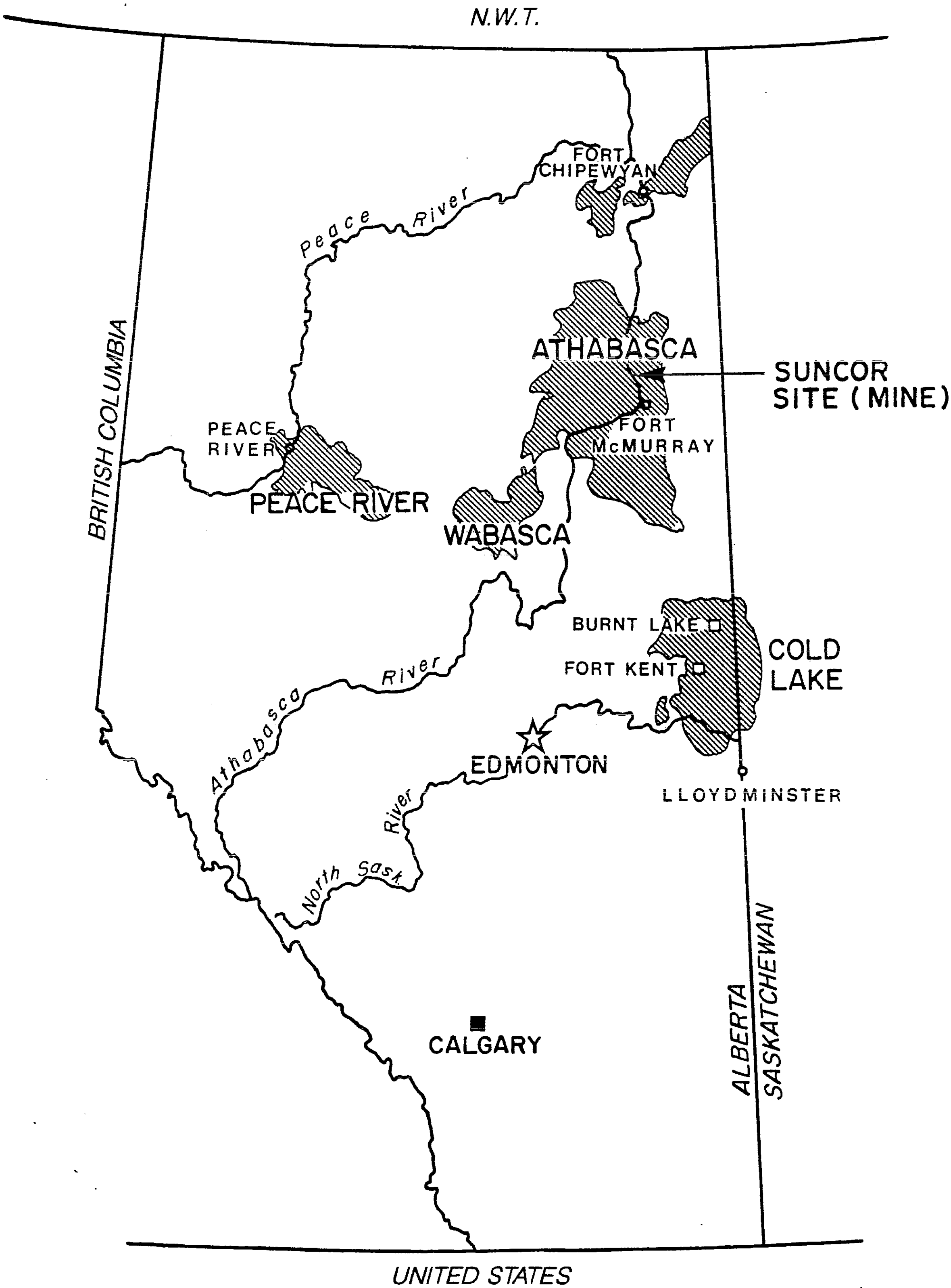


Figure 1. Alberta Map Showing Location of Suncor and Syncrude Leases

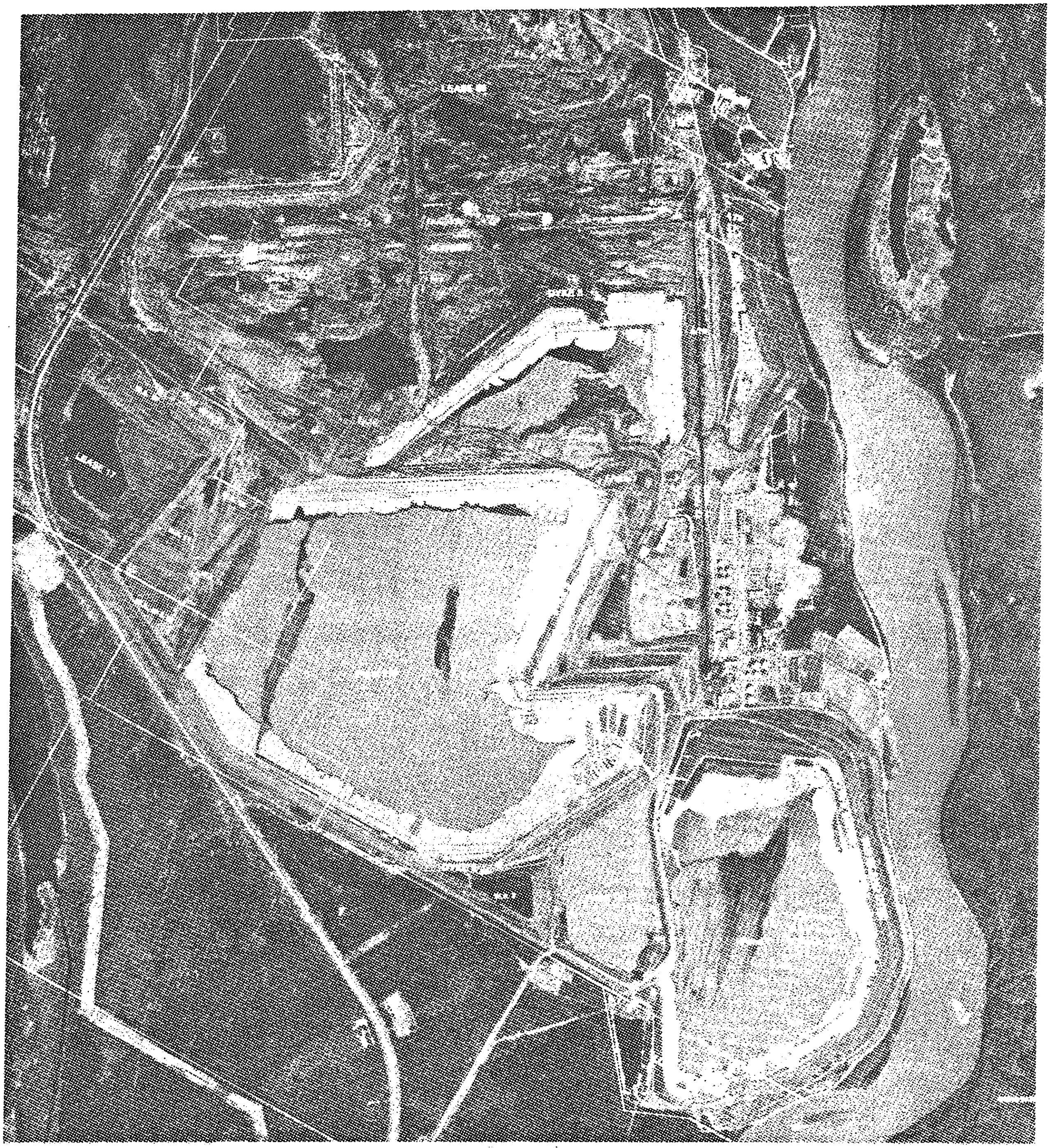
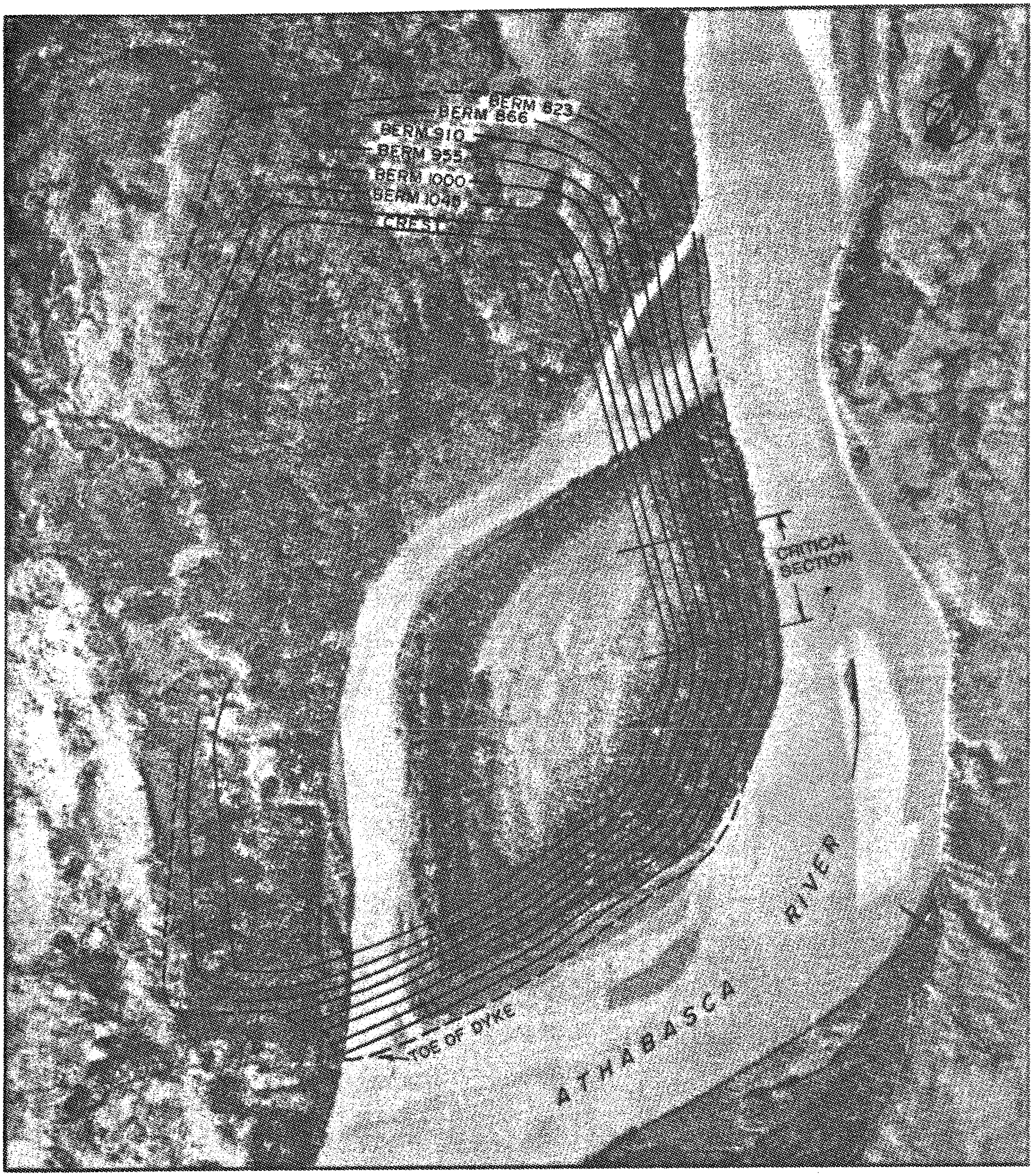


Figure 2. Suncor Site Plan. Tar Island Dyke
shown at the lower right hand corner.



