G.I.-30 SEPT. 1976

GEOCRES No.

DIST. 5 REGION

W.P. No.

CONT. No.

W. O. No. 93-11009

STR. SITE No. 30-664 W

HWY. No. Municipal

LOCATION City of Orillia
Atherley Rd. Retaining Wall

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

371-2895

MEMORANDUM

TO: A. Witecki
Municipal Engineer
Approvals Section
Structural Office

DATE: 93 09 03

Attn: Albert Lee

FROM: Foundation Design Section

RE: Atherley Road Retaining Wall
City of Orillia
FD Section WO 93-11020, Site NA
District 5, Owen Sound

Further to your memo of July 22/93, we have reviewed the proposal for a retaining wall at this site. The proposal consisted of a soldier pile/lagging permanent retaining wall.

Our original assessment was that the proposal was more than adequate, but we recommended gabions should be considered on a cost-effectiveness basis. Based on our review, we understood that the cost of gabions would be less than 50% that of the soldier pile proposal, and would result in savings in the order of $50K.

As you are aware, the city's consultants were requested to review the technical aspects of a gabion proposal submitted by Maccaferri Gabions. Their response was that gabions were not acceptable for this project.

In view of the scheduling constraints and the position adopted by the consultant, we recommend that you proceed with the project as originally proposed (soldier piles). In our opinion it is unlikely that a consensus for the gabion approach can be arrived at without considerable negotiations that would minimize any potential cost advantages.

Mike Kelly of District 5 was advised of our recommendation during a previous telephone conversation.

D. Dundas

D. Dundas, P.Eng.
Sr. Foundation Engineer

cc: Mike Kelly

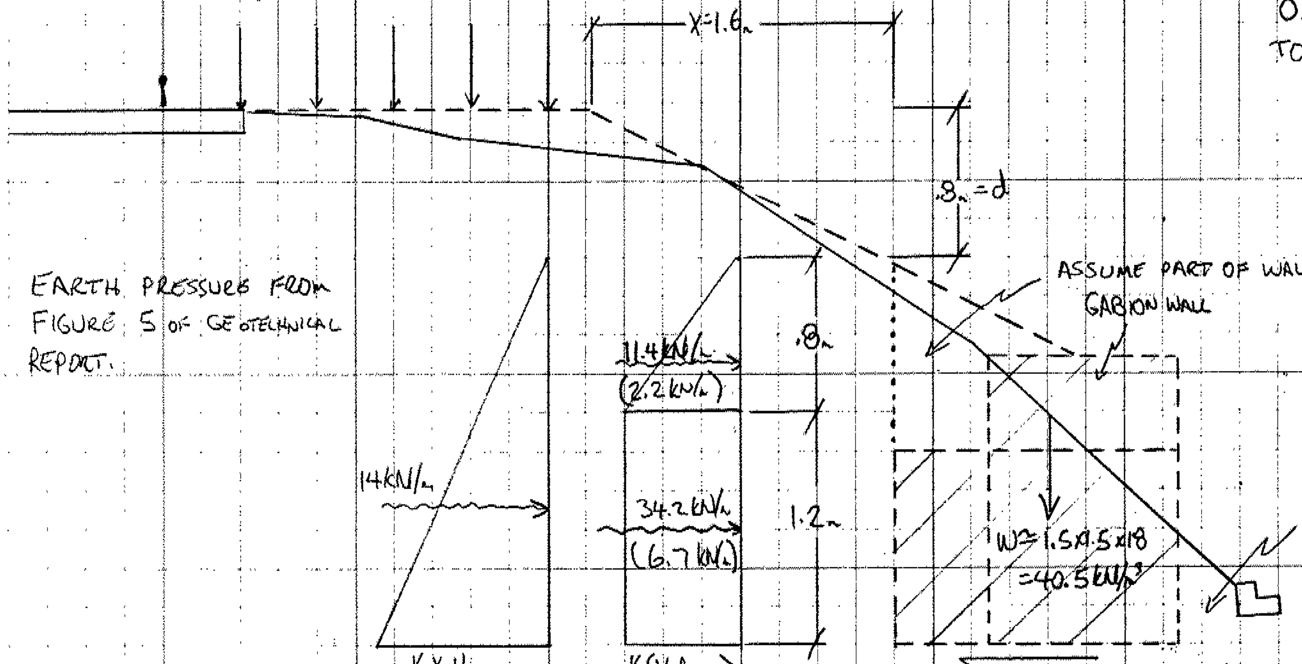
SECTION 2+80

TRIAL GABION - 0.3m SET BACK

0.3m BELOW G.S.
1.5m BASE WIDTH

NOTE: CITY REQUIRES 0.9m SET BACK; 0.3m USED IN AN EFFORT TO MAKE GABION "FIT"

CONCLUSION: NOT STABLE EVEN IF SURCHARGE IGNORED!



$K\gamma H$
$.35(20)(2.0)$
$= 14\,kPa$

$K(\gamma d + q)$
$= .35(20(.8)+65)$
$= 28.4\,kPa$

(IF NO SURCHARGE)

$K(\gamma d)$
$= .35(20(.8))$
$= 5.6\,kPa$

$T = .6^{*}(40.5)$
$= 24.3\,kN/m$

$^{*}\mu = .6$ FROM GEOTECHNICAL REPORT $(\delta = 31°)$

F.S. SLIDING $\dfrac{24.3}{(14 + 11.4 + 34.2)} = .41$

IF NO SURCHARGE

F.S. SLIDING $: \dfrac{24.3}{(14 + 2.2 + 6.7)} = 1.06$ NOT ACCEPTABLE.

SCALE 1:400

LICENSED PROFESSIONAL ENGINEER
Aug 9/93
J. WESTLAND
PROVINCE OF ONTARIO

Golder Associates

SUBJECT GABION CALCS
Job No. 931-1356
Ref. CITY/WALL/ORILLIA
Made by
Checked
Reviewed
Date AUGUST 9, 1993
Sheet 1 of 3

MACCAFERRI

Form G.A.—E—3

Form G.A.—E—3

Golder Associates

SUBJECT: GABION CALCS
Job No. 931-1356
Ref. CITY/WALL/ORILLIA
Made by
Checked
Reviewed
Date AUGUST 9, 1993
Sheet 2 of 3

