CHARACTERIZATION AND DESIGN FOR EXPANSIVE CLAY SOILS IN THE OKANAGAN VALLEY, BRITISH COLUMBIA

by

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ABSTRACT

The prime objective of this paper is to introduce consulting engineers to the presence and characteristics of swelling clay soils that occur in the Okanagan Valley.

While the presence of swelling clay in arid areas is a world-wide occurrence, to which extensive engineering research and analysis have been devoted over the last 50 years, there is still no apparent consensus on methods of swell estimation, or on design procedures for reasonably economic construction. Significant damage to foundations occurs world-wide as a consequence of behaviour of these soils.

As an introduction to these soils and the associated design challenges, the results of 13 years of engineering analysis, design, and practice on swelling clays in the Okanagan Valley are presented. These include charts summarizing over 150 swell tests.

To date, only a rough correlation of swelling behaviour to initial moisture content and atterberg limits has been found. Direct swell testing is still our principal method of characterizing and evaluating sites.

Our typical design procedure is presented with some examples. Alternate procedures by other consultants are also discussed, as well as anecdotal case histories from a particularly difficult area where several design procedures have been tried.

INTRODUCTION

Swelling clay deposits in the Okanagan Valley represent a considerable design challenge. Direct swell testing using the oedometer has been our preferred method of characterizing sites and carrying out designs for the last 10 years.

The susceptible soils are typically high plasticity clays, originally deposited in the Valley as lacustrine deposits. Naismith (1962) mapped the Okanagan Valley - his observations of the extent of lacustrine deposits are generally accurate in outlining the areas in which the high plasticity clays will be found, ranging from Enderby south to Westbank.

Figures 1 and 2 show Naismith's mapping for the area. Figure 1 shows mapping for the Kelowna and Westbank area, with Figure 2 showing the Enderby to Vernon area.

Liquid limits of these clay deposits are typically in the range of 60 to 80%, with the plastic limit normally at 30 to 40%. Direct swell testing of these clays will lead to results varying from 1 to 12% expansion when saturated at a seating load of 6.25 kPa (125 psf). The pressure required to return the clays to their initial height varies from as little as 50 kPa (1000 psf) to as much as 900 kPa (18,000 psf), which is referred to as the swelling pressure. Test results from a difficult site are shown on Figure 3.

ASSESSMENT AND TESTING PROCEDURES

On encountering these lacustrine deposits, the first question is to decide whether or not swell testing is necessary.

Our first screening test is to obtain the atterberg limits, to help identify the soil type and plasticity. This also allows us to compare the initial moisture content to the plastic limit, which has become our most important screening criterion.

It has been our experience that medium to high plasticity clays will swell significantly if they are drier than the plastic limit.

On identifying soils we consider likely to be a problem, we then turn to direct swell testing using a consolidation machine. The sample is trimmed into the ring with porous stones top and bottom, and loaded to a nominal seating load of 6.25 kPa. The sample pot is then flooded, and the vertical expansion is monitored over several days until the upward movement has slowed or stopped. The sample is then loaded as for a normal consolidation test.

Ultimately, the loading will recompress the sample to, or below, the original height. Results of the test are then plotted as percent expansion versus log (pressure) as shown on Figure 3.

We identify the pressure required to return the clay to the initial height as the "swelling pressure". The percent expansion at 6.25 kPa (or another selected pressure such as the overburden pressure) is noted as the "percent swell" of the sample.

Tests of this nature are typically referred to as "free swell" tests.

DESIGN PROBLEM

Much of the design difficulty in respect to these clays lies in the estimation of soil behaviour. It has been difficult to believe in estimates of soil swell that can be as much as 200 to 300 mm of total swell, particularly when there are few well-documented cases, and certainly no one advertising them.

Up until recently, I had been unable to find well-documented causes that actually identified significant heaves. This highlights another difficulty, in that much of the available literature is not easily available to a practicing engineer.

However, there is good reason to believe that significant soil swell can occur, despite the lack of well advertised cases. Some of these will be described later.

Having determined or estimated the total soil swell by methods I will describe, the next problem becomes deciding how much of this might occur as differential. This can turn on relatively minor details of which an extreme example is shown in Hamilton (1984), which I have only recently encountered, during research for this paper.

In this reported case, two adjacent houses in Winnipeg constructed on clay soils each rose approximately 150 mm as a result of heave, over a 16 year period. One house rose relatively evenly, with only roughly 30 mm of differential in a total of 150 to 180 mm heave. The house next door heaved only 50 mm on one side, and roughly 175 mm on the other side, for a differential of roughly 125 mm.

In this instance, the difference was attributed to the presence of a 1.2 meter thick silt layer on the lesser heaving side of the house.

The last design problem relates to the inevitable difference between housing construction and commercial or institutional construction.

In general, housing sites are investigated broadly, with investigations and reports carried out for large areas which will be developed in a piecemeal fashion. Therefore, variations in the soil profile as described above can easily occur between tested locations, with undesirable effects. Also, there is typically greater pressure to control the initial cost of construction, and the party carrying out the construction is often not the ultimate owner.

By comparison, commercial or institutional sites cover a limited area, and therefore can be investigated in more detail. Budgets for investigation, design, and construction are better, and there are often other consultants involved, as well as the ultimate owner, who are making the decisions with respect to the desired level of performance.

EARLY DESIGN EXPERIENCES

To my knowledge most design experience prior to 1985 in the Okanagan Valley was originally obtained by Vancouver consultants on higher profile sites.

Designs were typically based on isolation piling procedures which are common in many areas for clay soils, and described in widely available texts such as Peck et al (1974). These generally require the installation of piles to depth to avoid the movements of the upper clays, and the provision of structural elements supported on "void forms" (collapsible corrugated cardboard void boxes, usually 150 mm high) to avoid bearing directly on the near-surface clays. However, there was some unfamiliarity with the procedures, few experienced local contractors, and an understandable lack of experience with properties of the clay soils. Therefore, some designs were satisfactory, while others, sometimes for unexplained reasons, were less satisfactory. The less satisfactory designs were typically in the Vernon area.

As a junior engineer with a Vancouver consulting firm which had some experience and difficulties with sites in the Okanagan Valley, I was aware of some of these problems, and therefore was somewhat forewarned when it came to my turn. Having moved from Vancouver to the Okanagan in 1985, our first significantly swelling site was found in 1987. This was beside a project that had earlier problems, in the Vernon area. I was alerted to the problems with the adjacent site as a result of a conversation with a local structural engineer. He had been involved in reviewing the construction when it was unclear why the building was not performing satisfactorily.

At this stage, I found some design guidance within the NavFacs Design Manuals (1982). These contained procedures for estimating the swell for a site, and then calculating sub-excavation and replacement depths based on direct swell testing. This procedure is shown on Figure 4, with the depth of swelling soil based on the swell tests as indicated. An alternate procedure for estimating total heave called the "South African method" was also given and this procedure is shown on Figures 5 and 6. For this procedure the depth of potentially swelling soil is simply assumed as 6 meters. After identifying swell percentages of 3 to 7%, a 400 kPa swelling pressure, and a total estimated surface swell of roughly 200 mm, I prepared a design.

This called for a 2 or 3 meter subcut and replacement with compacted gravel, to be left with only 50 mm or 25 mm respectively of theoretical soil heave, depending on the building tolerance.

The prospective purchaser decided that better properties could be found elsewhere, so the site was not developed for that purpose. However, it was later used for housing - more on that later.

DESCRIPTION AND EXAMPLE OF DESIGN PROCEDURE

The next difficult site, which was constructed upon, came in 1990, with the construction of the new Okanagan University College campus.

On preliminary screening of the site, it was clear that relatively dry, potentially swelling clays existed in parts of the proposed site. As you might expect, site conditions at the locations chosen for the first phase of the campus were the most difficult.

Again, the NavFacs and South African method were chosen to estimate theoretical heaves based on swell tests. Results of the initial set of swell tests are as previously shown on Figure 3, with the associated calculations shown in Figure 7.

For the Navfacs method, the percentage swell (adjusted for the associated final overburden pressure) is integrated over the depth of swelling soil, normally taken as a maximum of 6 meters depth. For the South African method, the behaviour is classified as very high, high, medium, or low, based on the soil activity chart shown on Figure 16. We have modified this procedure, and classify the potential soil swell based on swell test results, as correlations to Figure 16 have been unsatisfactory locally. A calculated reduction (the F factor) is then applied to account for the overburden pressure.

For the College, these calculations suggested theoretical heave on the site could be as much as 175 mm to 200 mm. If a 3 meter subcut and replacement zone were constructed, the theoretical heave could be reduced to 33 to 36 mm. Accordingly, I recommended a 3 meter subcut below the foundations and replacement with compacted gravel.

There is a strong possibility this design was too conservative. However, the architects were all too familiar with one of the unsatisfactory buildings in Vernon. In addition to building movements, there had been severe heaving and cracking of sidewalks and pavement near that building, eventually leading to replacement of some areas.

Accordingly, to avoid difficulties with exterior site grades, and to allow for additions, the excavation and replacement zone was to extend 3 meters beyond the building footprint at depth, and then slope up at 1 Horizontal to 1 Vertical. The central courtyard area was to be underlain with 1.5 meters of compacted gravel.

