



memorandum

To: Peter Stuart, P. Eng.
Structural Engineer
Structural Section
Northern Region

1996 08 12

From: Pavements and Foundation Section
Room 315, Central Building
Downsview, Ontario

Re: Preliminary Foundation Recommendations
Proposed Grade Raise, Northwest Bay Crossing
W.P. 89-86-01, Str. Site 43-57
Highway No. 64, District 54, Sudbury

This is further to our meeting of July 23, 1996 that took place in North Bay to discuss the above mentioned project. The meeting was attended by Ken Ahmad, Tae Kim, Peter Stuart, Paul Lecoarer and Mary Young. Different alternatives for the detour alignment, construction methodology and feasibility of proposed grade raise at the existing alignment were discussed. The following outlines the history of the project, rehabilitation proposal and preliminary foundation recommendations for different alternatives:

Existing Highway and Structure:

Based on the information on the file, the initial construction of the bridge at the above site took place in early 1900's. In 1952 the existing wooden structure replaced the original bridge structure. The existing wooden trestle of the bridge is about 10m wide and 63.2m long and is founded on 14 pile bents. The wooden piles were spliced and driven to the bedrock (pile lengths about 15 to 27m). The existing embankment is about 11.3m thick.

Previous investigations revealed that the causeway was built with rock fill and was resting on a blanket of sand and gravel founded on a corduroy pad that was about 4.3m in width. In June 1994 Northern Region carried out a soil investigation (Evan clinche's memo dated March 1, 1995) but did not encounter any rock fill on the north half of the north causeway. On the south half of the north causeway a 2m of sand and gravel was encountered over the rock fill. The boreholes were advanced to a maximum depth of 6m. No corduroy was encountered. Settlement in the causeway has been occurring. Major grade restoration work took place twice in the past 32 years. Rock protection was placed along the side of the causeway in 1976.

Beside the existing road embankment (on the west side) the soil under the lake bed consists of a 2.5m thick very organic silt (peat) overlying a 5.2m to 14m thick very soft to soft normally consolidated silty clay underlain by a 3.4m thick loose sand and gravel. The sand and gravel is in turn underlain by a soft to firm varved clay up to 11.6m thick. The soft to firm varved clay is underlain by a thin layer of silty sand which rests on probable bedrock. Water depth in the bay, at the site is between 2m to 3m.

The existing and proposed grade of the embankment at the existing highway alignment are as follows:

	<u>North Approach</u>	<u>South Approach</u>
Existing elevation of the embankment:	197.6m	197.8m
Approved Elevation:	198.1m (grade raise 0.5m)	198.5m (raise 0.7m)
Alternative 1, Proposed Elevation:	199.6m (grade raise 2.0m)	199.7m (raise 1.9m)
Alternative 2, Proposed Elevation:	198.6m (grade raise 1.0m)	198.7m (raise 0.9m)

Proposed Detour:

The proposed detour will be constructed at an offset of about 20m west from the existing Hwy 64. The proposed length of the detour causeway is 74.0m. However, different alignment options for the detour are under consideration and the length of the detour causeway may vary. The proposed grade elevation at the detour will be 198.4m. The height of the detour causeway will be about 4.5m. The recorded high water level elevation is: 195.71m

RECOMMENDATIONS

For the proposed detour construction, a 4.5m high embankment will be constructed. Before the rock fill is placed a layer of sand and gravel up to 2m thick and 10m wide will be placed throughout the length of the embankment to prevent remoulding of the clay. The rock fill will be placed over the sand and gravel layer. The new embankment will be constructed beside the existing embankment, leaving no gap between the new and the old embankment. The rock fill placement will start right at the west side slope of the existing embankment and will move towards the west. This will ensure that the mud wave will move away from the existing embankment.

Construction should be closely controlled in the field. During the rock fill placement several points (stations) will be set on the existing causeway and monitored for any movement. If any movement is noticed, construction will be stopped and Foundation Section will be informed immediately. The existing causeway shows signs of lateral movement (longitudinal cracks in the pavement). The embankment of the detour will provide lateral support to the existing causeway. The west side slope of the rock fill embankment will be constructed at 1.25H:1V. No berm will be required.

If possible, the embankment of the detour should be left for one year and monitored for settlement. It is expected that immediate settlement up to 2.5m will occur during construction. Consolidation settlement up to 1.07m will be realized over a long period of time. 50 per cent of the settlement will occur in 10 years and 90 per cent in 30 years.