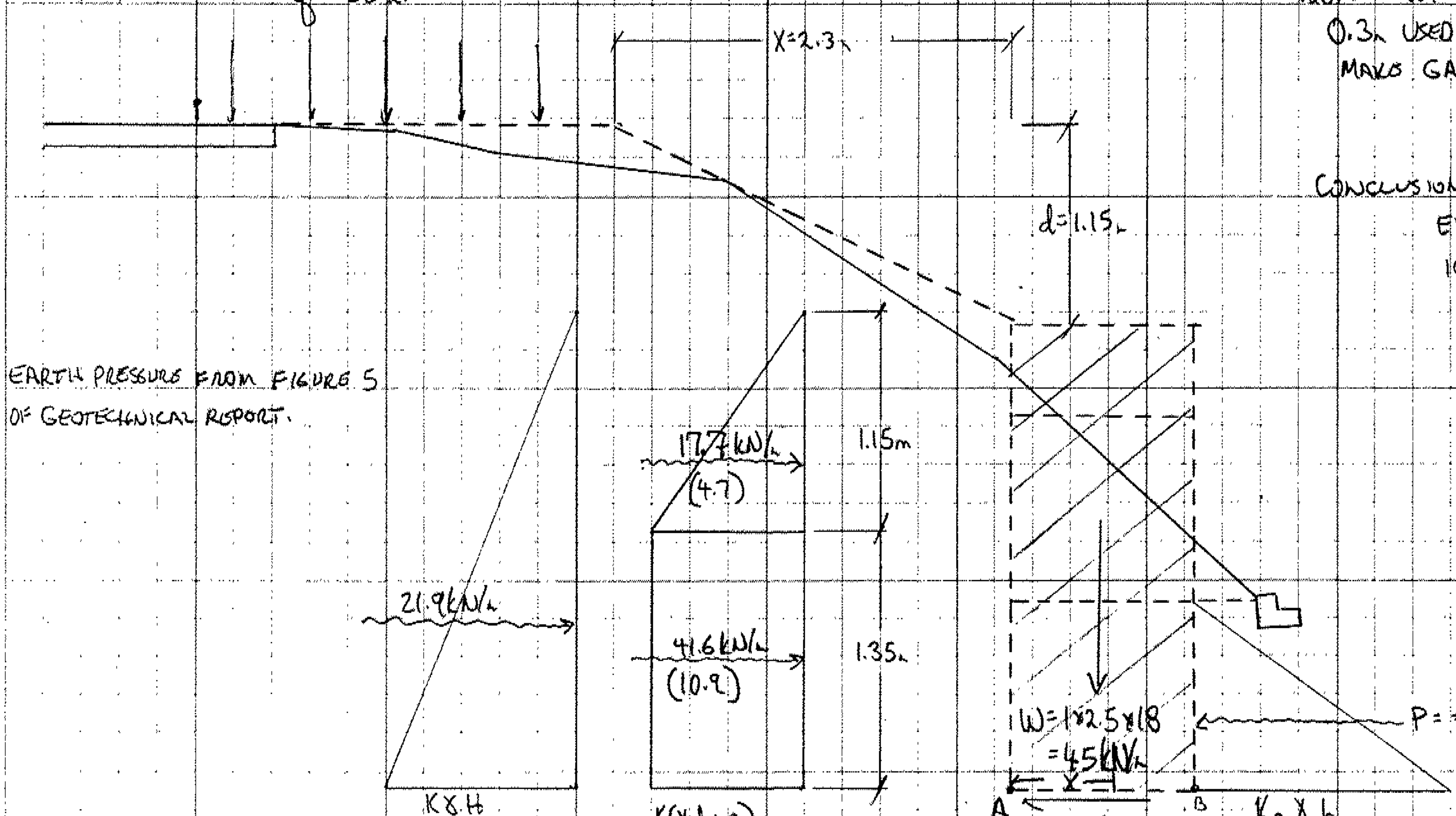
SECTION 2+80

TRIAL GABION - 0.3m SET BACK

1.0m BELOW G.S.
1.0m BASE WIDTH.

NOTE: CITY REQUIRES 0.9m SET BACK; 0.3m USED IN AN EFFORT TO MAKE GABION FIT.

CONCLUSION: NOT STABLE EVEN IF SURCHARGE IGNORED!



$K\gamma H$ = 17.5 kPa.

$K(\gamma d+q)$ = 30.8 kPa.

T = .6(45)
27 kN/m

$K_p \gamma h$
4(22)(1) = 88 kPa.

(IF NO SURCHARGE)
$K(\gamma d)$
8.1 kPa

F.S. SLIDING $= \frac{27+44}{41.6+17.7+21.9} = .87$ WITH SURCHARGE

F.S. SLIDING $= \frac{27+44}{21.9+10.9+4.7} = 1.9$ NO SURCHARGE

LOCATION OF RESULTANT $10.9\left(\frac{1.35}{2}\right)+4.7\left(1.35+\frac{1.15}{3}\right)+21.9\left(\frac{2.5}{3}\right)+45(.5)$
$-44(.33)-45x=0$

$x = .93$m !! OUTSIDE MIDDLE THIRD

CHECK OVERTURNING.
(NO SURCHARGE)

F.S. OVERTURNING (B) $= \frac{45(.5)+44(.33)}{10.9\left(\frac{1.35}{2}\right)+4.7\left(1.35+\frac{1.15}{3}\right)+21.9\left(\frac{2.5}{3}\right)} = \frac{37.02}{33.8} = 1.1 < 1.5$