Part of the concern on this site also related to the interlayering of occasional granular soils within the clay. There was no indication of a consistent moisture zone at depth where the clay appeared to be certain to be stable, possibly due to drainage provided by these seams. Once the site became developed, however, it could be expected that these seams might convey increased site drainage directly to depth under the buildings. Accordingly the design was intended to be conservative, partly due to my unease with these soils and the lack of information on actual heave behaviour, partly due to the architects concerns and experience, and partly because it could all be done at reasonable cost - a major gravel pit lay approximately 1 km away.

CHARACTERIZATION OF THE CLAY SOILS

A compilation of our swell test data for various sites and areas in the Okanagan Valley is shown on Figures 8 and 9 (Kelowna area), 10 and 11 (Westbank area), 12 and 13 (Vernon and Coldstream area), and 14 and 15 (Armstrong and Salmon Arm together). Some attempts at simple correlations to swelling behaviour have been tried as follows.

(a) Atterberg Limits and Initial Moisture Content

To date, the only rough correlation we have found to swelling behaviour is the relationship of the initial moisture content to the plastic limit, or at times, the moisture content alone. The suggestion to compare initial moisture content to the plastic limit was originally made in a text by Sowers to my recollection, but I no longer have that reference. Although I believe this suggestion was dropped in later editions, it does appear to be useful.

There is considerable scatter in the relationship between moisture content and percent swell (Figures 8, 10, 12, and 14). Some order begins to appear when swell is plotted against the moisture content as it compares to the plastic limit, with a more apparent upper bound (Figures 9, 11, 13, and 15).

In general, experience shows that little trouble seems to occur where the initial clay moisture contents are equal to, or greater than, the plastic limit. This may relate to the fact that at least 50% of the results would suggest less than 4% swell at the plastic limit, and less than 1% swell if initial moisture is 10% above the plastic limit. Also, if the soil is at the plastic limit, in most areas it tends to be wetter at greater depth, and there will be almost no thickness of significantly swelling soil present.

Other methods to assist in characterizing the clays have been tried, but have not significantly helped, as follows.

(b) Clay Activity

The clay activity, defined as the plasticity index divided by the percent clay fraction (based on a hydrometer analysis), has not been found to be useful. This is proposed as a screening test in the Canadian Foundation Manual (1985). The chart is shown on Figure 16, with some test results plotted for comparison.

Results have typically been 0.5, which lies on the boundary between each of very high, medium, low, and very low swell. No correlation has yet been indicated for the Okanagan soils, with some high swell soils (7%) plotting in the "low" swell zone as shown.

(c) Clay Mineralogy

We have carried out only two mineralogy tests. These indicated predominately chlorite, with illite, muscovite, some smectite, as well as quartz, albite, and anorthite, with some cordierite. The soils tested were from sites exhibiting moderate (4%) to very high (11%) swell.

Prior work in South Africa (Williams and Donaldson, 1980), which I have only recently found, indicates that little success has been found there with this approach, despite extensive attempts to correlate mineralogy to swelling behaviour.

As both chlorite and illite are suggested by the Canadian Foundation Manual (1985) to be low swelling soils, I do not expect any correlation to be likely, although some further research is planned.

(d) Other Methods

Blight (1998), on p. 121, refers to a comprehensive survey by Schreiner (1987) that listed 39 published methods of predicting heave in expansive clay soil profiles. I have not yet obtained a copy of that paper to see which methods I have missed, but I am aware of methods based on soil suction, moisture contents, or atterberg limits. One local proposal has been to use the plot proposed by Vijayvergiya and Ghazzaly (1973) to classify the local soils. However, this plot does not correlate well with field results. This plot is shown as Figure 17, with results of swell tests from Figure 18 plotted onto it. It can be seen that the correlation is poor, with only 3 results out of 9 matching. However, I do not have a copy of this reference to confirm that definitions of swell and swelling pressure are similar.

To date, I have not found a more useful indicator than moisture content either alone, or better yet, as it relates to the plastic limit.

GENERAL FEATURES OF DEVELOPED CLAY AREAS

We provide a generalized outline of areas developed as follows. These general areas are identified on Figures 1 and 2.

(a) Glenmore area of Kelowna

- typically a moderate to low risk area
- typically wet, non-swelling clay at 2 to 3 meters below grade
- upper clays normally swell on the order of 1 to 2%
- foundations are typically set directly on the clay (where damp to wet) , or on 300 mm of subcut and replacement gravels
- there are some exceptions, notably the Kane Road area, where a sample at the plastic limit showed 8.2% swell!
- performance has generally been good, although there are some reports of cracks
- roadway performance is generally good

(b) Hartman Road Area of Kelowna

- typically a moderate risk area
- typically wet, non-swelling clay at 3 to 4 meters below grade
- upper clays normally swell on the order of 3 to 5%

- foundations are typically set on 300 to 450 mm of subcut and replacement gravels
- performance has generally been good, although there have been some reports of cracks, and at least two houses have roughly 50 mm of elevation variation at present
- performance of some roadways has been poor, with up to 75 mm of heave measured by the civil engineers. This appears to have occurred due to drying during construction. After stripping the topsoils, the associated trenching and backfilling was carried out over the summer. The contractor ultimately brought in a sample of the clay soils from a stockpile, which was found to swell 17%, as compared to initial swells of roughly 2 to 4% prior to construction. This was the highest result we have ever recorded, and would easily explain the observed movements of the roadways.

(c) College Site

- high risk area
- dry to depths beyond 6 meters
- clays swell on the order of 5 to 10%
- foundations were set on 3 meters of subexcavated and replaced gravel, courtyard on 1.5 meters
- performance has been good

(d) Highway 6 Area of Vernon

- high risk area
- dry to depths beyond 6 meters
- clays swell on the order of 10%
- several foundation schemes tried by various consultants, which will be discussed further
- subexcavation and replacement depths of roughly 2 meters proposed for this area with 1 residence and 1 commercial building successfully constructed to date
- heaves of 75 mm to 200 mm are reported for some of the alternate foundation procedures, but this apparently varies widely, even between foundations on the same site
- recent roadway construction has performed poorly, with extensive cracking, as well as rolls, dips, and heaves of roughly 200 mm

ANCEDOTAL CASE HISTORY

At an intersection in the Highway 6 area of Vernon as noted above, there has been an unusual concentration of different consultants, procedures, and results.

This location is of interest, in that there are some records or anecdotes of performance, the areas have several well documented soils reports, and a number of different foundation systems have been tried. Each procedure correlates roughly to one corner of the intersection, with approximate details as follows.

(a) A piled system using piles, pile caps, voids under the grade beams, and a structural slab (corner 1).

(b) A residential foundation scheme, using a basement wall with concentrated point-loads on pad footings with void forms between, and a basement slab on grade (corner 2).

(c) A commercial building (gas station) where the "designed" excavation and replacement system was used, with the replacement depth based on swell test results (corner 3).

(d) A condo development, where a partial sub-excavation, replacement, and drainage scheme was employed. Coincidentally, this was constructed on the site I had previously investigated (corner 1).

(e) A heavily investigated site for which residential buildings were planned, but "designed" excavation and replacment requirements presently preclude economic development (corner 4). This also contains newly constructed roads which are performing very poorly.

Four soils reports for this area present a discouraging picture. Swell tests frequently give results of 10% swell or more, with swelling pressures on the order of 500 to 900 kPa (results for corner 4 are shown on Figure 18). By contrast to other areas, moisture contents of the clays do not reliably increase with depth, even to as deep as 9 to 12 meters.

For the piled building constructed on corner 1, it was apparently recognized that foundation movements were occurring at some time after construction, and it was later found that the exterior pile caps had heaved relative to the interior pile caps, apparently by as much as 75 mm. Thus, even with the load concentrated at the pile caps, significant movement had occurred, probably due to heave.

For the commercial building (corner 3), a "designed" 2 meter subcut and replacement was provided, and the concrete block building, constructed in 1992, appears to be performing quite satisfactorily. However, the clay subsoil was only 4 to 5 meters deep, so that conditions were somewhat less severe at this location.

At our original site later developed for condos (corner 1 again), a "guess-timated" 0.6 meter subcut and replacement was carried out for foundations placed near or below the original site grade, with drainage provided within the base gravel. Performance has apparently been satisfactory in roughly one-half of the units, with relatively poorer performance in a portion of the remainder. Apparently at least one unit is out of level by on the order of 125 mm, as compared to our original swell estimate of roughly 200 mm.

On the deep basement wall site (corner 2), located within a recent residential subdivision, apparently there was a break of an irrigation line in the owners absence. On their return several weeks later, it was found that the basement floor slab had heaved relative to the wall by approximately 200 mm, but the basement walls were in good condition. No prior heave estimate or soils information is known to me for this site for comparison.

The "designed" subcut and replacement appears to have performed satisfactorily where some of the other procedures have been less successful. However, it is clear that some of the other systems have been successful, at least in part, when they might not have been expected to be successful at all.

