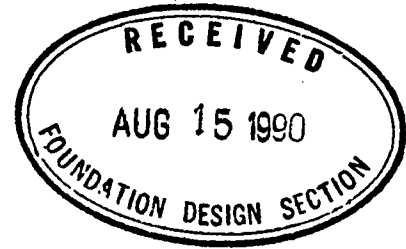


GEOCROS No:

31G5-156E-12



Golder Associates Ltd.
CONSULTING ENGINEERS



SERIES II - (2)

REPORT TO
MINISTRY OF TRANSPORTATION ONTARIO

4b
—

ENGINEERING STUDY
PROPOSED CUT AND RAILWAY UNDERPASS
HIGHWAY 416
DISTRICT NO. 9 (OTTAWA)
NEPEAN ONTARIO

W.P. 121 - 87 - 00

Distribution:

14 copies - Ministry of Transportation Ontario,
Toronto, Ontario

2 copies - Golder Associates Ltd.,
Mississauga, Ontario

August, 1990

901-1339

TABLE OF CONTENTS

	<u>Page No.</u>
1.0 INTRODUCTION	1
2.0 TERMS OF REFERENCE	1
3.0 LIST OF DATA REVIEWED	2
4.0 SUBSURFACE CONDITIONS	3
5.0 ASSESSMENT OF UNDRAINED SHEAR STRENGTH FOR USE IN DESIGN	4
6.0 ASSESSMENT OF EFFECTIVE STRESS PARAMETERS FOR USE IN DESIGN	4
7.0 LOCAL EXPERIENCE WITH SIMILAR CUT SLOPES	5
8.0 DETERMINATION OF SAFE CUT SLOPE	5
8.1 Undrained Analysis	5
8.2 Effective Stress Analysis	6
8.3 Recommended Slope	6
9.0 MODIFICATIONS TO CUT SLOPES	7
10.0 REPLACEMENT OF SLOPES BY VERTICAL DIAPHRAGM WALLS	8
10.1 Design Earth Pressure(s)/Anchor Loads	9
10.2 Wall/Tie Back Arrangement	9
10.3 Provision of Frost Protection	11
10.4 Availability of Contractors/Costs	11
11.0 GROUND WATER CONTROL	11
11.1 Temporary Dewatering	12
11.2 Permanent Dewatering	14
12.0 COMMENTS ON ROADWAY EXCAVATION AND CONSTRUCTION	14
13.0 COMMENTS ON BRIDGE FOUNDATIONS AND CONSTRUCTION	16
14.0 SUMMARY AND CONCLUSIONS	18
LIST OF TABLES	
LIST OF FIGURES	
APPENDIX A - SUMMARY OF SITE GEOTECHNICAL DATA	

LIST OF TABLES

1. Summary Data for Existing Local Experience
2. Assessment of Undrained Analysis

LIST OF FIGURES

1. Site Location Plan
2. Borehole Location Plan
- 3-A Section A-A
- 3-B Section B-B
4. S_u vs Elevation and Design Profile (Acres UU Test and Vane Data)
5. S_u vs Elevation (MTO/Golder Lynwood Vane Data)
6. S_u vs Elevation (Golder data on Jock River)
7. Composite Plot of All S_u Data and Design Profile
8. Mohr Circles and Envelope (Acres Interpretation)
9. Revised Interpretation for Effective Stress Parameters
10. "Normalized" Slope Data $\gamma H/S_u$ vs β (Slope Angle)
11. Stability Analysis (Undrained) using Design Profile (7 m Slope)
12. Stability Analysis (Undrained) using Design Profile (11 m Slope)
13. Summary of C' , ϕ' and F from Effective Stress Analysis
14. Approximate Design Lateral Pressure Distributions
15. Diaphragm Walls - Suggested Arrangement
16. Configuration of Caissons/Bridge Abutment
17. Temporary Excavation For Bridge Pier Construction

1.0 INTRODUCTION

The Ministry of Transportation Ontario (MTO) retained Golder Associates Ltd. (GA) to carry out an engineering study along the proposed Highway 416 adjacent to the Lynwood subdivision in Nepean, Ontario (see Site Location Plan, Figure 1). Highway 416 is to enter the Ottawa area as a six lane highway along the route of and just east of Cedarview Road, in the city of Nepean. It is planned to carry Highway 416 under the main CNR tracks which are immediately south of Baseline Road. The vertical alignment results in a cut of about 11 m at the railway, with the cut depth gradually decreasing towards the south as the highway approaches existing grade south of Bell High School at the top of an escarpment area. This cut section is adjacent to the Lynwood Subdivision. North of the railway track the ground surface falls off relatively quickly and the cut depth also decreases. This report presents the results for the study and also incorporates the discussion and comment during various meetings between MTO and GA, and between MTO, GA and FENCO.

2.0 TERMS OF REFERENCE

The scope of work for this project was outlined in Golder Associates' proposal to MTO, dated April 12, 1990.

After an assessment of available data on the shear strength parameters of the silty clay, the feasibility of the deep cuts along Highway 416 near the railway crossing is to be examined. The use of a diaphragm wall as a means of retaining the soil in the deeper sections of the cut (and also decreasing the volume of excavated clay to be wasted) is also to be examined. Considering the harsh-climate of the Ottawa area, means to protect the wall from frost action are to be considered in this study. The earth pressure distribution required for the wall design is to be

included in the study as well as drainage concerns. The study would address whether competent contractors are available to construct this wall and at what approximate cost.

The excavation of clay to the depths proposed will probably involve a two stage dig. The study is to comment on excavation methodology and on procedures that would be required to provide trafficability to construction equipment operating on the surface of the clay. The study is to comment on the difficulties of transportation and disposal of the sensitive clay.

3.0 LIST OF DATA REVIEWED

- o Geotechnical and Groundwater study. Proposed Highway 416. Cedarview Road Corridor Near the Lynwood Subdivision. W.P. 146-74-00-3 District 9 (Ottawa) Nepean, Ontario (Golder Associates Report to MTO, Report No. 891-2208). January, 1990.
- o Foundation Investigation. Proposed Jock River Bridges, Highway 416. W.P. 128-87-07/08 District 9 (Ottawa), Nepean, Ontario (Golder Associates Report to MTO, Report No. 891-2251). September, 1989.
- o Geotechnical Investigation. Proposed Diaphragm Walls and Stability of Cuts at Highway 416. W.P. 126-87-01-A District 9 (Ottawa), Nepean, Ontario (Golder Associates Report to MTO, Report No. 901-2256). August, 1990.
- o Foundation Investigation for Bridge Structure. Proposed Highway 416 and CNR Subway. District No. 9, Ottawa, WP 126-87-01, Site 3-544 (Acres Report to MTO). February, 1990.
- o Foundation Investigation Report. Geocon Limited, Consulting Engineers. Proposed Highway 7 and 15 Underpass, District 9 (Ottawa) - W.P. 908-64.
- o MTO Borehole data, 1966 and 1984.
- o Golder Associates in-house data regarding experience of local slopes.
- o Golder Associates Report No. 901-2115 on Pump Test. (In preparation).

4.0 SUBSURFACE CONDITIONS

The subsurface conditions in the study area, along the route of the proposed Highway 416, generally consist of sensitive silty clay of variable thickness overlying discontinuous deposits of silty sand and gravel which, in turn, overlies dolomitic limestone bedrock. In some areas, a layer of glacial deposit composed of sandy silt and clay and gravel follows the silty clay layer, and this glacial till layer overlies bedrock. The ground water level measured in standpipes sealed in the overburden and the bedrock indicates a general pattern of underdrainage.

The locations of boreholes put down in the area of deeper cut are shown on Figure 2, the borehole location plan. The boreholes were put down during current (1990) geotechnical investigations carried out by GA, and during previous investigations by MTO, GA and Acres. The soil profiles along section A-A (west of centreline of highway alignment) and B-B (east of centreline of highway alignment) are shown on Figures 3A and 3B, respectively. The subsoil conditions generally consist of a relatively deep deposit of sensitive silty clay, the upper layer of which is grey-brown and weathered, the lower layer being grey in colour and unweathered; this is followed by granular glacial and fluvial deposits which overlie dolomitic limestone bedrock. The bedrock profile was determined by boreholes where coring was carried out and inferred by boreholes which were augered to refusal. Boreholes were spaced generally at 20 m intervals. In general, the bedrock surface is relatively level, ranging from Elevation 71 m to Elevation 76 m across the site. Karst structures were not encountered at the borehole locations, though localized occurrence of such conditions has been reported in the Ottawa area. The soil

and rock conditions defined in the current investigations are consistent with the results of previous investigations.

During service clearance for drilling in the current 1990 investigations, a gas, a sanitary and 1,220 mm diameter watermain lines were reported in the general area of the deep cut section. The approximate locations for these lines are shown on Figure 2.

5.0 ASSESSMENT OF UNDRAINED SHEAR STRENGTH FOR USE IN DESIGN

Data concerning undrained shear strengths of the clay at the site and in the general area have been obtained by GA, MTO and Acres. The variation(s) of undrained strength, S_u , with elevation are plotted in Figures 4, 5 and 6, based on Acres data, MTO and GA Lynwood data, and GA Jock River data respectively.

A composite plot for all of the data is shown on Figure 7, which indicates the variation of S_u with elevation; from the S_u data, a design profile is delineated. This S_u profile comprises a strong top layer followed by a thin soft layer of 20 kPa and a thick lower layer of 30 kPa.

6.0 ASSESSMENT OF EFFECTIVE STRESS PARAMETERS FOR USE IN DESIGN

The Mohr circles for consolidated undrained triaxial tests with pore pressure measurements carried out by Acres are shown on Figure 8, which indicates an interpretation of the strength envelope ϕ' of 23° in the normally consolidated stress range. For the average stress range appropriate for 5 m to 15 m slope cuts, a strength envelope of $\phi' = 25^\circ$ to 26° can be drawn, together with a cohesion intercept, C' , which varies from about 0 to 12 kPa. This interpretation

for the effective stress parameters is shown in Figure 9. For purposes of analysis, a value of 5 kPa was used for C' .

7.0 LOCAL EXPERIENCE WITH SIMILAR CUT SLOPES

A synthesis of some local experience with safe slope cuts in sensitive soft clays is illustrated in Figure 10, which plots $\gamma H/S_u$ (where γ = total unit weight of soil, and H = height of slope cut) against the slope angle β . Tabulated slope data on which data in Figure 10 are based, are shown in Table 1.

On the basis of these data, a boundary limit can be set to represent "safe local slopes" in terms of strength, height and slope angle. However local experience has also shown that surface sloughing can occur even in "safe" areas, if the slope surface is steeper than 26° .

8.0 DETERMINATION OF SAFE CUT SLOPE

The stability of a proposed slope cut of 2.5(H):1(V) ($\beta = 22^\circ$) is examined below. Design analyses are based on the design strength parameters previously determined.

8.1 Undrained Analysis

The results of slip circle analysis for 7 m and 11 m slopes, using the design profile in Figure 7, are shown on Figures 11 and 12 respectively. For a cut slope of 11 m, the calculated factors of safety, F , for the cases of no surcharge and an applied rail surcharge of 75 kPa are 1.4 and 1.05 respectively. The calculated factors of safety are shown on Table 2. These calculated values of F may be compared with the local slope data on Figure 10 and the "calculated" safety factor, F , can thus be calibrated to

derive an "operational value", F' , which correlates with observational data. From Figure 10, the operational factor of safety, F' , for a 7 m cut is thus about 1.5. For an 8 m cut it is about 1.3 to 1.5 and, for a deeper cut of 11 m, it reduces to about unity.

8.2 Effective Stress Analysis

The results of effective stress analyses using the parameters derived from Figure 9, namely $C' = 5$ kPa and $\phi' = 26^\circ$, are shown on Figure 13. These parameters are consistent with local failures at the toe of steep slopes due to high r_u values (where r_u = ratio of pore pressure to total vertical pressure) where F approaches unity. The phreatic surface and potential slip surfaces suggest an average pore pressure ratio, r_u of about 0.3 over the entire slope with a local r_u of 0.5 at the toe. Computed factors of safety range from 1.3 for a 5 m to 6 m cut, to 1.2 for a 10 m to 11 m cut, and to 1.1 or less for 15 m cut. Reference to the normalized slope data shown on Figure 10, indicates that these effective stress analyses are consistent with the slope experience reflected in Figure 10.

8.3 Recommended Slope

Based on this assessment it is considered that both temporary and final cuts of between 7 m and 8 m can be safely excavated at side slopes of 2.5(H):1(V), which fits within established precedents in this area. For design, it is recommended that the final cuts be restricted to a maximum depth of 7.5 m, using side slopes of 2.5(H):1(V).

The drainage system in the slope is essential for long term stability and erosion protection. Treatment of the slope

surface for erosion protection should be provided by a granular cover consisting of a 0.5 m thick layer of gravel, cobble, or fine rockfill, overlying 0.5 m of concrete fine aggregate (sand). Drainage within the slope may be provided using conventional counterfort drains at 10 m centres and at least 1.8 m deep below surface grade to prevent freezing of the drains. Trenching should be carried out continuously, being backfilled immediately with concrete fine aggregate (sand).

