



Golder Associates Ltd.

CONSULTING ENGINEERS

Geocres No:

3145-1360

PRELIMINARY REPORT TO

MINISTRY OF TRANSPORTATION ONTARIO

**DRAFT FOR
COMMENTS**
ENGINEERING STUDY
PROPOSED CUT AND RAILWAY UNDERPASS
HIGHWAY 416
NEARBY, ONTARIO

Distribution:

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Mississauga, Ontario

April 1990

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TABLE OF CONTENTS

	<u>Page No.</u>
Terms of Reference	1
Subsurface Conditions	2
List of Data Reviewed	3
Assessment of Undrained Shear Strength for use in Design	3
Assessment of Effective Stress Parameters for use in Design	4
Local Experience with Similar Cut Slopes	4
Determination of Safe Cut Slopes	4
Undrained Analysis	4
Effective Stress Analysis	5
Modifications to Cut Slopes	5
Replacement of Slopes by Vertical Diaphragm Walls	6
Comments on Roadway Excavations and Construction	7
Further Items for Discussion	8
List of Tables	
List of Figures	

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 Soil Section A-A
- 3-B Soil Section B-B
4. Su vs Elevation and Design Profile (Across UU Test and vane data)
5. Su vs Elevation (MTO8/MTO6/Golder Lynwood Vane Data)
6. Su vs Elevation (Golder Associates data on Jock River)
7. Composite Plot of all Data and Design Profile
8. Mohr Circles and Envelope (Across' interpretation)
9. Revised Interpretations for Effective Stress Parameters
10. "Normalized" Slope Data $\gamma H/Su$ vs β (Slope Angle)
11. Stability Analyses (Undrained) using Design Profile (7 m Slope)
12. Stability Analyses (Undrained) using Design Profile (11 m Slope)
13. Summary of C' , ϕ' and F from Effective Stress Analysis
14. Approximate Design Lateral Pressure Distributions

April 27, 1990

Our ref: 901-1339

Ministry of Transportation Ontario
1201 Wilson Avenue
Central Building, Room 315,
Foundation Design Section
Downsview, Ontario
M3M 1J8

ATTENTION: Mr. M.S. Devata, P.Eng

RE: PRELIMINARY REPORT
ENGINEERING STUDY
SLOPE CUT AND RAILWAY UNDERPASS
HIGHWAY 416,
NEPEAN, ONTARIO

Dear Sirs:

The Ministry of Transportation of Ontario (MTO) retained Golder Associates (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). This letter reports the preliminary results for this study and is written in draft form for your discussion and comment at our meeting on April 30, 1990.

Terms of Reference

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

After an assessment of available data on the shear strength parameters of the 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. A preliminary report on this study is to be available on April 30, 1990. This report will provide the background required for a meeting with Kingston region staff. Following this meeting, the final report would be prepared and submitted in early June.

This preliminary report assesses only the design practicability and optimum arrangements for cut slopes and diaphragm walls. These are the key issues; other aspects of lesser importance will be covered in the final report.

Subsurface Conditions

The locations of boreholes put down during previous geotechnical investigations at the site by MTO and by Golder Associates are shown on Figure 2, Borehole Location Plan.

The soil profiles for sections A-A and B-B along the proposed highway alignment are shown on Figures 3A and 3B respectively. The subsurface conditions in the area of the deeper cuts

generally consist of weathered sensitive silty clay and clayey silt of variable thickness, followed by discontinuous deposits of granular soil and underlain by dolomitic limestone bedrock.

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)
- o Foundation Investigation, Proposed Jock River Bridges, Highway 416. W.P. 128-87-07/08 District 9 (Ottawa). Nepean, Ontario.
- 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).
- o Slurry wall construction for BART Subway Stations. ASCE Annual Meeting and National Meeting on Structural Engineering, Pittsburgh, Pa. 1968. Reprint 746.
- o Golder Associates in-house data regarding experience of local slopes.

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 previously by Golder Associates, MTO and Acres as noted above.

The variation(s) of undrained strength, S_u with elevation are plotted in Figures 4, 5 and 6, based on Acres' data, MTO/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 profile with elevation. From the S_u profile, a design profile is delineated. This S_u

profile comprises a strong top layer followed by a thin very soft layer of 20 kPa and a thick lower layer of 30 kPa.

Assessment of Effective Stress Parameters for use in Design

The Mohr circles for consolidated undrained triaxial tests 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.

Local Experience with Similar Cut Slopes

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

This normalized slope data plot provides a basis on which stability of a proposed slope cut may be assessed by comparisons with known local experience.

Local experience has shown that surface sloughing occurs even in "safe" areas of slopes, if the slope surfaces are steeper than 26° .

Determination of Safe Cut Slope

The stability of a proposed slope cut of 2.5H:1V is examined below. Design analyses are based on the design strength parameters previously determined.

Undrained Analyses: The results of slip circle analysis for 7 m and 11 m slope, using design profile in Figure 7 are shown on

Figures 11 and 12 respectively. For a cut slope of 11 m, the calculated factor of safety (F) for the cases of no surcharge and applied soil surcharge of 75 kPa are 1.4 and 1.05 respectively. The calculated factor of safety is shown on Table 2. These calculated values of F may be compared with the local slope data on Figure 10. The calculated safety factors, can thus be calibrated to derive an operational value, F' , which correlates with observational data. From Figure 10, the operational F' for a 7 m cut is thus about 1.5; F' for an 8 m cut is about 1.3 to 1.5, and while for a deeper cut of 11 m, F' reduces to about unity.

Effective Stress Analyses: 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 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 5 to 6 m cut, to 1.2 for 10 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.

Based on this assessment it is considered that cuts of 7 m less can be safely excavated at side slopes of 2.5(H):1(V), which fits within established precedents in this area.

Modifications to Cut Slopes

Cuts deeper than 8 m are beyond most precedent data and will require flatter side slopes or modifications to the slopes to ensure stability. 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 materially 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 very 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 till or bedrock. Thus both of these types of modifications are not recommended. As an alternative, it is recommended that considerations be given to replacing the cut slope.

Replacement of Slope by Vertical Diaphragm Walls

To replace cut slopes deeper than 7 m, vertical diaphragm walls with tie backs may be used. The conditions are favourable for such a retaining system. Bedrock is at a relatively shallow depth, vertical walls may be taken into rock and inclined tie backs may also be anchored in rock. A further advantage is that concrete wall taken to rock can readily be modified to act as load bearing bridge abutments.

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 that 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 for preliminary design purposes, lateral coefficient of earth pressure 0.5 is provided for use in design. Possible design loadings are illustrated for a cut of 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 bedrock at the site.

COMMENTS OF ROADWAY EXCAVATION AND CONSTRUCTION

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

No equipment should be permitted to operate on or near (within 1.5 metres 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.

All the excavation should be planned using large hydraulic shovel(s) with trucks to haul the soil away.