SCALE 1:400

LICENSED PROFESSIONAL ENGINEER
Aug. 9/93
J. WESTLAND
PROVINCE OF ONTARIO

SECTION 2+80

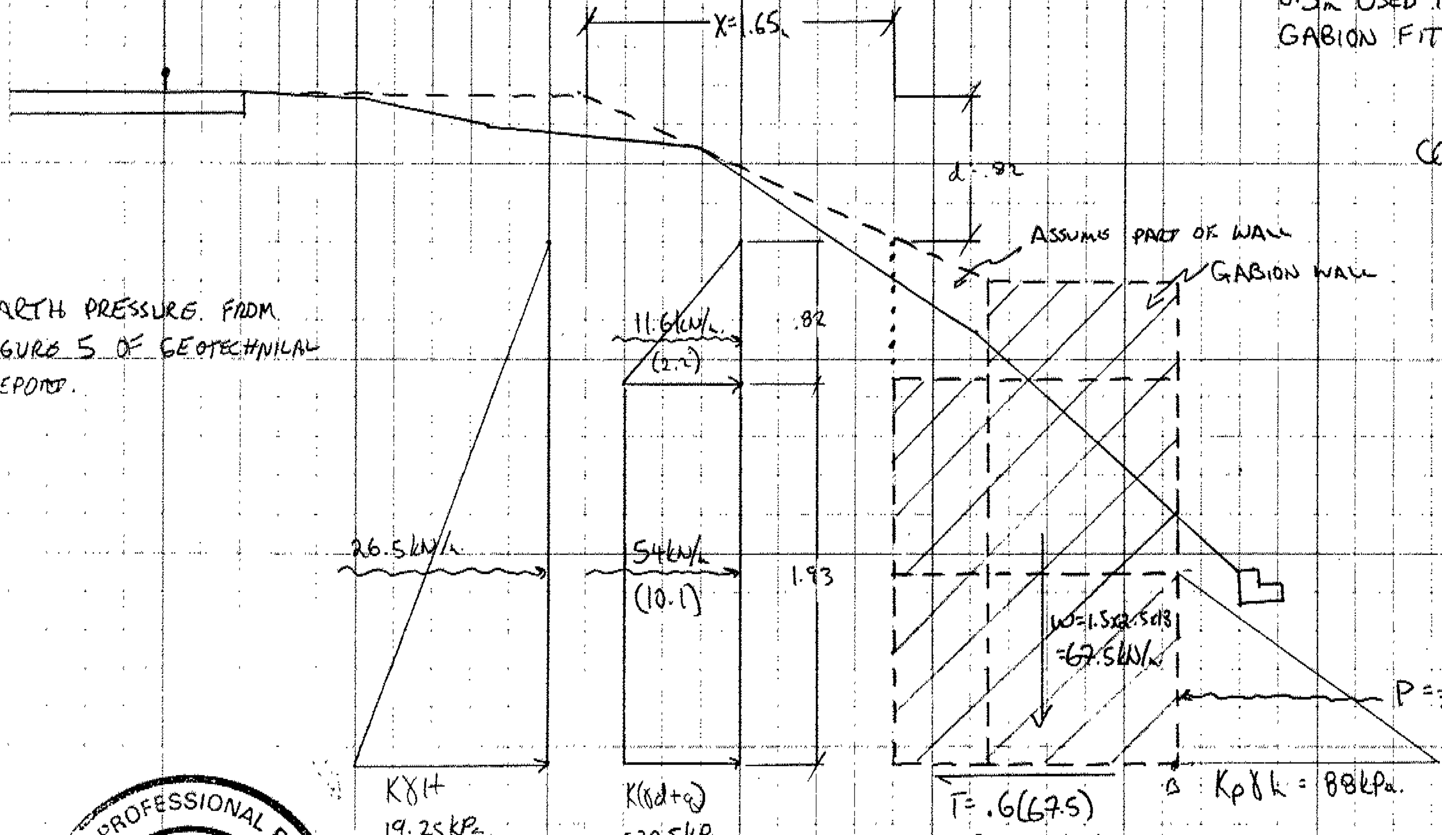
TRIAL GABION – 0.3m SET BACK

1.0m BELOW G.S.
1.5m BASE WIDTH

NOTE: CITY REQUIRES 0.9m SET BACK; 0.3m USED IN AN EFFORT TO MAKE GABION FIT.

CONCLUSION: NOT STABLE UNDER SURCHARGE. STABLE WITHOUT SURCHARGE.

EARTH PRESSURE FROM FIGURE 5 OF GEOTECHNICAL REPORT.



(IF NO SURCHARGE)
Kγd = 5.25kPa

T = .6(67.5)
= 40.5kN/m

$F.S._{SLIDING} = \frac{40.5+44}{26.5+54+11.6} = .92$ ✗ SURCHARGE

$F.S._{SLIDING} = \frac{40.5+44}{26.5+10.1+2.2} = 2.2$ ✓ OK NO SURCHARGE

$F.S._{OVERTURNING} = \frac{67.5(.75)+44(.33)}{10.1(\frac{1.93}{2})+2.2(1.93+\frac{.82}{3})+26.5(\frac{2.75}{3})} = \frac{65.1}{38.9} = 1.67$ ✓ OK

SCALE 1:400

LICENSED PROFESSIONAL ENGINEER
Aug. 9/93
J. WESTLAND
PROVINCE OF ONTARIO

Golder Associates

SUBJECT	GABION CALCS
Job No.	931-1356
Ref.	CITY/WALL/ORILLIA
Made by	JW
Checked	
Reviewed	
Date	AUGUST 9, 1993
Sheet	3 of 3

Form G.A.–E–3

MINISTRY OF
TRANSPORTATION

FAXGRAM

May 14/93

TO: DAVE DUNDAS FOUNDATION SECTION FAX 416-235-5240	FROM: M. J. KELLY DIST.MUN.ENGINEER DISTRICT #5, OWEN SOUND TEL. 519-376-7350 FAX 519-371-2895

RE:

Soils Report

As discussed.

memo To file. May 14/93

I discussed project with Mike Kelly, Municipal Eng Dist 5. He wanted my opinion on recs. Apparently Dist. 5 is reviewed job & subsidy. I told Mike

1. recs will provide workable wall
2. recs are very conservative
3. depth of embedment of soldier pile should be reviewed
4. consideration should be given to simple gabion wall
5. contact Maccaferri for design for Gabion wall
6. if gabion wall used cant it back at 15°
   - 2 rows on bottom, 1 on top
   - concurrent excavation to wall construction
   - limit strip excavation to say 10 to 12'
   - get rid of crest detail
   - no drain at bottom but ensure no runoff across way
   - construct at times when no train traffic
   - have contingency to repair track if necessary & monitor
7. I advised Mike to be cautious not to assume liability from McCormick Rankin but to have them reevaluate the cost-effectiveness of their proposal in light of limitations on subsidy and compare possibility of gabion structure to their ~~soldier pile~~ wall

RECEIVED
MAY 14 1993
FOUNDATION DESIGN SECTION

8. I advised Mike to use sensitivity & tact in dealing with the players since the Town will want ~~Cad~~ Cadillac wall & the consultants will want to justify their initial recs.

D. Dundas
Sr. Fdn Eng

The terms of reference for this investigation are outlined in our proposal (Ref. P31-1015) dated January 18, 1993. Authorization to proceed with the investigation was received from the City of Orillia, on March 5, 1993.

2.0 SITE DESCRIPTION

The site is located along the north side of Atherley Road between Millard Street and Moffat Street in Orillia, Ontario (see Figure 1). The section of affected embankment extends from about Sta. 2+55 to about Sta. 3+10 (City of Orillia, Contract Chainage), a distance of about 55 m. As a result of the Atherley Road reconstruction, the CN Rail embankment toe has been over-steepened to a gradient of about 1 horizontal to 1 vertical over this length. The north curb of Atherley Road now forms the toe of the 2.5 m to 3 m high embankment.

The ground surface at the site slopes slightly from east to west with ground surface elevations ranging from 220.6 m to 220.2 m along the length of the proposed retaining wall. The general topography of the surrounding area lying near the shores of Lake Couchiching is gently undulating. The site is located in a physiographic region known as the Simcoe Lowlands situated in an area termed the Lake Simcoe Basin. (Chapman, L.J. and D.F. Putman, "The Physiography of Southern Ontario, 2nd Edition", 1973). The region is generally bordered by shorecliffs, beaches and bouldery terraces and floored by sand, silt and clay deposits. Geological mapping of the area indicates that the site is situated in an area of sand plains and drumlins.

Based on the results of a previous subsurface investigation carried out along Atherley Road by Peto MacCallum Ltd., the stratigraphy is mainly comprised of a silt till deposit containing traces of gravel, clay and sandy silt seams. The silt till deposit is overlain by layers of fill in some areas.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on March 19, 1993, during which time four boreholes (BH 1 to BH 4) were put down at the locations shown on Figure 2. The boreholes were advanced to depths of between 6.6 m and 8.2 m below ground surface using a truck mounted CME 75 power auger drill rig supplied and operated by a local specialist drilling contractor.

Boreholes were advanced through the overburden using solid stem augers. When a relatively soft zone was encountered at the bottom of Borehole BH 3, a dynamic penetration cone was driven until refusal at 9.45 m.

- wrong equipment

An additional borehole (BH-4A) was drilled at about 7 m from Borehole BH-4 when auger refusal was encountered in BH-4 at a depth of 1.9 m below ground surface.

The field work was supervised throughout by a geotechnical engineer who located the boreholes, cleared the locations of buried services, directed the drilling and sampling operations, logged the boreholes and placed the samples in labelled, air-tight containers. In each boring soil samples were obtained at selected depth intervals as part of the Standard Penetration Test (SPT), using conventional 50 mm outside diameter split-spoon sampling equipment. All of the soil samples were visually identified in the field and returned to our geotechnical laboratory for further examination and classification testing. Representative soil samples were selected and tested for index properties such as water content, grain size distribution and Atterberg Limits.

Piezometers were installed in Boreholes BH-1 and BH-4A on completion of the drilling operation to monitor groundwater levels at the borehole locations. The piezometer tips were surrounded by pea gravel and sealed with bentonite into the bottom of the boreholes which were then backfilled with native soil auger cuttings.

The locations of the completed boreholes were surveyed by our staff and are referenced to stations as laid out by the City of Orillia for the Atherley Road reconstruction project. Ground surface elevations for the completed boreholes were surveyed by the City of Orillia Engineering Department, and it is understood that the elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

The detailed subsurface conditions encountered in each of the boreholes, together with the results of the field and laboratory tests carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report. It should be noted that the stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and represent a transition between soil types rather than an exact plane of geological change. Conditions will vary between and beyond the borehole locations.