For the existing embankment and replacement structure, the Alternative No 1 which requires 2m raise on the north side and 1.9m on the south side of the bridge is only feasible if one metre of the existing fill is removed and replaced with 3m of Elastizell (Unit weight 5 kN/m³). Elastizell is has been widely used in USA. However, MTO has experience using this material on one project only. The existing one metre material which will be replaced by the lightweight material will be removed from under the road subbase to allow construction of the road way above the Elastizell.

The alternative 2 which requires 1m grade raise on the north side and 0.9m on the south side is possible if 1m of existing fill is replaced with 2m of slag. Both solutions will have no increase in the net embankment load. The existing one metre material which will be replaced by the lightweight material will be removed from under the road subbase to allow construction of the road way above the slag.

For the pile driving at the abutment locations all the rock fill should be removed. However, any rock fill sunk below the river bed will not provide much resistance to the pile driving.

We need to drill at least two boreholes (one borehole at each abutment location) to determine the exact depth to the bedrock and any changes in the soil properties due to long period of consolidation.

Please send a request for the supplementary Foundation Investigation.

A handwritten signature in black ink, appearing to read 'K.S.Q. Ahmad', written in a cursive style.

K.S.Q. Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

cc: Paul Y. Lecoarer - P&D
Mary Young - Environmental Office

749-7384

TO DISCUSS IN THE MEETING ON JULY 23, 1996

W.P. 89-86-01
Hwy 64, Northwest Bay
Lake Nipissing @ Lavigne

Location : on HWY 64, about 15Km South
of HWY 17, Lavigne

FACTUAL INFORMATION

Length of Causeway : 420m
Bridge : 65m

Existing Highway and Structure:

The initial construction of the bridge at the above mentioned site took place in early 1900's. In 1952 the original bridge structure was replaced by the existing wooden structure. Existing wooden trestle of the bridge is about 10m wide and 63.2m long and is founded on 14 pile bents. Wooden piles were spliced and driven to the bedrock (pile lengths about 15 to 27m). The existing embankment is about 11.3m thick

Previous investigations indicated that the causeway was built with rock fill resting on a blanket of sand and gravel founded on a corduroy pad about 4.3m in width. In June 1994 Northern Region carried out a soil investigation (Evan clinche's memo dated March 1, 1995) but did not encounter any rock fill on the north half of the north causeway. On the south half 2m of sand and gravel was encountered over the rock fill. The boreholes were advanced to a maximum depth of 6m. No corduroy was encountered.

Settlement in the causeway has been occurring. Major grade restoration work took place twice in the past 32 years. Rock protection was placed along the side of the causeway in 1976.

Beside the existing road embankment (on the west side) the soil under the lake bed consists of a 2.5m thick very organic silt (peat) overlying a 5.2m to 14m thick very soft to soft normally consolidated silty clay underlain by a 3.4m thick loose sand and gravel. The sand and gravel is in turn underlain by a soft to firm varved clay up to 11.6m thick. The soft to firm varved clay is underlain by a thin layer of silty sand which rest on probable bedrock. Water depth in the bay, at the site is between 2m to 3m.

The existing and proposed grade of the embankment at the existing highway alignment are as follows:

	<u>North Approach</u>	<u>South Approach</u>
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Alternative 1, Proposed Elevation:	199.6m (grade raise 2.0m)	199.7m (raise 1.9m)
Alternative 2, Proposed Elevation:	198.6m (grade raise 1.0m)	198.7m (raise 0.9m)

Proposed Detour:

The length of the proposed structure is: 74.0m.

The proposed grade elevation at the detour is: 198.4m

The recorded high water level elevation is: 195.71m

The height of the fill required for the detour is about 4.5m.

The offset of the detour alignment is about 20m west from the existing Hwy 64.

PROPOSAL FOR CONSTRUCTION

For the proposed detour construction, a 4.5m high embankment will be constructed. Before the rock fill is placed a layer of sand and gravel up to 2m thick and 10m wide will be placed throughout the length of the embankment to prevent remoulding of the clay. The rock fill will be placed over the sand and gravel layer. The new embankment will be constructed beside the existing embankment, leaving no gap between the new and the old embankment. The rock fill placement will start right at the west side slope of the existing embankment and will move towards the west. This will ensure that the mud wave will move away from the existing embankment.