The granular material placement should follow excavation. As a rule, the lowest lift of granular material should be placed in a thickness of at least 0.75 metres 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. Even with good construction, considerable total and differential frost heaving will occur due to the wet conditions.

Where the subgrade is more than about 1 metre below the ground surface in frost susceptible soil such as sensitive silty clay, silty fine sand, clayey silt, etc. we suggest that 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 reduced somewhat.

Where the concrete diaphragm walls is used it must be insulated to prevent freezing of the frost susceptible soils behind it in order to inhibit lateral heaving forces. Such insulations can probably be incorporated with the outer final facings on the wall.

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 metre of ground surface can be considered for use in embankment fill construction. Since these soil types are sensitive to changes in moisture content and wetting, earthwork using the native weathered silty clay may only be carried out during consistent dry weather. Reuse 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.

Further Items for Discussions

- Definition of EXACT bedrock profile and soil conditions along the proposed diaphragm walls; definition of wall lengths required. Details of transitions between walls and slopes.
- Frost protection to walls.
- Implication of groundwater lowering on pressures, on subgrade and pavements. Definition of drainage measures required.

- Design of railway bridge structure:
 - Abutments, piled relieving platform behind abutments
 - Load bearing units incorporated in diaphragm walls
 - Foundation and construction of bridge piers.
- Diversion of railway tracks during construction:
 - Definition of arrangements, stages in wall construction and in excavation.
 - Temporary support of railway tracks.
 - Maintenance of grades, speed limits.

This report is intended to act as an agenda for discussions at our meeting on April 30, 1990. At that time your comments will be welcome.

Yours truly,

GOLDER ASSOCIATES LTD.

R. Ng, P.Eng.

V. Milligan, P.Eng.

RMN/vm/spc

TABLE 1
SUMMARY DATA FOR EXISTING LOCAL EXPERIENCE

	LOCATION	SLOPE HEIGHT H (m)	SLOPE β (deg)	ANGLE 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 2.0H: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	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

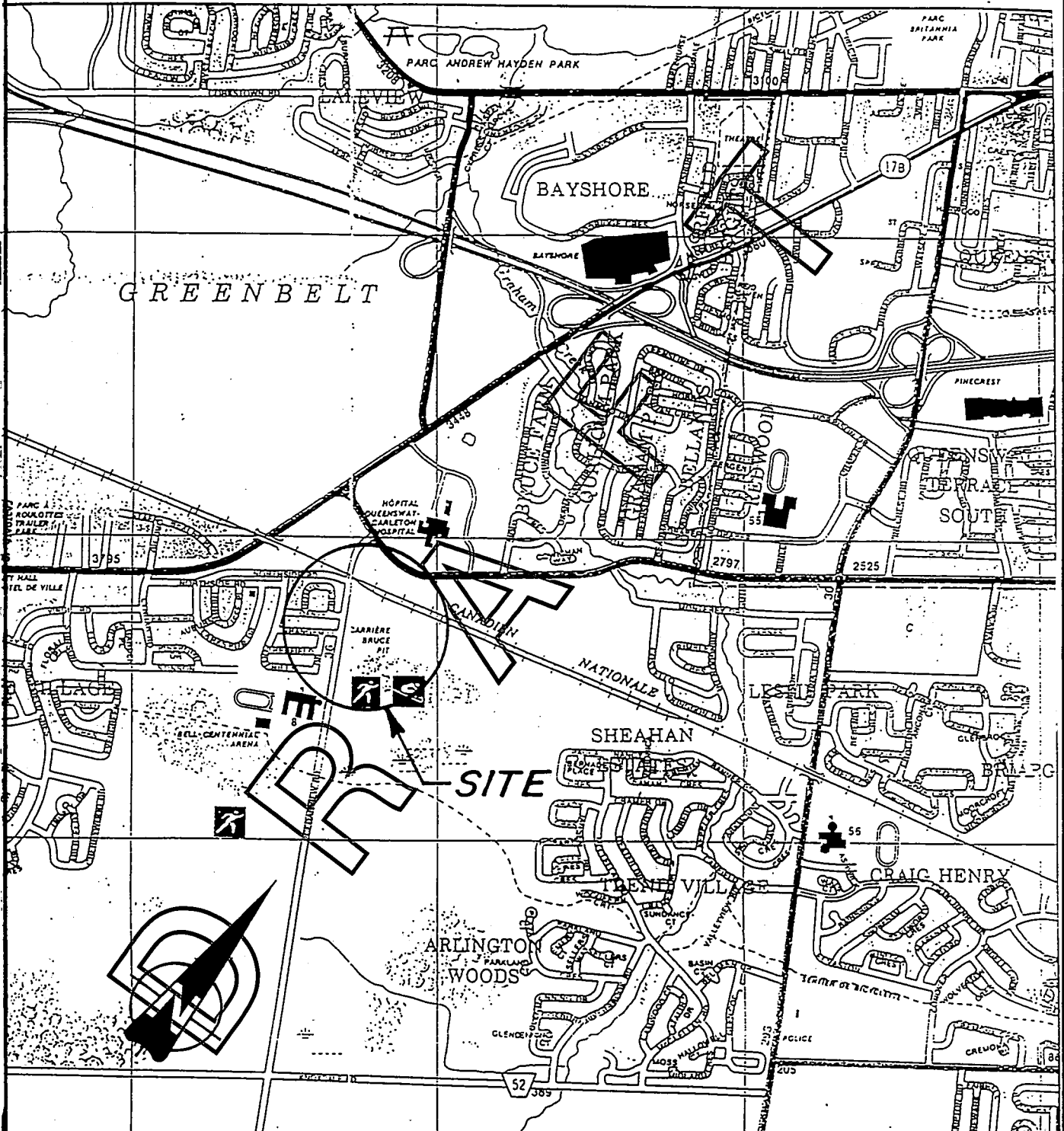
ASSESSMENT OF UNDRAINED ANALYSIS*

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.