9.0 MODIFICATIONS TO CUT SLOPES

Cuts within Leda clay deeper than 7.5 m at 2.5 (H):1(V) are beyond most precedent data and will require flatter side slopes or modifications to the slopes to ensure stability. Accordingly, the maximum depth of cut in clay for this project should be 7.5 m. This critical height corresponds to the distance from ground surface to the maximum excavated level, namely, the ditch invert (Refer to inset in Figure 10). Various possible modifications of cut slopes through the use of berms and partial retaining walls have been examined; the intent being to steepen the cut slopes without reducing stability. Both upper walls and toe walls have been examined, but are considered to be unsuitable options for the site. An upper wall will have to be founded in the soft clay layer; a toe wall requires a substantial initial excavation at the toe of the slope, thereby reducing the overall stability of the slope during construction; a toe wall for a deep slope cut will have also to be relatively massive, and to be effective, must be founded on bedrock. Thus both of these types of modifications are not recommended. As an alternative, it is recommended that consideration be given to replacing the cut slope with a vertical wall, as discussed below.

10.0 REPLACEMENT OF SLOPE BY VERTICAL DIAPHRAGM WALLS

Adequate precedents for permanent anchored walls exist. Case histories of the construction and behaviour of such anchored walls are documented in the literature, and confirmed by our experience with diaphragm walls.

To replace cut slopes deeper than 7.5 m, vertical diaphragm walls with tie backs are recommended. The site conditions are favourable for such a retaining system. Bedrock is at a relatively shallow depth so that vertical walls could be taken to rock and inclined tie backs could also be anchored in rock. A further advantage is that concrete walls taken to rock can be incorporated into the design of railway bridge abutments, if required.

At locations where the cut depth is marginal, namely, at or only slightly exceeding 7.5 m, the following may be considered:

- o Raising the road grade to reduce the required depth of cuts;
- o flattening the slope to 3.5(H):1(V) to increase stability should space permit;
- o removing soil at the top of the slope for a distance of 15 m to 20 m behind the crest to reduce height of slope; and,
- o constructing a mid-height berm.

Where soil is removed from the top of slope, the depth of excavation preferably should not exceed 0.5 m, so as to leave as much stiff clay crust as possible. The option for constructing a mid-height berm is not desirable since construction and equipment traffic will disturb the soft clay berm.

10.1 Design Earth Pressure(s)/Anchor Loads

The prediction of lateral earth pressures for this type of construction involves assumptions concerning both the soil parameters and the method of construction. Estimates of the possible range of lateral pressures to be expected, based on established prior experience, is illustrated in Figure 14. Possible envelopes of wall loading, based on both rectangular and triangular earth pressure distributions and on possible hydrostatic pressures are shown in the figure. (Note: Recorded observations and data for tied back walls show wide variations in loading. However, since in practically all cases the anchors were pre-stressed to approximately the design loads, the measurements only indicate that the design loads were not exceeded).

Taking the long term loading conditions into account, a lateral coefficient of earth pressure of 0.5 should be used in design. Possible design loadings are illustrated in Figure 14 for a cut with 10 m depth. For the wall loadings estimated, tie back anchor loads of the order of 500 kN to 1,000 kN are possible, depending on the pattern and number of anchors used. Such anchor loads can readily be developed by anchorage within the dolomitic limestone bedrock at the site. Some design parameters are presented in Appendix A.

10.2 Wall/Tie Back Arrangement

Deep excavations in soft to firm clays are subject to base heave failure if the foundation soil is overstressed in shear. Based on the shear strength data, vertical deep cuts at the site would be subject to basal instability if founded in the soft to firm clay layer. However, if rigid

walls are extended below the bottom of the clay cut, the wall stiffness reduces the tendency for the clay to be displaced toward the excavation and consequently inhibits heave. In areas where the vertical wall cut is to be located, the stratigraphy generally consists of soft to firm clay overlying a layer of loose sandy silt till, unsuitable for use as a founding stratum. It is, therefore, necessary to extend walls, a minimum of 0.5 m below the surface of the dolomitic limestone to unweathered rock.

Provided that the wall is rigid and good workmanship is maintained, horizontal deformations during construction should be modest and less than 0.5 per cent of the depth of excavation. Adequate rigidity is normally obtained by using a concrete wall thickness of 0.7 m and sufficient reinforcement to give a wall stiffness approaching 1,000 MN m²/m of wall.

0.005

EI = 1000 MN/m²

A typical tie back diaphragm wall system is shown on Figure 15. In this example, the diaphragm wall shown (Part(a) of Figure 15) is supported by four levels of rock anchors, a row of anchors being installed at the toe of the wall for stability in preference to excavating for deep embedment in bedrock. Where the cut extends below bedrock due to road pavement requirements, the wall section may be founded in rock and provided with a rock-bench set-back as shown on part(b) of Figure 15.

To inhibit corrosion, it is recommended that all tie backs be double-proofed. This is now standard practice for anchors and experience to date has been good.

10.3 Provision of Frost Protection

Nepean is in an area with a Freezing Index of about 1000 degree (Centigrade) days with a possible depth of frost penetration of 1.8 m. Where concrete diaphragm walls are used, insulation is required to prevent freezing of frost susceptible soils behind the walls which could induce excessive lateral forces due to frost action. It is suggested that Styrofoam SM, or equivalent, be used as insulation, with a minimum thickness of 150 mm being provided behind the wall facing.

10.4 Availability of Contractors/Costs

In our opinion, there are adequate numbers of contractors who are experienced in construction of this kind of wall system.

Recent contracts in Toronto and Ottawa suggest that a unit cost of about \$1000 to \$1100 per square metre of wall face is a reasonable estimate for costing in 1991 dollars. (This figure assumes minimal chiselling of the wall into bedrock and includes the cost of tie backs, insulation and facing panels. A cost surcharge may need to be added for contractors' mobilization to Ottawa).

11.0 GROUND WATER CONTROL

Ground water control will be essential during construction and positive permanent under-drainage will be needed below the final pavement. To maintain stability of the base of the cuts, drainage of slope faces and at the toe of the slopes should be directed to a permanent longitudinal drainage system. It is likely that pumping with adequate controls will be required during construction.

The proposed highway cut will be carried out through sand and gravel, and through silty clay underlain, in areas, by pervious sandy deposits. To facilitate construction of the roadway through the pervious deposits, the ground water level will have to be lowered to below the level of the bottom of the excavation, so as to allow construction and excavation in the dry. Where the bottom of the excavation is underlain by impervious silty clay which is, in turn, underlain by a pervious stratum under hydrostatic pressure, basal heave or a "blow" may occur if the net hydrostatic pressure on the bottom of impervious stratum is greater than the weight of the overlying soil. To prevent basal heave and disturbance of the sensitive clay, the piezometric pressure of the bottom pervious layer must be lowered.

11.1 Temporary Dewatering

The piezometric water levels in the lower deposits and bedrock, immediately beneath the base of the open cuts and within diaphragm walls, should at all times during excavation and following completion of excavation, be below the level of the base. The water level in the lower deposits and bedrock should be maintained at this lowered level throughout the construction period of the structure and until a permanent under-drainage system is installed and operating.

It is recommended that a "performance specification" be used in that only the results which must be achieved by the temporary dewatering system are specified; the type of dewatering system to be installed at the site being the responsibility of the contractor.

The design, installation and maintenance of the temporary dewatering system should be the responsibility of the contractor, but the system installed at the site should be acceptable to the Engineer/MTO and should meet certain minimum requirements. Consequently, the contractor should be instructed to submit a detailed description of the proposed temporary dewatering system for approval at least 2 weeks before he intends to begin installation of the system. The system should be designed to lower and maintain the piezometric water level in the bedrock and lower pervious deposits to below the base(s) of excavation at all stages of excavation. The system should be designed and installed by a recognized company or person experienced in the design and operation of dewatering systems.

The bedrock and lower pervious deposits will recharge if pumping is stopped. Furthermore, because a large part of the base of the excavation is within granular soil which may "pipe", if subjected to any appreciable upward seepage force, standby pumps and alternative power sources should be available at the site.

To enforce the performance specification, piezometric levels below the base must be monitored during excavation and throughout the period that the temporary construction excavation will be open. The drawdown should be monitored by means of piezometers and by standpipes. The location of piezometers or standpipes will depend on the locations at which wells or well-points are installed and consequently can only be selected after the contractor's particular dewatering system has been approved. Provision should be made in the specifications for installing monitoring piezometers on at least 50 m centres within the area of the excavation bottom. All of these piezometers need not be installed initially, but could prove to be necessary should

the drawdown pattern beneath the area of the excavation bottom be erratic. These piezometers should be installed near the toe of the temporary excavation side slopes so that they will not be covered by the structure and thus be destroyed during construction.

11.2 Permanent Dewatering

In the long term, it is necessary to control the ground water inflow in the slopes and prevent buildup of hydrostatic pressure below the roadway.

The underdrainage system for permanent dewatering may consist of continuous embedded horizontal drains along the length of the roadway together with vertical relief wells, as required. Such wells could be located within the east and west ditches of the roadway cut in the deep sections.

Details of the permanent drainage system can best be defined after assessment of the pump test data (Report 901-2115, in preparation).

12.0 COMMENTS ON ROADWAY EXCAVATION AND CONSTRUCTION

Two stage excavation will likely be required where the depth of the cut is about 7 m to 8 m; where the depth reduces to about 6 m, the excavation could readily be carried out in one stage.

All the excavation should be planned using large hydraulic shovel(s) with trucks for haulage. No heavy equipment should be permitted to operate within 1.5 m of the final subgrade. Any excavation equipment must be fitted with a smooth excavation blade. Conventional excavator buckets

with teeth will cause unnecessary and excessive disturbance to the subgrade.

In general, the soft clays are weak for road subgrade and require sub-excavation and replacement with granular material. The sands and till are suitable for road subgrades and do not require sub-excavation. Localized loose granular native soils may require compaction by a few passes of a roller.

Temporary slopes may be cut to a maximum depth of 7.5 m; excavations below this depth, such as for the construction of pavement subbase and base courses, should only be carried out in strips of 5 m perpendicular to the slope which are then immediately backfilled. No storage of equipment or stockpiles should be permitted at the head of slopes.

As a possible example for consideration, it is suggested that, where the subgrade is more than about 1 m below the ground surface in frost susceptible soils such as silty clay or silty fine sand, the subgrade be protected against frost penetration with at least 50 mm of high strength polystyrene insulation installed beneath the subbase. The insulation should be underlain by a levelling layer of granular material where required. Insulation sheets should all be overlapping or ship-lapped. With insulation, the total pavement thickness may possibly be somewhat reduced.

The placement of granular material should follow excavation. The lowest lift of granular material should be placed in a thickness of at least 0.75 m to allow haulage vehicle access, although thinner lifts have been used in the past. The subgrade must be properly shaped and graded to promote drainage to the sub-drains.

Based on the water content and Atterberg limit data available, most of the silty clay and clayey silt will not be suitable for use as engineered fill; however, the weathered silty clay within about 1.0 m of ground surface can be considered for use in embankment fill construction. Since these soil types are sensitive to changes in water content and wetting, earthwork using the native weathered silty clay may only be carried out during consistent dry weather. Re-use of this weathered material may not be practical economically, since it would involve two stage excavation. The wet, weathered silty clay and the grey silty clay, although sensitive to disturbance, should remain intact ("solid") throughout haulage and disposal and could probably be piled to a low height at the disposal site ("liquid-like" spoil is not expected). Haulage and spreading equipment will probably not be able to travel on any of the excavated, disturbed, wet clay material until the material has had a considerable drying period. Even then, only a bulldozer could operate on the fill; a thick granular road would have to be constructed on the clay to enable haulage vehicle access.

13.0 COMMENTS ON BRIDGE FOUNDATIONS AND CONSTRUCTION

The bridge abutment for the CNR at the crossing may consist of a relieving platform with cassions founded in bedrock. It is recommended that cassions be embedded at least 0.5 m in bedrock and provided with sockets in unweathered good rock. Recommended geotechnical design parameters are given in Appendix A. A typical section showing a configuration of the bridge abutment and tie back wall system at the CNR crossing is shown in Figure 16.

For the temporary excavations required for the construction of bridge piers, where the total excavation is greater than

7.5 m, it is recommended that, to maintain stability, the section shown in Figure 17 be specified for construction. Temporary slope protection should be provided to the exposed clay face.

During construction the existing railway tracks will be temporarily diverted. An outline of possible sequence of construction events follows:

- o Construct diaphragm walls north of existing rail track.
- o Relocate tracks to north of existing tracks. (Railway tracks supported on native ground).
- o Dismantle existing (old) tracks.
- o Construct the remaining diaphragm walls.
- o Construct caisson supported platform at the crossing section (see Figure 16).
- o Install dewatering systems and monitoring piezometers.
- o Begin excavation south of relocated (new) track after pre-drainage of site is in progress.
- o Install tie back anchors as excavation proceeds.
- o Install additional dewatering units as required.
- o Construct bridge piers for the permanent track crossing. (See Figure 17).
- o Relocate rail tracks to the bridge piers.
- o Dismantle temporary tracks and begin excavation north of permanent tracks.
- o Construct permanent dewatering system and highway pavement structure.