Construction should be closely controlled in the field. During the rock fill placement several points (stations) will be set on the existing causeway and monitored for any lateral movement. If any movement is noticed, construction will be stopped and Foundation Section will be informed immediately. The existing causeway shows signs of lateral movement (longitudinal cracks in the pavement). The embankment of the detour will provide lateral support to the existing causeway. The west side slope of the rock fill embankment will be constructed at 1.25H:1V. No berm will be required.

If possible, the embankment of the detour should be left for one year and monitored for settlement. It is expected that immediate settlement up to 2.5m will occur during construction. Consolidation settlement up to 1.07m will be realized over a long period of time. 50 per cent of the settlement will occur in 10 years and 90 per cent in 30 years.

For the existing embankment and replacement structure, the Alternative No 1 which requires 2m raise on the north side and 1.9m on the south side of the bridge is only feasible if one metre of the existing fill is removed and replaced with 3m of elastizell (Unit weight 5 kN/m³)

The alternative 2 which requires 1m grade raise on the north side and 0.9m on the south side is possible if 1m of existing fill is replaced with 2m of slag. Both solutions will have no increase in the net embankment load.

For the pile driving at the abutment locations all the rock fill should be removed. However, any rock fill sunk below the river bed will not provide much resistance to the pile driving.

We propose to drill at least two boreholes (one borehole at each abutment location) to determine the exact depth to the bedrock and changes in the soil properties due to long period of consolidation. Structural Section should initiate a request for the supplementary investigation.

June 19, 1996

W.P. 89-86-01

Hwy 64. Northwest Bay
Lake Nipissing, @ Lavigne.

Meeting with Tae Kim and Ken Ahmad. at the
Site on June 19, 1996. The following was discussed
and agreed:

For the detour $\pm 4.5m$ fill rock fill will be used.
The method will be end dumping. The dumping will
start from one side (north end or south end) leaving
no gap between the new and old embankment. The
rockfill placement will also start from the
existing embankment westwards. This will ensure any
underwater away from the existing embankment. During
the rockfill placement several points (stakeouts) will
be set on the existing embankment ~~at~~ and monitored
~~with the~~ (curved) to see any lateral movement.
If any movement, foundation sections will be in place.
The embankment of the detour will provide lateral
support to the existing embankment which shows signs
of lateral movement (longitudinal cracks in the
pavement).

The embankment of the abutment will be left for one year if possible and monitored for settlement. It is expected that the rockfill will sink in the peat and clay but due to temporary nature of the structure more expensive proposal is not suitable.

For the existing embankment and replacement structure, the Alternative No 1 which requires 2m raise on the north side and 1.9m on the south side of the bridge is only possible if one metre of existing fill is removed and replaced with 3m of earthfill (5.0 kN/m^3 material).

The Alternative 2 which requires 1m grade raise on the north side and 0.9m on the ^{South} side ~~can be~~ is possible if $\pm 1\text{m}$ of existing fill is replaced with 2m of slag. Both alternatives will have no increase in the net embankment load.

For the pile driving at the abutment locations after it will be recommended to remove all the rockfill. Any rockfill such ^{as} the river bed will not provide much resistance to the pile driving.

we will recommend to drill two boreholes,
one borehole at each abutment location to
determine the exact depth ^{to} an bedrock and
^{changes in} soil property after a long period of consolidation

Ken Almag

Northwest Bay (Lake Nipissing)

begin of proposed structure 74.0m.

Need structure for proposed detour.

Grade elevation 198.4m HWL = 195.71m

for Detour fill required $\pm 4.5m$

Detour $\pm 20m$ offset from existing Hwy 64.

Existing Hwy

Existing elevation of
~~Approved height~~ embankment North +197.6
South +197.8

Approved elevation North = 198.1 raise = 0.5m
South = 198.5 raise = 0.7m

Alternative 1 elev. North = 199.6 raise = 2.0m
South = 199.7 raise = 1.9m

Alternative 2 elev. North = 198.6 raise = 1.0m
South = 198.7 raise = 0.9m

Original Construction more than 70 years ago. In 1952 wooden bridge was replaced.

Existing embankment rockfill constructed on a blanket of sand and gravel founded on corduroy pad about 5.0m in width.

Existing wooden trestle about 10m in width is founded on 14 pile bents.

Wooden piles were spliced and driven to bedrock. (Depth 15 to 27m)

— The Thickness of the existing embankment ± 11.2 m.