SITE LOCATION PLAN

FIGURE 1



SCALE 1:25000

SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

Date APRIL 27 1990

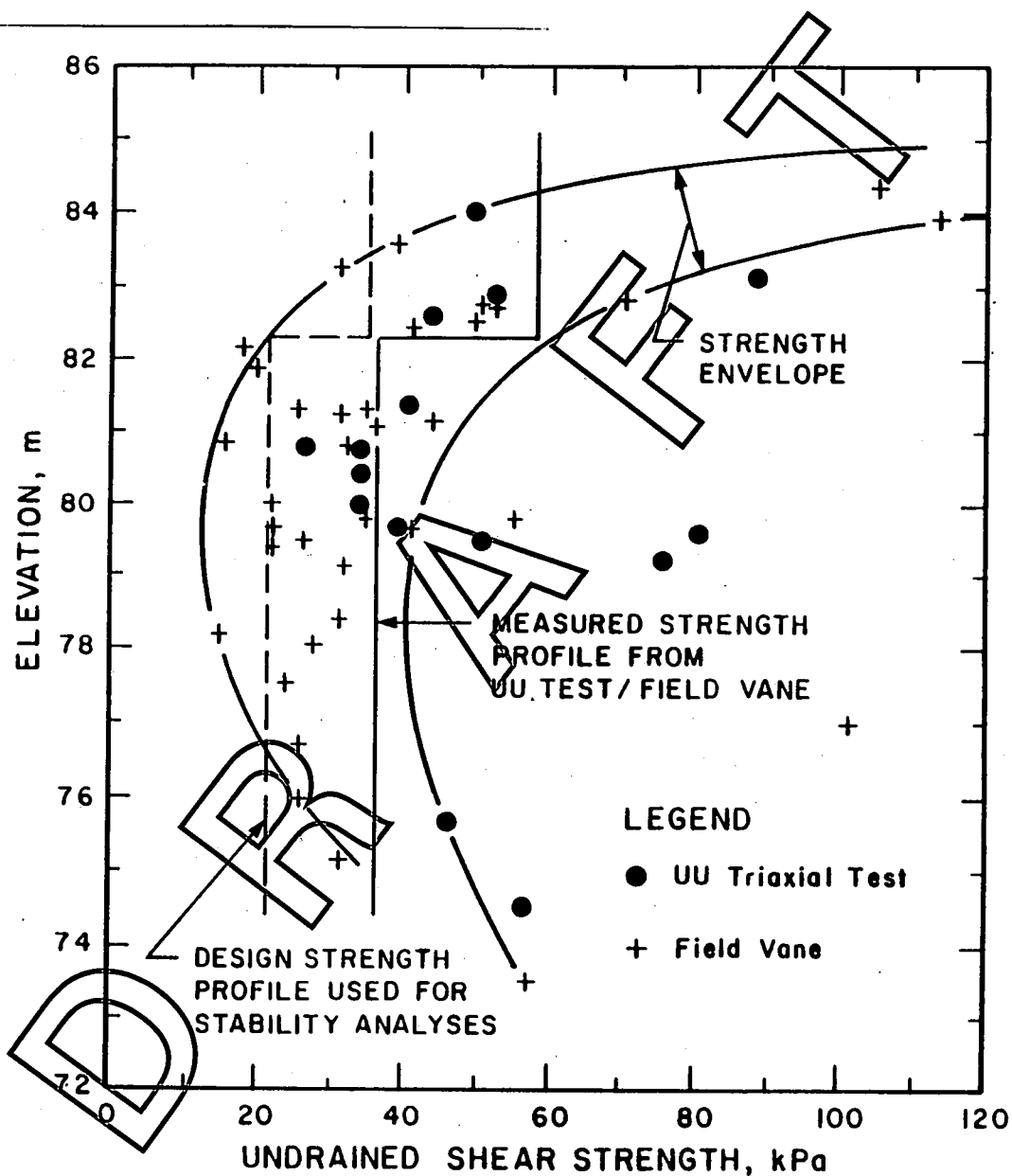
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Drawn PW
Chkd. _____

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

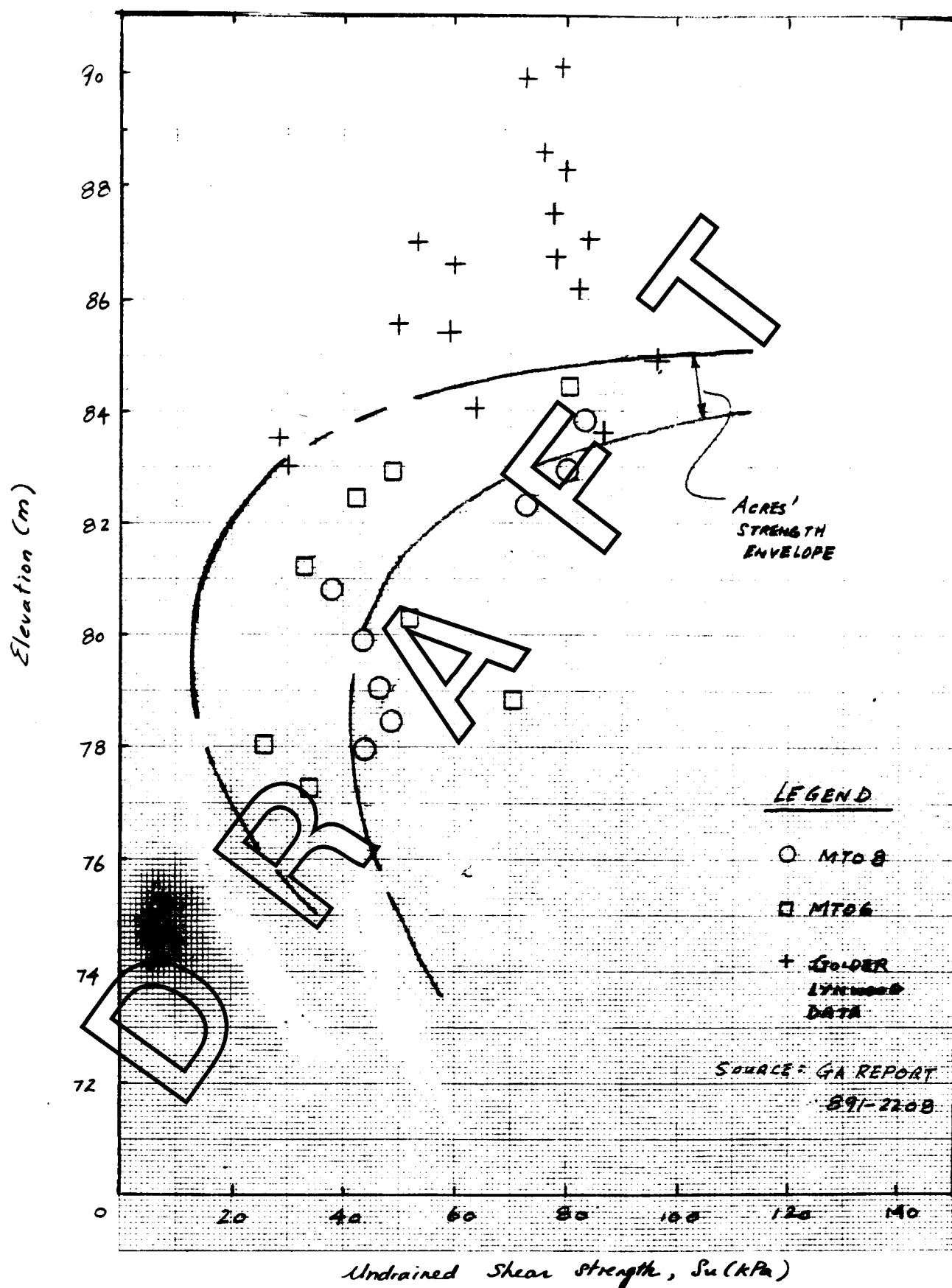
Date APRIL 27 1990
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Drawn RW
Chkd. _____