14.0 SUMMARY AND CONCLUSIONS

- o From a review of available geotechnical data and subsurface information gathered in our 1990 field investigation, an assessment of undrained shear strength and effective stress parameters for use in design is made.
- o Based on the strength data and our experience with similar cut slopes in clay, it is considered that slopes at 2.5(H):1(V) can be excavated safely to 7.5 m depth.
- o The results of undrained and effective stress stability analyses carried out using the design profile and interpreted effective stress parameters respectively are consistent with data regarding local slopes.
- o Cuts in Leda clay deeper than 7.5 m at 2.5(H):1(v) are beyond most precedent data and require modifications to ensure stability. Alternatives were examined and replacement of slope by a tie back diaphragm wall is considered the most feasible, effective system, for the given site condition.
- o Groundwater control is essential during construction and permanent underdrainage system below the final pavement is necessary to maintain stability of the base of cut. Drainage of slope faces is also needed. Groundwater monitoring by piezometers are necessary during and after construction.

GOLDER ASSOCIATES LTD.

R. Ng, P.Eng.

V. Milligan, P.Eng.

RMN/VM/dh

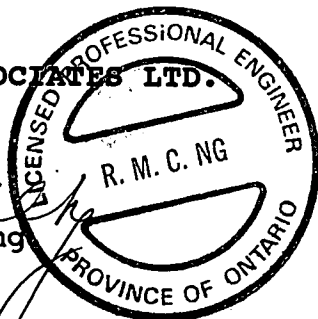


TABLE 1
SUMMARY DATA FOR EXISTING LOCAL EXPERIENCE

	LOCATION	SLOPE HEIGHT H (m)	SLOPE ANGLE β (deg) H:V		UNDRAINED STRENGTH Su (kPa)	REMARK OR REFERENCE
1	Heron Road at CN Beachburg Overpass-Confederation Heights	6	21.8	2.5H:1V	40 to 50	3.5 m weathered crust overlying firm grey silty clay.
2	Billings Bridge Transitway Station	7.5	21.8 to 26.6 ----- 27.8	2.5H:1V to 2H:1V ----- 1.9H:1V (some sloughing occurred)	70	(GA Report 821-2039) 6.4 m weathered crust overlying low plasticity grey silty clay.
3	Hunt Club Road, Underpass at CP Prescott Railway	5	18.4	3H:1V	24 to 43	(GA Report 73748) 2 m weathered crust overlying stiff grey silty clay
4	Orleans Blvd. Cut South of St. Joseph Blvd.	5	26.6 or flatter	2H:1V or flatter	25 to 30	(GA Report 821-2279) 2.3 m of sand overlying highly plastic firm grey silty clay
5	Prestone Drive Access Rd. - South of St. Joseph Blvd.	9 to 12 -----	18.4 -----	3H:1V -----	50 to 60	(GA Report 752160, 871-2254) weathered crust over silty clay
5a		5.5	32	Average 1.6H:1V		----- Quarried rock retaining wall
6	Highway 7 and 15 Underpass	9.2	26.6 ----- 18.4	2H:1V (in the top stiffer grey brown clay layer) ----- 3H:1V	35 to 100	(Geocon Report WP908-64) Fill underlain by 4.5 to 7 m stiff to firm silty clay overlying silt till overlying rock.
7	Duford St. Access Road	H<9.1 H=12.2 H=15.2	26.6 21.8 18.4	2H:1V 2.5H:1V 3H:1V	-----	(GA Report 69783) Recommended side slope angles to enhance slope stability

TABLE 2

ASSESSMENT OF UNDRAINED ANALYSIS*

H (m)	Calculated F (based on Field Vane S_u)	Operational F' (S_u reduced by 0.7)	Remark
11	1.4	~1	See Figures 10, 12
8	2.0	~1.3 to 1.5	F' interpolated from Figure 10 for 22° slope
7	2.2	~1.5	See Figures 10, 11

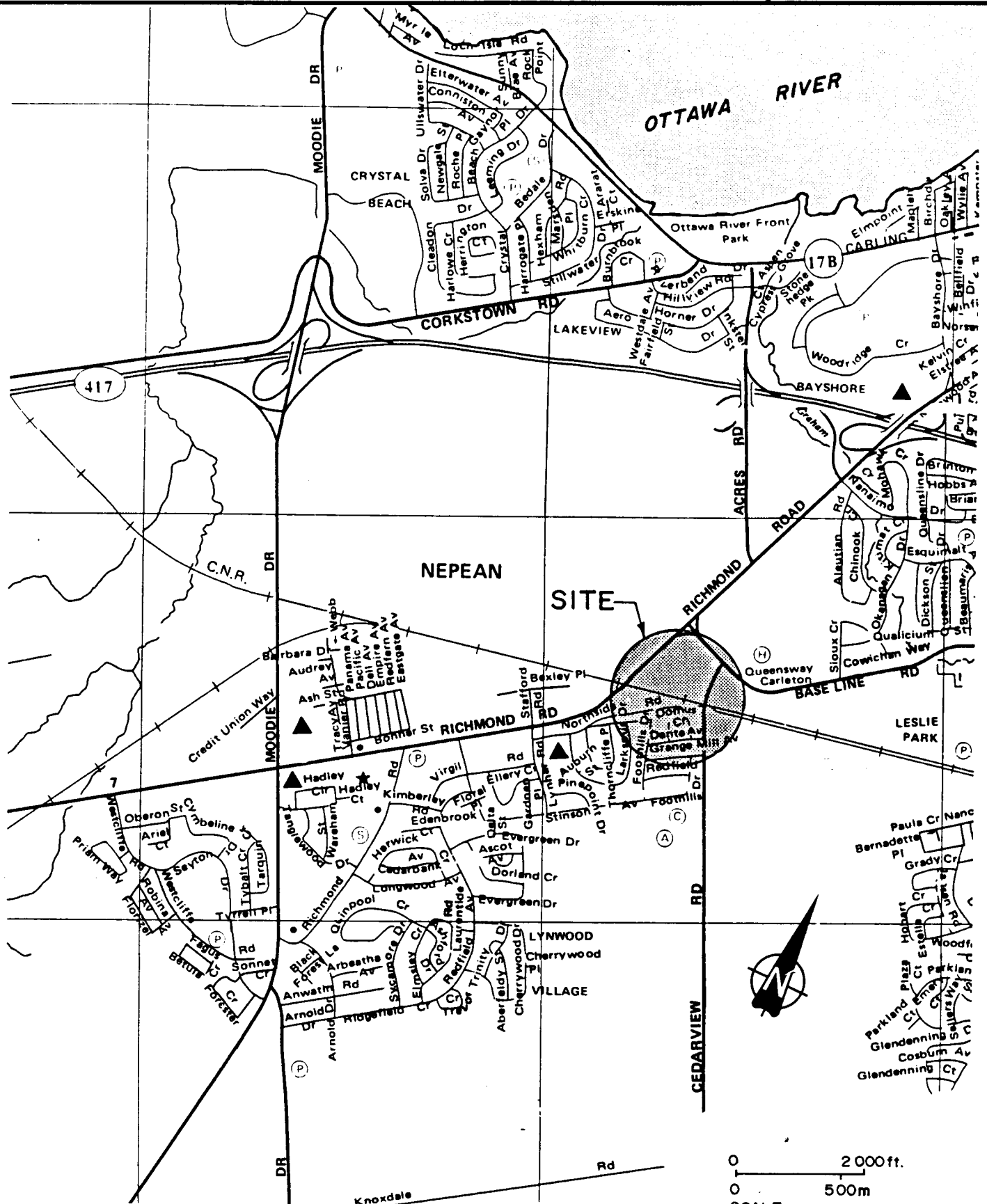
* See figures 10, 11 and 12 for undrained analyses and normalized undrained slope data.

August, 1990

901-1339

SITE LOCATION PLAN

FIGURE I



Date JUNE 12, 1990.

Project 901-1339

Golder Associates

Drawn R.B.C.

Chkd. *[Signature]*

FORM PRODUCED JUNE 1986

LEGEND

- CORED BOREHOLE
86.6 — GROUND SURFACE ELEVATION
75.3 — BEDROCK SURFACE ELEVATION
- AUGER BOREHOLE TO REFUSAL
86.4 — GROUND SURFACE ELEVATION
73.3 — ELEVATION OF AUGER REFUSAL (INFERRED TOP OF BEDROCK)
- BOREHOLE TERMINATED IN OVERBURDEN
87.3 — GROUND SURFACE ELEVATION

BOREHOLE IDENTIFICATION TABLE

BOREHOLE SERIES No.	INVESTIGATION BY AGENCY	YEAR OF STUDY
90-W1	GOLDER ASSOCIATES	1990
89-1	GOLDER ASSOCIATES	1989
88-1	GOLDER ASSOCIATES	1988
103	ACRES	1990
MT08	MT0	1984

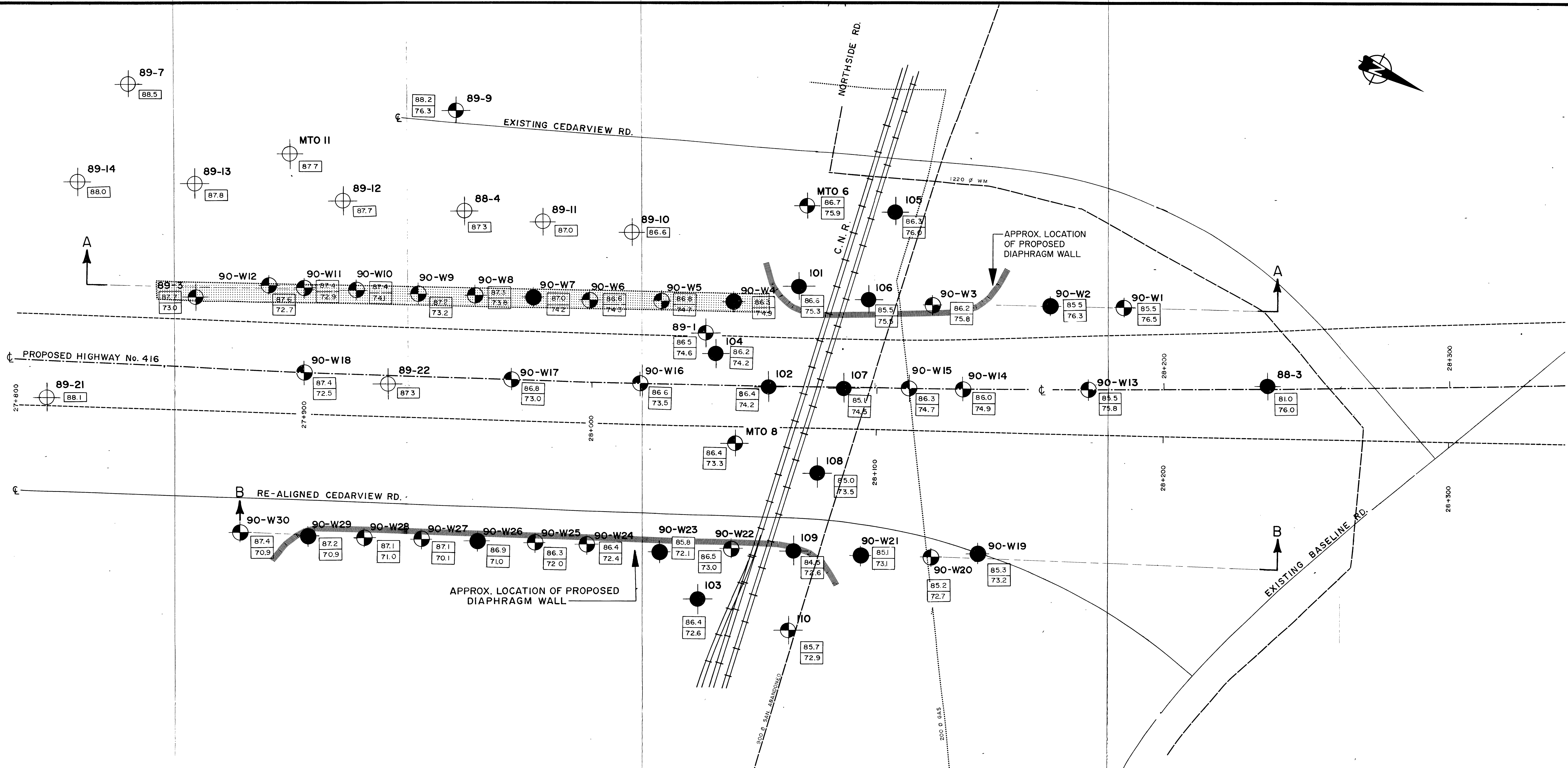
APPROXIMATE LOCATION WHERE DIAPHRAGM WALL IS PROPOSED.