In reviewing the corner 4 site for foundation design, a 1.5 to 2.4 meter subcut and replacement depth was proposed for footings. Although 1 house and 2 mobile homes have been successfully constructed this requirement has virtually precluded ordinary residential house construction due to expense.

At this site, the recently constructed roadways are also showing signs of distress. Severe cracks have occurred, and have re-occurred in some areas even after initial removal and replacement of the failed sections. Significant dips and rises due to heaving are also apparent.

This appears to be related to the provision of gravel backfill within the trenchs as required by the City by-laws. Sections of the road are still underlain by clays with severe differential movements as a consequence. This roadway, and another constructed on a difficult site in Armstrong, at nearly the same time, appear to be splitting in tension, as the cracks in the asphalt and road base have extended vertically down into the ground as much as 0.6 meters or more.

On sites with 10% swell, it appears ordinary roadway designs, with the associated trenching and services, can not be expected to perform well, even though the ground is relatively hard when construction occurs.

CURRENT PRACTICE FOR RESIDENTIAL CONSTRUCTION

Current practice in the Okanagan Valley varies substantially between consultants, contractors, and municipalities. In part, this is due to the varying awareness, theories, and expectations of clays and their potential behaviour; it is also due to an ongoing desire on the part of engineers and contractors to find effective relatively low cost measures if possible.

The objective is to find moderate expense methods that will result in satisfactory housing construction, without movements sufficient to cause noticeable cracking or heaving of floors and walls. It would be ideal if movements could be limited to roughly 25 mm or less, and ordinary foundation procedures could be used. If differential movements exceed 30 to 50 mm, there tends to be some evidence of cosmetic cracking in drywall construction, and doors will often stick or bind. While such movements are generally not serious, they can be quite irritating to the owner of a new house.

Approaches that have been tried for residential foundations, and the approximate date of their first use, to my knowledge, are as follows.

(a) Foundations supported directly on the clay soils. This has been standard procedure for many years with varying degrees of success. Fair to poor in Vernon on difficult sites, typically successful in Kelowna and Westbank areas on simpler sites. This continues, and could fairly be considered to be "established local practice" as described in the B.C. Building Code, although it is not always satisfactory. Cosmetic cracking and binding doors are occasionally reported, usually due to heave under the lightly loaded interior walls.

(b) Partial Excavation and Replacement - first residential application in approximately 1989, Vernon area, partially successful, even on a very difficult site.

(c) Calculated Excavation and Replacement based on Swell Testing - this is our typical design procedure, first used for residences in approximately 1991 to 1992. Generally successful to date on either easy or difficult sites, although at much greater expense if the site is considered difficult. This can require as much as 2.4 meter subcut on soils where roughly 10% swelling is expected. Replacement thicknesses of 0.3 to 0.45 meters are commonly used on sites of moderate (3% to 5%) swelling potential with good success to date.

(d) Moisture barrier method, approximately 1994 - used mainly in Vernon by others, generally successful to date.

(e) Combination of limited excavation and replacement, with subgrade shaping and drainage, approximately 1992, reasonably successful, Armstrong area.

(f) Spreading of lime over clay subsoil before placing foundations. Only 2 Kelowna applications known, approximately 1996. Results unknown to date. Applied in areas of moderate swelling potential (3 to 5%).

(g) Pre-wetting of the foundation clay. Tried in 1993 and 1996 when foundations were in place prior to engineering input. Heaves of zero, and 27 mm were induced within 1 week. Appears successful to date, although both sites were not particularly difficult. In particular, the 27 mm heave occurred where only the top 600 mm was expected to swell.

(h) Hold down pressure trying to provide dead load to prevent heave of the clay, Vernon area, ± 1994. Generally successful for stiff walls, not so good for the floor.

(i) Mobile home pads, with allowance for periodic adjustment, 1997, Vernon area, on difficult site. Successful to date, but only 1 year old.

Numerous other foundation treatments have been proposed, in other jurisdictions, notably pier foundations with void forms, stiffened slabs, pre-wetting, and a host of other procedures. Most of these have not been tried yet in the Okanagan.

In my opinion, the single unifying element is the estimated behaviour of the clay soil, based on swell testing.

On a site with low swell potential, <u>all</u> of the methods listed above will work, and I suspect a false air of confidence will prevail. However, on a site with high swell potential (say 10%), I suspect most of the systems will not work as well as desired. If the site has not been characterized by the use of swell tests, I believe the critical difference between apparently similar sites will not be known. As an indicator test, I believe review of the moisture content as compared to the plastic limit will suggest if there is reason to be concerned.

However, even at present, it is not and has not been, standard practice locally to carry out swell testing for individual lots. As many residential sites containing clay subsoils are not within subdivisions where engineering reports are available to characterize the soils, it is still common for local consultants to rely on judgement and to propose foundation treatments that seem sensible for site conditions.

This is primarily due to the time involved in carrying out swell tests, and the cost of a full engineering review and design, which is comparable to simply selecting a reasonable depth of subcut.

Our firm makes extensive use of swell tests primarily because I believe it is the single most useful result possible. However, even we will periodically propose an excavation and replacement thickness we believe is reasonable based on local experience rather than requiring a full formal report.

At a 1997 meeting of 5 local consulting firms, it was confirmed that "guess-timated" procedures are typical for individual foundations in areas where overall reports are not available. In fact, depending on the experience of the individual attending the site, it is possible it would be simply confirmed that the clay bearing strata are hard enough to support a normal house, without any further consideration of the soil behaviour. Even for very experienced engineers, relatively little further work is typically carried out, particularly if, for whatever reason, there are no previous reports or concerns in the immediate area which would suggest further review is important.

CONCLUSION

It is still unknown how accurate heave predictions based on swell tests are, although it is suggested in the literature that they are quite conservative.

Generally our residential designs for moderate risk sites call for 300 to 450 mm of subcut and replacement, and are based on roughly 50 to 60 mm of predicted total heave. For this, we anticipate actual heave will be on the order of 25 mm to 30 mm or less, with differential heave an even lesser proportion given the replacement procedure. This has been quite successful on hundreds of houses to date, and has represented only a moderate cost increase to the contractors or owners ($2000 to $3000).

However, on difficult sites, this procedure can require 1.5 to 2.4 meters of subcut. This represents an additional $10,000 in foundation expense, even though we will allow the total heave estimate to rise to roughly 100 mm, given the thickness of the replacement pad (deeper subcuts would be required to reduce this value).

We believe these procedures are reasonably rational, and have a good record of success. However, they are by no means yet the universal method for design, as there continues to be innovation and different procedures carried out in the industry. Only time will tell which procedures are most effective.

With these soils, and their potential variation, it is impossible for engineers to always find a solution that is both economic and which has no risk. We will inevitably find some sites that will not behave as well as expected.

We are still searching for the "Holy Grail" which is a simple index test that would give us the swelling behaviour without the need for direct testing.

For those who are interested, I have tabulated an extensive data base for our swell test results. This includes the atterberg limits, sample depth, calculated density, seating loads, swell percentage, swelling pressure, initial moisture, final moisture and a few specific gravities and percent clay sizes for over 150 swell tests.

If you want a chance to try developing some correlations, I will be happy to provide a copy.

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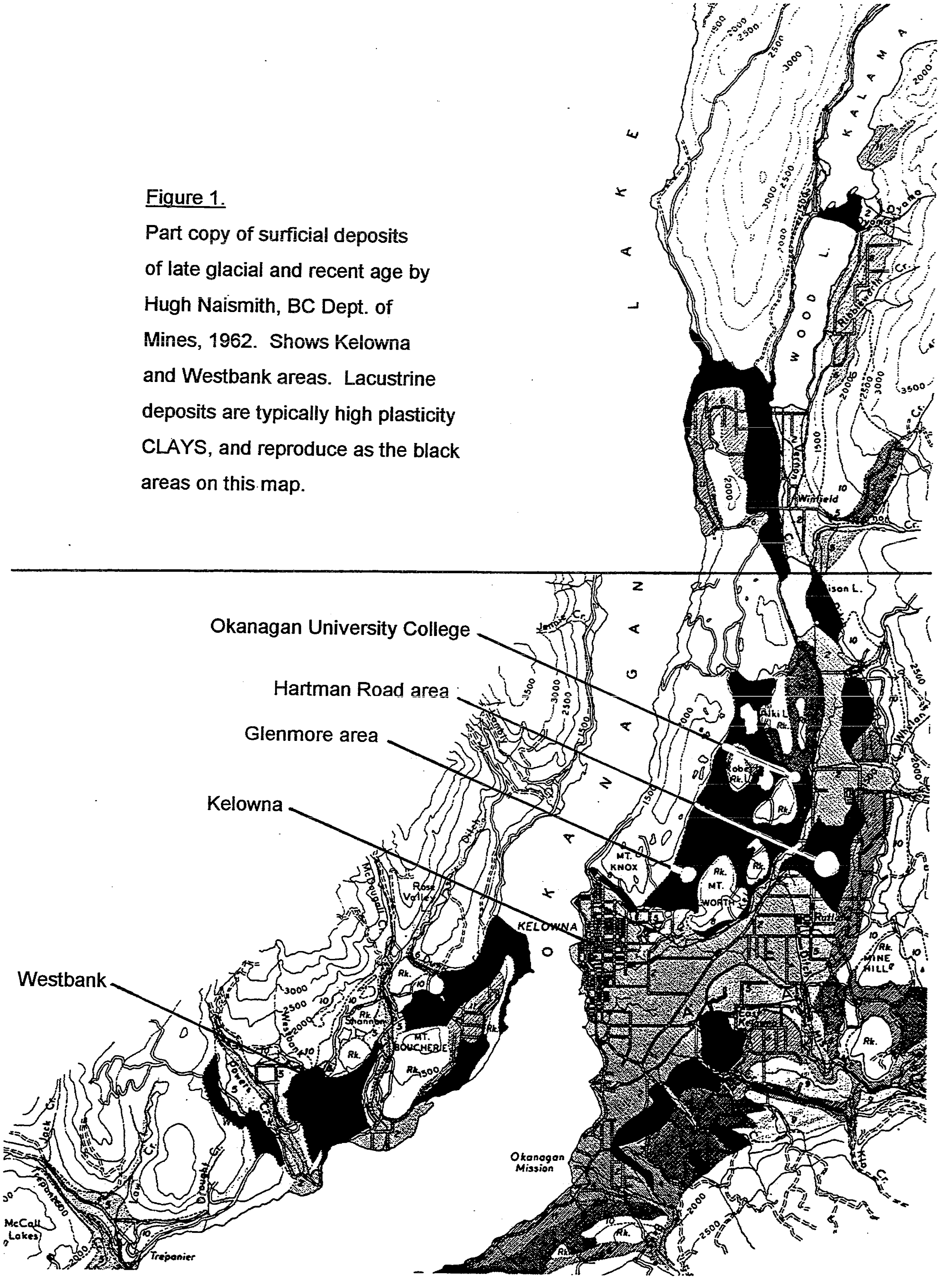
Figure 1.