Su VS ELEVATION
(MTO8/MTO6/Golder Lynwood Vane Data)

FIGURE 5



Date APRIL 27 1990

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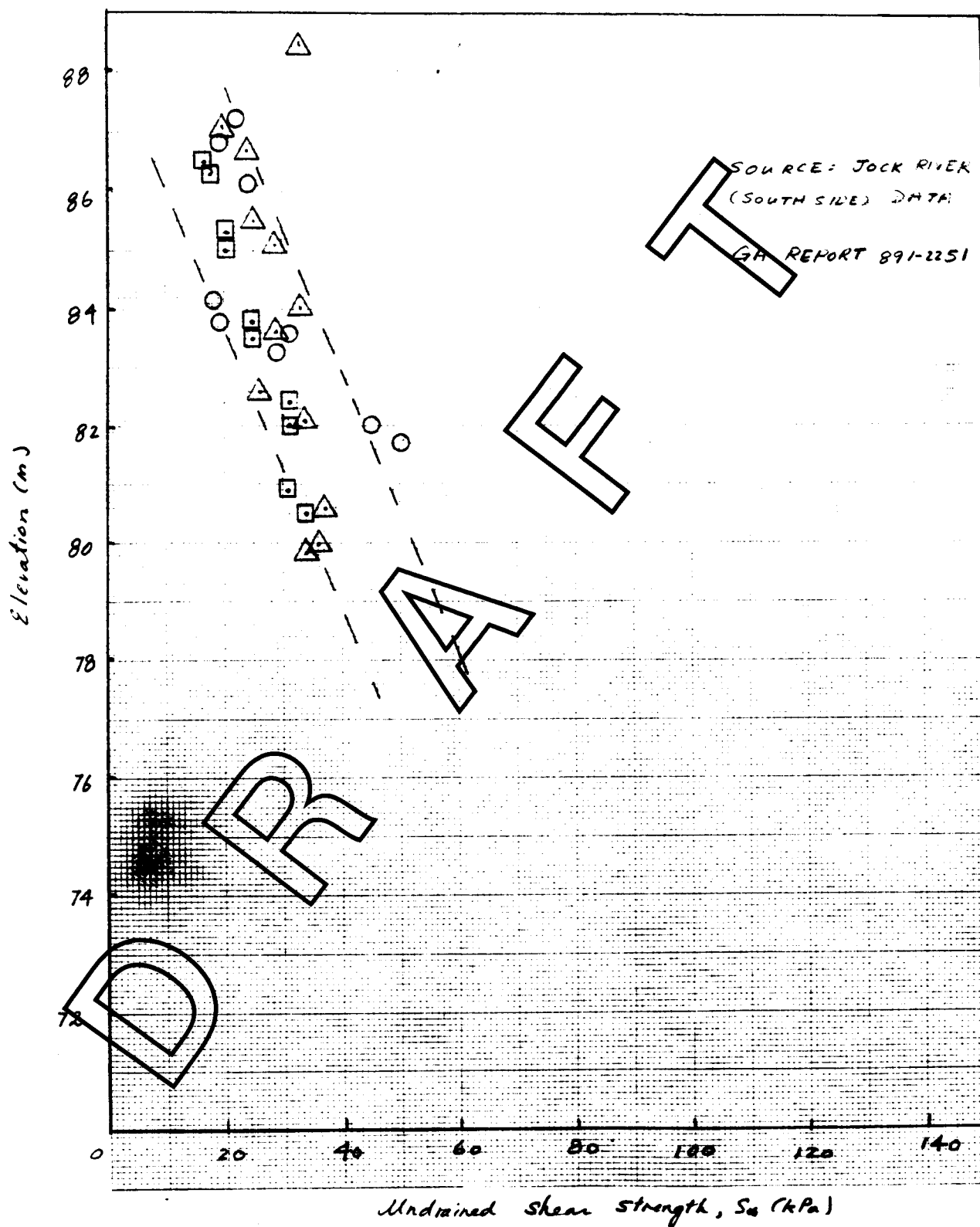
Golder Associates

Drawn RN

Chkd. _____

Su VS ELEVATION
(GA Data on Jock River)

FIGURE 6



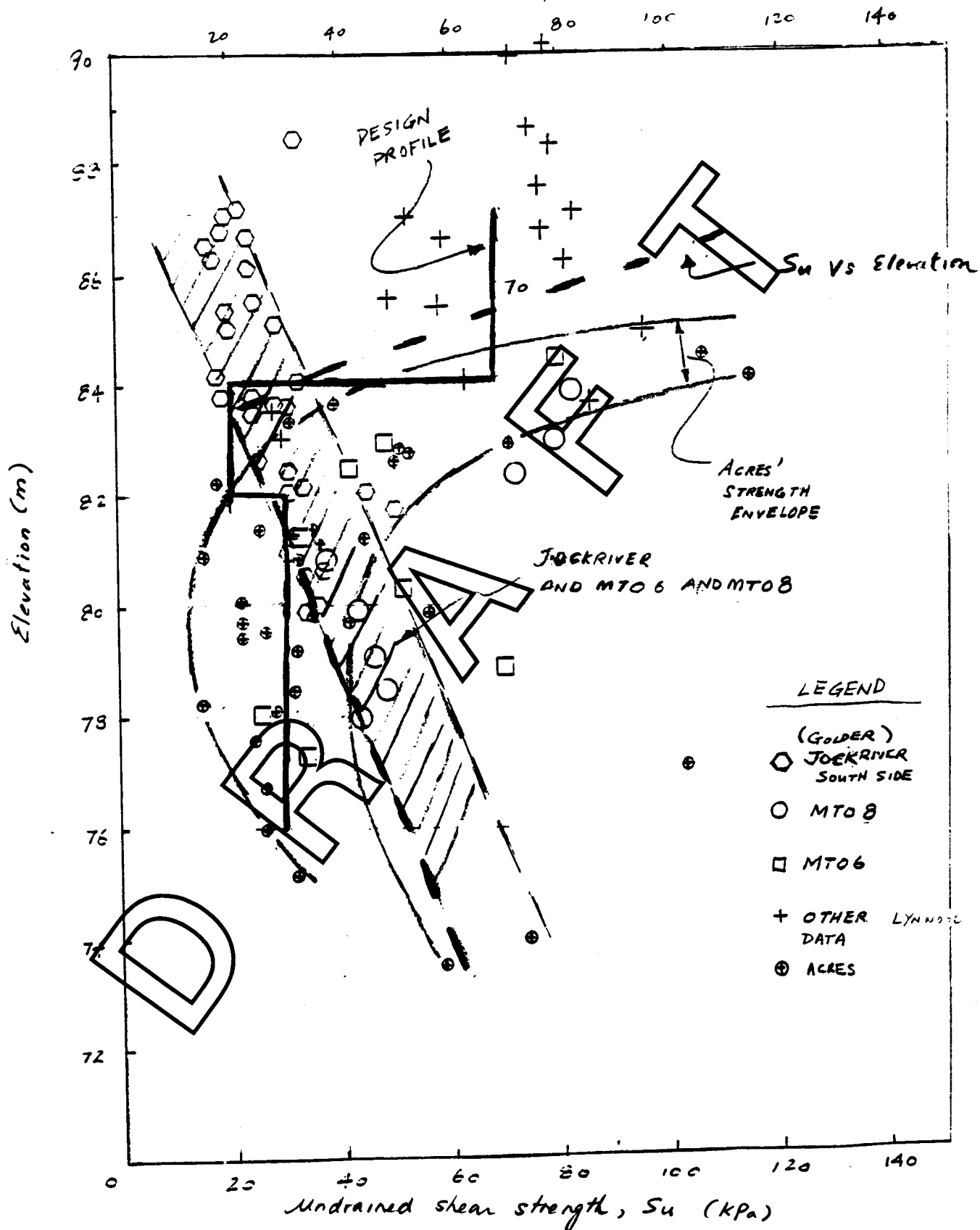
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Chkd. _____