— Beside the embankment Soil Compose consists of 2.5m Thick ^{Consolidated} very soft peat, underlying 5.2 m to 14m very soft to soft ~~consolidated~~ ^{consolidated} mucky silty clay underlain by a stratum of loose sand gravel up to 3.4m Thick underlain by soft to firm varved clay up to 11.6m underlain by thin layer of silty sand resting on granitic bedrock.

memorandum



To: J. McDougall, P. Eng. 6
Head, Geotechnical Section
Northern Region

Attention: E. Clinch

From: Pavements and Foundations Section
Room 315, Central Building

Subject: Highway 64
W.P. 89-86-01, Site No. 43-57
Causeway Realignment and
Structure Rehabilitation at Lavigne
District 54, Sudbury


Date: 95 05 23

1. Lightest fill
2. bridge
X, Cost of Lightest fill
① basic
② Hauling
③ deep of
XGout
over
backfill
based
on provided
grade

We refer to your memorandum dated 95 03 01 and the site meeting held on 95 04 25 attended by E. Clinch, P. Stuart, T. Kim and the undersigned. The overall status of the above-noted project and the various design problems and constraints were discussed in the meeting as follows:

1. The background of the project was reviewed. During the route selection process, it was decided that the new structure would be constructed generally at the same location as the existing bridge with only minor shift ($1 \pm m$) to the west. There would also be a minor grade raise of 0.5 to 1.0 m. The construction would be carried out in stages while maintaining traffic along the existing causeway and bridge. Removal of rock fill in the area where new piles are to be driven would also be necessary. No boreholes were advanced along the existing bridge alignment during the previous investigations carried out by the consultants.
2. It was agreed in the meeting that additional investigation is necessary to fully define the subsurface conditions along the existing alignment. It was also considered that the roadway protection work required for the staged excavation and removal of rock fill can be substantial. P. Stuart suggested that a Bailey bridge be built on one side of the bridge resting on supports that are set back from the existing abutments to avoid the rock fill, to carry the traffic, so that the new bridge structure can be constructed in stages.
3. Questions were raised regarding the details of the proposed embankment given in the previous foundation recommendations. It was agreed that the embankment details will be reviewed and revised if necessary, based on the upcoming foundation investigation results.