Reference:
Govt. of Alberta Air Photo
No. AS 83-51, Flown Sept. 1950

0 1000 2000 3000 ft.

Figure 3. Aerial Photograh of Tar Island with Outline of Dyke

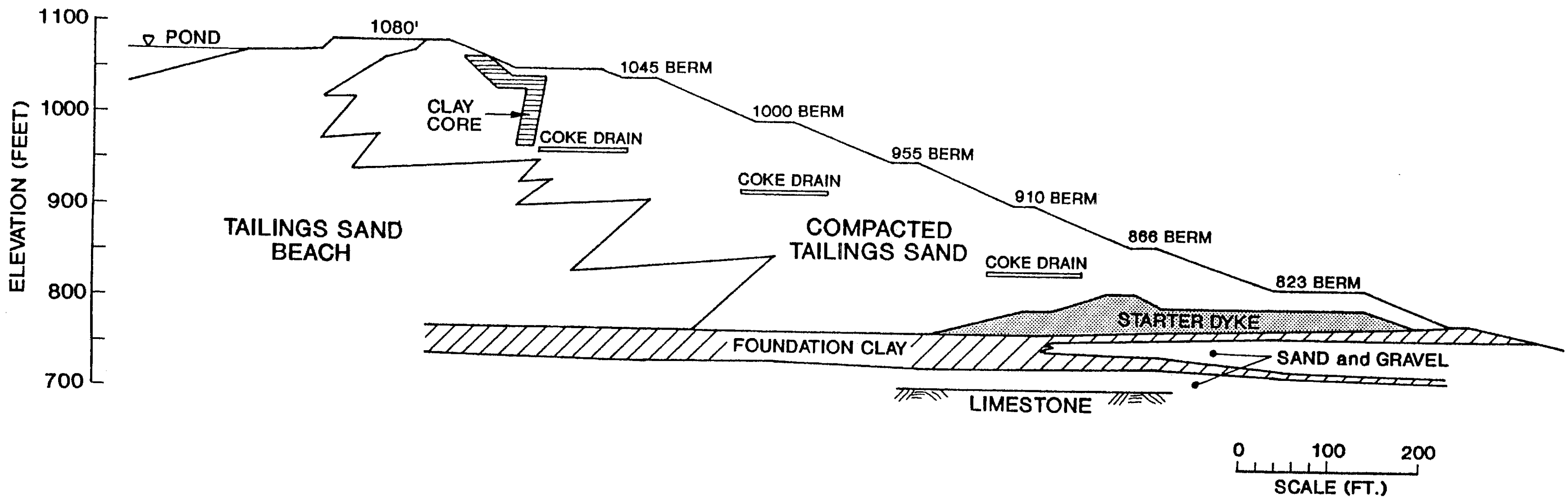


Figure 4. Cross Section of TID

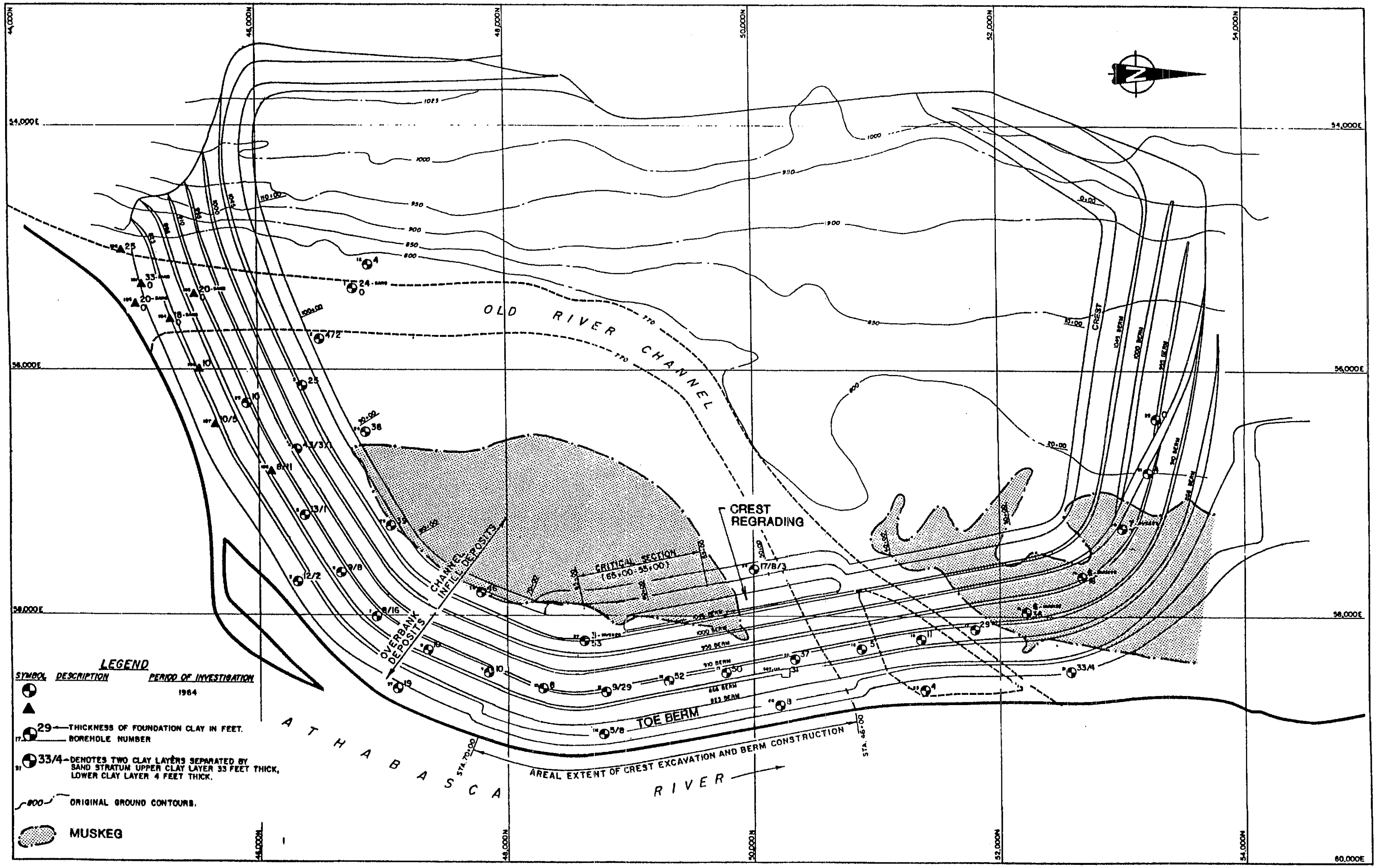


Figure 5. Plan of TID

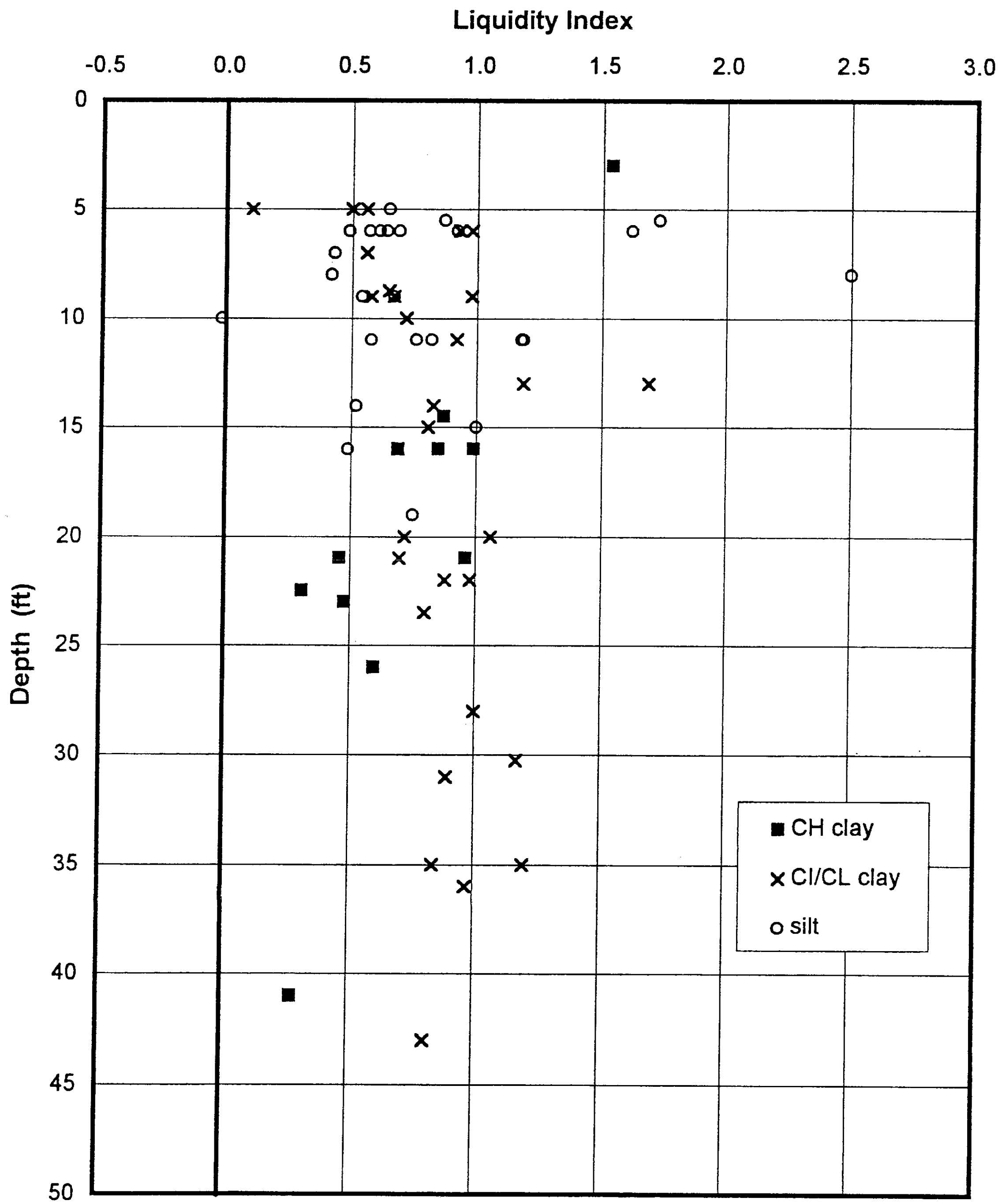