COMPOSITE PLOT OF ALL DATA AND DESIGN PROFILE

FIGURE 7



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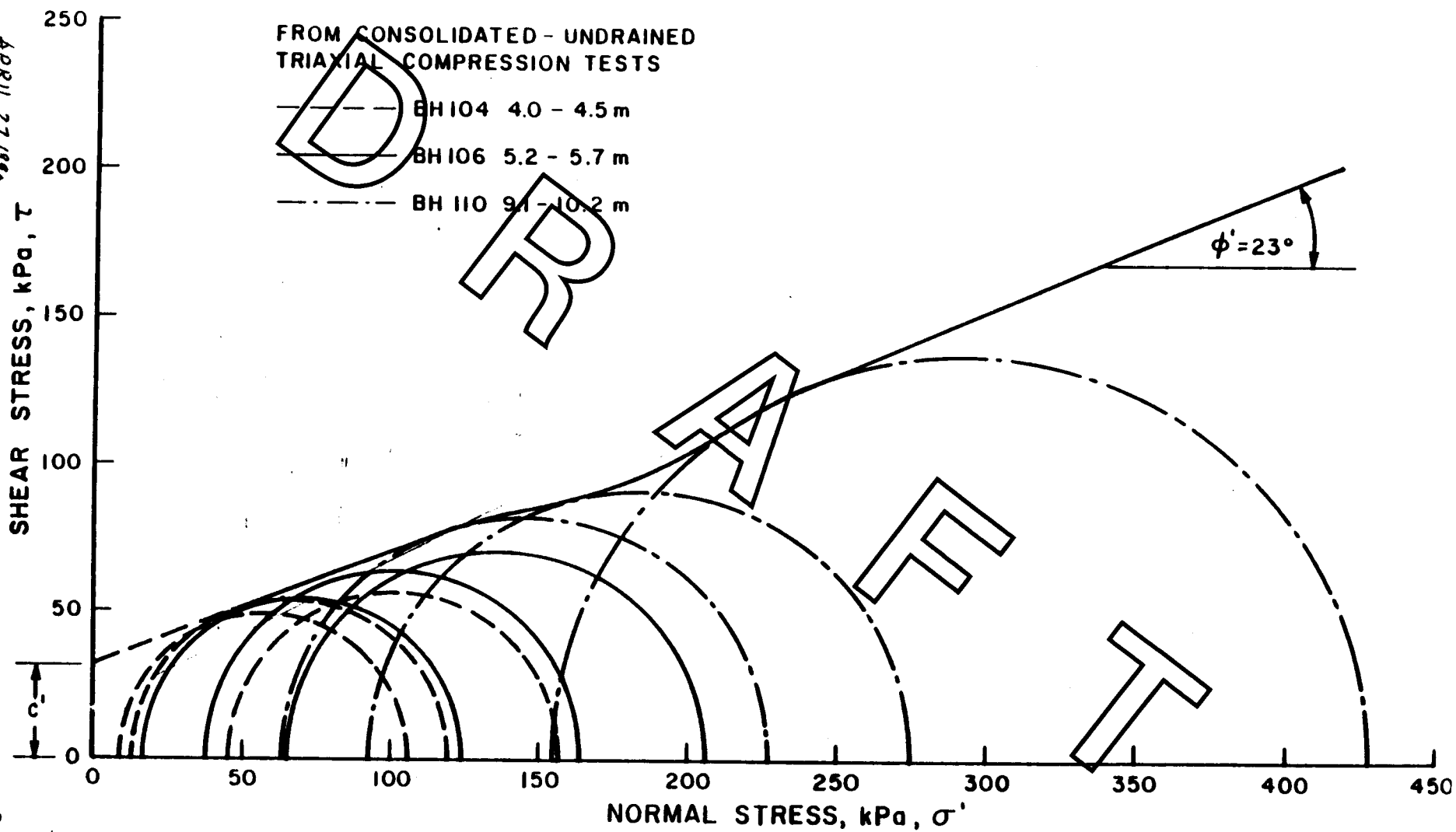
Drawn KL

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Date APRIL 27 1990
Project Pot-1339