Part copy of surficial deposits of late glacial and recent age by Hugh Naismith, BC Dept. of Mines, 1962. Shows Kelowna and Westbank areas. Lacustrine deposits are typically high plasticity CLAYS, and reproduce as the black areas on this map.



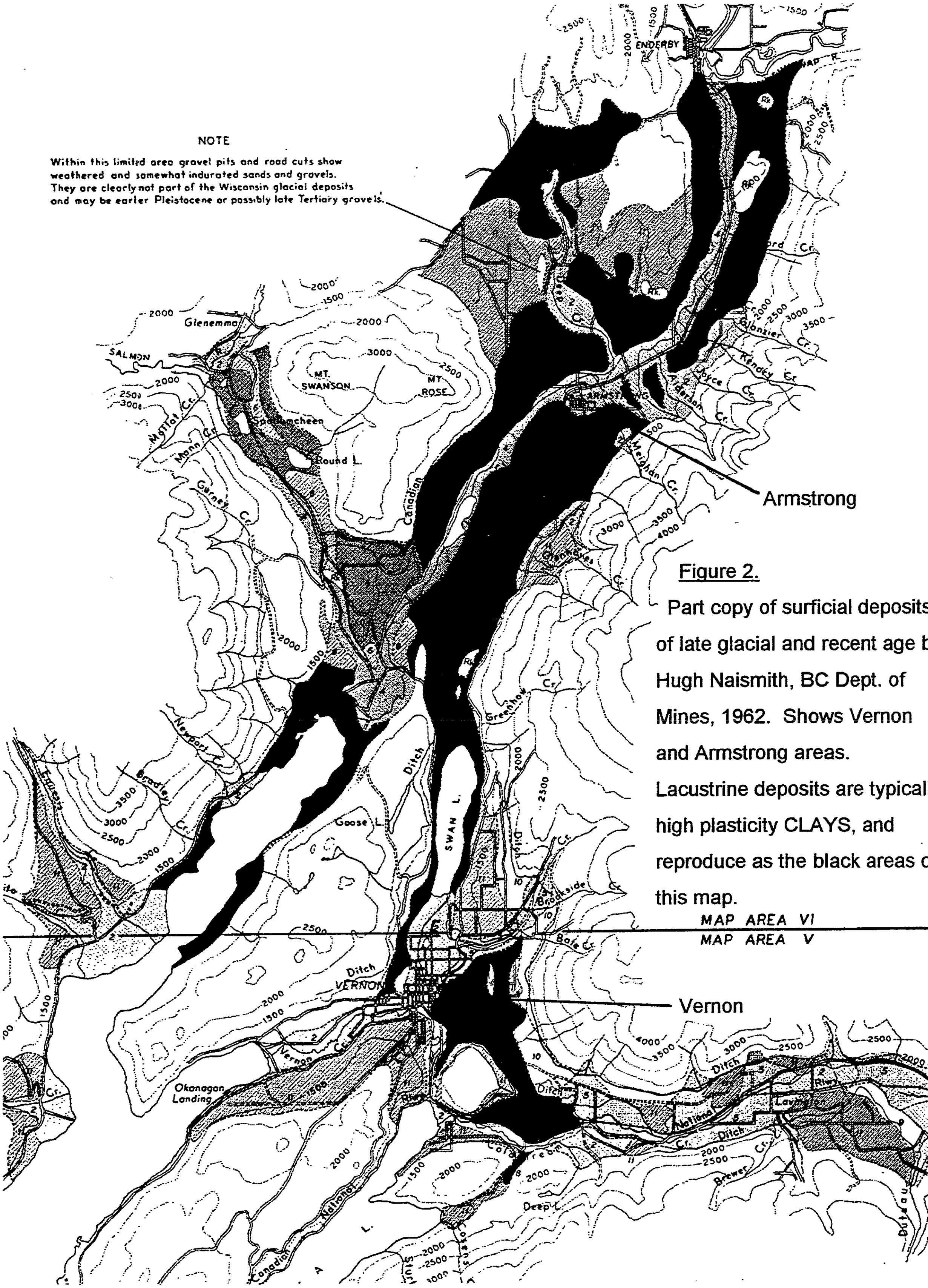
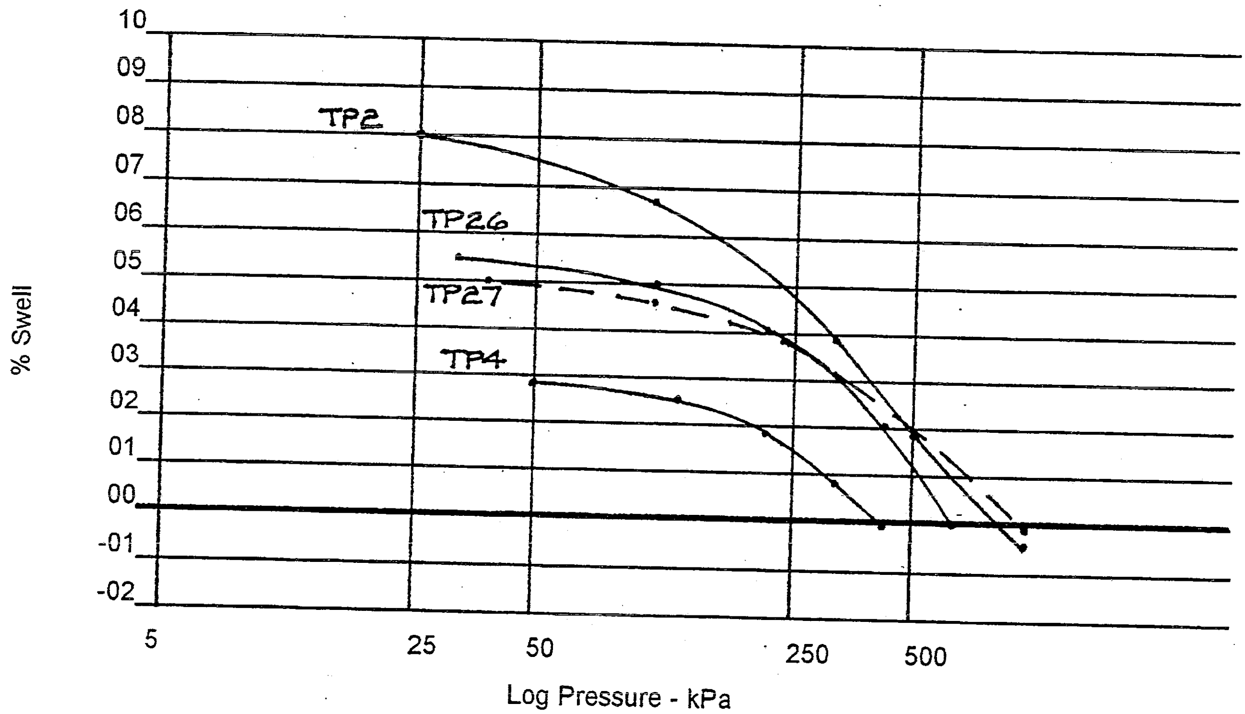


Figure 2.