Figure 6. Liquidity Index Data From 1964 Investigation

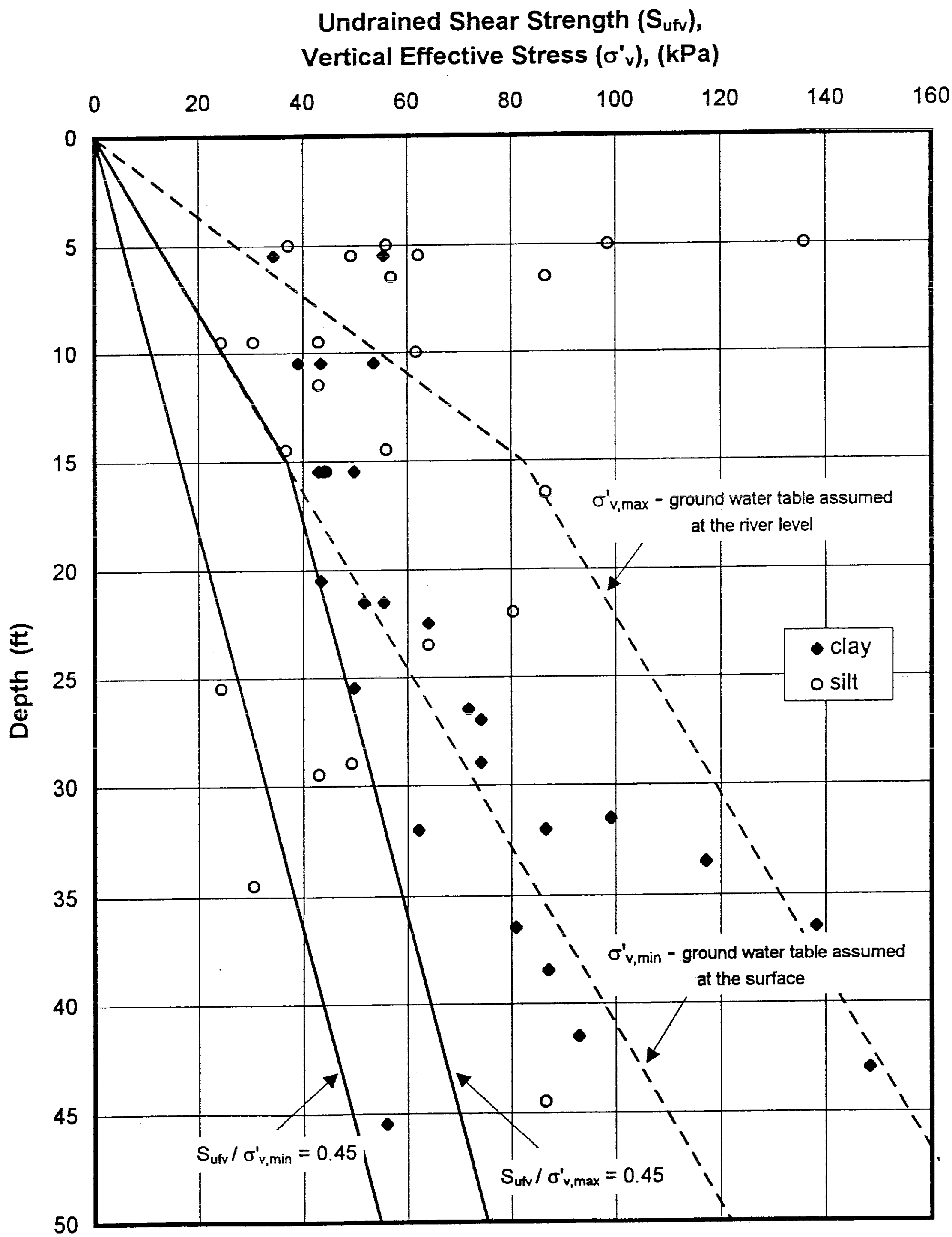


Figure 7. Field Vane Shear Test Data From 1964 Investigation

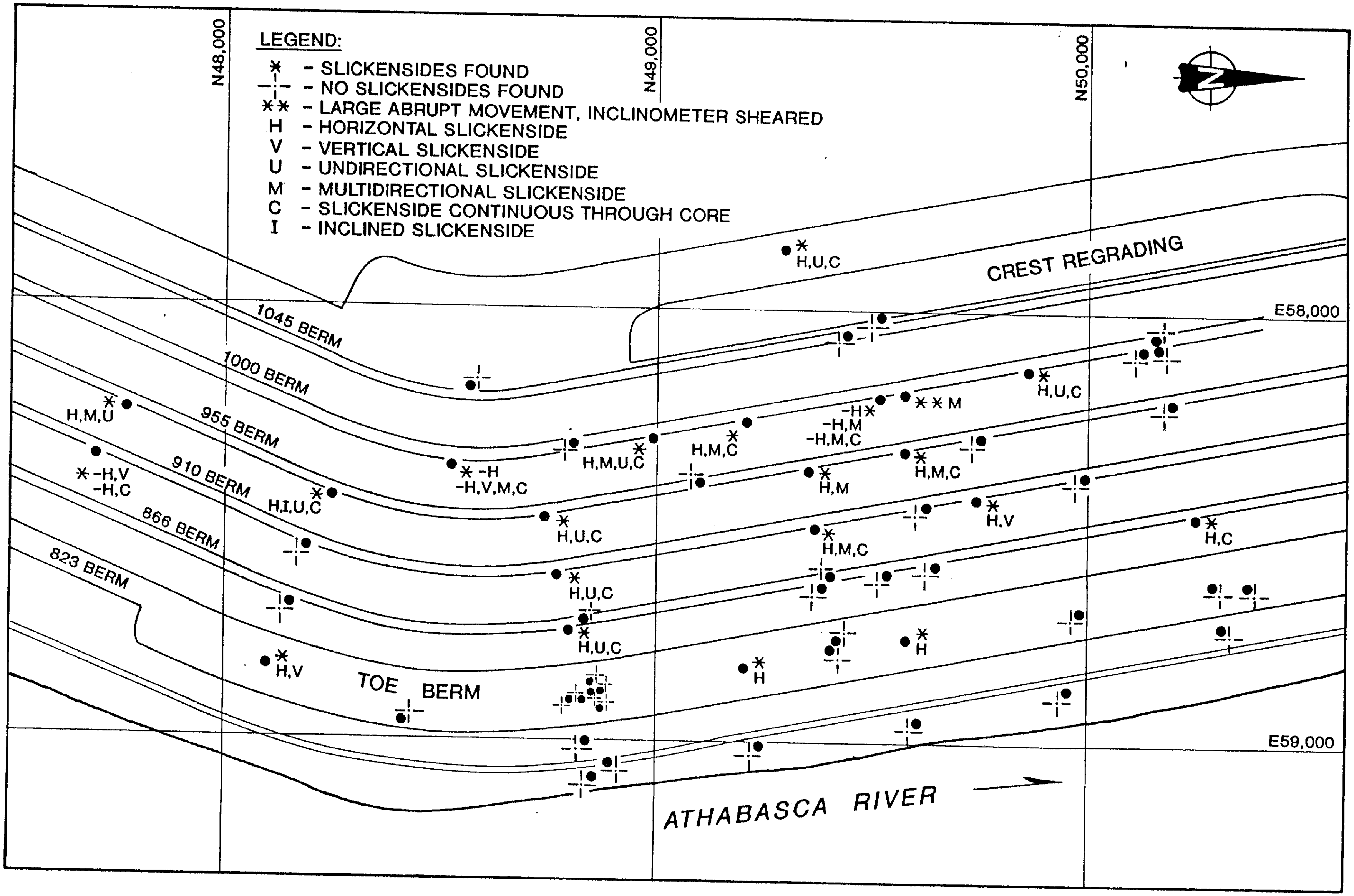


Figure 8. Location of Slickensides

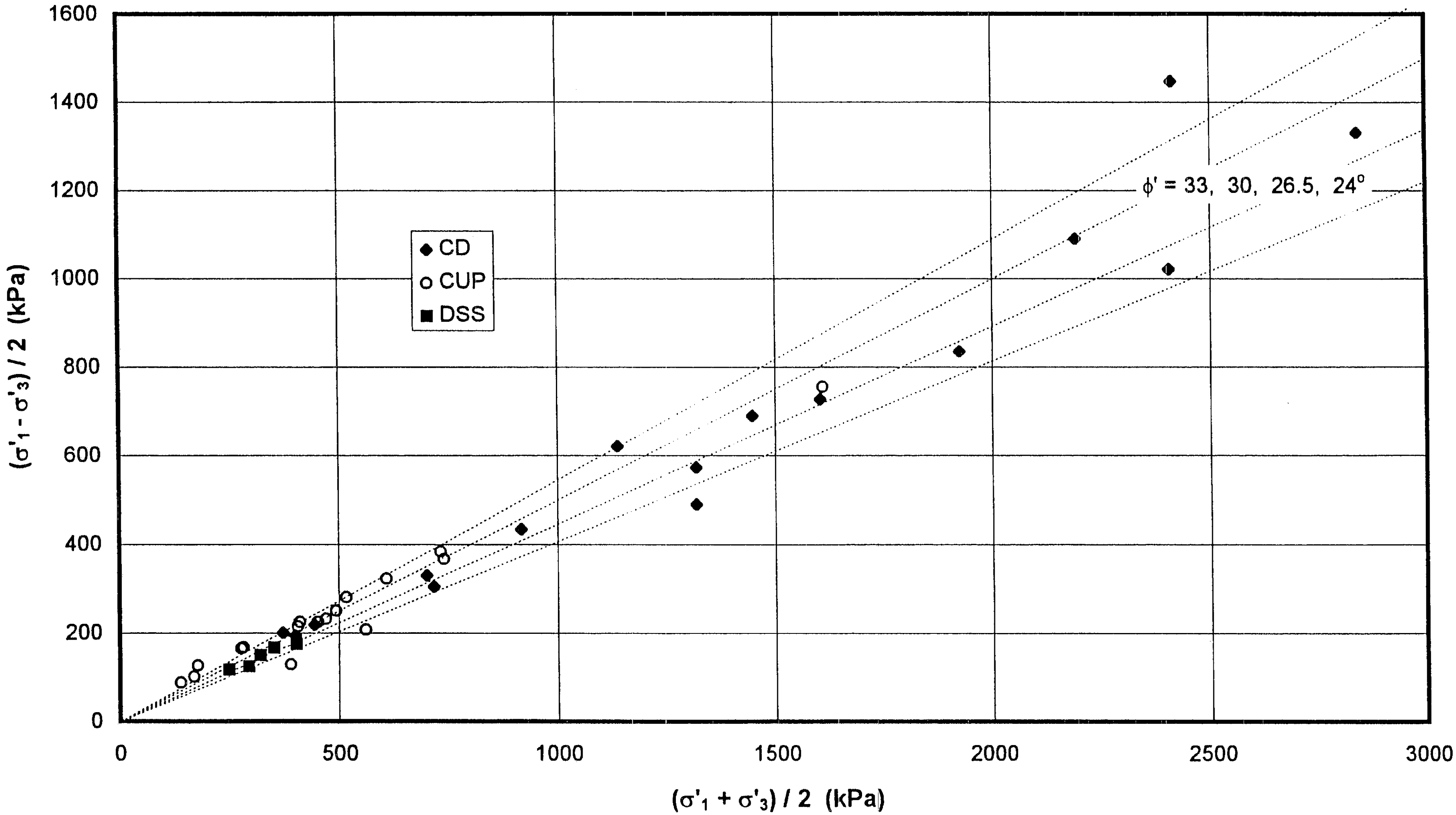


Figure 9. Results of Triaxial Tests (CUP and CD) and Direct Simple Shear Tests (DSS)