Golder Associates

Drawn AN
Chkd _____



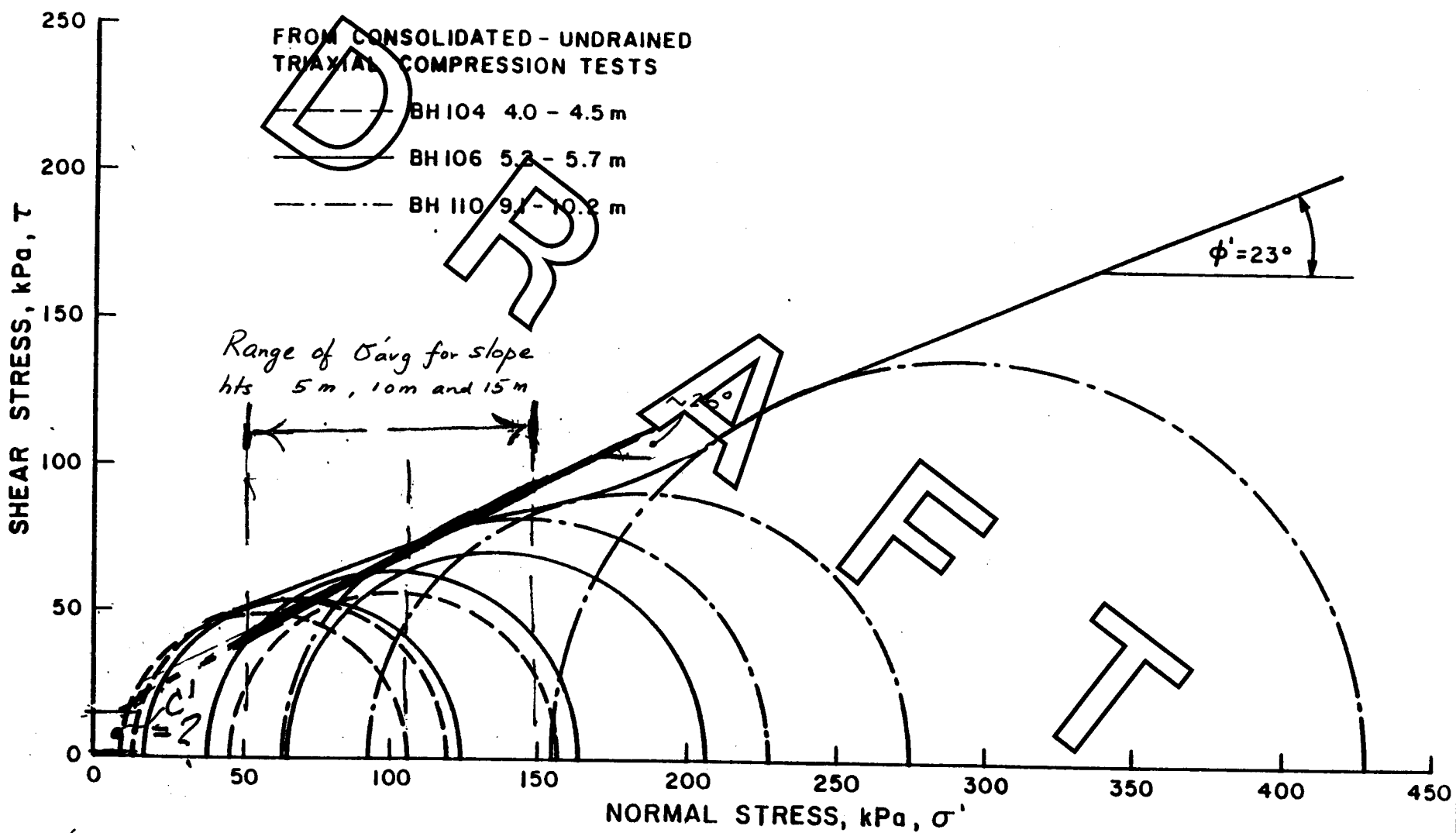
SOURCE: ACRES REPORT TO MTO
WP 126-87-01
FIGURE NO 2

**MOHR CIRCLES AND ENVELOPE (ACRES'
INTERPRETATIONS)**

FIGURE 8

REVISED INTERPRETATIONS FOR EFFECTIVE
STRESS PARAMETERS

FIGURE 9



$c' \sim (?)$
 $\phi' \sim 25^\circ - 26^\circ$

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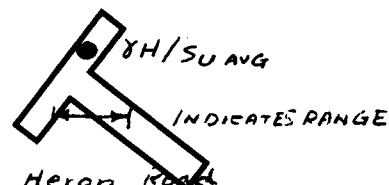
Golder Associates

Drawn RAI
Chkd _____

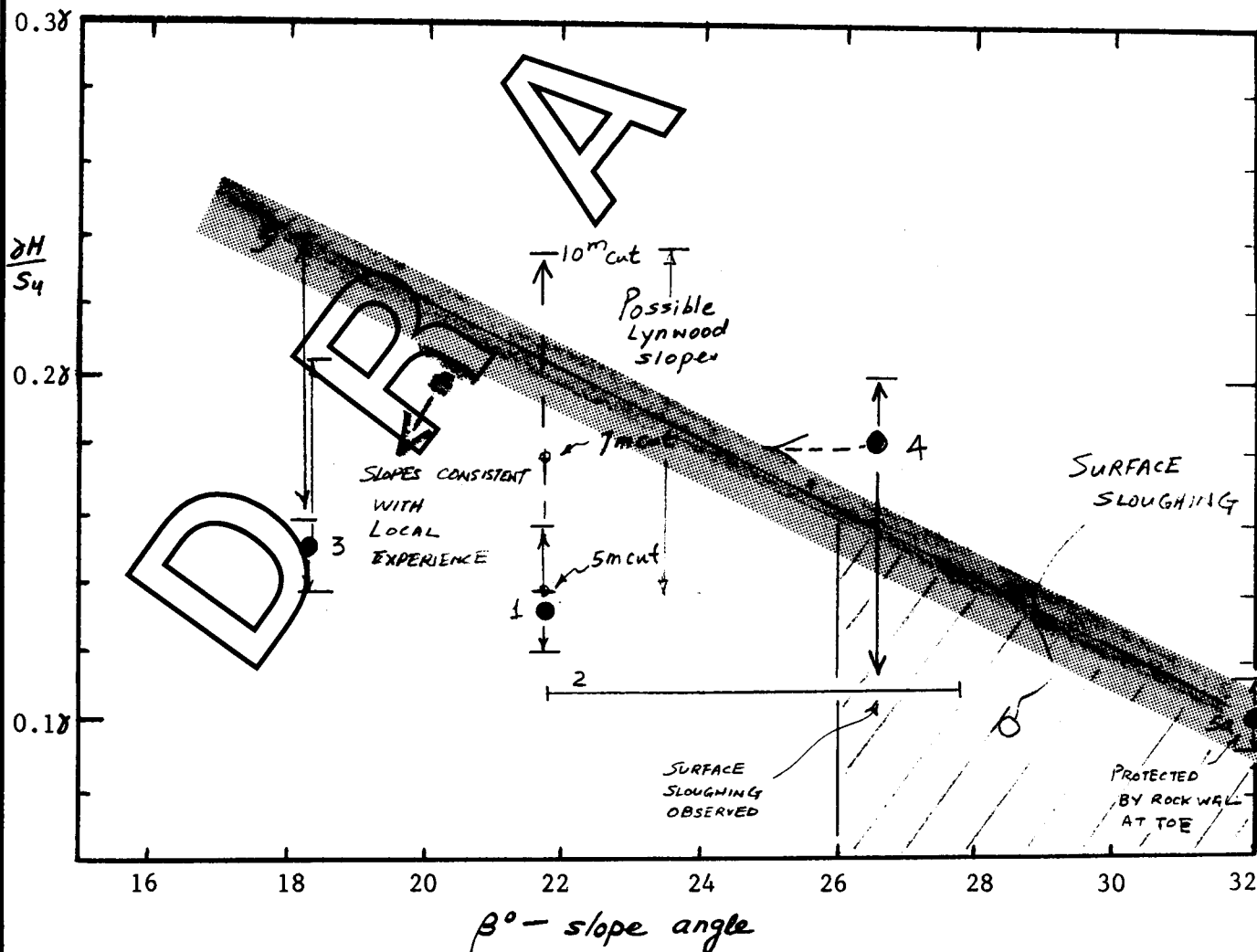
"NORMALIZED" SLOPE DATA $\delta H/S_u$ VS β (SLOPE ANGLE)

FIGURE 10

KEY



1. Heron Road
2. Billing Bridge Transitway Station
3. Hunt Club Road Underpass
4. Orleans Blvd. Cut
- 5 and 5a. Prestone Dr. Access Rd.