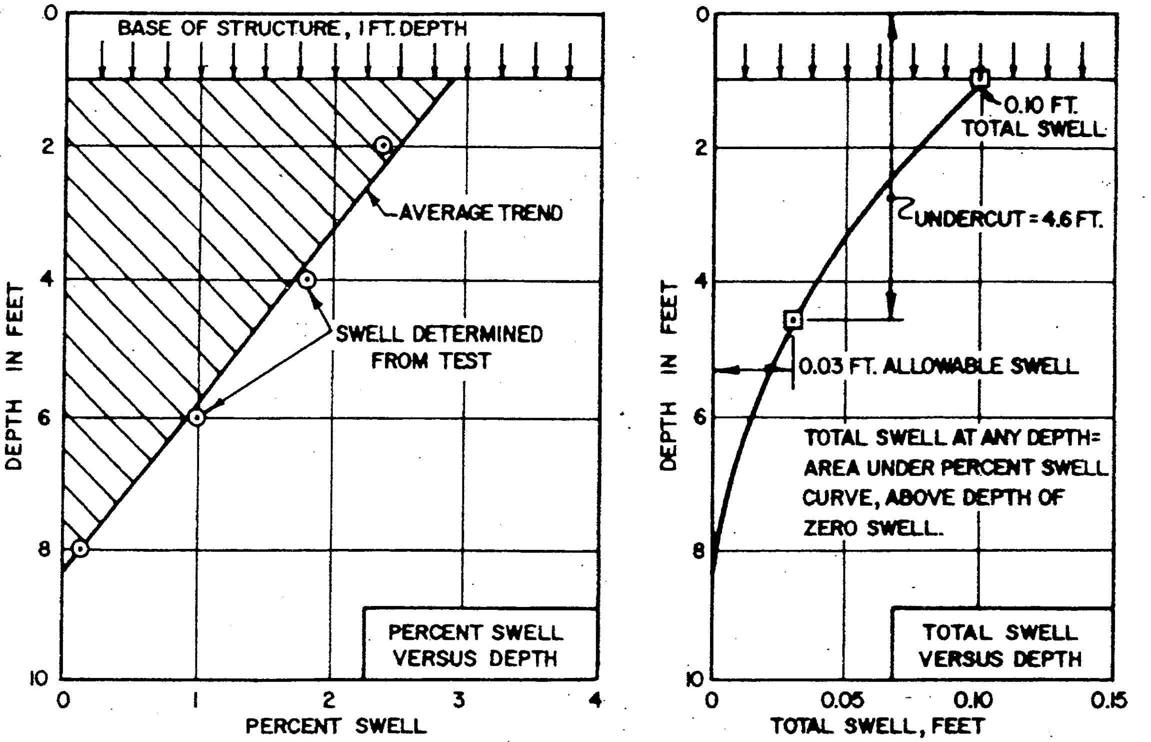
Part copy of surficial deposits of late glacial and recent age by Hugh Naismith, BC Dept. of Mines, 1962. Shows Vernon and Armstrong areas. Lacustrine deposits are typically high plasticity CLAYS, and reproduce as the black areas on this map.

Figure 3. Kelowna, Okanagan University College, 1990: Swell Test Results



Swell Test Data

Test Pit	Depth (m)	Initial Moisture (%)	Swell (%)	Final Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Swelling Pressure kPa)
2	1.5	22.6	8.0	34.0	76.4	34.4	800
4	3.4	31.1	2.9	41.3	64.7	22.5	400
26	1.8	37.6	5.6	43.7	96.0	31.5	600
27	2.4	37.3	5.0	41.8	85.5	29.7	850



MATERIALS INVESTIGATED ARE CLAYS, HIGHLY OVERCONSOLIDATED BY CAPILLARY STRESSES THAT ARE EFFECTIVE PRIOR TO THE CONSTRUCTION OF THE STRUCTURE UPON THEM.

PROCEDURE FOR ESTIMATING TOTAL SWELL UNDER STRUCTURE LOAD.

1. OBTAIN REPRESENTATIVE UNDISTURBED SAMPLES OF THE SHALLOW CLAY STRATUM AT A TIME WHEN CAPILLARY STRESSES ARE EFFECTIVE; I.E., WHEN NOT FLOODED OR SUBJECTED TO HEAVY RAIN.
2. LOAD SPECIMENS (AT NATURAL WATER CONTENT) IN CONSOLIDOMETER UNDER A PRESSURE EQUAL TO THE ULTIMATE VALUE OF OVERBURDEN FOR HIGH GROUND WATER, PLUS WEIGHT OF STRUCTURE. ADD WATER TO SATURATE AND MEASURE SWELL.
3. COMPUTE FINAL SWELL IN TERMS OF PERCENT OF ORIGINAL SAMPLE HEIGHT AND PLOT SWELL VERSUS DEPTH, AS IN THE LEFT PANEL.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO THE AREA UNDER THE PERCENT SWELL VERSUS DEPTH CURVE. FOR THE ABOVE EXAMPLE:

   TOTAL SWELL = $1/2\ (8.2 - 1.0) \times 2.8/100 = 0.10$ FT.

PROCEDURE FOR ESTIMATING UNDERCUT NECESSARY TO REDUCE SWELL TO AN ALLOWABLE VALUE.

1. FROM PERCENT SWELL VERSUS DEPTH CURVE PLOT RELATIONSHIP OF TOTAL SWELL VERSUS DEPTH AT THE RIGHT. TOTAL SWELL AT ANY DEPTH EQUALS AREA UNDER THE CURVE AT LEFT, INTEGRATED UPWARD FROM THE DEPTH OF ZERO SWELL.
2. FOR A GIVEN ALLOWABLE VALUE OF SWELL, READ THE AMOUNT OF UNDERCUT NECESSARY FROM THE TOTAL SWELL VERSUS DEPTH CURVE. FOR EXAMPLE, FOR AN ALLOWABLE SWELL OF 0.03 FT, UNDERCUT REQUIRED = 4.6 FT. UNDERCUT CLAY IS REPLACED BY AN EQUAL THICKNESS OF NONSWELLING COMPACTED FILL.

Figure 4: Computation of Swell of Dessicated Clays (after NavFacs, DM 7.1, 1982)

Procedure

1. Classify swell potential of soil using chart in DM-7.1, Chapter 1 (Very high, high medium or low).

2. Assign potential expansion (P.E.) as in./ft. of thickness based on

Swell Potential	Potential Expansion (P.E.) In./ft.
Very high	1
High	1/2
Medium	1/4
Low	0

3. Assume depth of lowest level of the groundwater table = 20 ft. Divide this thickness of 20 feet to several soil layers with variable swell potential. Assume thickness of individual layer = ΔD.

4. Calculate the factor $F = \log^{-1}(-\frac{D}{Z}) = \log^{-1}(-\frac{D}{20})$ for each soil layer. D is depth in feet to mid point of each layer.

5. Compute expansion for each individual layer $\Delta_e = (P.E.)(\Delta_D)(F)$

6. Compute total expansion $(\Delta H)_S$

$$(\Delta H)_s = \sum_{e=1}^{n} \Delta_e$$ where n is number of soil layer.

FIGURE 3
Estimating Swell Using the South African Method

Figure 5: Procedure for South African Method (after NavFacs DM 7.3, 1982)

EXAMPLE:

LAYER	THICKNESS (FT.) Δ_D	P.E.	D (FT.)	F	Δe (INCHES)
1	5	0	2.5	0.75	0
2	8	1	9	0.35	2.80
3	2	1/2	14	0.20	0.20
4	5	1	17.5	0.13	0.65
					Σ = 3.65