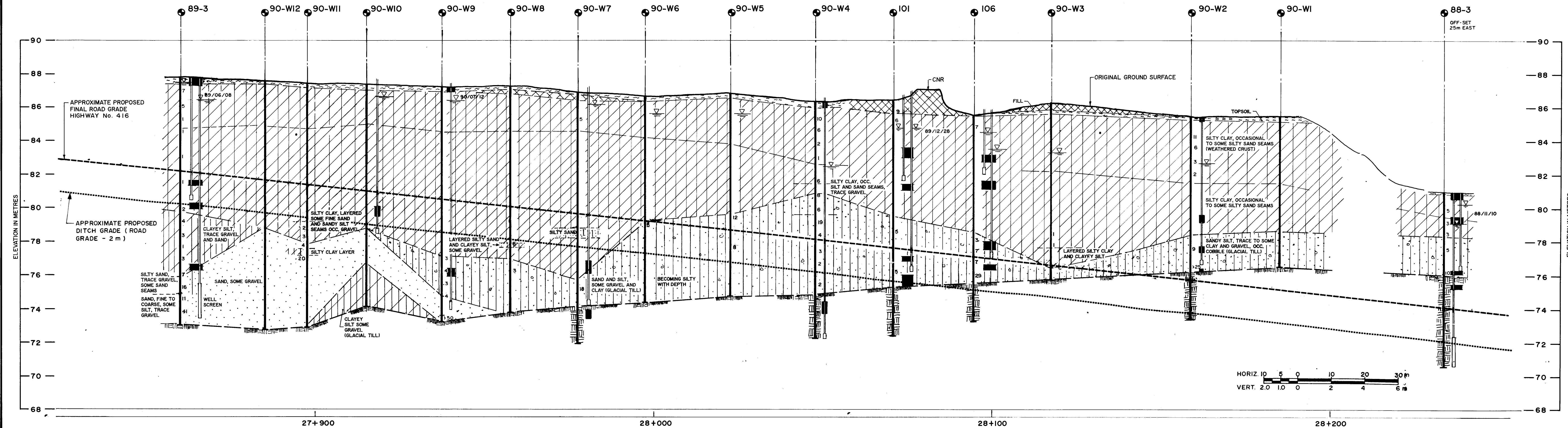
APPROXIMATE LOCATION WHERE DEPTH OF CUT IN CLAY IS AT OR MARGINALLY EXCEEDS 7.5m. REFER TO SECTION 10.0 IN THE TEXT FOR POSSIBLE OPTIONS INSTEAD OF DIAPHRAGM WALL.

NOTE: BEDROCK AND GROUND SURFACE ELEVATIONS ARE APPROXIMATE ONLY.

NOTE: THIS FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

0 5 10 20 30 50m
SCALE, METRES





LEGEND

90-W19 BOREHOLE NO.

- N BLOWS/0.3 m (Standard Penetration Test)

WL in piezometer (All measurement taken on 1990 July 27, unless otherwise noted).

Piezometer


 WL in open hole

SOIL UNITS

 FBI

Topsoil

 **Silty Clay**

 **Sandy silt, some gravel and clay (glacial till)**

 **Clayey silt, some sand and gravel
(glacial till)**

Sand, trace to some gravel

 Bedrock

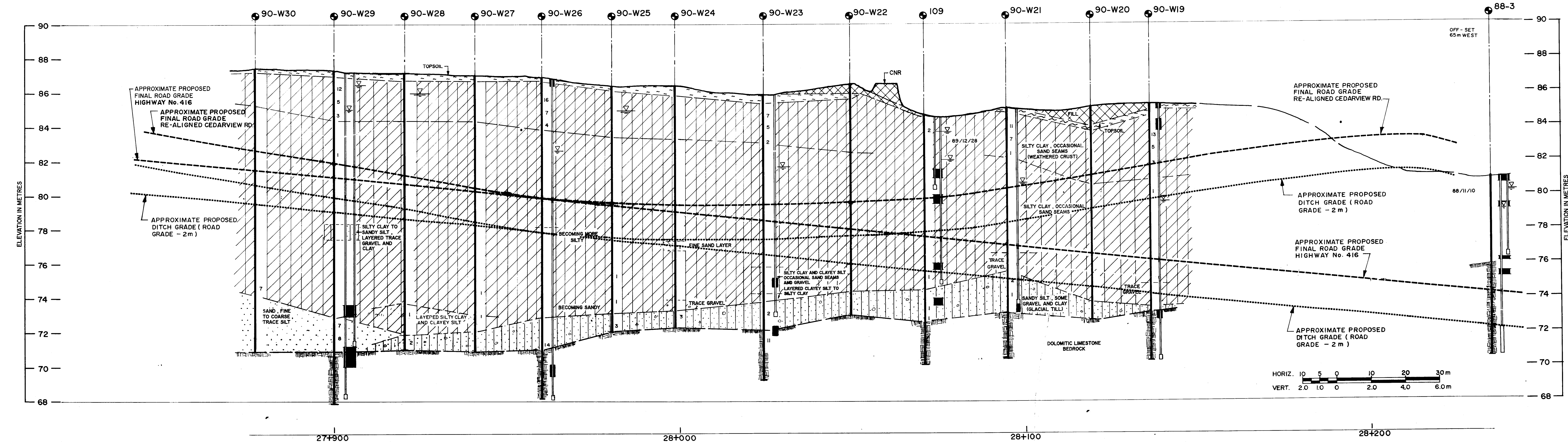
NOTE

1. Refer to Figure 2 for Borehole Location Plan.
2. The boundaries between soil units have been established only at borehole locations. Between boreholes the boundaries are assumed.

Date. **AUG. 3, 1990**
Project. **901 - 1339**

Golder Associates

Drawn.....D.M.
Chkd.....*[Signature]*



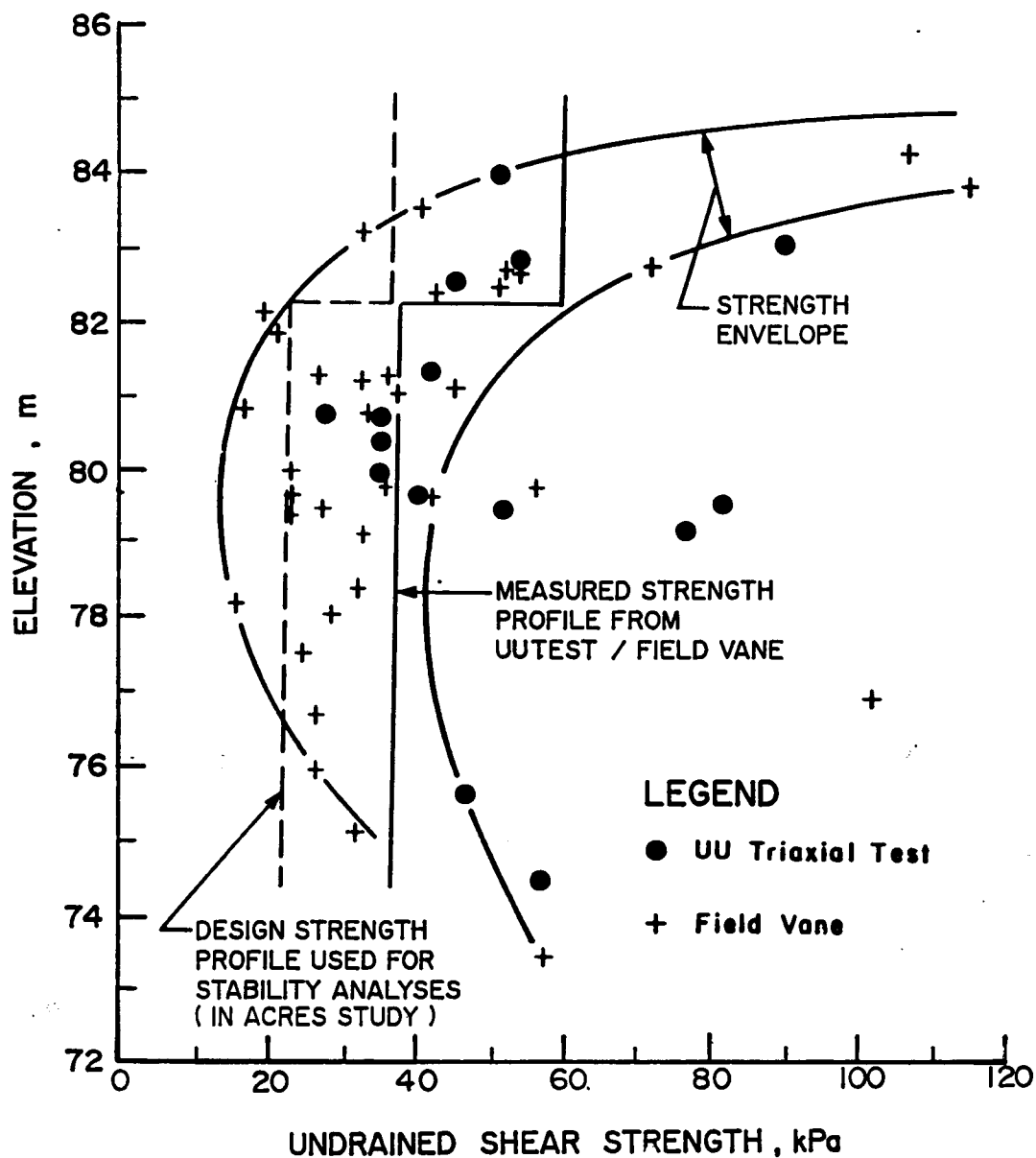
Date: AUG. 1990
Project: 90-1339

Golder Associates

Drawn: R.B.C./D.J.R.
Chkd: [Signature]

**Su VS ELEVATION AND DESIGN PROFILE
(ACRES UU TEST AND VANE DATA)**

FIGURE 4.



SOURCE : ACRES REPORT TO MTO
WP 126 - 87 - 01
FIGURE No. 1

Date JUNE 12, 1990.

Project 901 - 1339

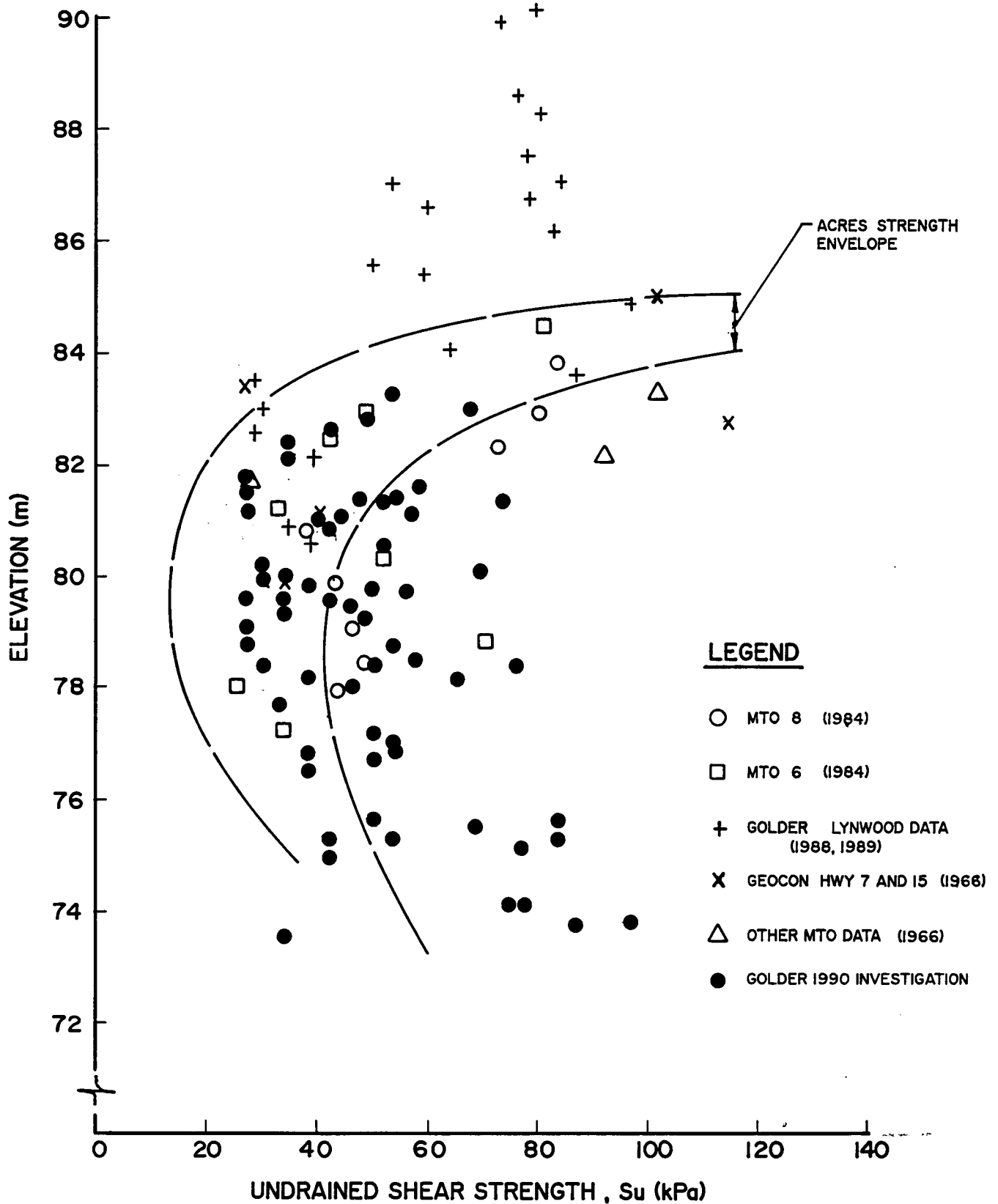
Golder Associates

Drawn R.B.C.

Chkd. R.B.C.

Su VS ELEVATION
(MTO / GOLDER LYNWOOD VANE DATA)

FIGURE 5 .



Date JUNE 13, 1990.