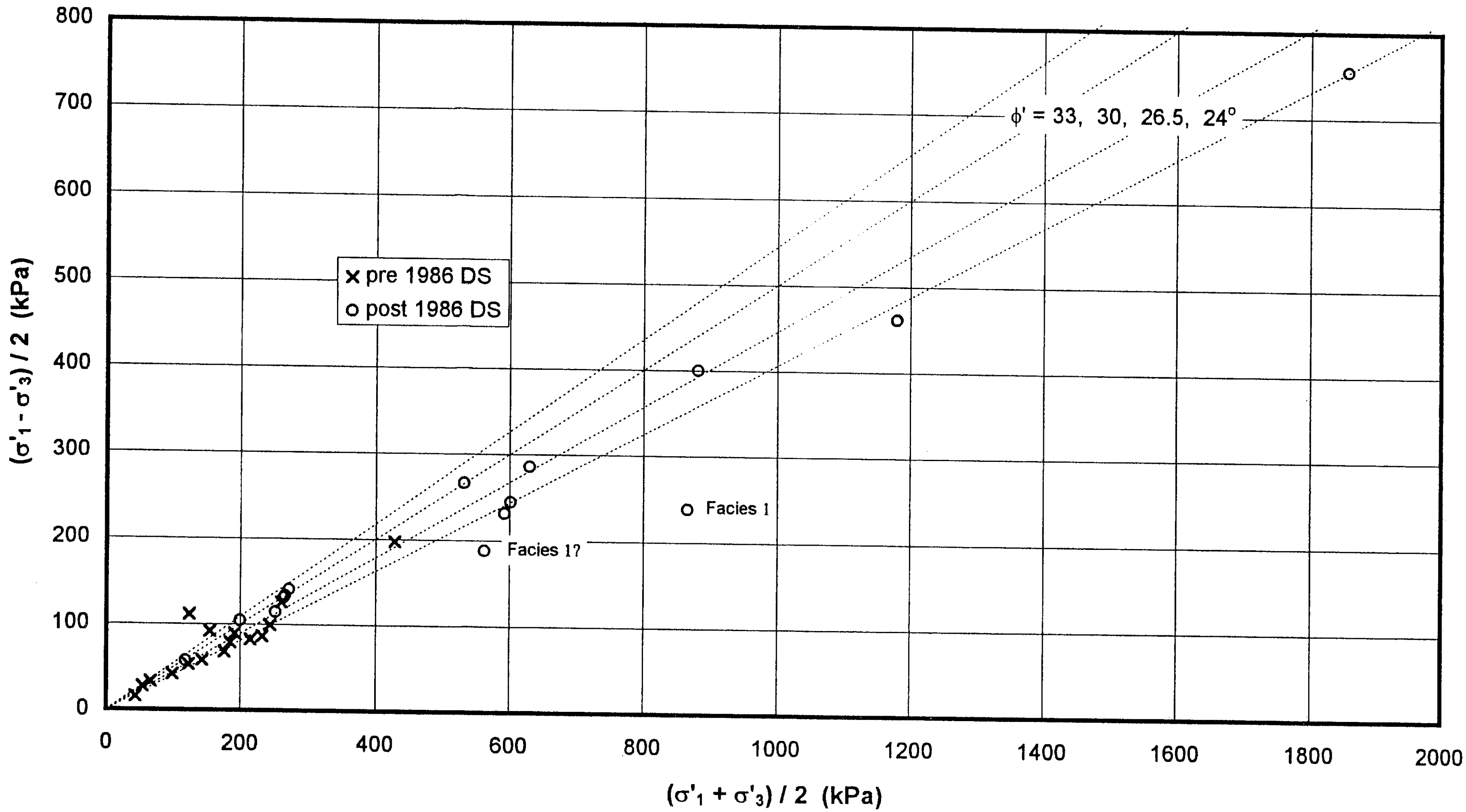


Figure 10. Results of Direct Shear Tests - Peak Strength

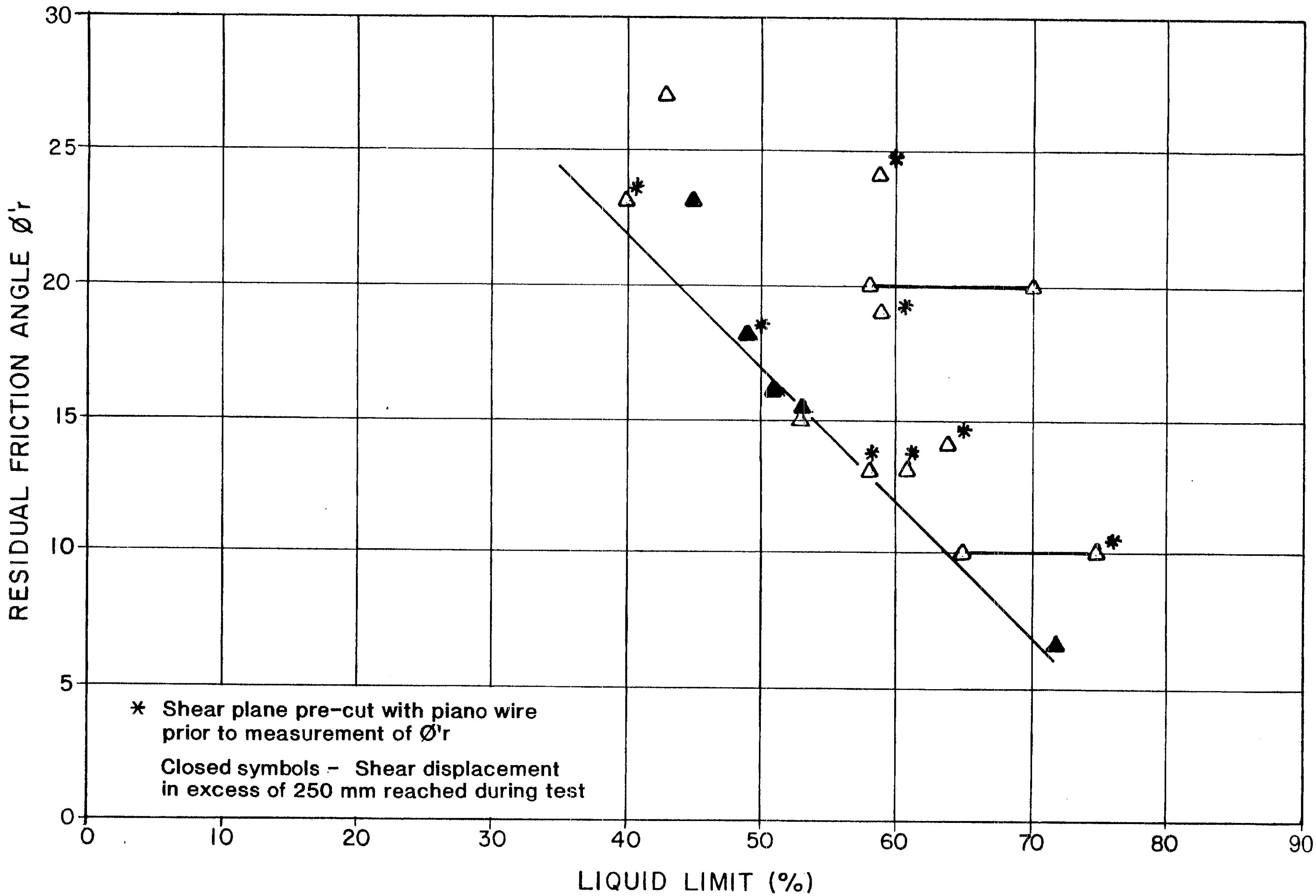