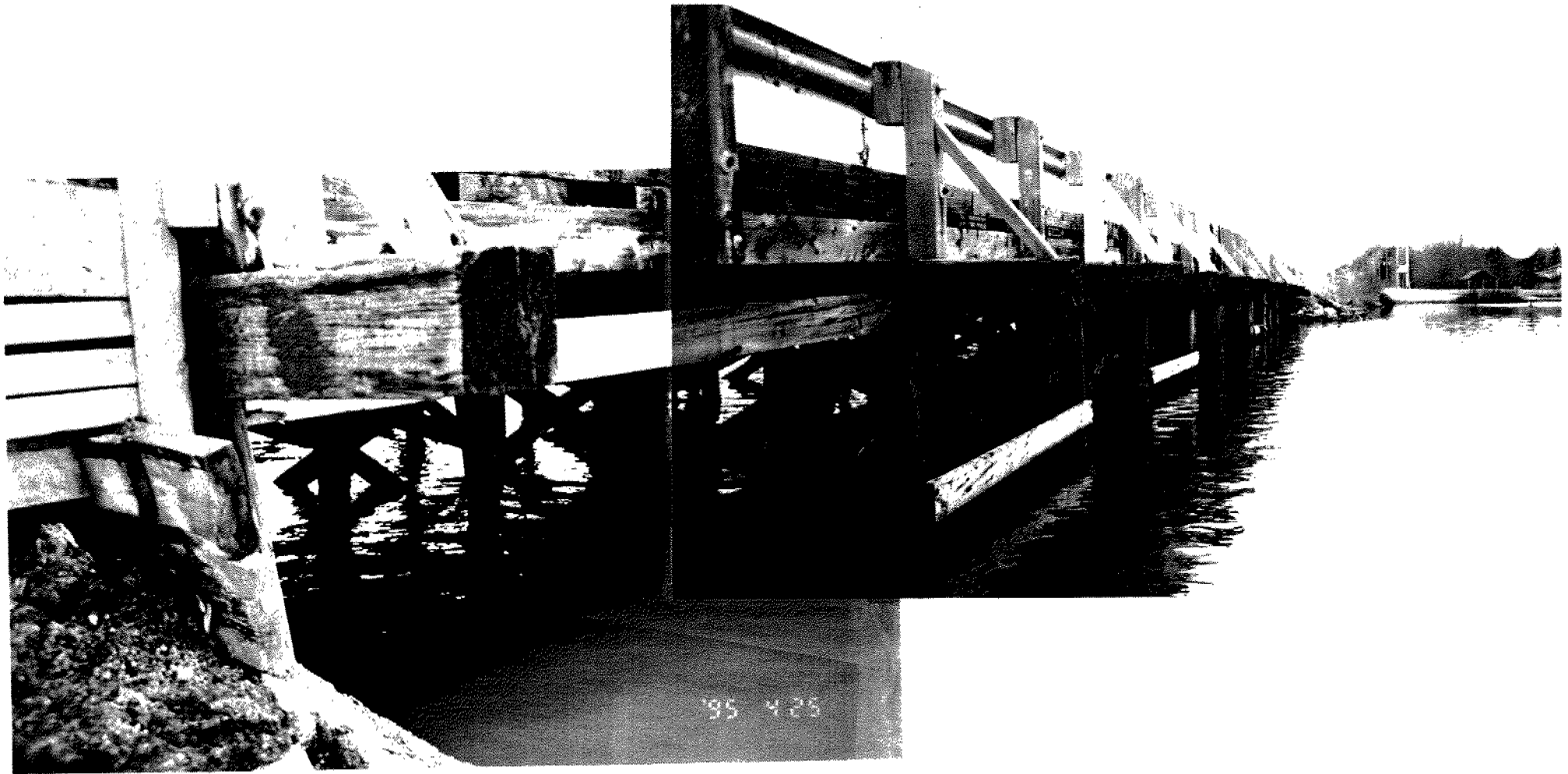
4. Regarding the question on possible restriction on rock blasting on an existing rock cut some 500 m south of the existing bridge for the supply of material for this project, our site observations did not reveal any reason why a blasting restriction is required. We therefore concur with your views that blasting at this location can be safely carried out without causing failure at the causeway, assuming that it will be controlled in the usual manner.
5. We understand that Structural Section will initiate the request for the supplementary investigation and provide us with tentative footing element locations so that the boreholes can be properly located to suit. In view of the number of projects anticipated from the region this summer, an early action by Structural Section would be very much appreciated.



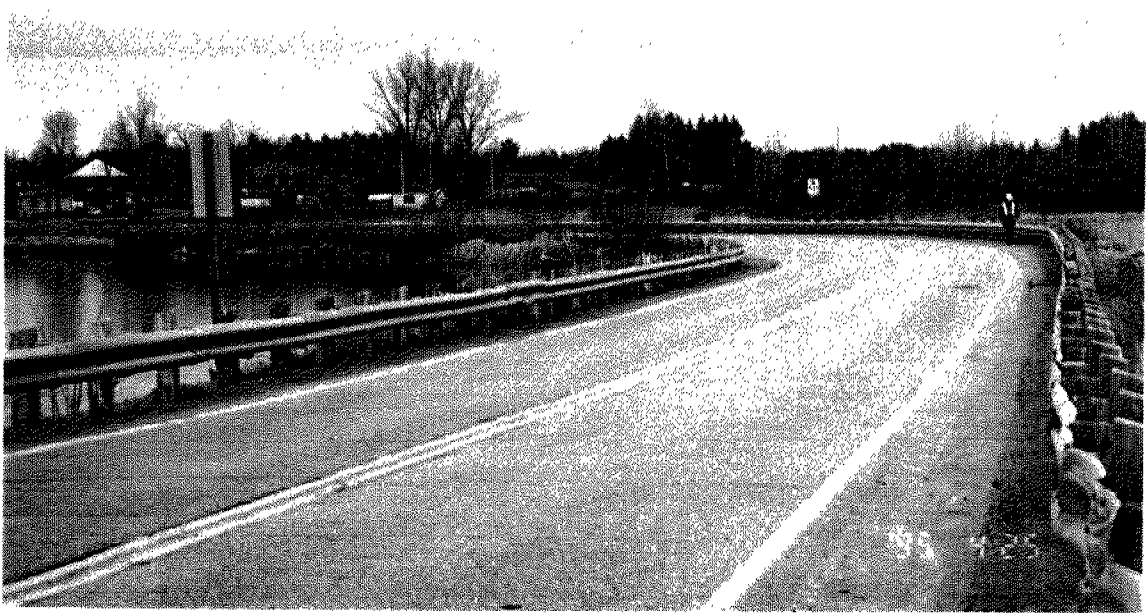
D. Kwok, P. Eng.
Project Foundation Engineer
for
T. Kim, P. Eng.
Senior Foundation Engineer