Project 901 - 1339

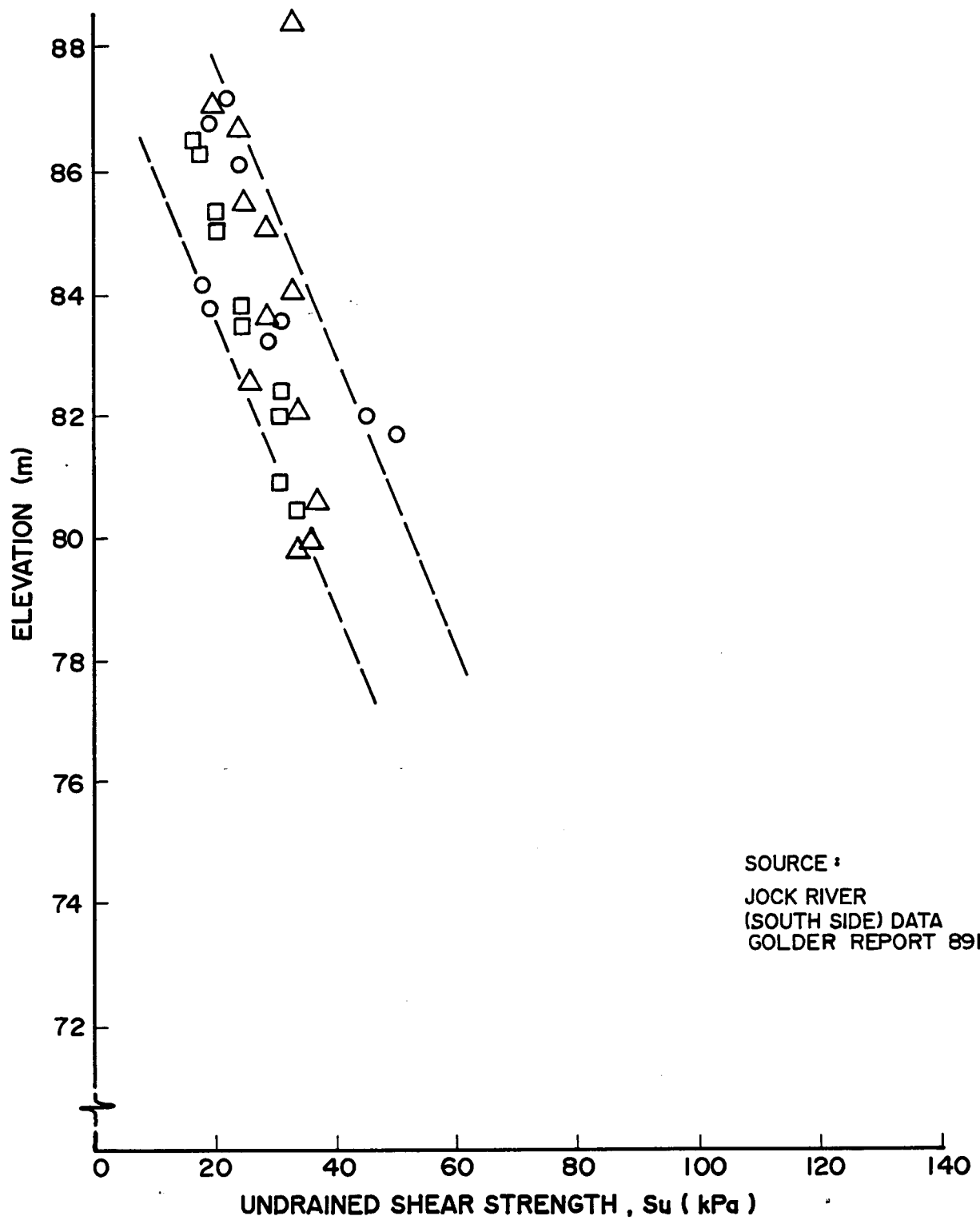
Golder Associates

Drawn R.B.C.

Chkd. *[Signature]*

Su VS ELEVATION (GOLDER DATA ON JOCK RIVER)

FIGURE 6 .



SOURCE :
JOCK RIVER
(SOUTH SIDE) DATA
GOLDER REPORT 891-2251

Date JUNE 13, 1990

Project 901 - 1339

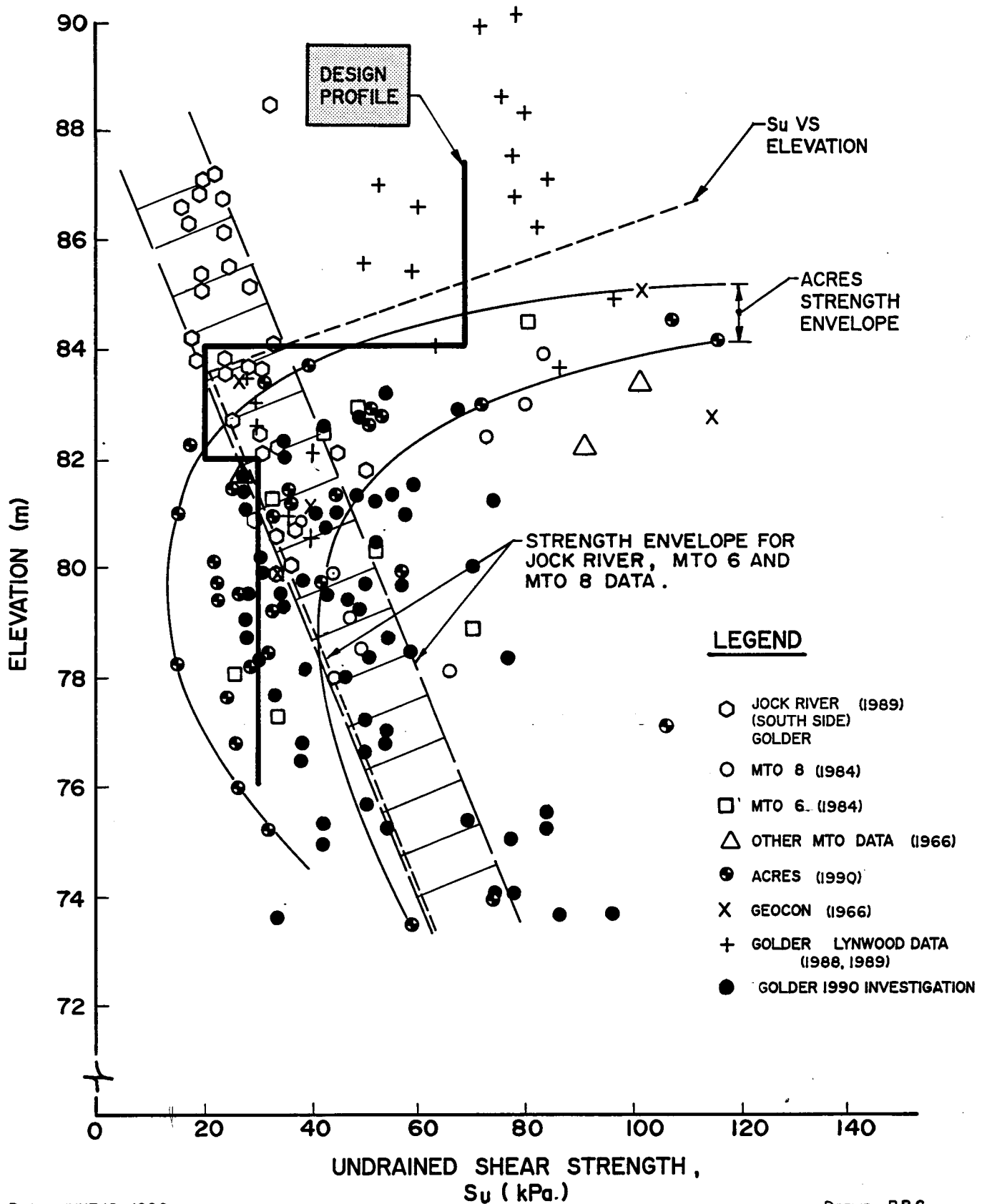
Golder Associates

Drawn R.B.C.

Chkd. *[Signature]*

COMPOSITE PLOT OF ALL S_u DATA AND DESIGN PROFILE

FIGURE 7.



Date JUNE 12, 1990

Project 901 - 1339

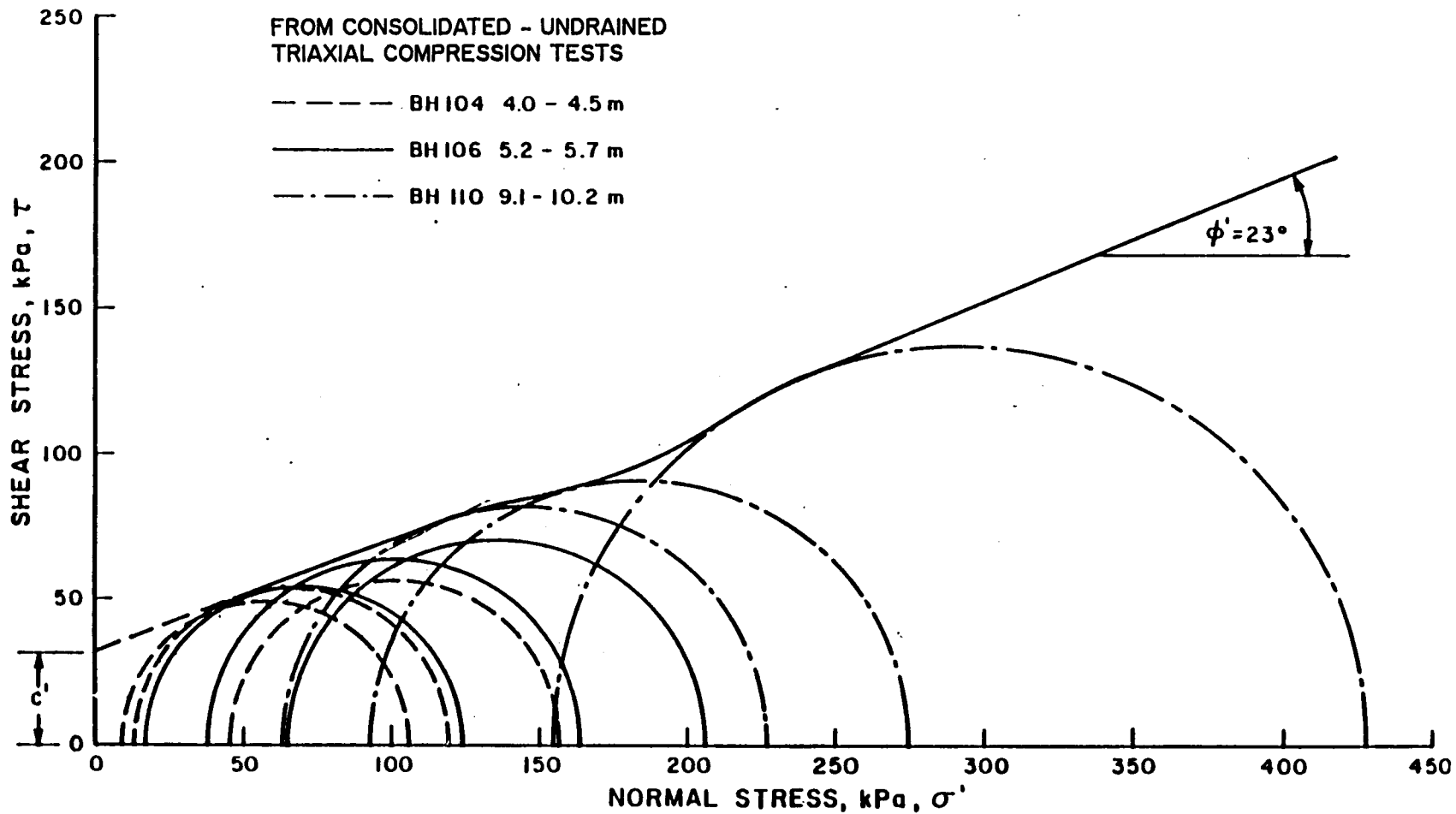
Golder Associates

Drawn R.B.C.

Chkd. R.J.

MOHR CIRCLES AND ENVELOPE
(ACRES INTERPRETATION)

FIGURE 8.