Figure 11. Design Correlation of Liquid Limit and Residual Angle

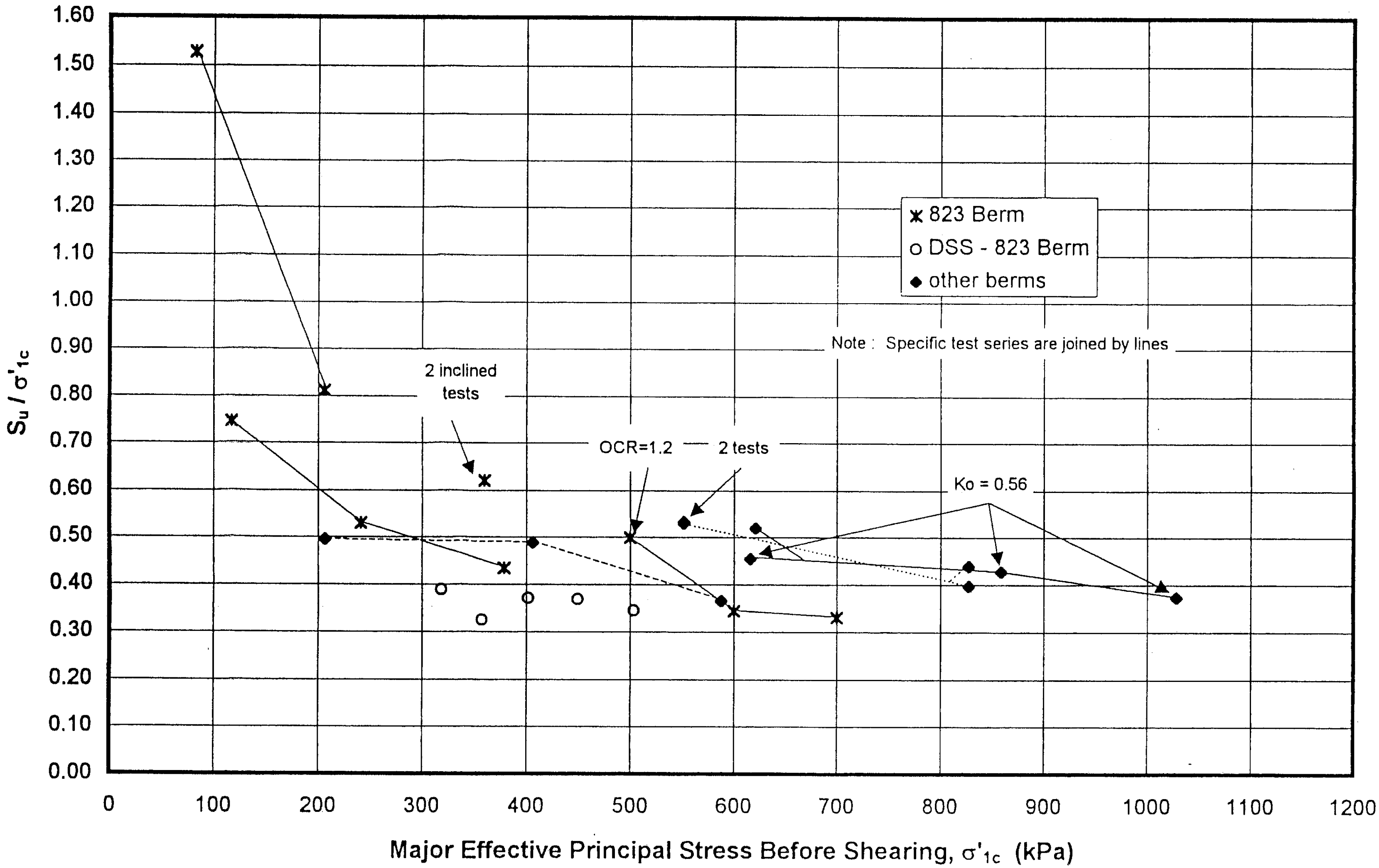
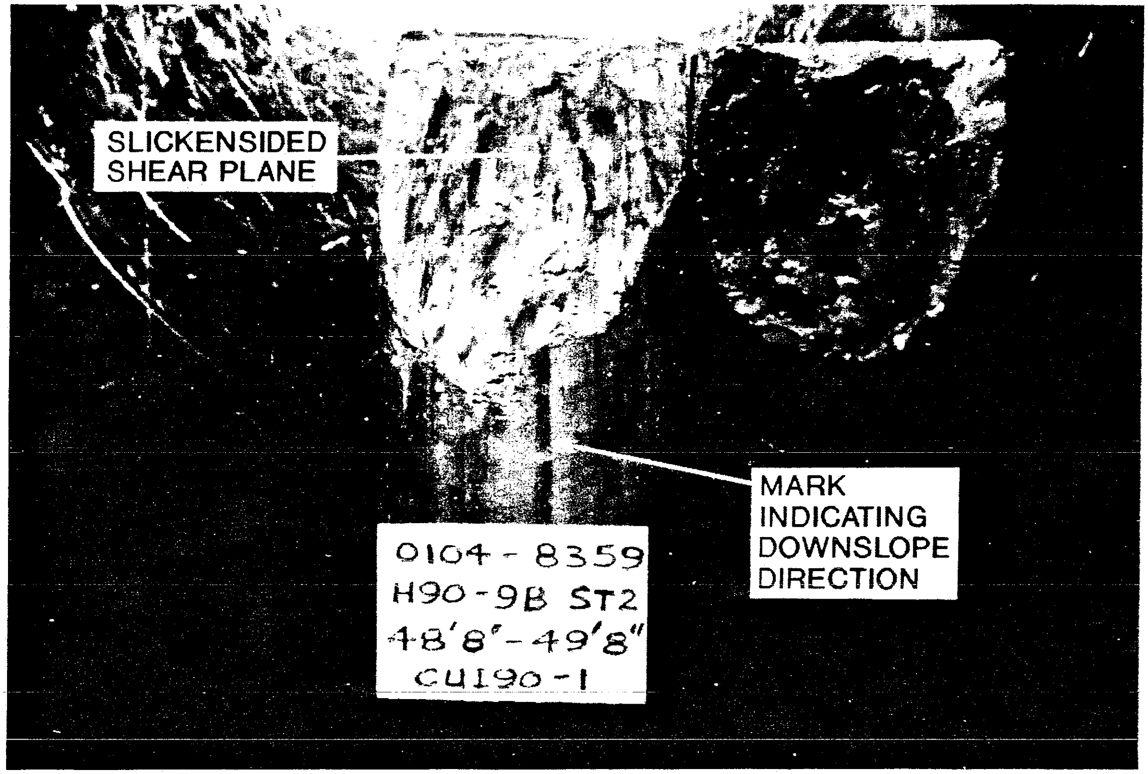