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Project 90.1-133.9

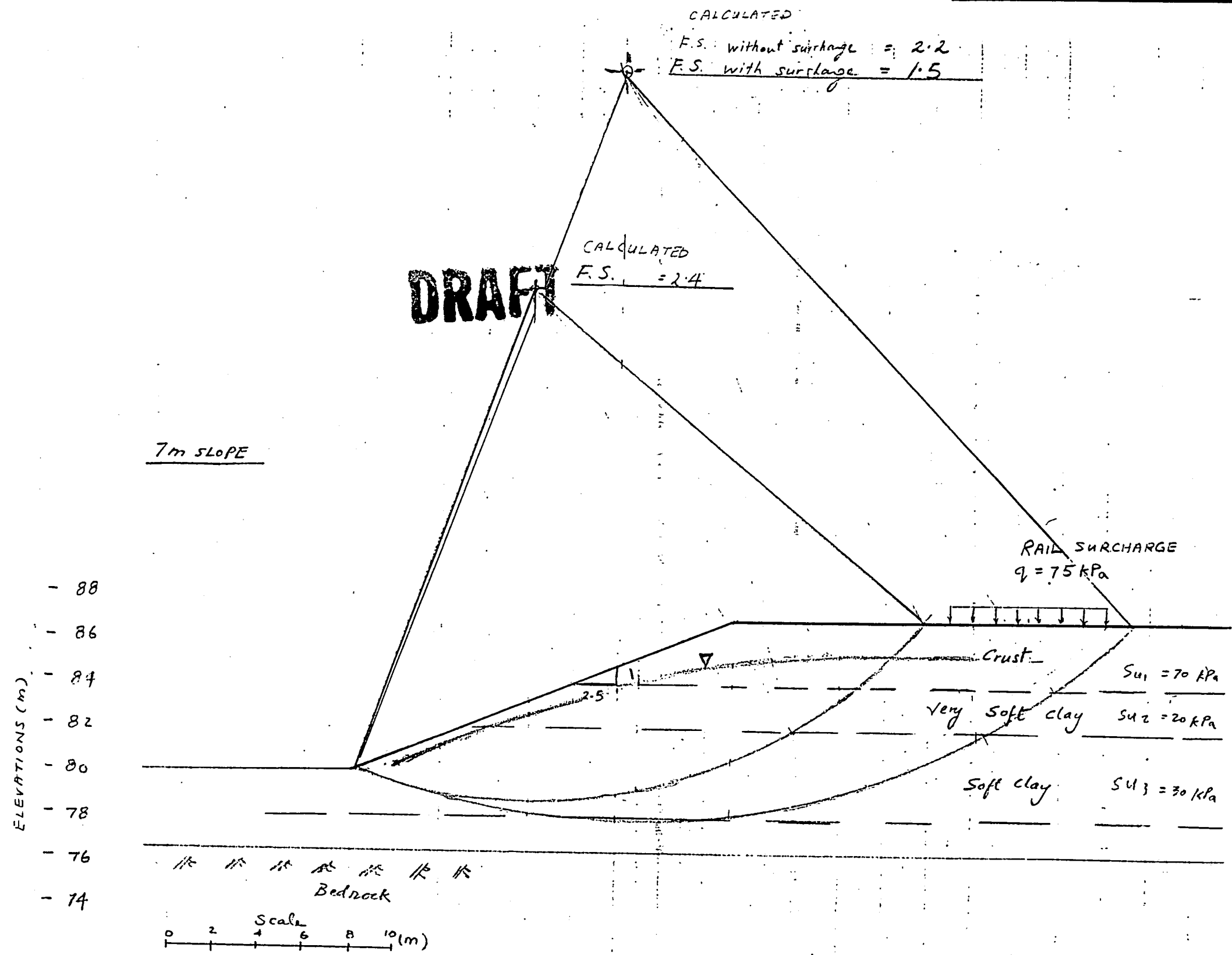
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STABILITY ANALYSES (UNDRAINED) USING
DESIGN PROFILE (7 m SLOPE)

FIGURE II



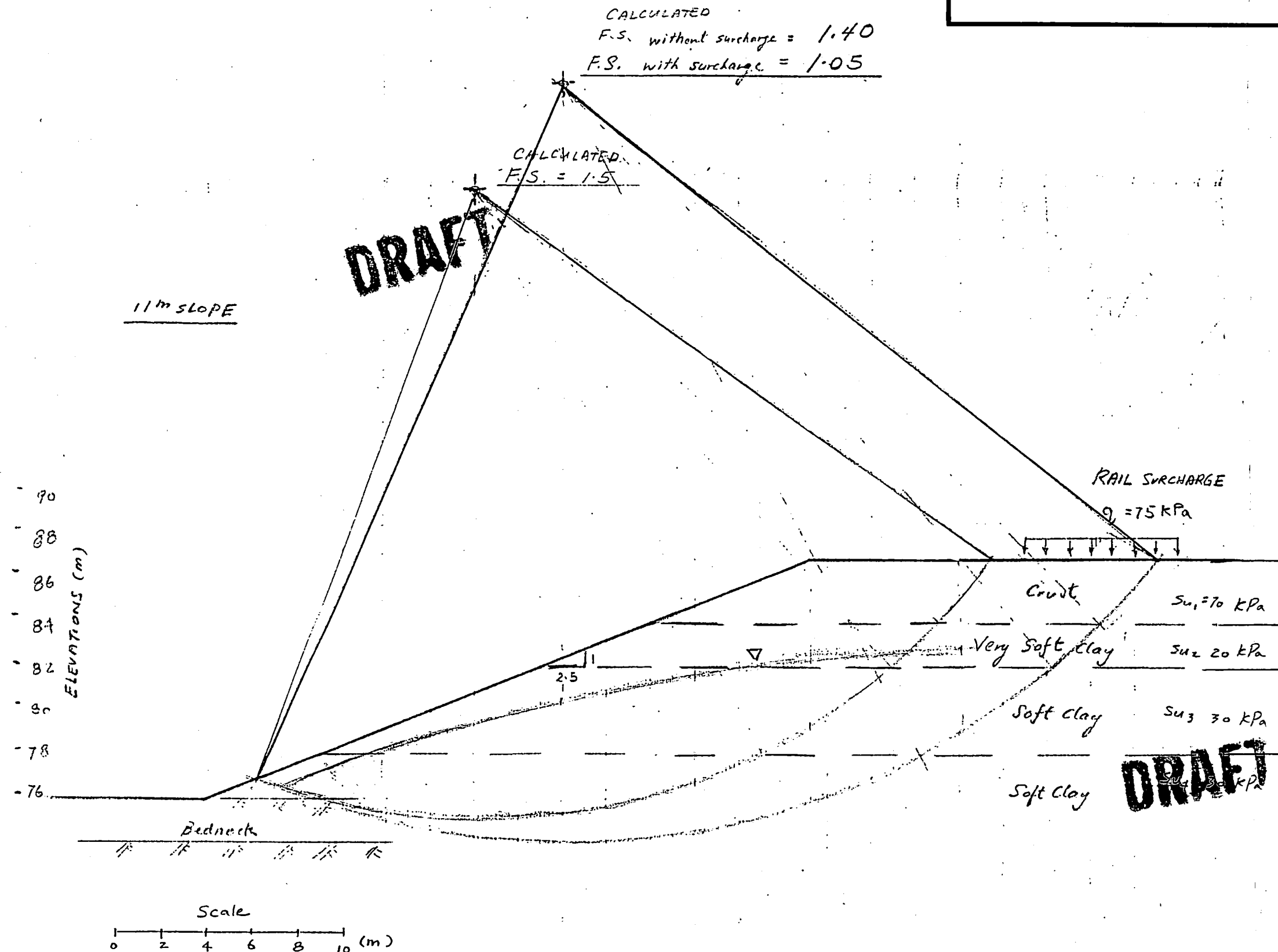
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 Draw: R.H.
 Chkd:

STABILITY ANALYSES (UNDRAINED) USING
DESIGN PROFILE (11 m SLOPE)

FIGURE 12



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Drawn: PH
Chkd:

FIGURE 13



SUMMARY OF C' , ϕ' - ANALYSIS

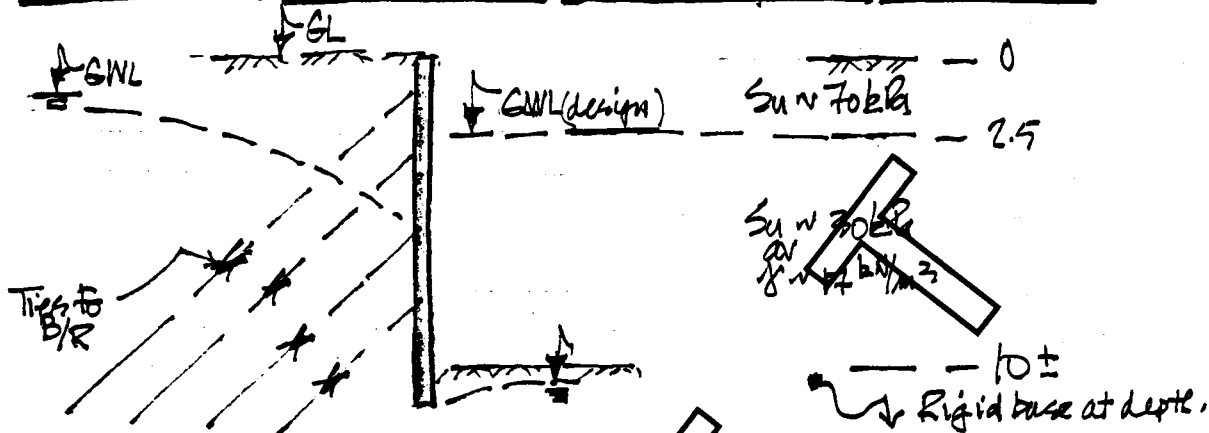
H(m)	C'(kPa)	ϕ'	$C'/\gamma H$	FACTOR OF SAFETY
5 to 6	4 to 5	26°	0.05	1.3 ($r_u=0.3$, overall slope) 1.0 ($r_u=0.5$, local toe area)
10 to 11	4 to 5	26°	0.025	1.2 ($r_u=0.3$)
15	4 to 5	26°	0.02	1.1 ($r_u=0.3$)

Date APR 12, 27, 1990
Project 901-1339

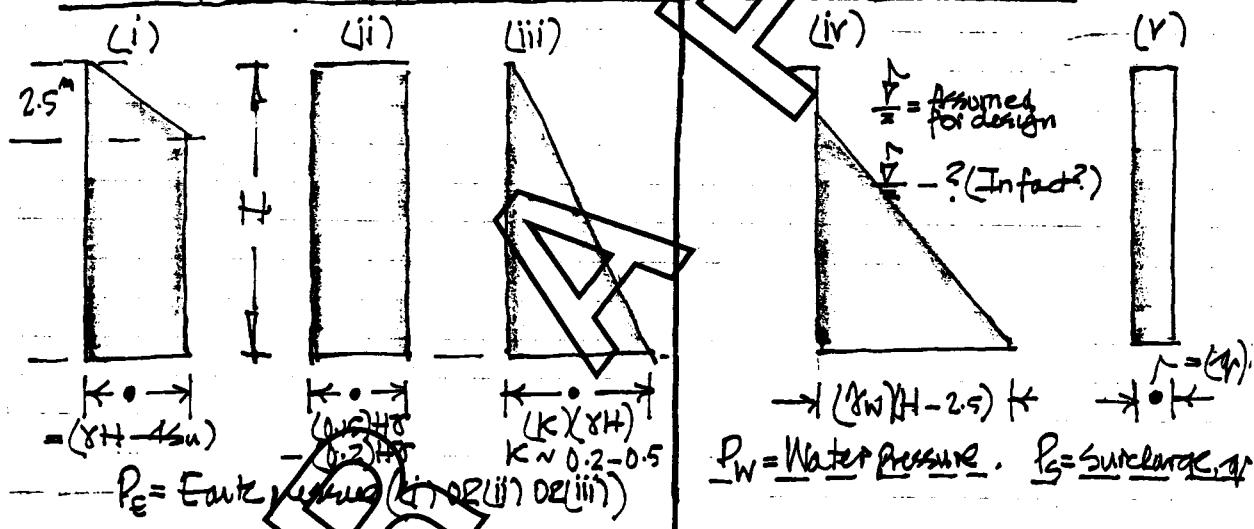
Goldier Associates

Drawn *RJ*
Chkd.

CONDITIONS ASSUMED FOR PRELIMINARY WALL DESIGN.



ASSUMED RANGE(S) OF DESIGN PRESSURES ON THE WALL.



EXAMPLE DESIGN PRESSURE(S) WHERE $H = 10^m$

For case (i) $\sum P_E \sim 500 \text{ kN/m.}$
 $\sim 340 \text{ kN/m.}$ (where $K = 0.2$)
 $\sim 425 \text{ kN/m.}$ (where $K = 0.5$)

For case (iv) $\sum P_W = 375 \text{ kN/m.}$ ($H = 10^m$)
 $\sum P_S = 50 \text{ kN/m.}$ ($K = 0.5$)

Case (iii) probably more appropriate
 LONG TERM CASE,
 CASE $\sum P_E = 425 \text{ kN/m.}$

Thus, $\sum P = \sum P_E + \sum P_W + \sum P_S = 425 + 375 + 50 = 850 \text{ kN/m.}$
 for design