SOURCE : ACRES REPORT TO MTO
WP 126 - 87 - 01
FIGURE No. 2

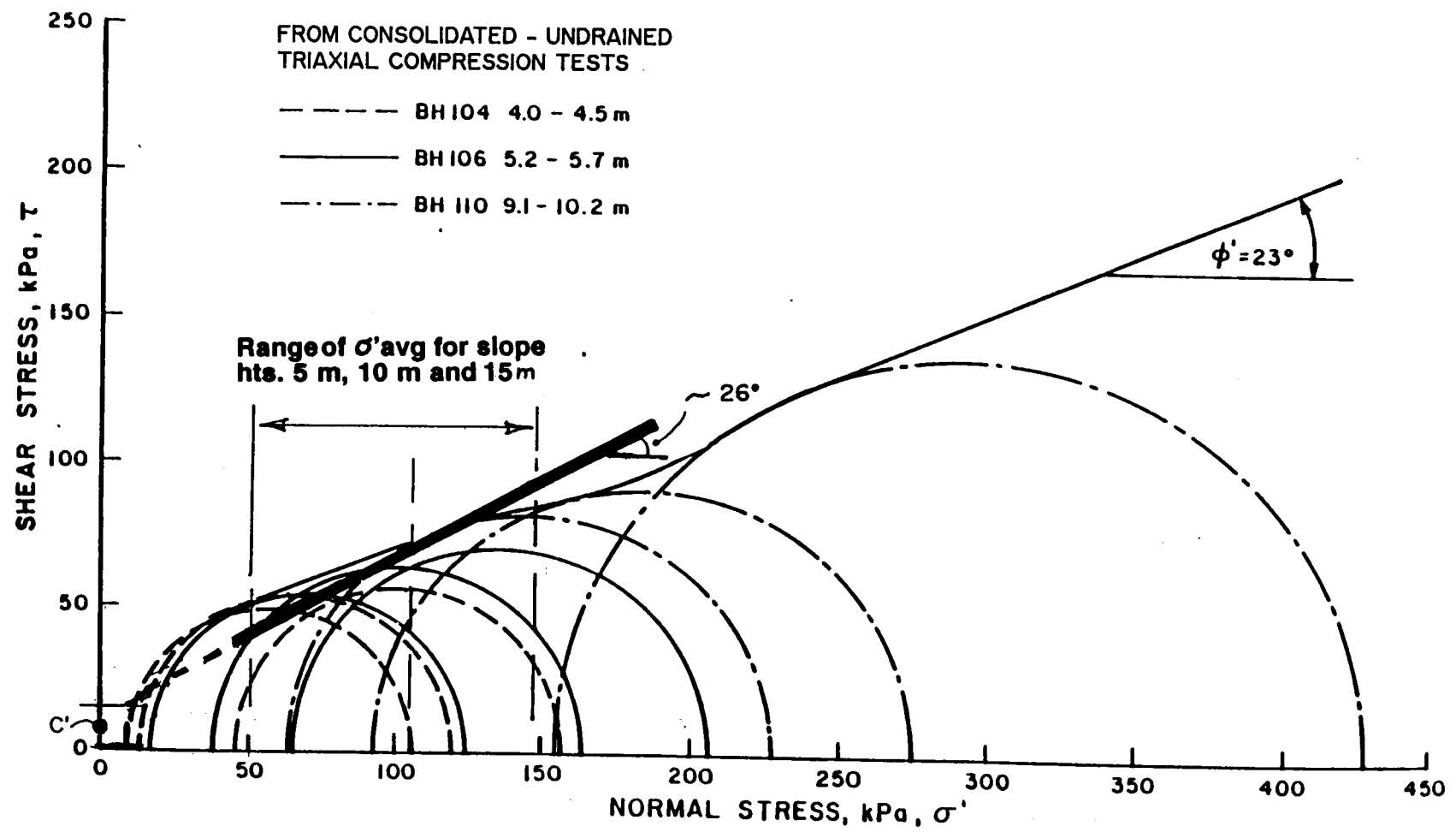
Date JUNE 12, 1990.
Project 901 - 1339

Golder Associates

Drawn R.B.C.
Chkd. *RL*

REVISED INTERPRETATION FOR
EFFECTIVE STRESS PARAMETERS

FIGURE 9.



$C' \sim 0^\circ \text{ TO } 12 \text{ kPa}$
 $\phi' \sim 25^\circ - 26^\circ$

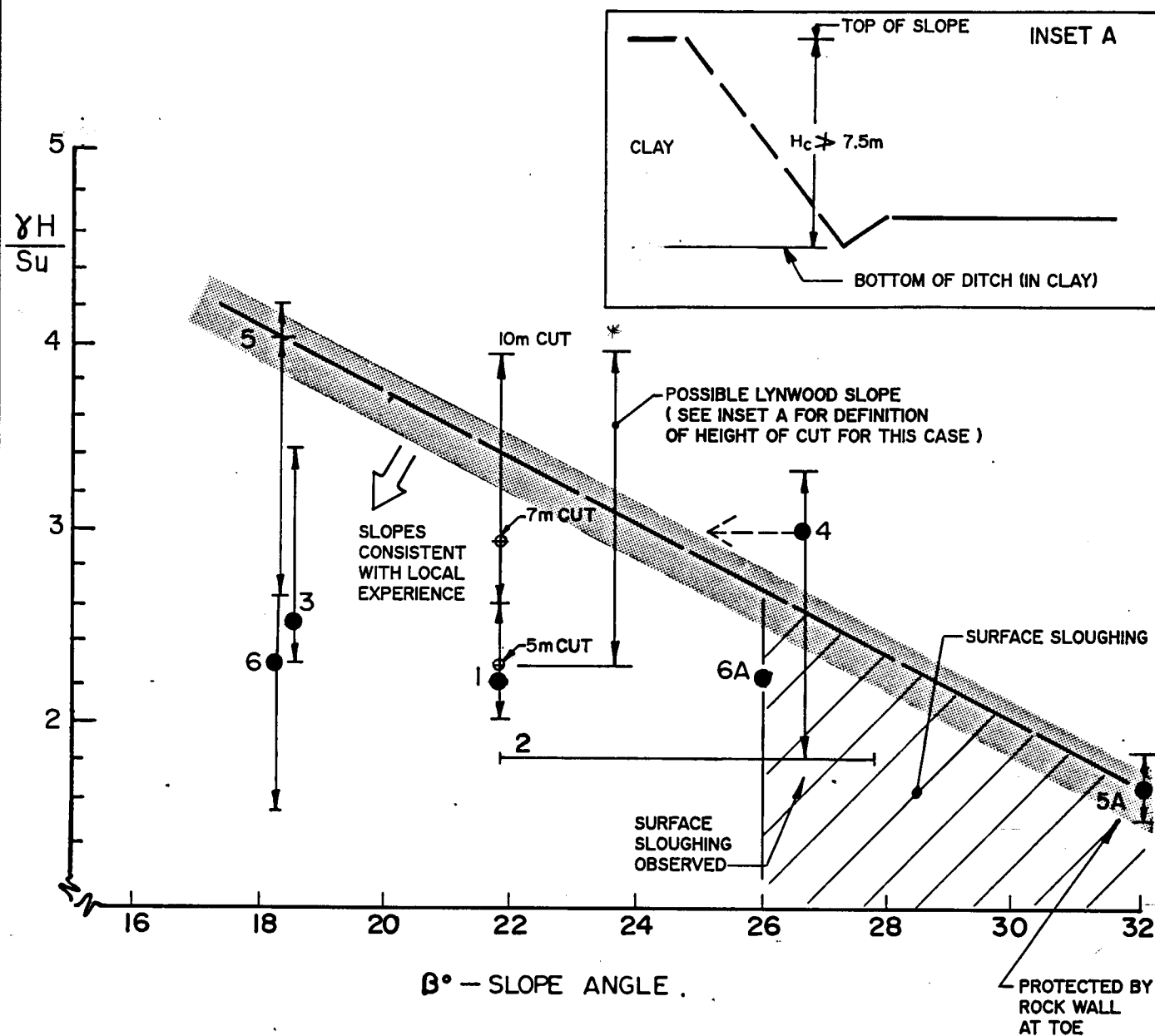
Date JUNE 12, 1990
Project 901-1339

Golder Associates

Drawn R.B.C.
Chkd. *[Signature]*

"NORMALIZED" SLOPE DATA $\gamma H/S_u$ VS β
(SLOPE ANGLE)

FIGURE 10.



LEGEND

- $\gamma H/S_u$ AVERAGE
- — — INDICATES RANGE

- 1. HERON ROAD
- 2. BILLING BRIDGE TRANSITWAY STATION
- 3. HUNT CLUB ROAD UNDERPASS
- 4. ORLEANS BOULEVARD CUT
- 5. AND 5A PRESTON DR. ACCESS ROAD
- 6. AND 6A HIGHWAY 7 AND 15 UNDERPASS

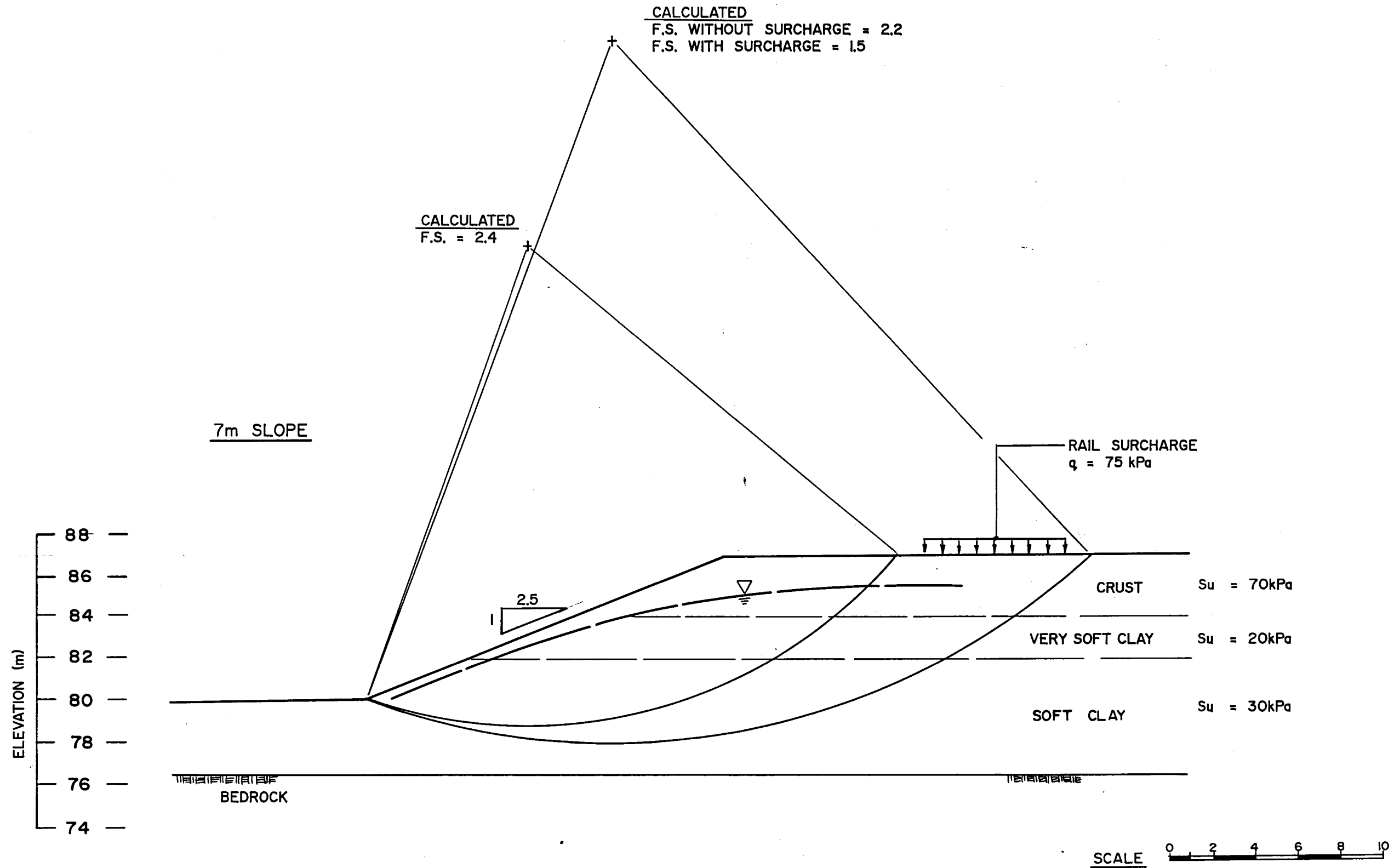
Date JUNE 13, 1990.
Project 901 - 1339

Golder Associates

Drawn R.B.C.
Chkd. *Ral*

STABILITY ANALYSES (UNDRAINED)
USING DESIGN PROFILE (7m SLOPE)