The subsurface conditions, as inferred from the boreholes, generally consist of asphalt and road base fill overlying a clayey silt deposit. This clayey silt stratum was found to contain pockets of silt and sandy silt in some places and lenses of sandy gravel and silty clay in others. The measured groundwater levels, at the time of the investigation, vary from 2.4 m to 2.7 m below ground surface (Elevation about 217.8 m).

4.1 Asphalt and Road Base Fill

The asphalt on Atherley Road, encountered at the borehole locations, ranges in thickness from 0.10 m to 0.14 m and is underlain by a road base fill. The road base fill is composed of a brown sand containing some gravel and trace silt. It varies in thickness from 0.5 m to 0.7 m with measured SPT "N" values ranging from 26 to 93 blows per 0.3 m penetration, indicating a compact to very dense relative density.

4.2 Clayey Silt (Till)

Light brown and grey clayey silt with some sand, trace to some angular to sub-angular gravel, occasional cobbles and occasional lenses of sandy silt was encountered in all of the boreholes along the retaining wall alignment. At most locations this clayey silt deposit extended from below the road base fill to the bottom of the borehole. However, at Borehole BH-3, below about 5.2 m the clayey silt is interlayered with silts and sandy silts.

Measured SPT "N" values in this deposit range from 6 to 58 blows per 0.3 m penetration indicating a firm to hard consistency. The natural water content of the soil samples tested varies from 7 per cent to 15 per cent with an average of about 11 per cent. The measured plastic limits are 11 per cent and 12 per cent and the measured liquid limits are 17 per cent and 21 per cent. Grain size distribution curves for the soil samples tested are presented on Figure 3.

The soils in this area are considered to be of glacial origin, and Borehole 4 met auger refusal at a depth of about 2.0 m. Therefore, the presence of cobbles and/or boulders should be anticipated in this glacial till deposit.

4.3 Sandy Gravel

Layers of light brown sandy gravel containing some silt and trace clay were encountered in BH-1, BH-2 and inferred in BH-3. The thickness varied between 0.8 m and 1.4 m. Measured SPT "N" values for this material range from 10 to 23 blows per 0.3 m penetration, indicating a compact relative density. The natural water content of the soil samples tested varies from 8 per cent to 12 per cent. A typical grain size distribution curve for a sample of this deposit is shown on Figure 4.

4.4 Silty Clay

A 0.8 m layer of firm, grey silty clay with trace sand and gravel was encountered at the bottom of BH-3. This layer had a measured SPT "N" value of 6 blows per 0.3 m penetration and the natural water content of the sample tested was 28 per cent.

4.5 Groundwater Conditions

A piezometer was installed in borehole BH-1 and BH-4A. The groundwater levels measured on March 30, 1993 are shown on the Record of Borehole sheets, and vary from 1.0 m to 1.4 m below the ground surface (Elevation 219.1 m).

It should be noted that the groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions will vary seasonally.

5.0 DISCUSSION AND DESIGN RECOMMENDATIONS

This section of the report provides our interpretation of the field and laboratory data gathered to date and provides our assessment of the current condition and long-term stability of the CN Rail embankment within the affected over-steepened section along Atherley Road. A discussion of retaining wall alternatives for the site is provided together with preliminary engineering design recommendations. A preliminary cost estimate is also provided for the recommended alternative.

It is noted that the subsoil conditions have been determined at a limited number of borehole locations along the proposed retaining wall alignment. The interpretation of the data is intended for the guidance of the design engineer only. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the works should make their own interpretation of the subsurface information provided as it may affect their proposed construction methods, equipment selection, scheduling and the like.

The professional services of Golder Associates Ltd. retained for this project include only the geotechnical aspects of the subsurface conditions and design. The presence or implication of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

5.1 Stability of Embankment

The section of CN Rail embankment affected by the reconstruction and widening of Atherley Road extends from about Sta. 2+55 to about Sta. 3+10 (City of Orillia, Contract Chainage), a distance of about 55 m. Over this length, the north curb of Atherley Road now forms the toe of the embankment. As a result of the road widening, the embankment has been over-steepened; the toe is now lower by a maximum of about 0.4 m and is shifted northward by a maximum of about 1 m. This has resulted in a steepening of the lower portion of the embankment slope to a gradient of about 1 horizontal to 1 vertical.

Survey data acquired prior to construction indicates that the embankment originally had side slopes of 1.8 horizontal(h) to 1 vertical(v) to 1.9h to 1v. Surveyed profiles of the current embankment condition, prepared by the City of Orillia, indicate that the railway ties lie behind a line projected at a 2 horizontal to 1 vertical gradient from the embankment toe. Based on the observed embankment materials, this setback is sufficient for short-term stability and the condition of the embankment does not pose an immediate threat to rail traffic.

It is not feasible, however, to maintain the embankment in its present form over the long-term. The lower slope is too steep and subject to sloughing and ravelling especially during thaws and when it is raining. This would result in further over-steepening and would eventually threaten the safety of the rail tracks. Until a permanent wall is constructed, regular monitoring and inspection of the embankment condition should continue. Such a program should involve visual monitoring of the geosynthetic filter fabric erosion control system to ensure its adequate performance in the prevention of surface sloughing. In addition, the established bi-weekly survey monitoring program of the rail tracks should continue.

In order to enhance the long-term stability of the affected area of the embankment, measures must be taken to return the profile to a 2 horizontal to 1 vertical slope. If the alignment of the newly reconstructed Atherley Road is to be unaltered, then this can only be accomplished by the construction of a retaining wall.

5.2 Retaining Wall Design Considerations

The retaining wall proposed for the Atherley Road rail embankment must extend over the 55 m length of over-steepened embankment. In addition, the ends of the wall must taper, and for preliminary design and cost estimating purposes, it is assumed that there will be a 5 m long taper at either end of the wall, providing a total wall length of 65 m. For purposes of traffic safety and snow removal, the wall must be setback from the curb of the road. We understand from the City of Orillia that this setback could be between 0.9 m and 1.2 m. For preliminary design it is assumed that the setback will be 1.2 m from the back of the curb. Again, for traffic safety purposes, a barrier wall or impact wall may be necessary between the toe of the wall and the curb. The proposed setback of the wall from the curb would allow construction of a barrier or impact wall and such a structure may be incorporated into the final wall design, if required by the City.

The retaining wall should be designed to resist the lateral earth pressure as illustrated on Figure 5, calculated using the following design parameters:

Coefficient of Lateral Earth Pressure	$K = 0.35$
Unit Weight of Embankment Fill	$\gamma = 20$ kN/m^3
Height of Groundwater Table	$h_w = 0$
Uniformly Distributed Surcharge	$q = 65$ kPa

The surcharge loading due to rail traffic is based on the Cooper E90 locomotive axle loading, as obtained from CN Rail Engineering Services. The design loading condition is a line load of 40,820 kg (90,000 lbs) per axle, for 4-axles spaced 1.5 m apart. For design purposes, it is assumed that the axle loads are spread to the crest of the embankment and are treated as a uniform surcharge of 65 kPa, or the equivalent of about 3 m of earth.