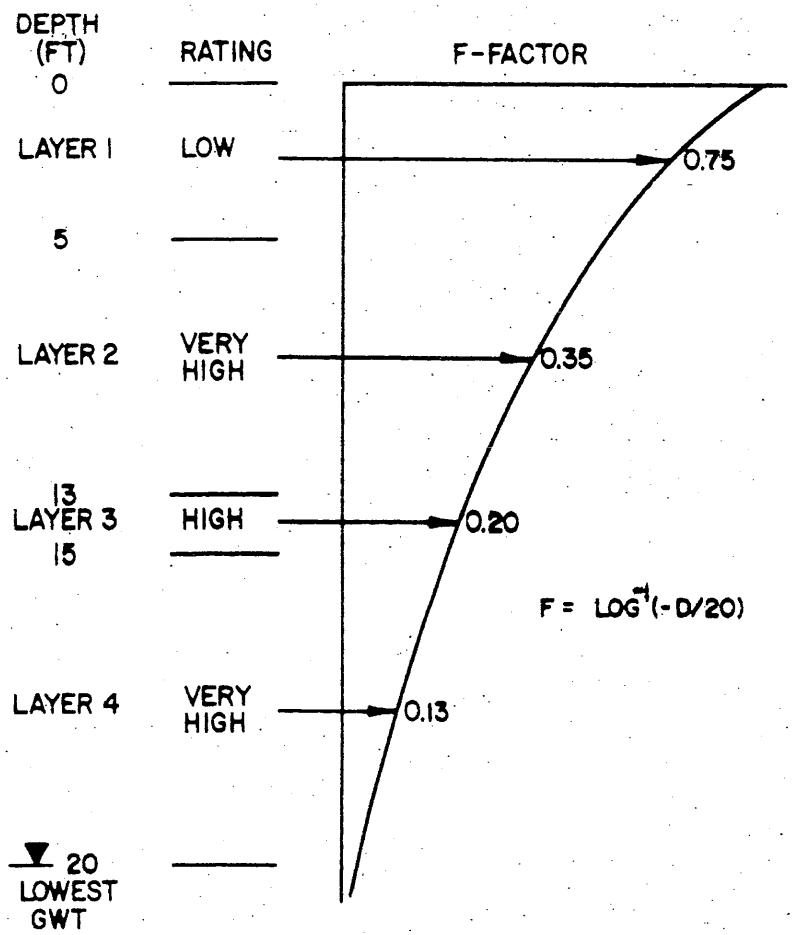


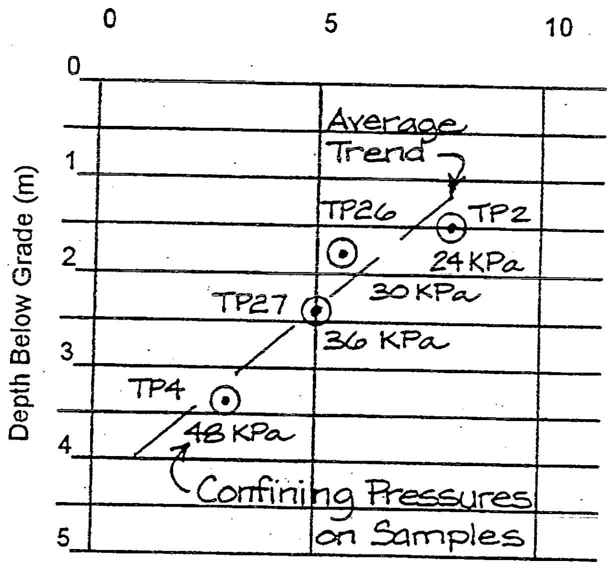
FIGURE 3 (continued)
Estimating Swell Using the South African Method

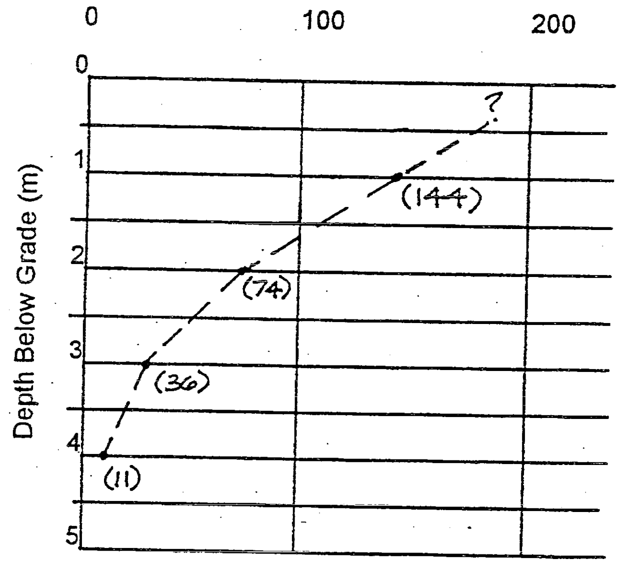
Figure 6: Procedure for South African Method (after NavFacs DM 7.3, 1982)

Figure 7. Okanagan University College: Swell Design Calculations

(Navfacs DM 7.1 (1982) method, and South African (Navfacs DM 7.3, 1982) method)

Estimated Swell for 6 metre Clay Layer Based on Lab Tests





Estimated Swell for 6 metre Clay Layer using South African Method

Depth (m)	Swell Potential	Thickness (m)	Potential Expansion (mm/m)	D (m)	F Factor	Computed Swell (mm)	Total Swell From Base (mm)
0							174
	High	1	80	0.5	0.83	66.4	
1							108
	High	1	80	1.5	0.56	44.8	
2							63
	High	1	80	2.5	0.38	30.4	
3							33
	High	1	80	3.5	0.26	20.8	
4							12
	Medium	1	40	4.5	0.18	7.2	
5							5
	Medium	1	40	5.5	0.12	4.8	
6							0

F Factor: F = Anti-log (-D/6)

Swell = F x Potential Expansion x Thickness

Figure 8. Kelowna: Percent Swell vs. Initial Moisture Content

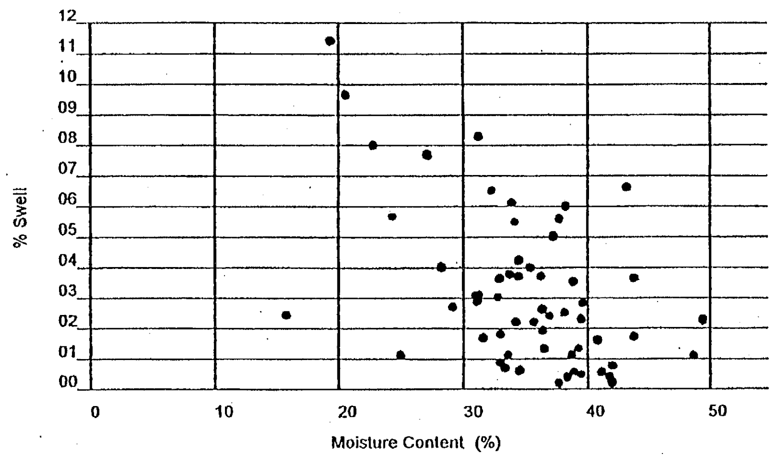


Figure 10. Westbank: Percent Swell vs. Initial Moisture Content

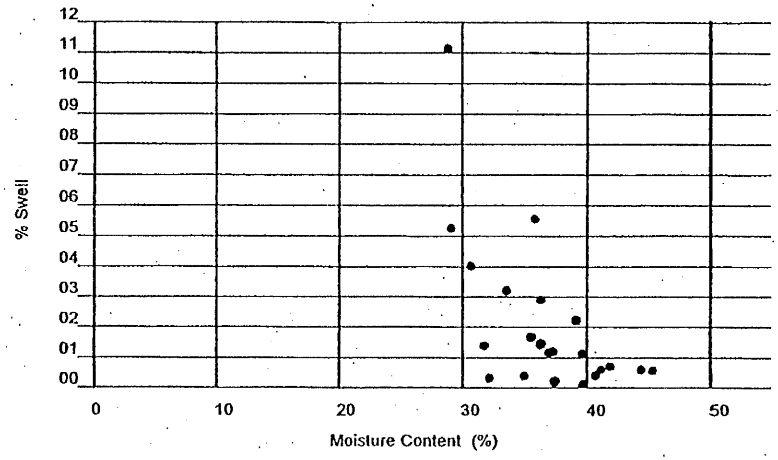


Figure 9. Kelowna: Percent Swell vs. (Moisture Content - Plastic Limit)

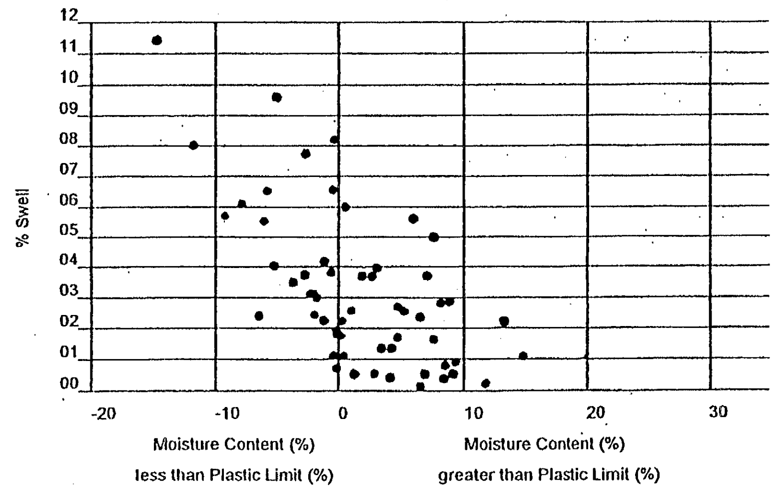


Figure 11. Westbank: Percent Swell vs. (Moisture Content - Plastic Limit)