Figure 12. Normalized Undrained Test Results



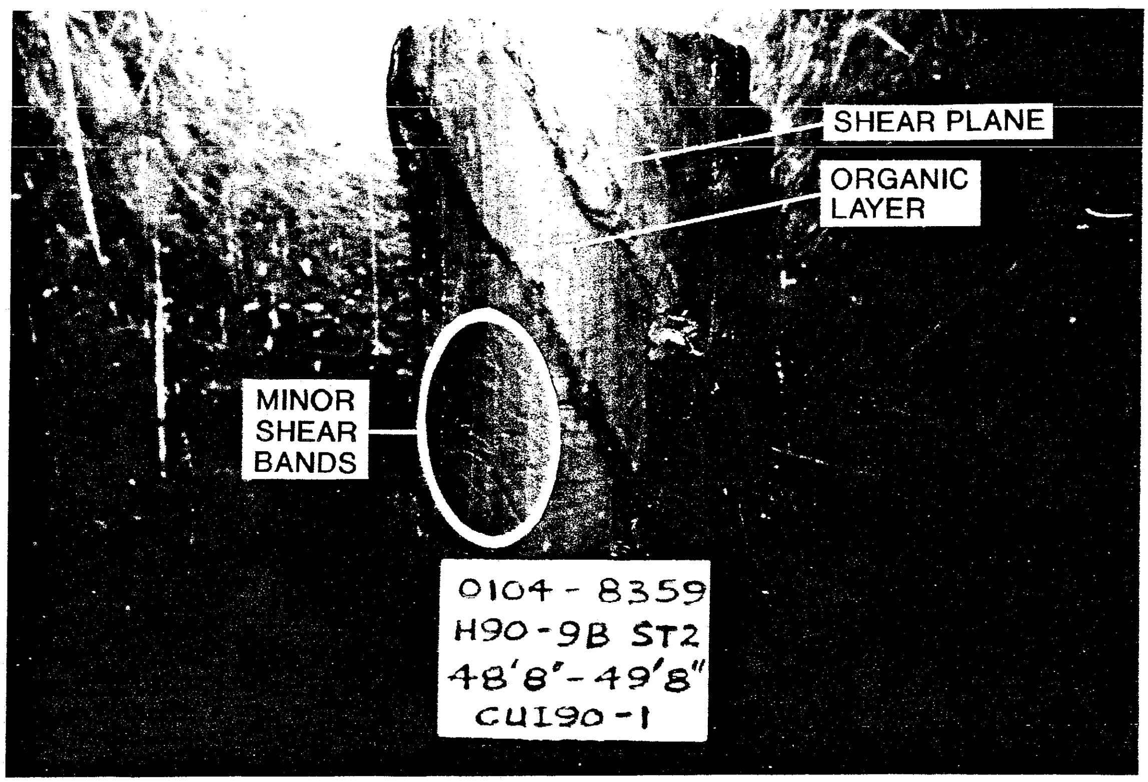


Figure 13. Photo of Inclined Sample

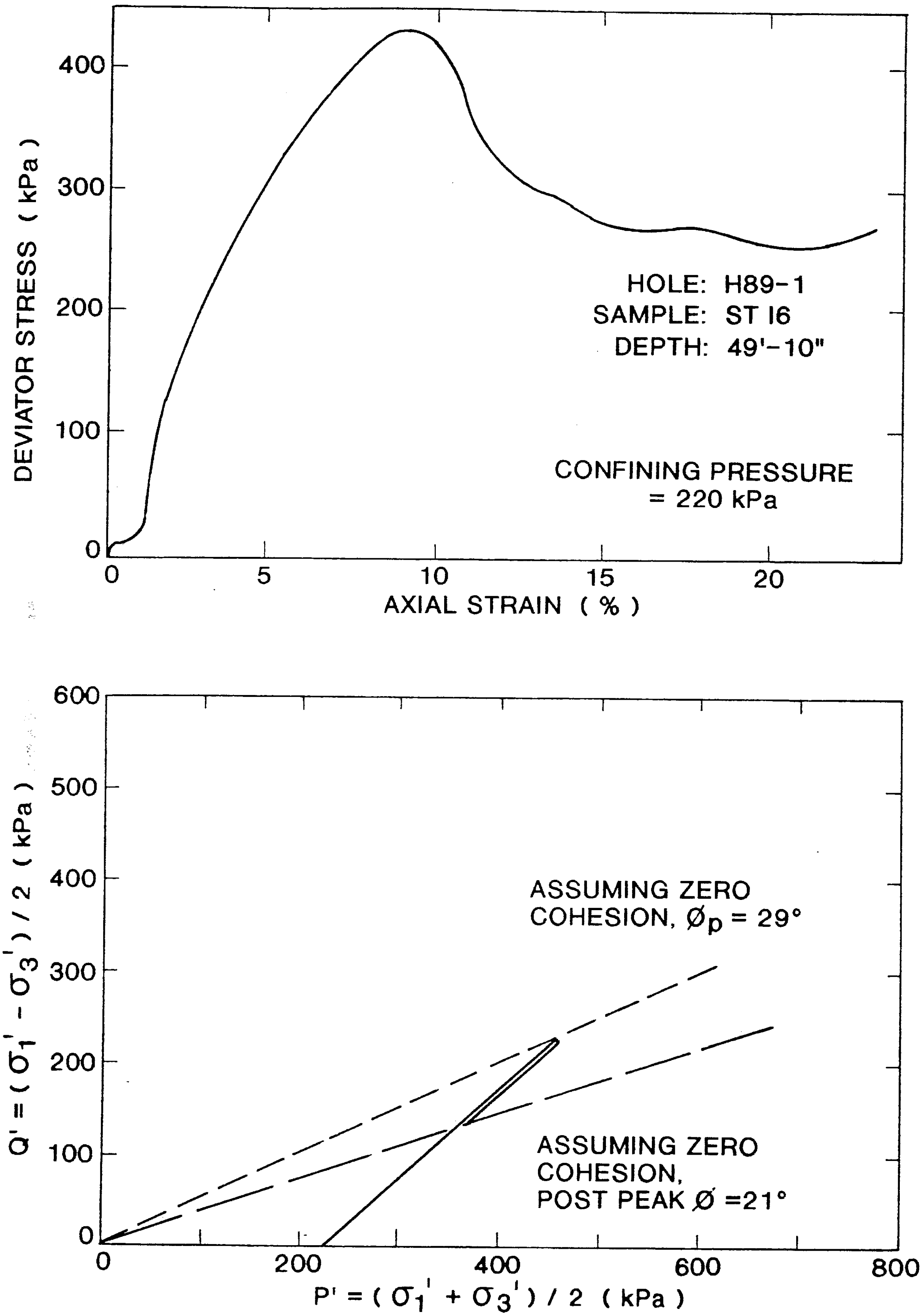


Figure 14. Results of Consolidated Drained Test on Inclined Sample

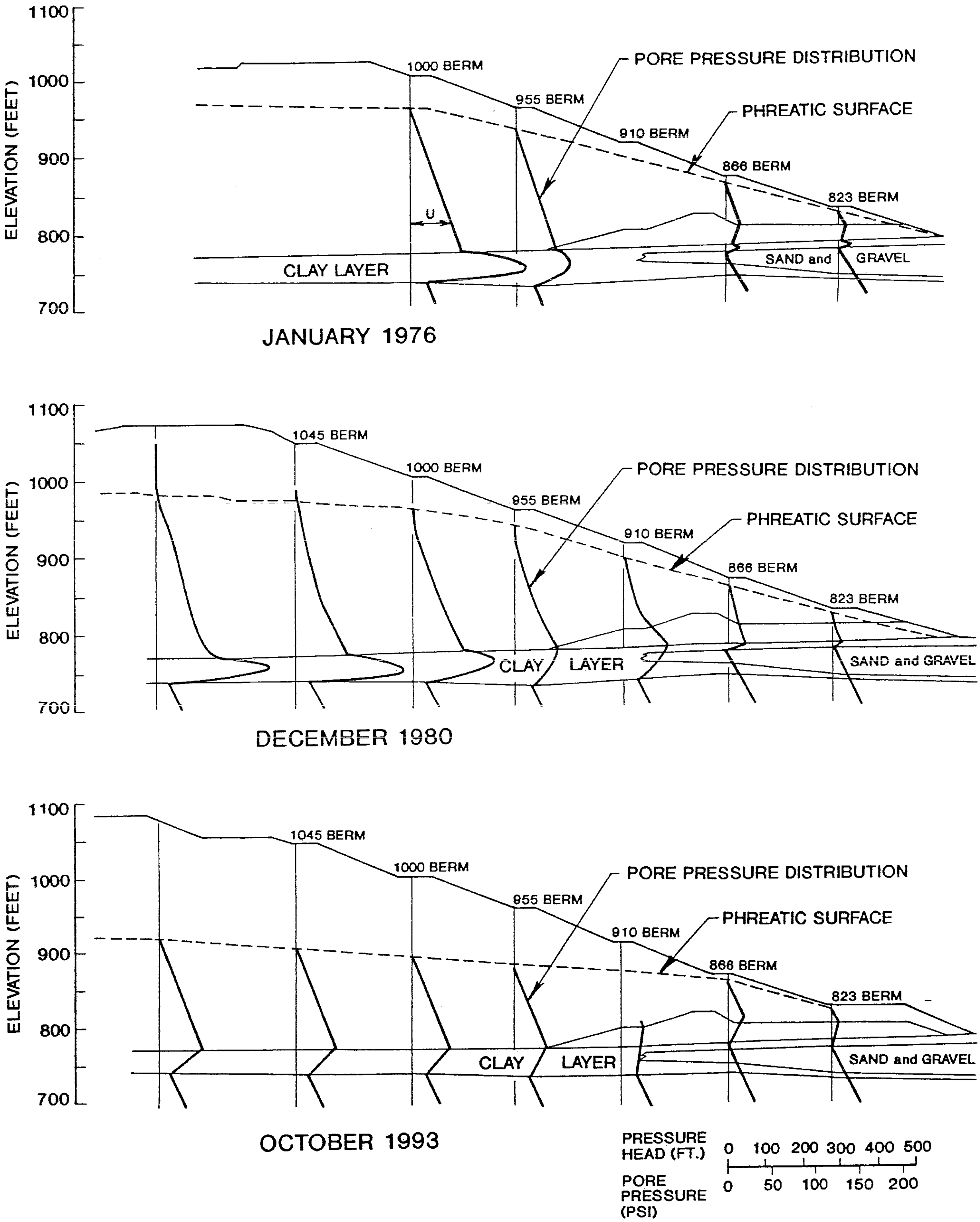


Figure 15. Piezometric Surfaces Summary

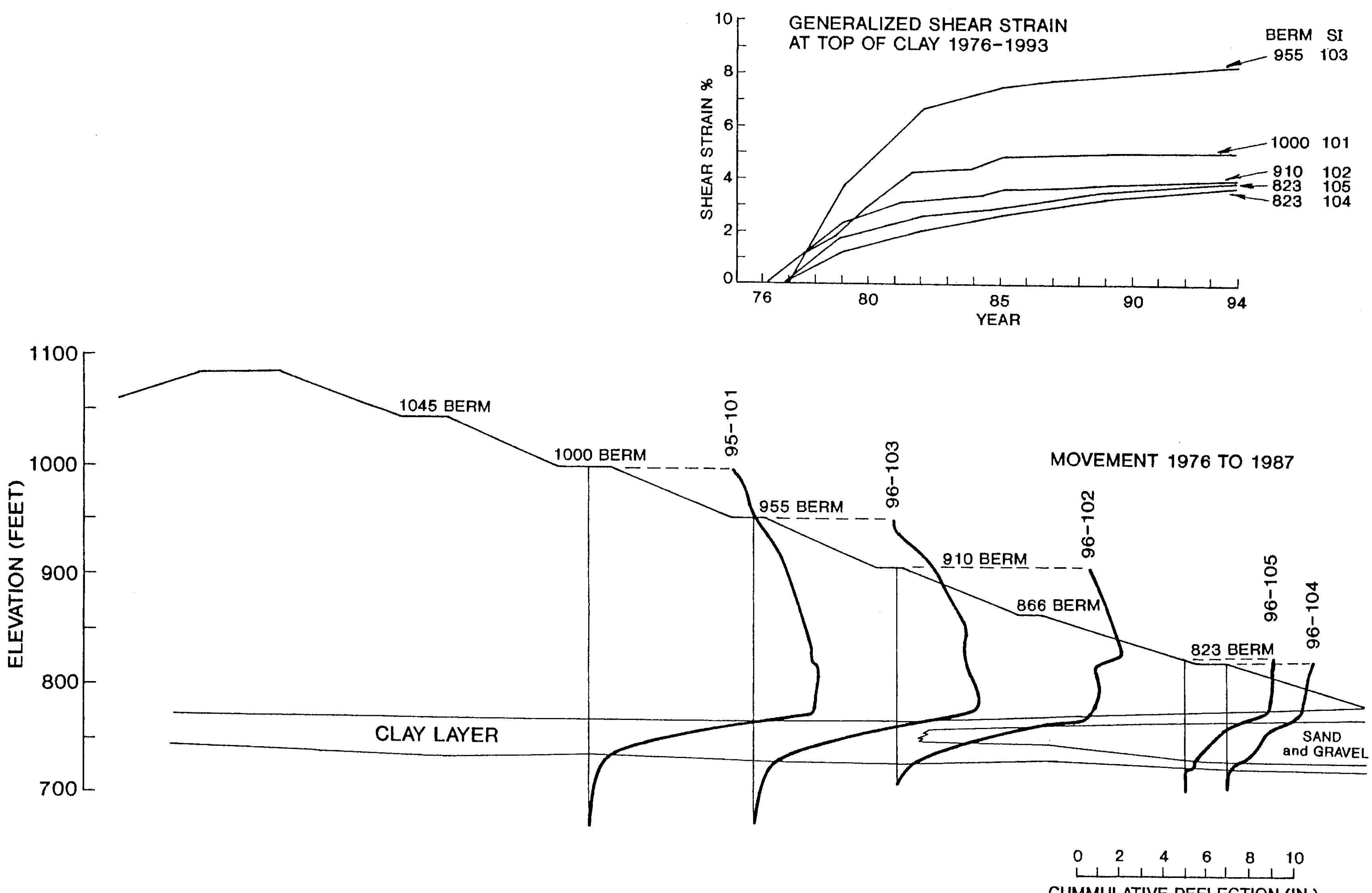


Figure 16. Summary of Slope Inclinometer Installed in 1976

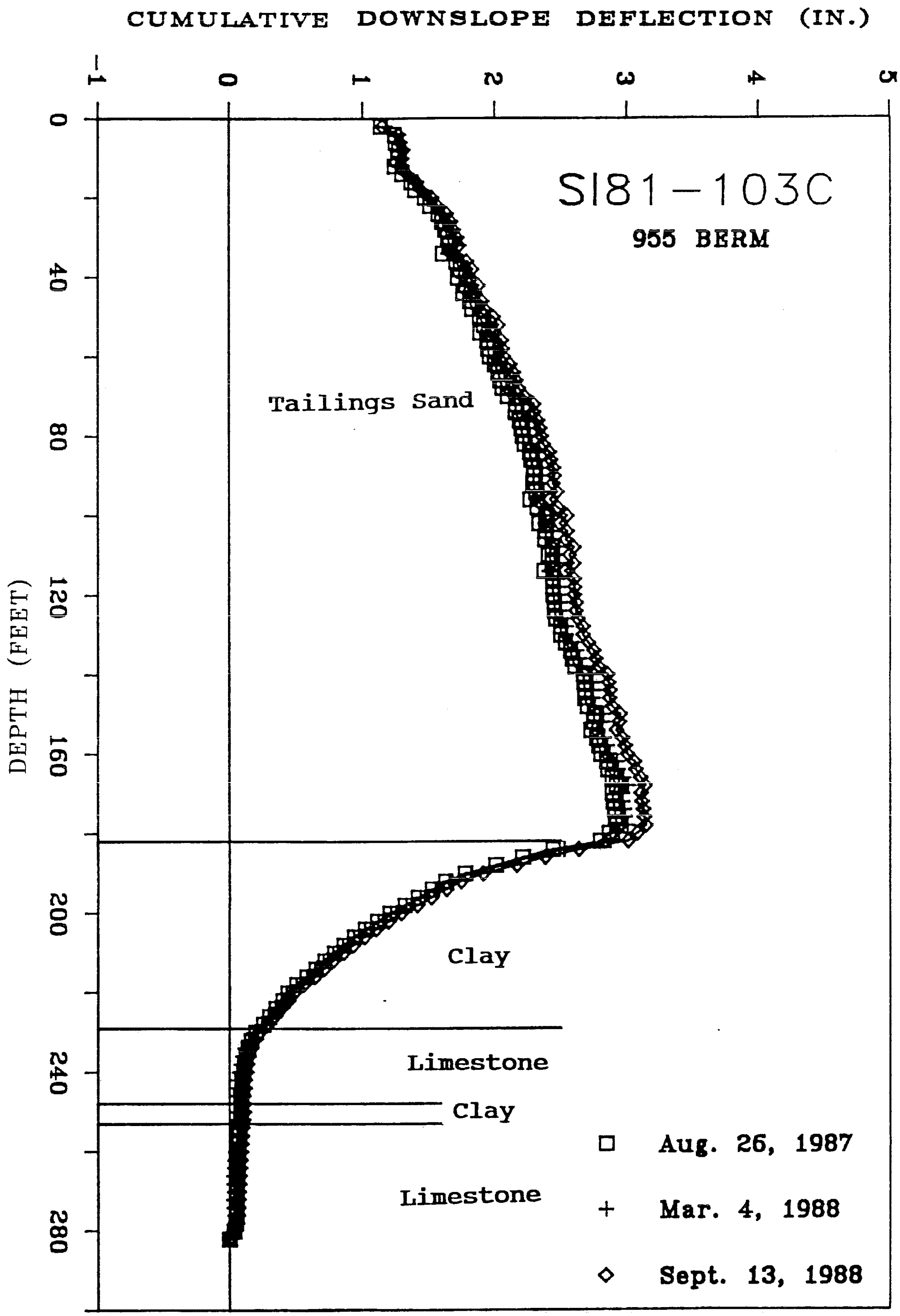


Figure 17. Slope Inclinometer SI81-103C