5.3 Retaining Wall Alternatives

5.3.1 Gravity Wall

Gravity retaining walls resist forces imposed by retained soils by the nature of their size and self weight. Common types of gravity walls include:

Gabion Walls
Stone Block Walls
Concrete Cantilever Walls
Soil Reinforced Walls

The internal design of each of these wall, types varies; however, each wall type must satisfy the same global stability requirements:

- adequate sliding resistance
- adequate overturning resistance
- sufficient bearing capacity of the underlying soils

For preliminary design of a gravity wall, an allowable bearing capacity of 150 kPa is appropriate for the site soils. The coefficient of sliding resistance appropriate for the design of various gravity wall types is as follows:

Gabion Wall, Soil Reinforced Wall:	0.6
Concrete Cantilever Wall, Stone Block Wall:	0.4

Based on the surcharge loading conditions and the embankment geometry, a gravity retaining wall would have to be at least 1.2 m wide at the base. A schematic design sketch of a gabion wall is illustrated on Figure 6. As illustrated on Figure 6, the wall width, its set back from the road and the space required for construction would mean that an open cut excavation for the gravity wall construction would require closure of the rail tracks, and therefore, this is an unacceptable alternative.

Closure of the tracks could be avoided by the construction of a temporary soldier pile and lagging shoring wall, the location of which is illustrated on Figure 6. While this construction measure would avoid complete closure of the rail track, access to and temporary closure of the tracks would be required to install the soldier piles. Furthermore, the required shoring wall would significantly increase the cost of construction. For these reasons, a gravity wall alternative is not considered to be as economical as other alternatives as discussed below.

5.3.2 Soldier Pile Wall

A soldier pile retaining wall is a system composed of evenly spaced vertical load bearing steel "H"-sections which support precast reinforced concrete panels or a cast-in-place concrete facing; it could be constructed with minimal disruption to railway operations. Installation of the soldier piles could be carried out from roadway level as could the installation of the wall facing.

The soldier pile wall should be designed to resist the lateral earth pressure distribution as discussed in Section 5.2. Resistance to the earth pressure would be provided by the passive restraint offered to the soldier pile sockets by the surrounding soil. The distribution of passive pressure may be calculated based on the pressure diagram given on Figure 7, using the following design parameters:

Coefficient of Passive Earth Pressure	$K_p = 3.5$
Unit Weight of Soil	$\gamma = 22.5$ kN/m³
Unit Weight of Water	$\gamma_w = 9.8$ kN/m³
Depth to Water Table	$d_w = 1.0$ m

In calculating the allowable passive restraint on the soldier pile socket:

- the effective width of the socket may be taken as 3 times the diameter
- the pressure over the upper 1.5 m should be neglected
- a Factor of Safety of 2 should be applied

Preliminary calculations indicate that a soldier pile wall set-back from the curb of 1.2 m and having a maximum height of about 2 m would require an embedment of about 4.5 m below ground surface. This will require augering the sockets to below the groundwater table at the site. The water bearing gravelly sand lenses encountered within the clay silt deposit may slough and ravel during drilling of the solider pile sockets, and therefore, provision should be made during construction to provide temporary liners during soldier pile installation. Such liners may be withdrawn during concrete placement, which must be carried out using tremie placement techniques. During concreting and liner removal, the bottom of the liner must remain below the level of concrete.

? driving

It should be noted that auger refusal was met at the location of Borehole 4 at a depth of about 2 m during the investigation program. Furthermore, till deposits such as that encountered at the site are inherently broadly graded and commonly contain cobbles and boulders. Provision should be made for the breaking and/or removal of cobbles and boulders during drilling of the soldier pile sockets.

5.3.2.1 Soldier Pile Wall - Facing Alternatives

The soldier pile wall design discussed may incorporate either a precast concrete panel facing or a cast-in-place concrete facing. Both of the alternatives are considered feasible for the site and preliminary designs have been developed for each alternative. In preparing the preliminary design, structural design input was obtained from McCormick Rankin Associates Ltd. Typical sections of each alternative are provided on the figures contained in Appendix A.

Alternative 1, consisting of a cast-in-place concrete facing would be constructed first as a temporary timber lagging wall. The reinforced concrete face would be secured to soldier piles spaced at 2.4 m centres. As shown on the typical section the temporary lagging would allow installation of a deep drain at the base of the wall. Any seepage from between the lagging and the face would be collected and discharged through a geocomposite drain placed between the lagging and the concrete face. Surface run-off from the embankment would be collected in a shallow ditch behind the wall and carried to either end of the wall.

The principal advantage of this alternative is that it provides the least risk of disruption to railway operations. The installation of timber lagging would be carried out as the toe of the existing embankment is excavated, essentially providing continuous support to the embankment. The cast-in-place wall face provides an opportunity to create a sculpted facing if desired. Similarly, if a traffic barrier at the toe of the wall is required, it could be integrally cast into the wall face.

Alternative 2, consisting of a precast concrete facing and galvanized soldier piles would require cutting the embankment back to the limit of the backfill (as illustrated on the typical section in Appendix A) prior to installing the facing. To minimize the risk to the rail track, the cut would have to be made in short sections, limited to the distance between two soldier piles. The embankment cut and panel placement would have to be coordinated with the train schedule such that a full cycle of excavation and panel placement could be completed without a train passing.

As shown on the typical section, drainage would be provided by a layer of free-draining Granular 'B' backfill placed behind the wall. Given the wall geometry, placement and compaction of the drainage layer would be difficult.

The precast panels themselves could consist of stacked interlocking panels, each 0.3 m to 0.45 m high and 2.4 m long. Given the relatively low wall height, 2.4 m long panels spanning the full wall height could also be used. Given the requirement to install a wall section quickly to avoid interruption to rail traffic, a full-height wall panel is considered more appropriate for the site.

5.4 Preliminary Cost Estimate

Preliminary cost estimates and breakdowns for the two soldier pile retaining wall alternatives are provided in Tables 1 and 2. In summary, the estimated cost of Alternative 1 is about $91,000 and that of Alternative 2 is about $77,000. The cost estimates do not include the costs for a traffic barrier between the wall and the curb.

The estimates are based on the typical sections as provided in Appendix A and on the length of the wall as discussed in Section 5.2. The unit price estimates for the various wall components were obtained from consultation with contractors and from project records where similar walls have been constructed. Considerable variation in unit prices was found which is probably a reflection of the current economic situation. The estimates provided are weighted toward the prices established from previous jobs, as opposed to those presently provided by contractors.

It is our understanding that rail traffic adjacent to Atherley Road is relatively infrequent, and therefore, the cost of Alternative 2, which requires construction in short sections between the passage of trains, will not be significantly affected by the rail schedule. Should rail traffic increase to a level of about three of four trains per working day, then the wall construction cost for Alternative 2 may be expected to increase.

5.5 Conclusions

A cantilevered soldier pile wall is considered an appropriate long-term solution to support the 55 m length of over-steepened embankment adjacent to Atherley Road. The two alternative soldier pile wall designs presented are both feasible and would allow construction with minimal risk to the rail line. Alternative 2, utilizing precast concrete panels as a facing, is about $14,000 cheaper than the cast-in-place concrete facing alternative. However, construction of Alternative 2 will require greater coordination of schedules between the contractor and the railway. Further, because Alternative 2 will require an organized sequence of excavation, panel placement and backfill with train traffic, there is a greater risk of delaying rail traffic due to equipment breakdowns or other construction problems. The consequences and costs associated with the increased risk of rail traffic delay must be considered against the cost saving associated with Alternative 2 when selecting the preferred alternative.