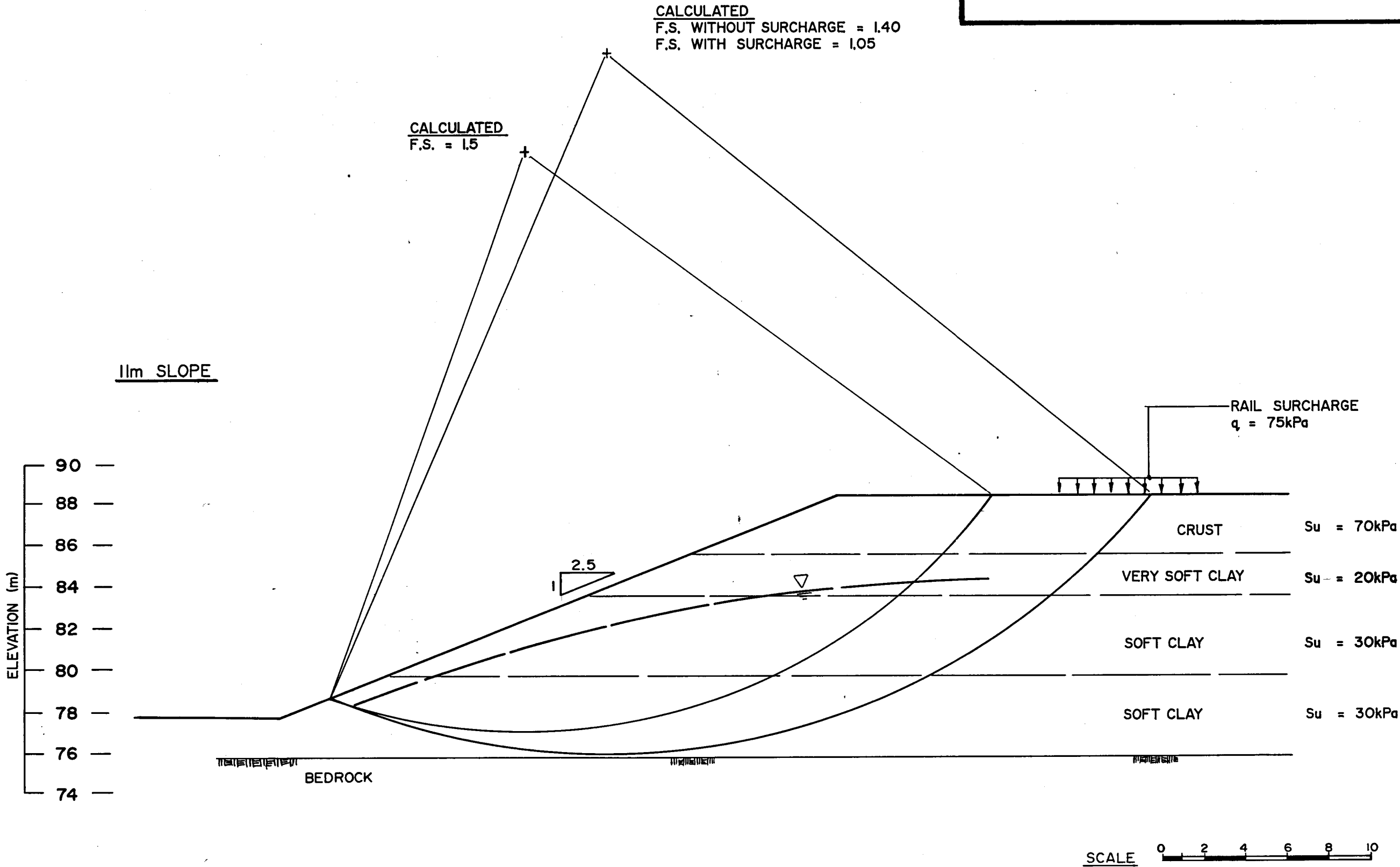
FIGURE 11.



Date..... JUNE 13, 1990.....
Project..... 901 - 1339.....

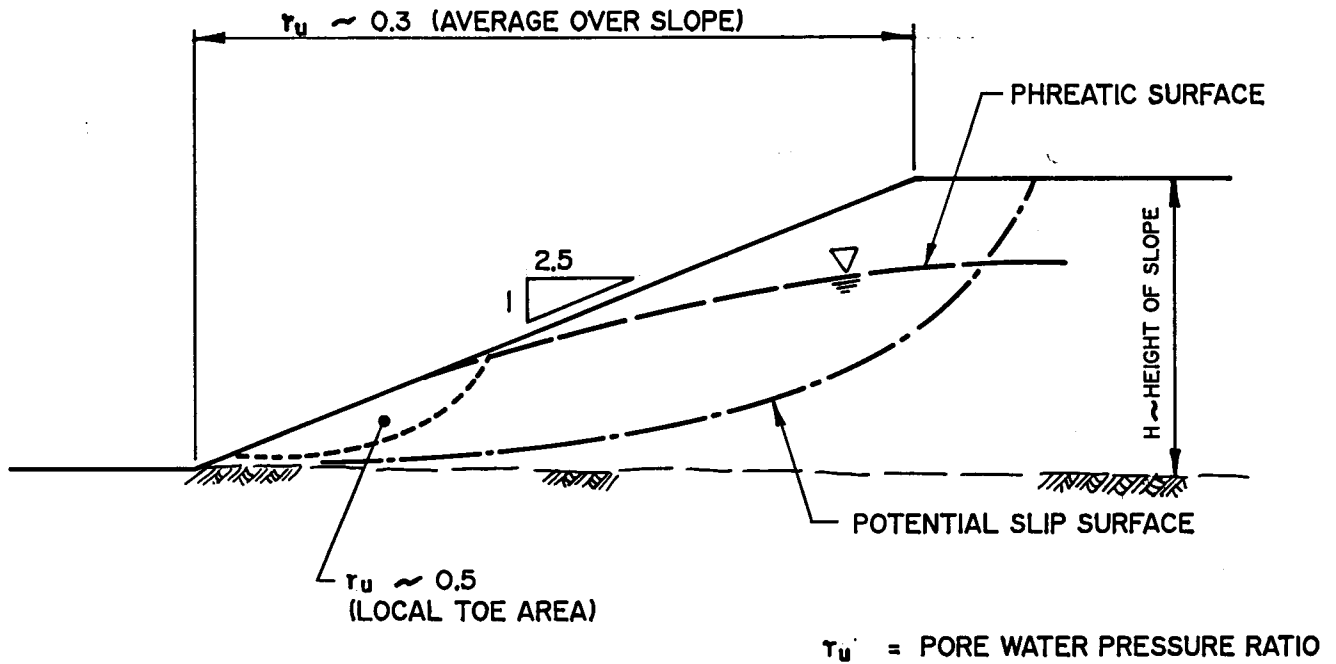
Golder Associates

Drawn..... R.B.C.....
Chkd..... *KL*.....



SUMMARY OF C' , ϕ' AND F FROM EFFECTIVE STRESS ANALYSES

FIGURE 13.



SLOPE CONFIGURATION

SUMMARY OF C' , ϕ' - ANALYSIS

H (m)	C' (kPa)	ϕ'	$C'/\tau H$	FACTOR OF SAFETY = F
5 TO 6	5	26°	0.05	1.3 ($r_u = 0.3$, OVERALL SLOPE) 1.0 ($r_u = 0.5$, LOCAL TOE AREA)
10 TO 11	5	26°	0.025	1.2 ($r_u = 0.3$)
15	5	26°	0.02	1.1 ($r_u = 0.3$)

Date JUNE 13, 1990.

Project 901 - 1339

Golder Associates

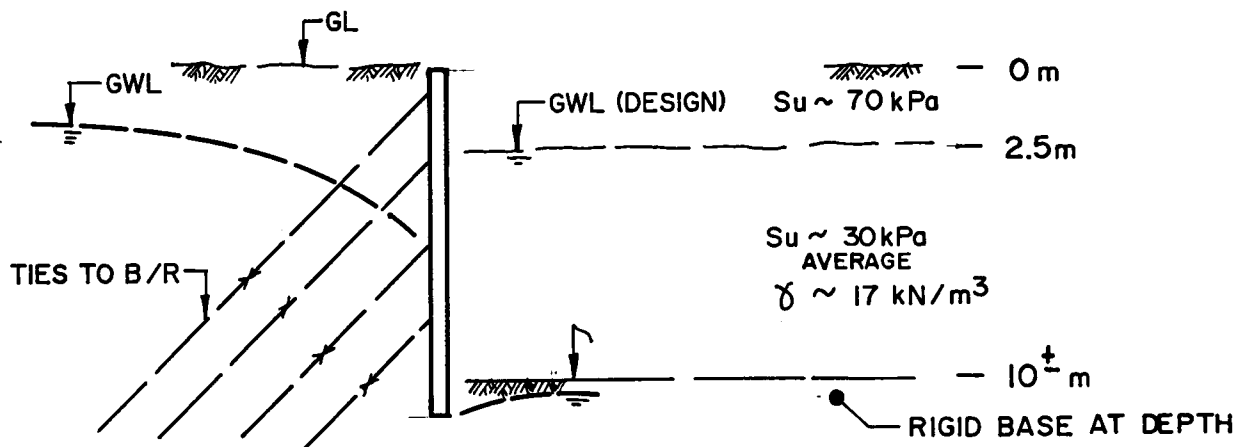
Drawn R.B.C.

Chkd. *R.B.C.*

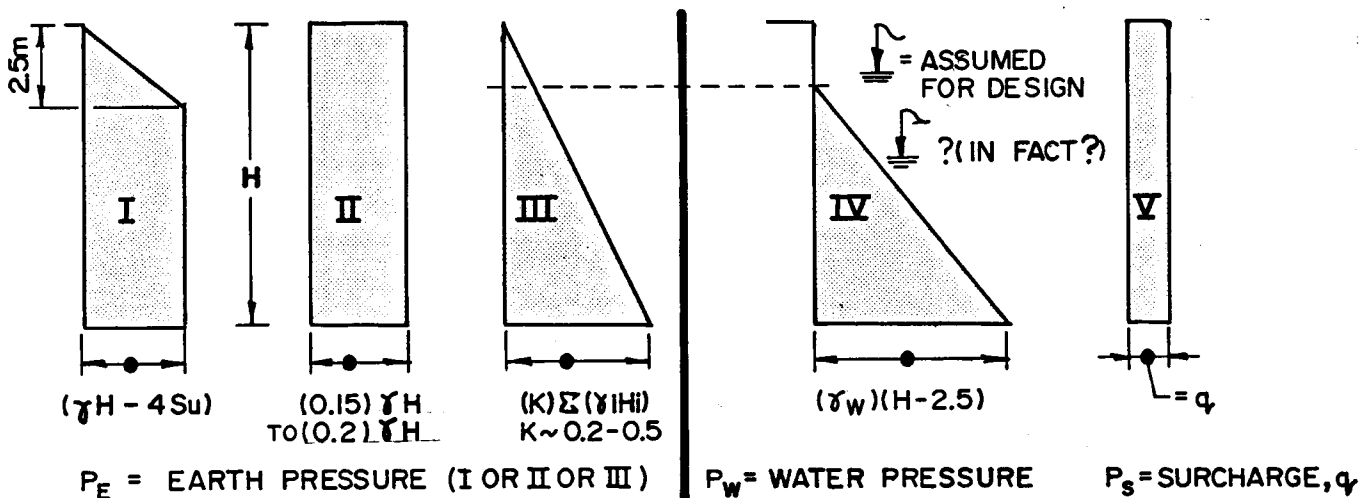
APPROXIMATE DESIGN LATERAL PRESSURE DISTRIBUTIONS

FIGURE 14 .

CONDITIONS ASSUMED FOR PRELIMINARY WALL DESIGN



ASSUMED RANGE (S) OF DESIGN PRESSURES ON THE WALL



EXAMPLE DESIGN PRESSURE(S) WHERE H = 10m

FOR CASE I $\Sigma P_E \sim 440 \text{ kN/m}$

FOR CASE II $\Sigma P_E \sim 350 \text{ kN/m}$

FOR CASE III $\Sigma P_E \sim 280 \text{ kN/m}$

(WHERE K = 0.5)

$\gamma' = 7 \text{ kN/m}^3$

FOR CASE IV $\Sigma P_w \sim 275 \text{ kN/m}$ (H = 10m)

FOR CASE V $\Sigma P_s \sim 50 \text{ kN/m}$ (K = 0.5)

CASE III PROBABLY MORE APPROPRIATE FOR LONG TERM CASE ,

$\Sigma P_E = 280 \text{ kN/m}$

THUS , ΣP (FOR DESIGN) = $\Sigma P_E + \Sigma P_w + \Sigma P_s = 280 + 275 + 50 = 605 \text{ kN/m}$