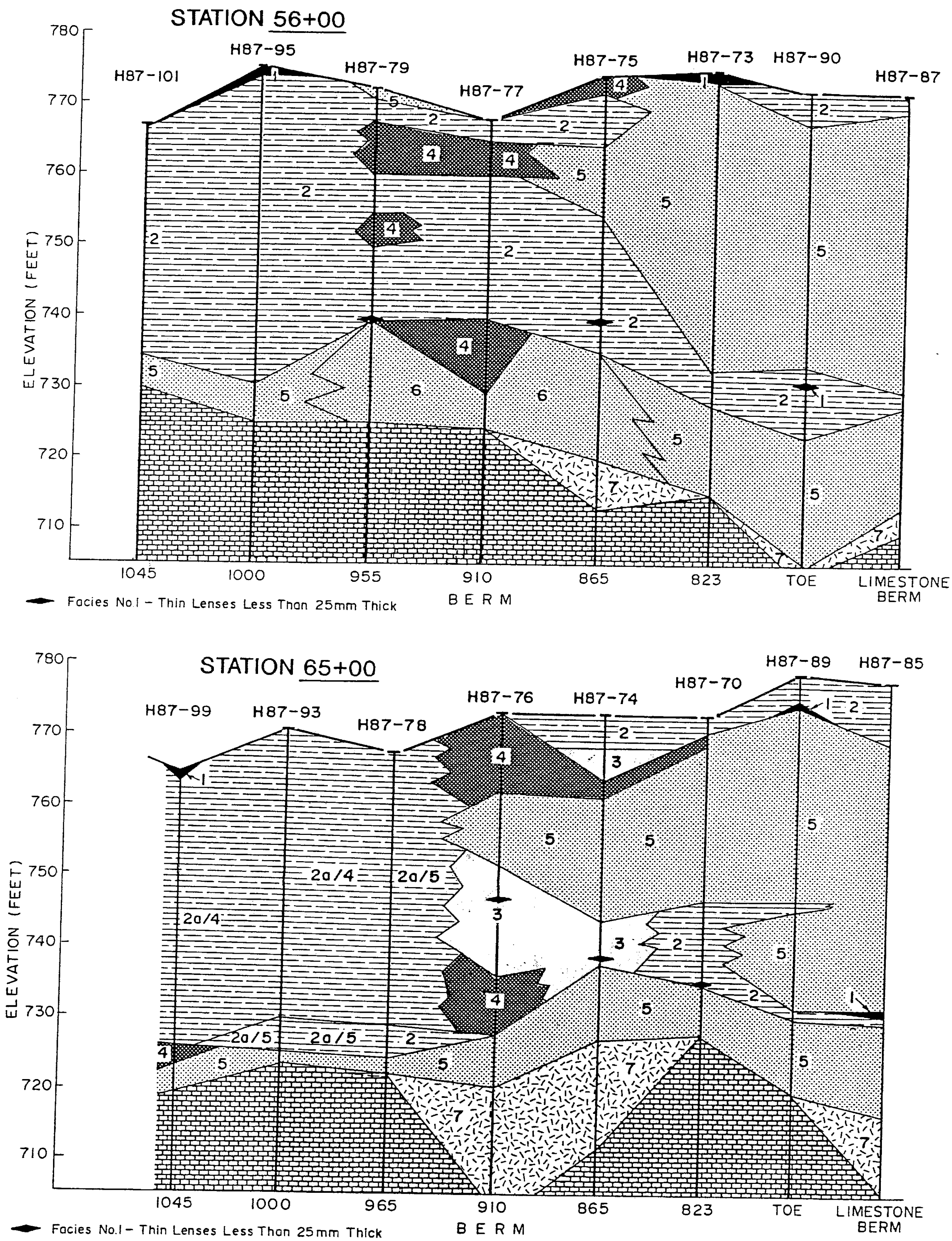
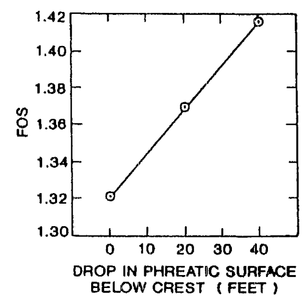


Figure 18. Facies Summary on Critical Sections, Adapted from Catto (1988)

SOIL TYPE	DENSITY (pcf)	C'	Ø OPERATIONAL	Ø RESIDUAL
COMPACTED TAILINGS SAND	120	0	36	N/A
OVERBURDEN STARTER DYKE	120	0	33	N/A
BBW SAND	120	0	30	N/A
CLAY U/S 1045 BERM, RESIDUAL	120	0	11	11
CLAY 1045 TO 910 BERM, RESIDUAL	120	0	11	11
CLAY 910 TO 866 BERM	120	0	15	15
CLAY D/S 866 BERM	120	0	24	15
BASE OF PROBLEM	N/A	N/A	N/A	N/A