GOLDER ASSOCIATES LTD.

for John Westland, P.Eng.

LICENSED PROFESSIONAL ENGINEER 26/04/93 J. WESTLAND PROVINCE OF ONTARIO

Dennis E. Becker, P.Eng.
Associate

PD/JW/DEB/pds

PROJECT: 921-1397

RECORD OF BOREHOLE 1

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: MAR. 19, 1993

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	CME-75 POWER AUGER BORING, 100mm SOLID STEM AUGERS	ROAD SURFACE	220.18					
		ASPHALT	220.04 / 0.14					CONCRETE
		Dense, brown gravelly sand, trace silt. (ROAD BASE FILL)		1	50 DO	50		
1		Stiff to very stiff, light brown CLAYEY SILT, some sand, trace to some angular to subangular gravel and occ. cobbles. Contains occ. lenses of sandy silt.	219.30 / 0.88	2	50 DO	16		BENTONITE SEAL
				3	50 DO	13		BACKFILL
2		Compact, light brown SANDY GRAVEL, some silt, trace clay.	218.05 / 2.13	4	50 DO	23	MH	
3		Very stiff, light brown becoming grey around 3.7m, CLAYEY SILT, some sand, trace to some angular to subangular gravel and occ. cobbles. Contains occ. lenses of sandy silt.	217.28 / 2.90	5	50 DO	29		BENTONITE SEAL
4				6	50 DO	16		
5				7	50 DO	20		PEA GRAVEL
6				8	50 DO	23		
		END OF BOREHOLE	213.63 / 6.55					
7								WATER LEVEL IN PIEZOMETER AT 1.2m DEPTH ON COMPLETION OF DRILLING.
								WATER LEVEL IN PIEZOMETER AT ELEV. 219.14m ON MAR. 30/93.
8								
9								
10								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat.V - + Q - ● rem.V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, K cm/s

WATER CONTENT, PERCENT — Wp —○W— Wl — 10 20 30 40

DATA INPUT: SK APR 21/93

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: PD
CHECKED: JW

LOCATION: SEE FIGURE 2 BORING DATE: MAR. 19, 1993 SHEET 1 OF 1

SAMPLER HAMMER, 63.5kg; DROP, 760mm DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME-75 POWER AUGER BORING, 100mm SOLID STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
0	ROAD SURFACE	220.19				
	ASPHALT	220.09 / 0.10				
	Very dense, brown sand, some gravel, trace silt. (ROAD BASE FILL)	219.58 / 0.61	1	50 DO	86	
1	Stiff to very stiff, light brown becoming grey around 1.07m CLAYEY SILT, some sand, trace to some angular to subangular gravel. Contains occ. lenses of sandy silt.	218.06 / 2.13	2	50 DO	14	
2			3	50 DO	17	MH
	No sample recovery - SANDY GRAVEL lens inferred.	217.29 / 2.90	4	50 DO	19	
3	Firm to very stiff, grey CLAYEY SILT, some sand, trace to some angular to subangular gravel. Contains, occ. lenses of sandy silt.	215.01 / 5.18	5	50 DO	6	
4			6	50 DO	15	
5			7	50 DO	20	
	Compact, light brown SANDY GRAVEL, some silt, trace clay.	214.25 / 5.94	8	50 DO	18	
6	Very stiff, grey CLAYEY SILT, some sand, trace gravel.	213.84 / 6.35	9	50 DO	15	
	END OF BOREHOLE					
7						
8						
9						
10						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp —○W— Wl 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION

WATER LEVEL IN OPEN BOREHOLE AT 2.4m DEPTH DURING DRILLING.

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: PD

CHECKED: JW

PROJECT: 921-1397

LOCATION: SEE FIGURE 2

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 3

BORING DATE: MAR. 19, 1993

SHEET 1 OF 1

DATUM: GEODETIC

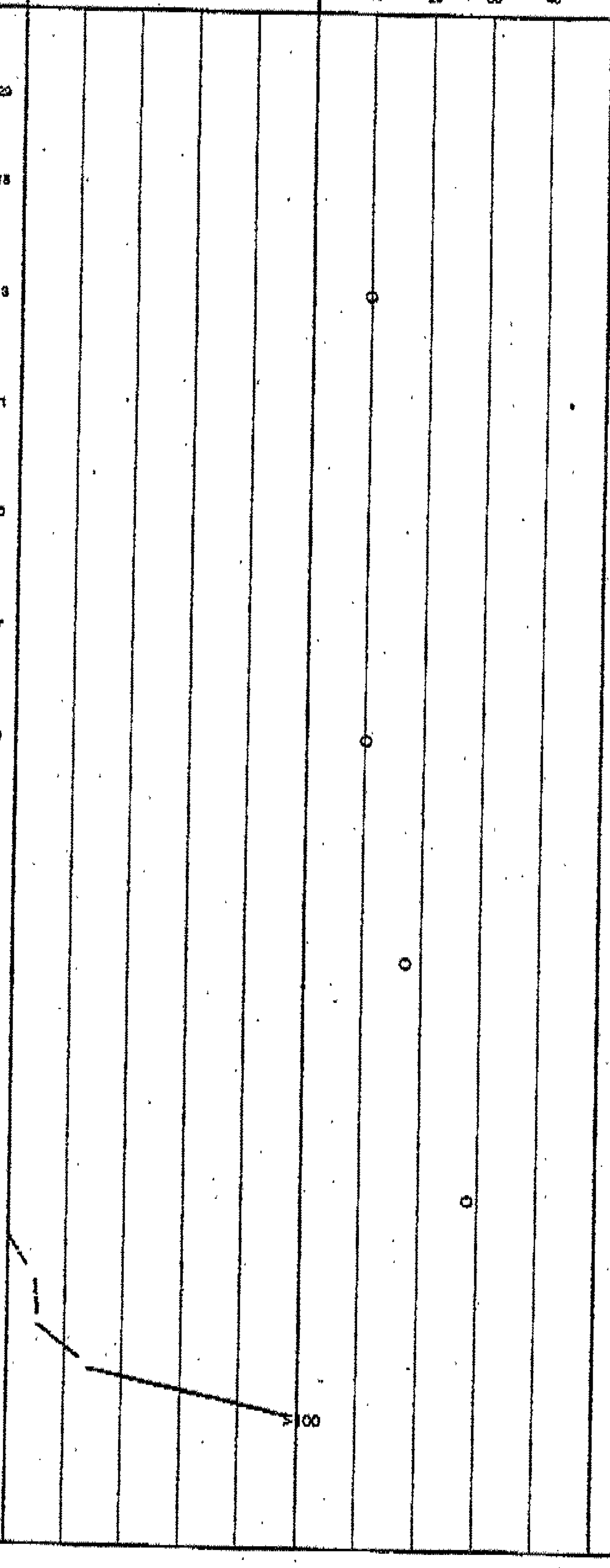
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME-75 POWER AUGER BORING, 160mm SOLID STEM AUGERS

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
ROAD SURFACE	220.41
ASPHALT	220.31 / 0.10
Compact, brown sand, some gravel, trace silt. (ROAD BASE FILL)	219.56 / 0.85
Stiff to very stiff, light brown CLAYEY SILT, some sand, trace to some angular to subangular gravel. Occ. lenses of sandy silt inferred.	218.28 / 2.13
No sample recovery - SANDY GRAVEL lens inferred.	216.90 / 3.51
Stiff to very stiff, light brown, becoming grey at around 4.3m CLAYEY SILT, some sand, trace to some angular to subangular gravel.	215.23 / 5.18
Very stiff/compact, interlayered deposit of CLAYEY SILT, trace sand, SILT and SANDY SILT, trace gravel.	212.94 / 7.47
Firm, grey SILTY CLAY, trace sand, trace gravel.	212.18 / 8.23
END OF BOREHOLE	
REFUSAL TO CONE PENETRATION	210.96 / 9.45

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	20
2	50 DO	18
3	50 DO	13
4	50 DO	21
5	50 DO	10
6	50 DO	27
7	50 DO	10
8	50 DO	16
9	50 DO	17
10	50 DO	13
11	50 DO	6

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80)
SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - O
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT Wp —O W— Wl (10 20 30 40)
ADDITIONAL LAB. TESTING
PIEZOMETER OR STANDPIPE INSTALLATION



WATER LEVEL IN OPEN BOREHOLE AT 2.7m DEPTH DURING DRILLING.