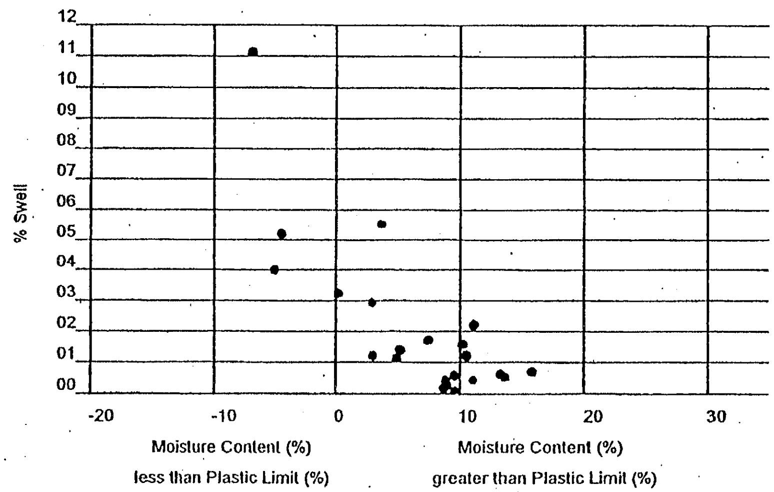


Figure 12. Vernon and Coldstream: Percent Swell vs. Initial Moisture Content

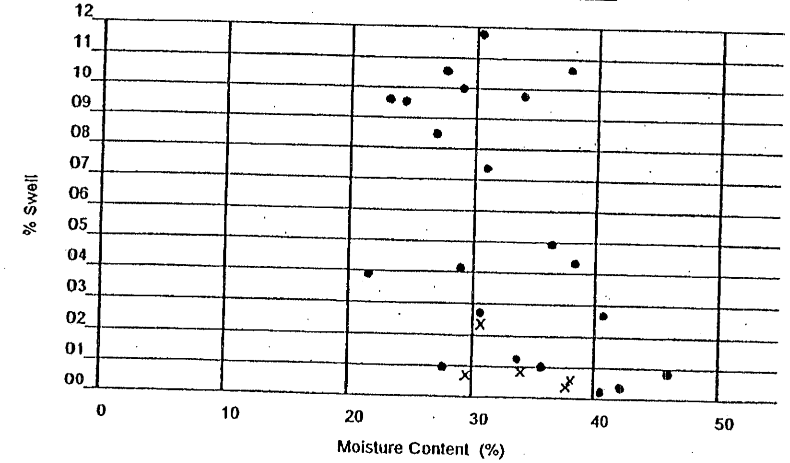


Figure 13. Vernon and Coldstream: Percent Swell vs. (Moisture Content - Plastic Limit)

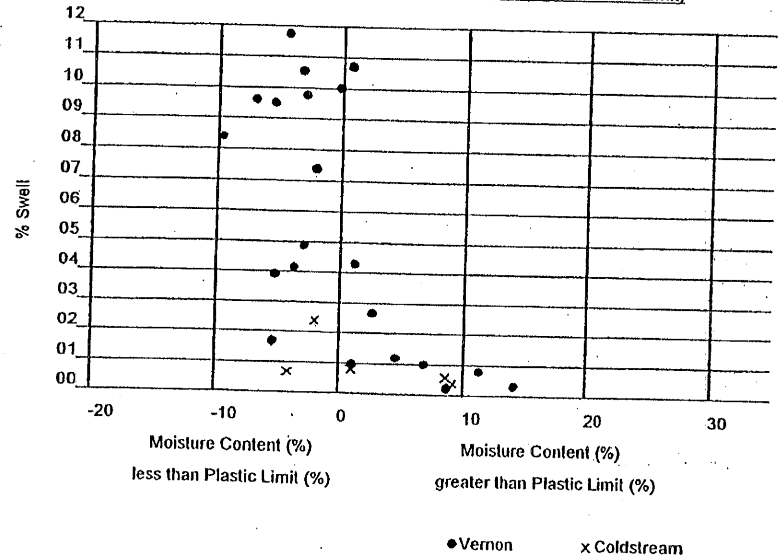


Figure 14. Armstrong and Salmon Arm: Percent Swell vs. Initial Moisture Content

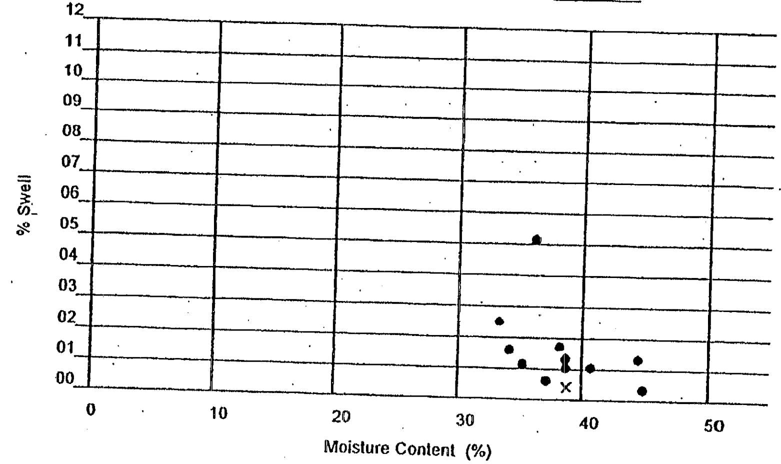
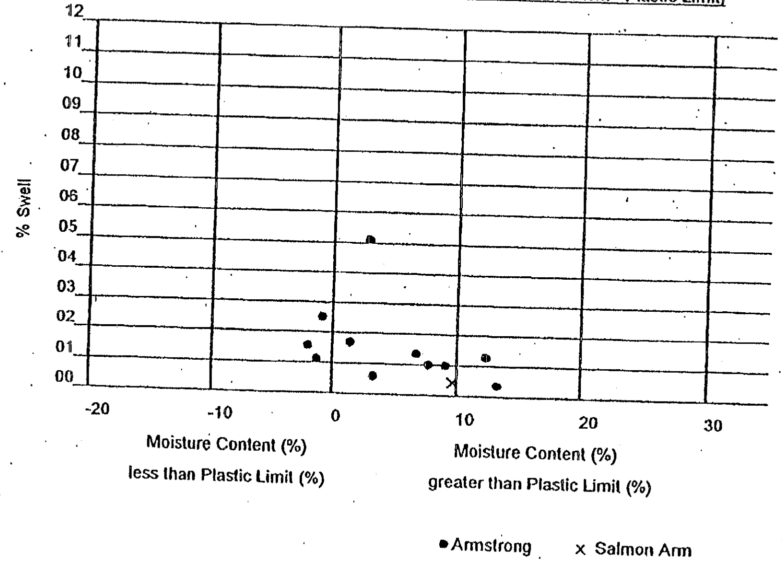
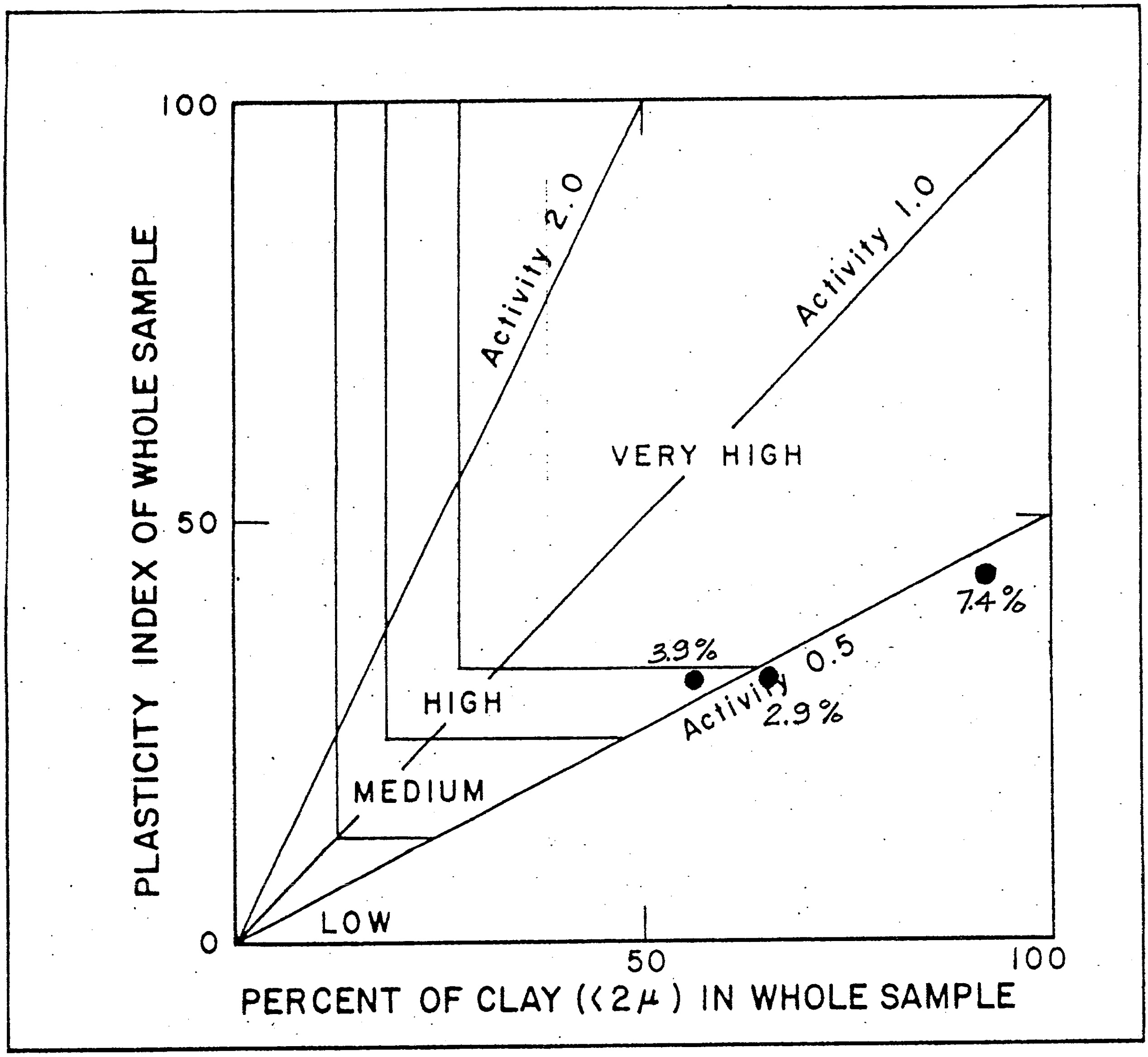