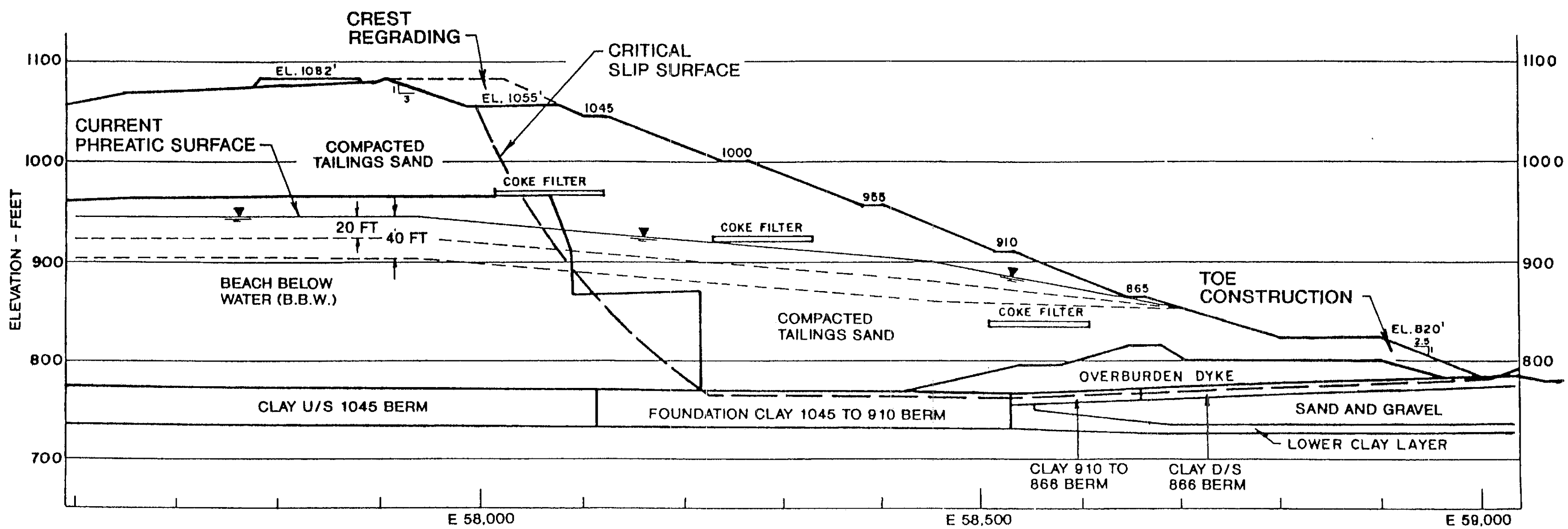


Figure 19. Example of Slope Stability Calculations at Critical Section

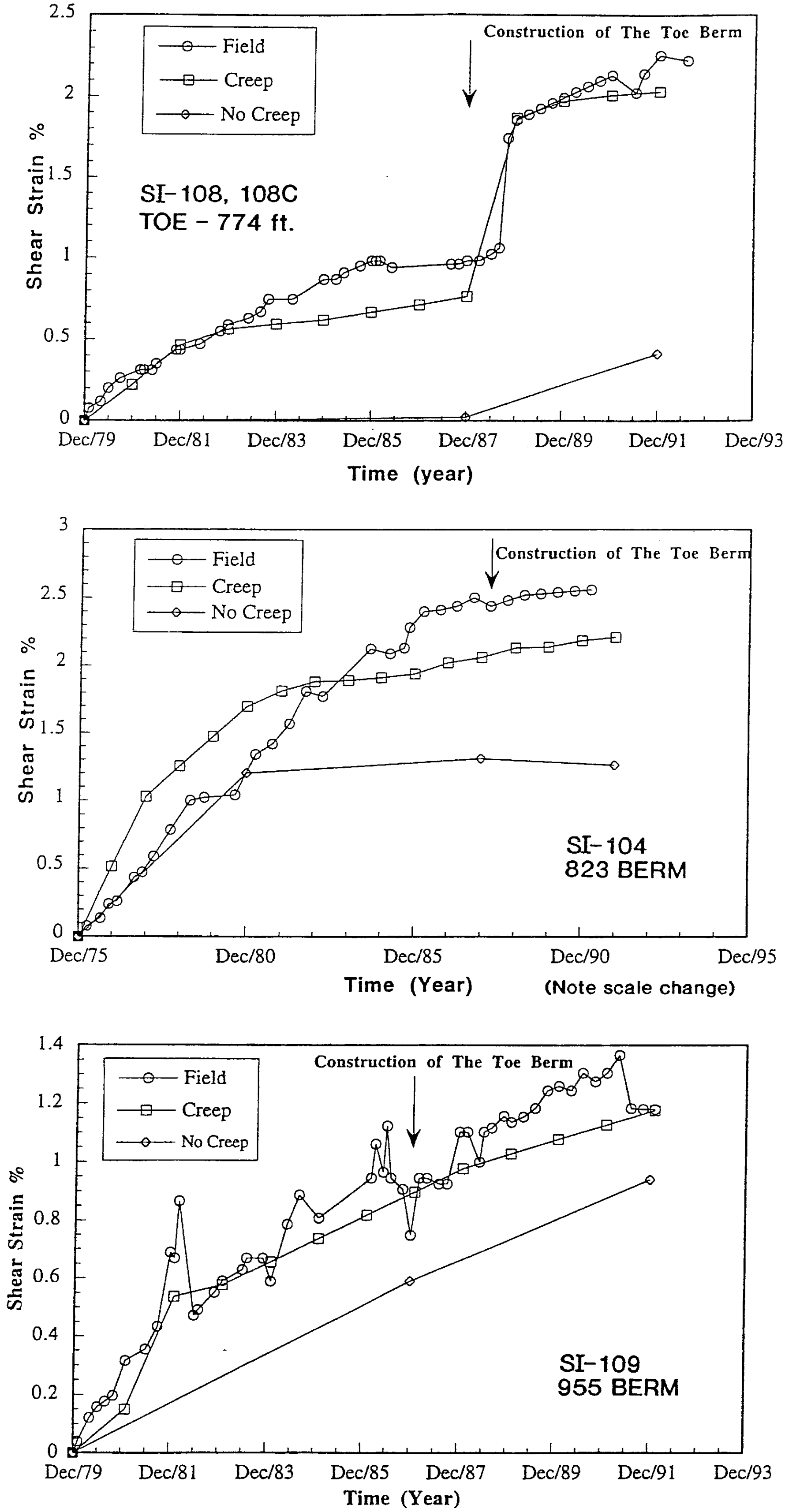


Figure 20. Measured and Predicted Shear Strain, Adapted from Morsy (1994)

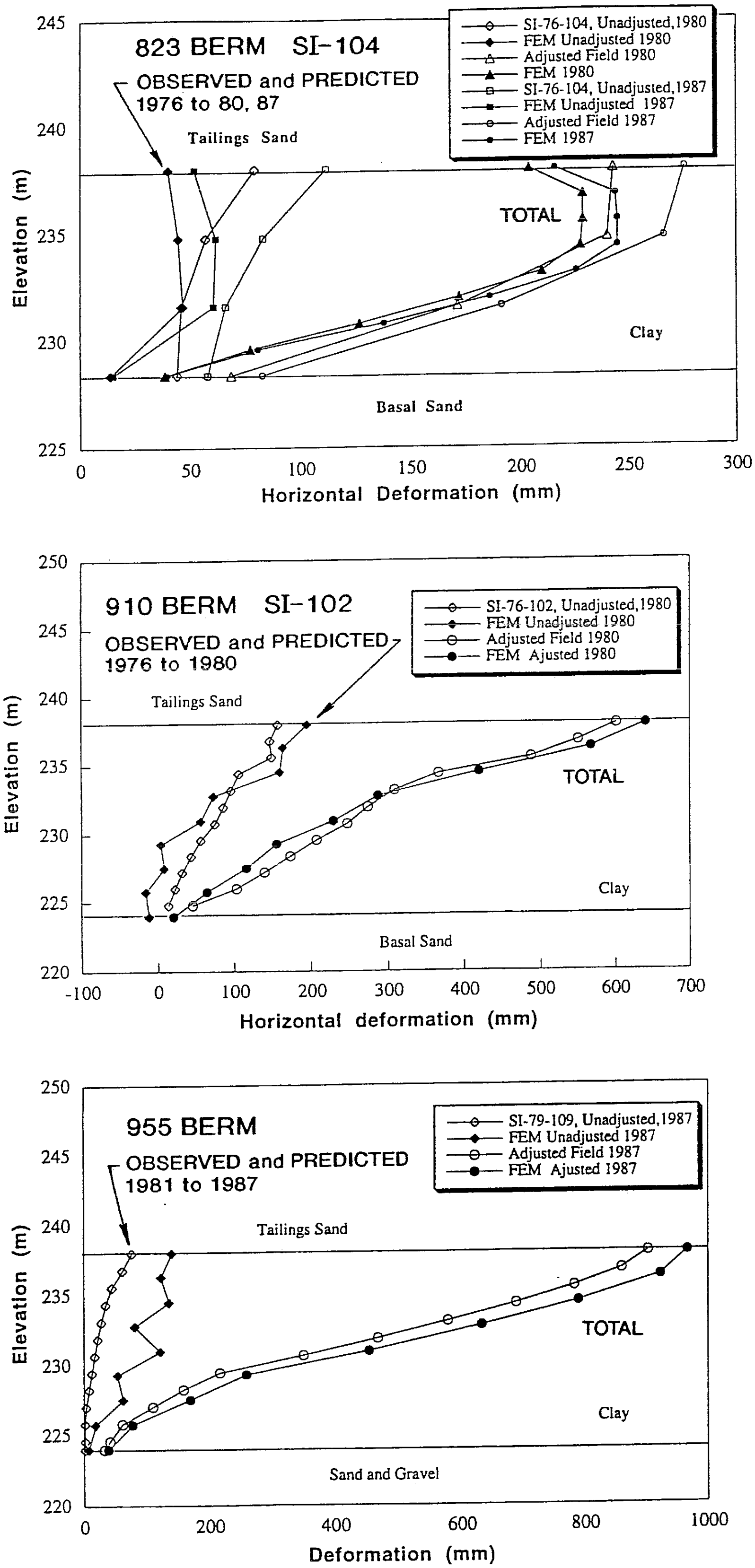


Figure 21. Measured and Predicted Horizontal Movements, Adapted from Morsy, Morgenstern, and Chan (1995).