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: PD

CHECKED: JW

DATA INPUT: SK APR 21/93

SAMPLER HAMMER, 63.5kg; DROP, 760mm

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	CME-75 POWER AUGER BORING 100mm SOLID STEM AUGERS	ROAD SURFACE	220.52					
		ASPHALT	220.39 / 0.13					CONCRETE & CASING
		Brown sand, some gravel, trace silt. (ROAD BASE FILL)	219.91 / 0.61					
1		Very stiff to hard, light brown becoming grey at around 3.7m, CLAYEY SILT, some sand, trace to some angular to subangular gravel.		1	50 DO	16		BACKFILL
2				2	50 DO	17		BENTONITE SEAL
3				3	50 DO	20		
				4	50 DO	56		PEA GRAVEL
4				5	50 DO	51		
5				6	50 DO	17	MH	
6				7	50 DO	19		
		END OF BOREHOLE	213.97 / 6.55					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, K, cm/s

WATER CONTENT, PERCENT Wp W Wl 10 20 30 40

BOREHOLE DRY ON COMPLETION OF DRILLING.

WATER LEVEL IN PIEZOMETER AT ELEV. 219.14m ON MAR. 30/93.

DEPTH SCALE 1 : 50

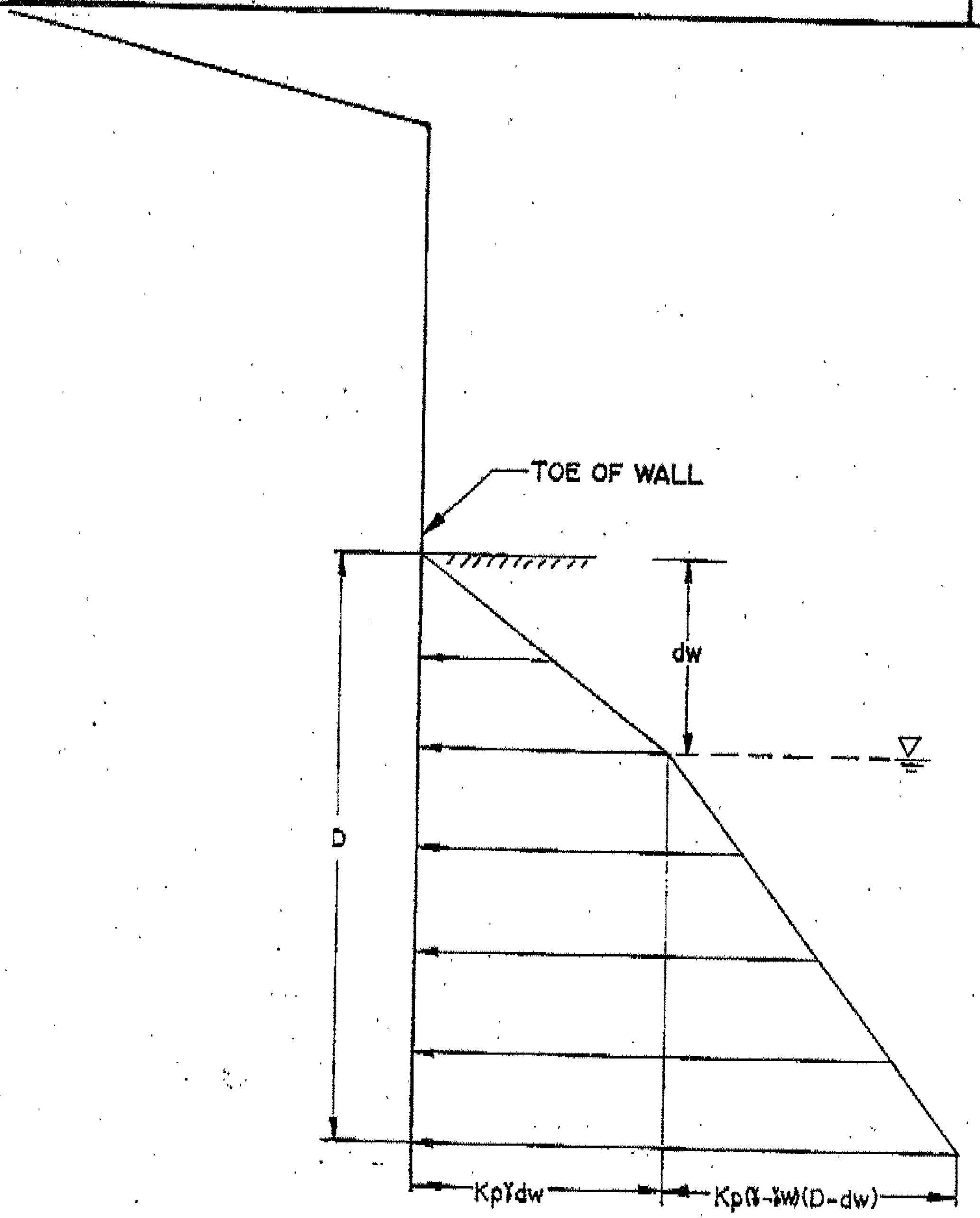
Golder Associates

LOGGED: PD

CHECKED: JW

DESIGN PASSIVE EARTH PRESSURE ACTING ON SOLDIER PILE SOCKET

FIGURE 7



Kp = COEFFICIENT OF PASSIVE EARTH PRESSURE

ɣ = UNIT WEIGHT OF SOIL

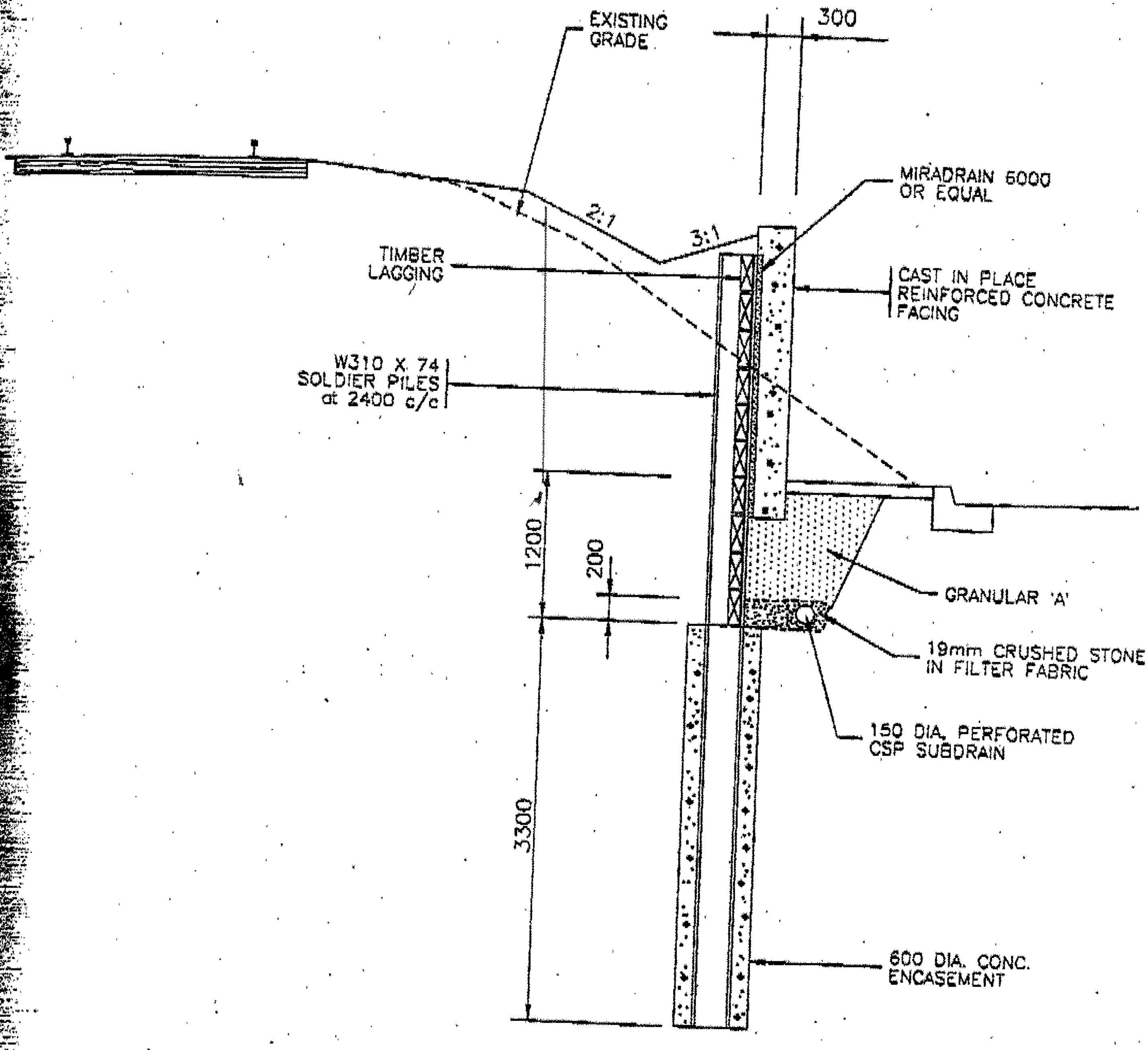
ɣw = UNIT WEIGHT OF WATER

(REFER TO TEXT OF REPORT FOR DESIGN VALUES)

Date APRIL / 1993
Project 921-1397

Golder Associates

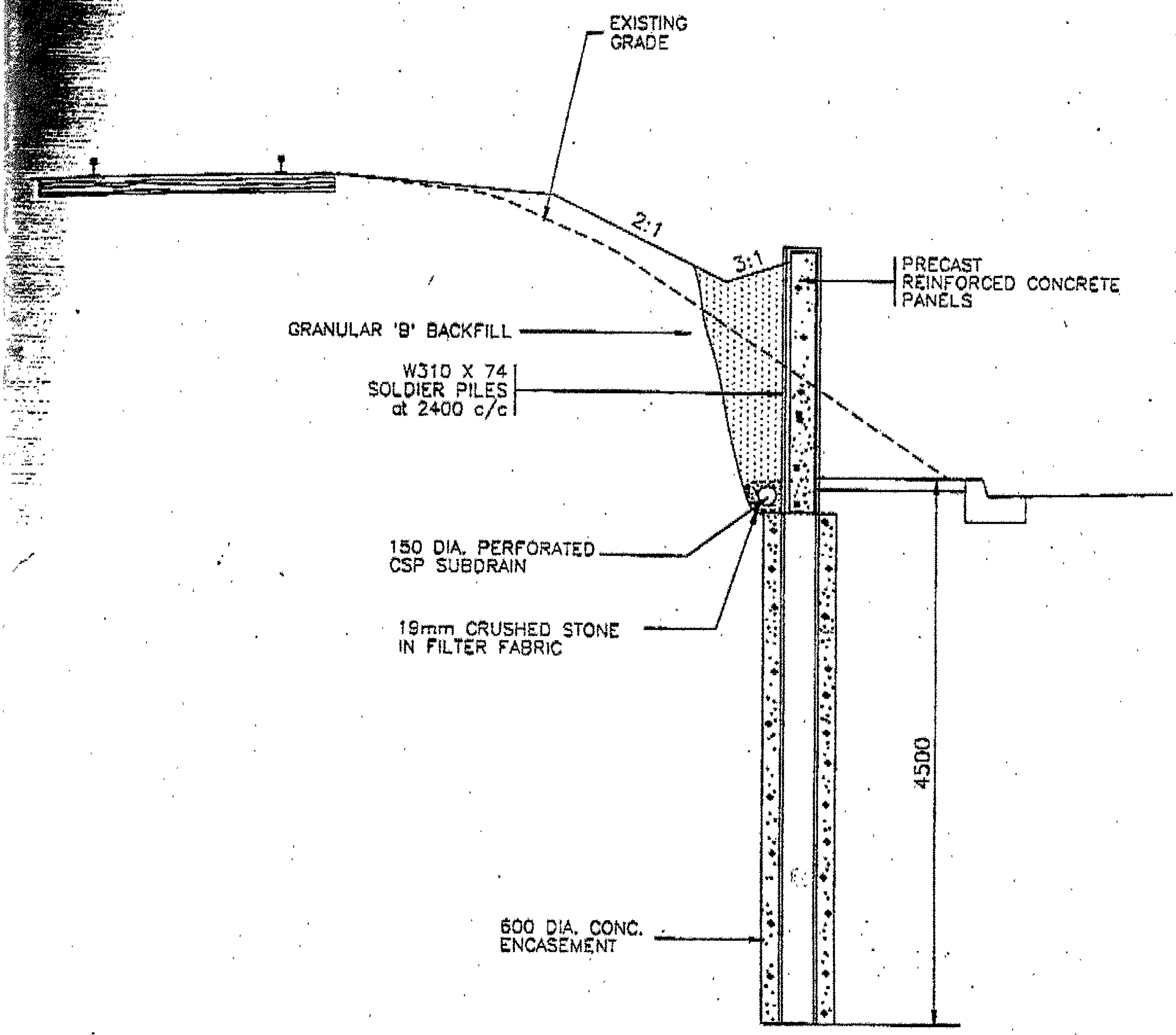
Drawn DV
Chkd. DEB



TYPICAL SECTION

SCALE: 1:50

McCORMICK RANKIN
GOLDER ASSOCIATES
ATHERLEY ROAD RETAINING WALL **CITY OF ORILLIA**
SOLDIER PILE WALL ALTERNATIVE 1   **April 1993**



TYPICAL SECTION

SCALE: 1:50

McCORMICK RANKIN
GOLDER ASSOCIATES
ATHERLEY ROAD RETAINING WALL **CITY OF ORILLIA**
SOLDIER PILE WALL ALTERNATIVE 2 **April 1993**