Figure 15. Armstrong and Salmon Arm: Percent Swell vs. (Moisture Content - Plastic Limit)





Volume Change Potential Classification for Clay Soils

Sample	Plastic Index	% Clay	Activity	Swell (%)	Seating Load (kPa)
TP1 - S5	43.0	92.0	0.47	7.4	6.2
TP1 - S8	31.0	56.0	0.55	3.9	12.5
TP1 - S10	31.0	65.0	0.48	2.9	25.0

Figure 16. Volume Change Classification for Clay Soils (after Canadian Foundation Manual, 1985)

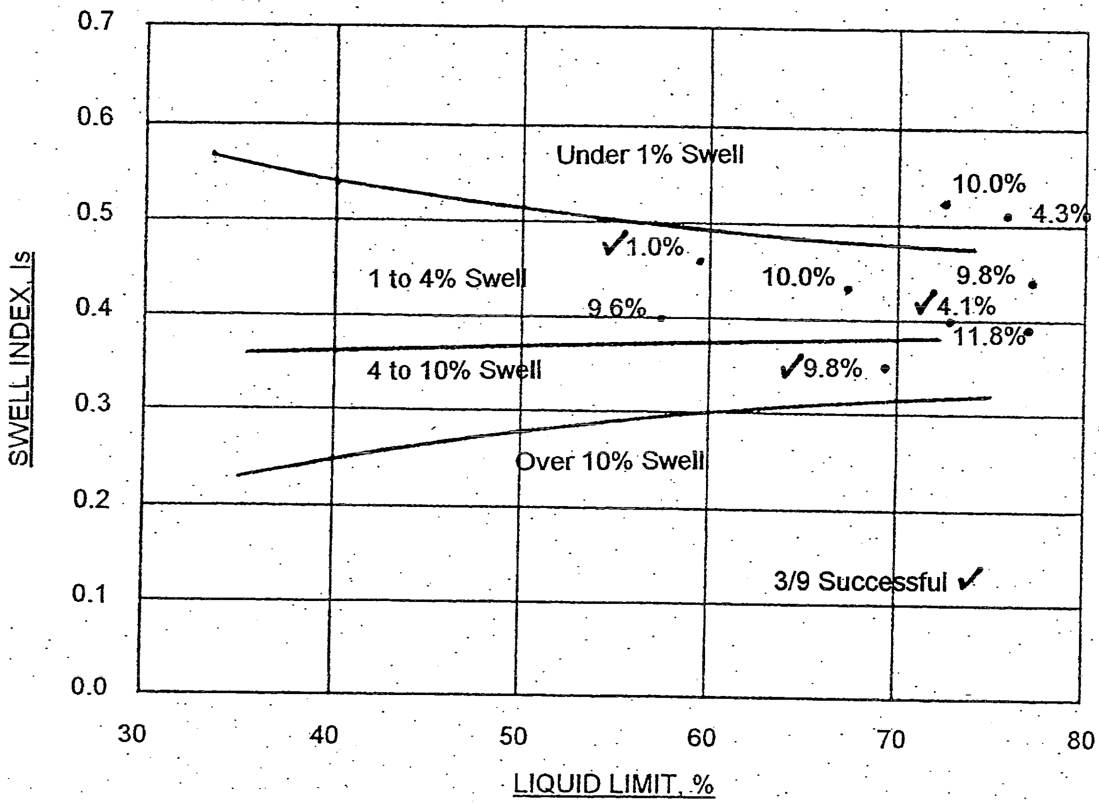
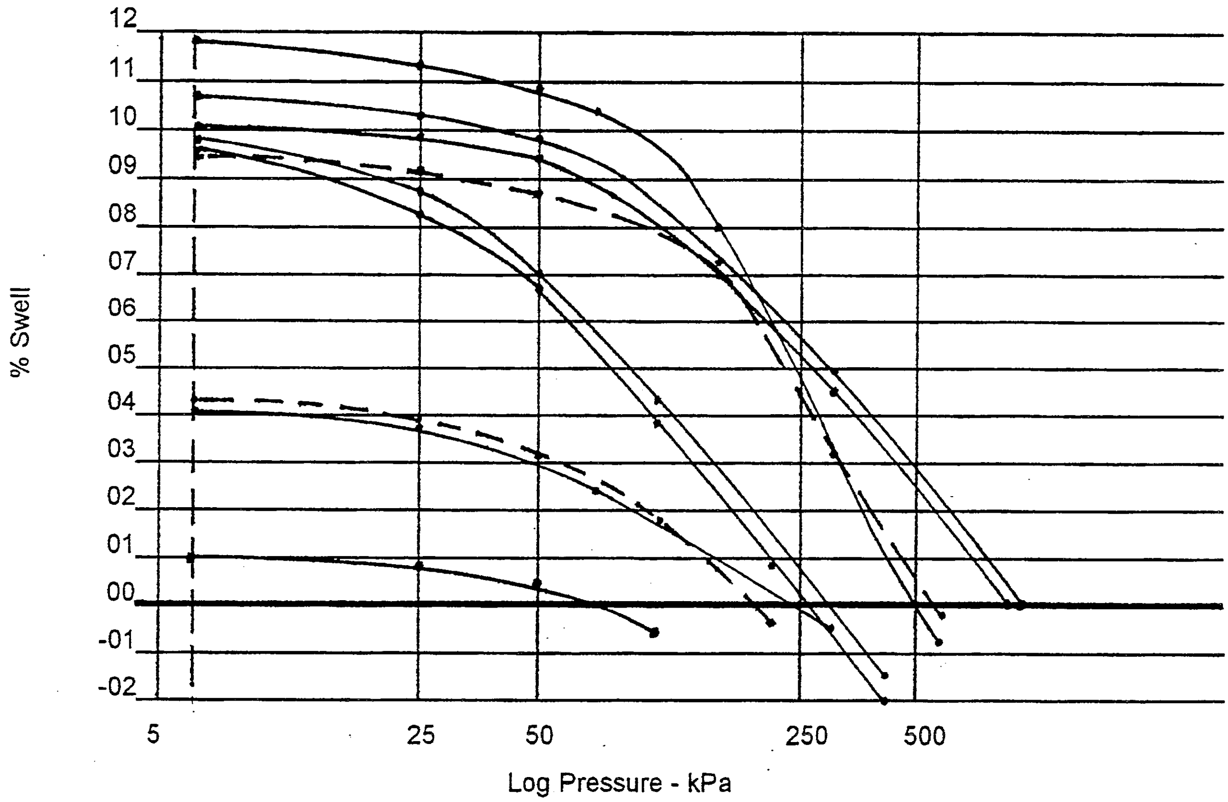


Figure 17: Relationship between Water Content and Liquid Limit for Expansive Clays (after Vijayvergiya and Ghazzaly, Proceedings, 3rd International Conference on Expansive Clay Soils, 1973)

SAMPLE	DEPTH, m.	W.C., %	P.L., %	L.L., %	Is = W.C./L.L.	% SWELL (6.25 KPa)
AH1	2.4	30.2	34.7	77.0	0.39	11.8
AH2	2.4	37.9	37.2	72.5	0.52	10.7
AH2	4.1	33.9	36.9	77.2	0.44	9.8
AH3	2.4	38.3	37.0	75.8	0.51	4.3
AH4	3.5	29.2	32.9	72.8	0.40	4.1
AH5	2.6	24.3	29.7	69.5	0.35	9.5
AH6	1.8	27.6	26.5	59.7	0.46	1.0
AH7	2.1	23.0	30.0	57.4	0.40	9.6
AH8	2.4	28.9	29.0	67.3	0.43	10.0

Table One: Data from Figure 18 for Comparison Plot with Proposed Relationship on Figure 17

Figure 18. Vernon, 15th and Hwy 6. 1996: Typical Swell Test Results



Swell Test Data

Auger Hole	Depth (m)	Initial Moisture (%)	Swell (%)	Final Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Swelling Pressure (kPa)
1	2.4	30.2	11.8	32.7	77.0	34.7	500
2	2.4	37.9	10.7	41.5	72.5	37.2	900
2	4.1	33.9	9.8	39.8	77.2	36.9	275
3	2.4	38.3	4.3	41.1	75.8	37.0	190
4	3.5	29.2	4.1	32.0	72.8	32.9	230
5	2.6	24.3	9.5	27.1	69.5	29.7	550
6	1.8	27.6	1.0	28.0	59.7	26.5	85
7	2.1	23.0	9.6	39.5	57.4	30.0	250
8	2.4	28.9	10.0	31.4	67.3	29.0	900