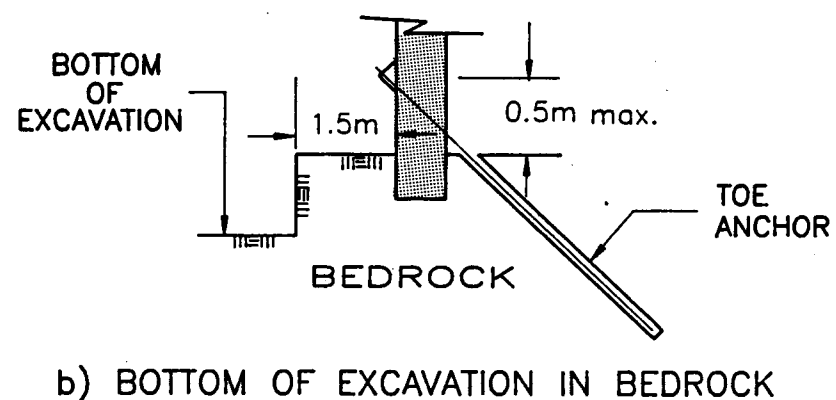
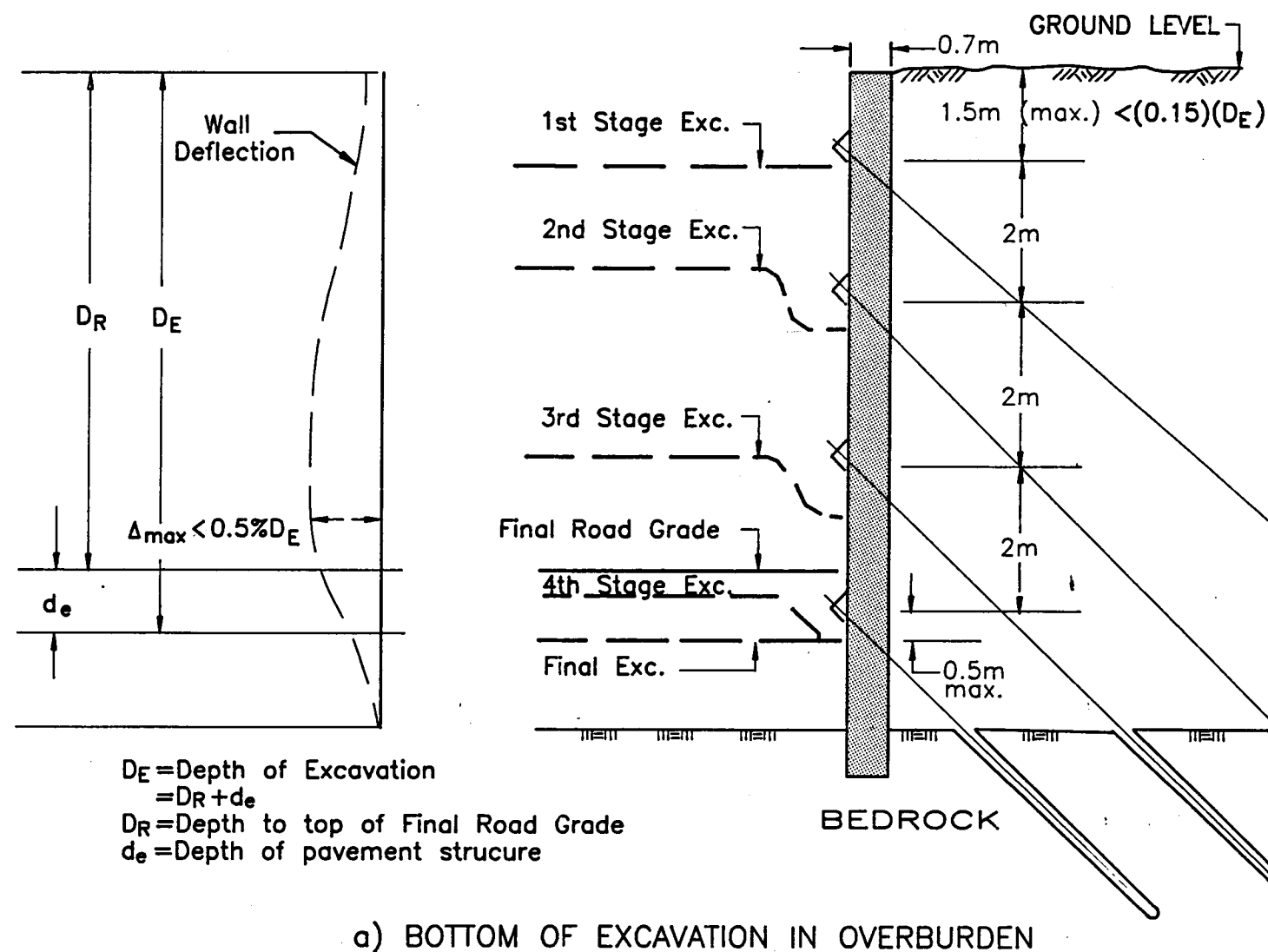
Date JUNE 13, 1990.

Project 901 - 1339

Golder Associates

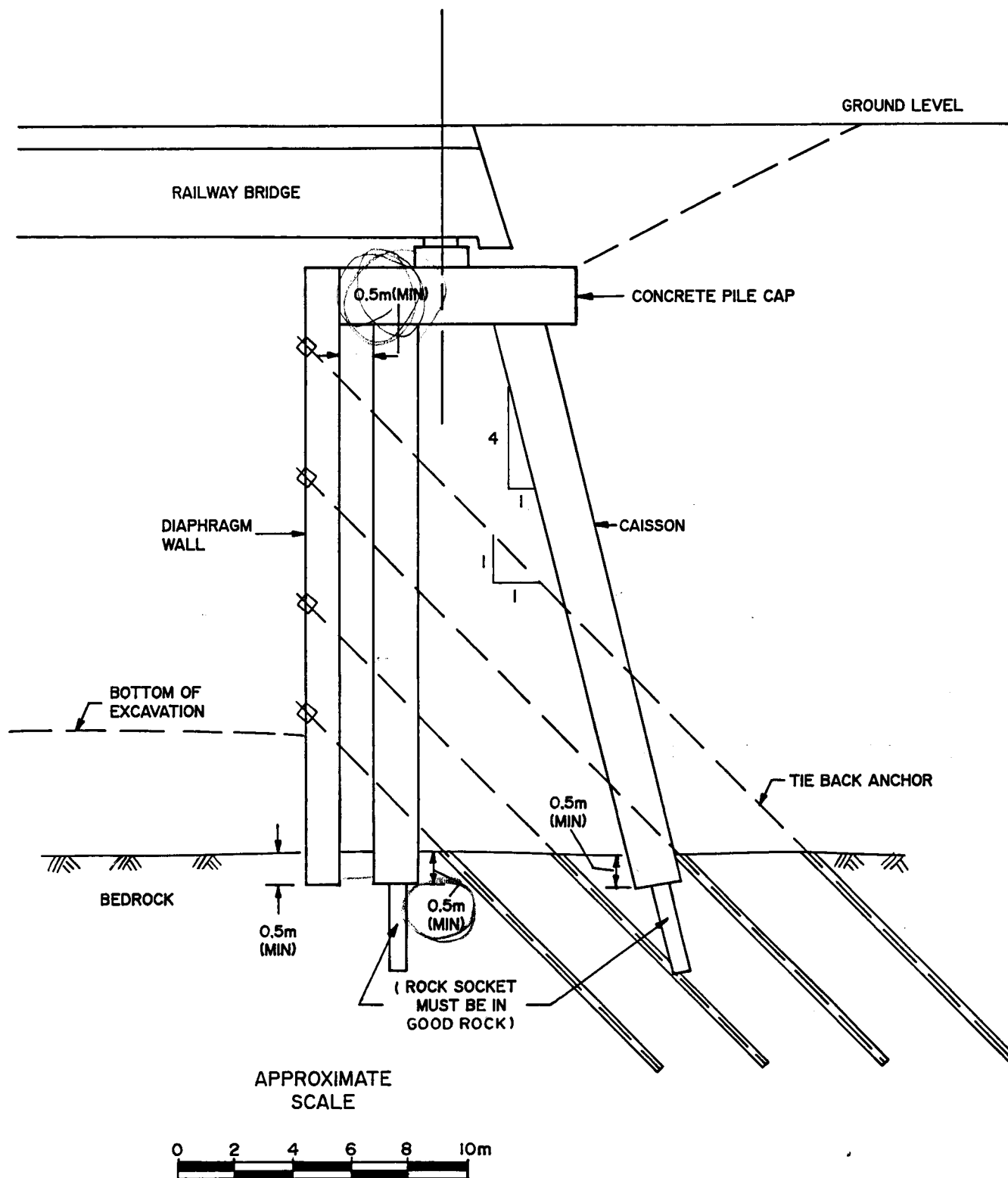
Drawn R.B.C.

Chkd. *[Signature]*



NOTE

1. 1st level anchor at 0.15D_E or 1.5m.
2. Subsequent anchor levels at 2m spacing.
3. Bottom support at 0.5m above base of excavation
4. Target maximum wall deflection $\nless 0.5\%D_E$
5. Wall thickness ~70cm.
6. Wall should penetrate B/R ~0.5m (minimum)



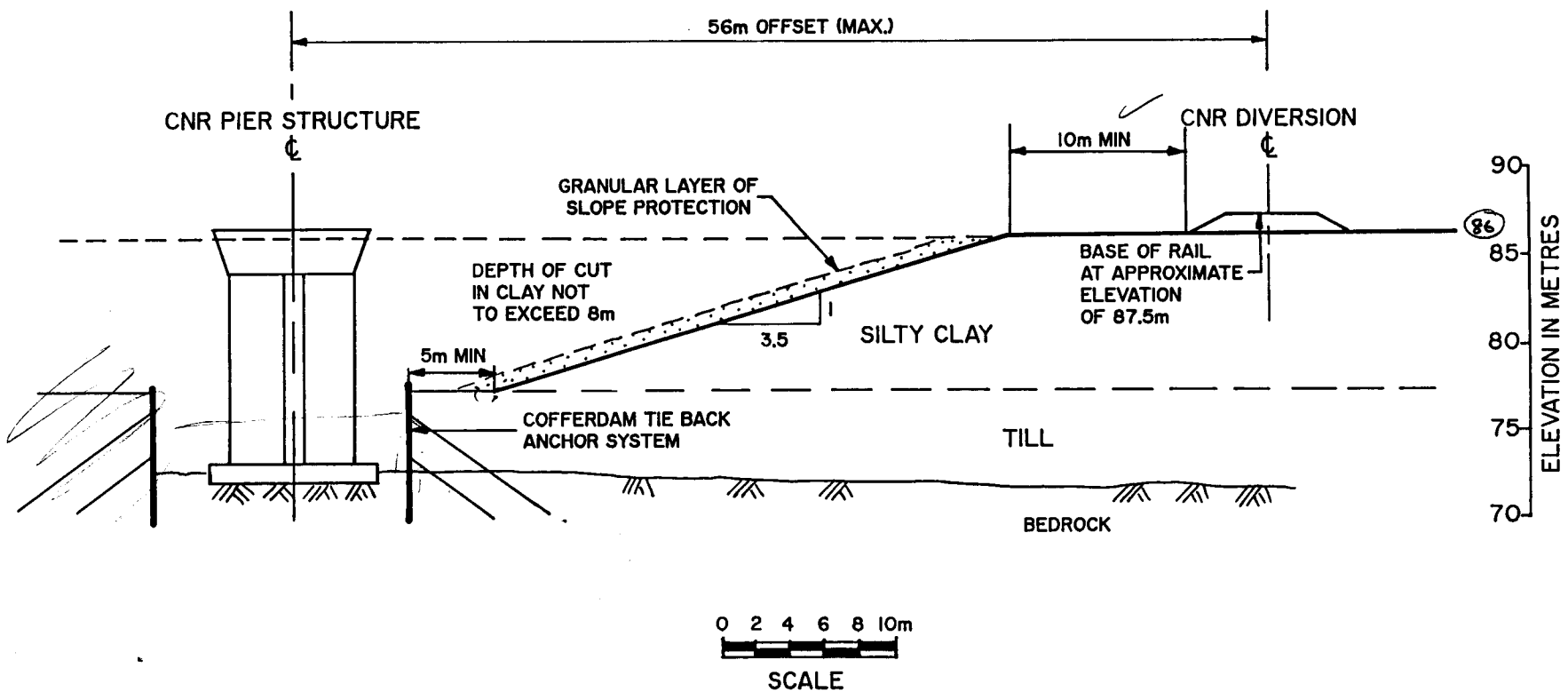
Date AUG. 14, 1990
 Project 90I-1339

Golder Associates

Drawn DJR
 Chkd. RW

TEMPORARY EXCAVATION FOR BRIDGE PIER CONSTRUCTION

FIGURE 17



Date AUG. 14, 1990
Project 90-1339

Golder Associates

Drawn D.M.R.
Chkd K.M.

APPENDIX A
SUMMARY OF SITE GEOTECHNICAL DATA

August, 1990

901-1339

SUMMARY OF SITE GEOTECHNICAL DATA(1)

PROPERTIES	REMARKS
SOIL/ROCK PROPERTIES	
o CLAY	
Undrained shear strength, S_u	Top Layer 70 kPa Thin soft layer 20 kPa Lower Layer 30 kPa See Figures 3A, 3B and Figure 7
Unit Weight, γ	17 kN/m ³
ϕ'	25° - 26°
c'	0 - 12 kPa See Figure 9
o TILL	
Unit Weight, γ	20.4 kN/m ³
ϕ'	28°
c'	0 5 kPa used for analysis
o ROCK	
Unconfined Compressive Strength	216 MPa to 245 MPa 231 MPa (average) Acres Data
o GROUNDWATER CONDITIONS	Underdrainage conditions. (Top of bedrock probably more permeable than overlying clay & till) See Figures 3A and 3B

RECOMMENDED DESIGN PARAMETERSDIAPHRAGM WALL

o Height	>7.5 m	
o Thickness	0.7 m	
o Stiffness (EI)	~ 1000 MNm ² /m	with reinforcement
o Founding level	0.5 m into Bedrock	minimum
o Target horiz. Deflection	<0.5% of depth of excavation	See Figure 15
o Pressure Distribution	Triangular	See Figure 14
o Earth Pressure Coefficient, K	0.5	
o Water pressure	Assume 2.5 m below ground level	See Figure 14
o Design Lateral Load/m of wall	500 kN - 1000 kN	See Figure 14
o Set-back of wall from rock face	1.5 m	Set-back required only if excavation for roadway extends to bedrock

ROCK ANCHOR

o Grout to Rock Bond	Ultimate resistance	
o Grout Strength	1000 kPa in good rock	
	30 MPa	See Figure 15

BEARING CAPACITY(2)

o Footings on good rock	3500 kPa at ULS ⁽³⁾ SLS	Do Not Govern Design Since Foundation is Assumed Unyielding
o Coefficient of Friction between good rock and concrete	ULS = 0.56	
o Caissons founded in good rock	3500 kPa at ULS ⁽³⁾	To be Designed as End Bearing Column

INSULATION

1.8 m soil cover or equivalent
or
150 mm (minimum) styrofoam SM or equivalent

NOTE: (1) The final geotechnical design parameters used for a specific design should be reviewed and approved by a geotechnical engineer

(2) Bearing Capacities are subject to reduction for inclined loads

(3) ULS = Ultimate Limit State
SLS = Service Limit State