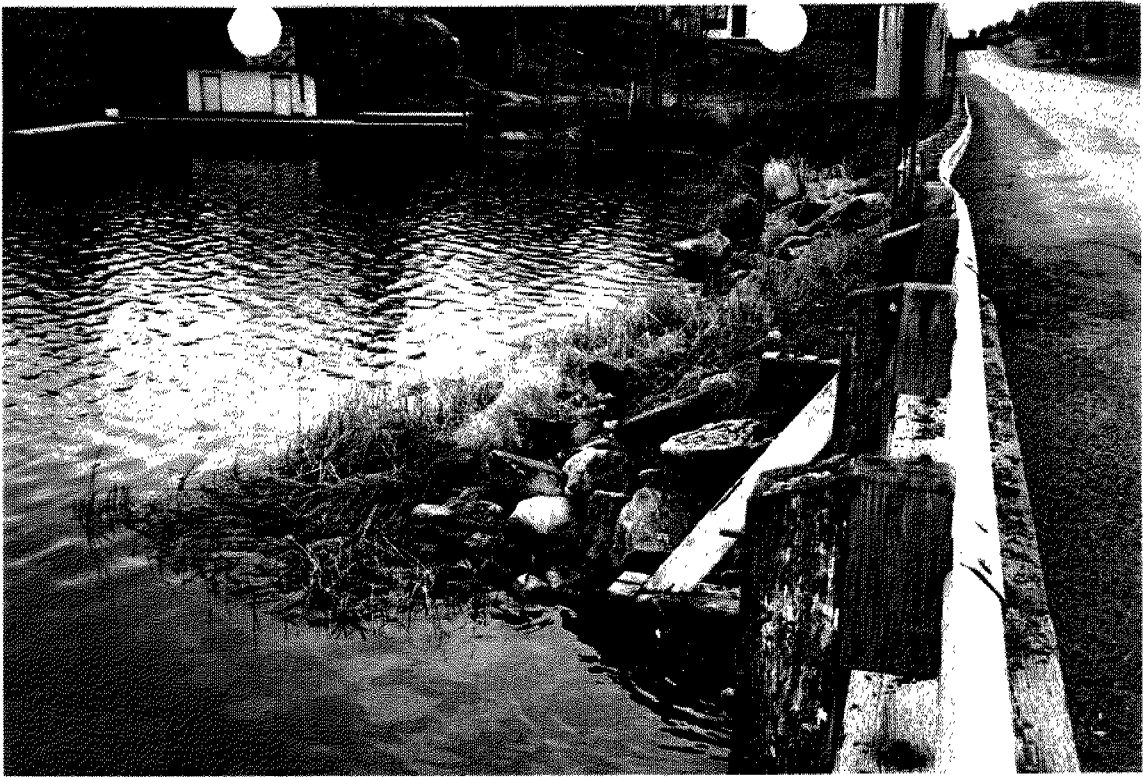
c.c. P. Stuart



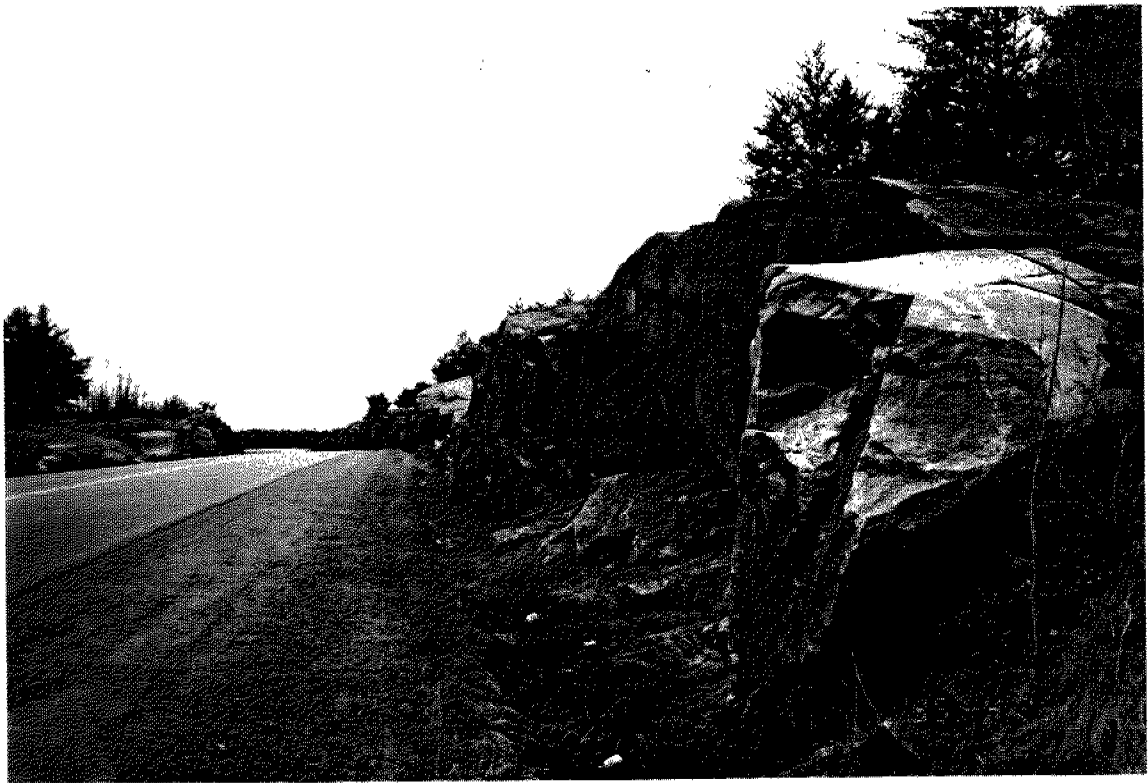
















memorandum



To: T. Kazmierowski, Manager
of the Pavements and
Foundations Section

Date: 1 March 1995

Attention: T. Kim, Senior
Foundation Engineer

From: Geotechnical Section,
Northern Region

Phone: 1-705-497-5478

Subject:

WP 89-86-01
Highway 64, Causeway Realignment and
Structure Rehabilitation at Lavigne
Site 43-57
District 54, Sudbury

On March 3rd, 1991, Dr. B. Iyer issued a memo. report which contained recommendations for the construction of a new structure and approaches across the Northwest Bay of Lake Nipissing at Lavigne. The field investigation was carried out by B.P. Walker Associates Ltd. No boreholes were placed in the existing causeway during that investigation.

In Dr. Iyer's memo., he states that: 'Based on previous investigations carried out at the site, it is considered that the highway embankment consists of rock fill resting on a blanket of sand and gravel founded on a corduroy pad about 4.3 m in width'. The recommendations for the new construction work on the causeway, given in that memo., require a 2.0 m, minimum, thick sand layer placed on the lake bed from the west side of the existing road, to the west over a width of 10.0 m. This sand layer is to be overlain by rock fill. Removal of the rock fill material in the area where the new piles are to be driven is also recommended.

The assumed composition of the existing causeway that is outlined in Dr. Iyer's memo. seems to be based on an investigation carried out by Geocon Ltd. in 1958. In June of 1994, we carried out a Soils Investigation at this site. Our borings show that on the south half of the causeway there is 2.0 m \pm of sand and gravel over rock fill. On the north half of the causeway no rock fill was encountered. Some of the test holes were advanced to a depth of 6.0 m. No corduroy was encountered at these depths. A copy of our Log of Boreholes is attached for your information.

Major grade restoration work to correct settlement of the causeway has been carried out twice in the past 32 years. Once under Contract 63-162 and again under Contract 76-126. Granular grade raises were placed under both contracts. Rock protection was placed along the sides of the causeway in 1976.

Before placing the granular layer that has been recommended by Dr. Iyer, the existing rock protection along the west side of the road will have to be removed. To construct the 10.0 m wide granular layer on the existing lake bed, it is likely that the material will be placed slightly above the water level at the site so that it can be pushed into place. The water depth at the site is between 2.0 m and 3.0 m according to the Project Appraisal Report.

After the granular layer has been placed, there will be very little space to place rock fill as is recommended. Normally rock fill is not placed in a fill that is less than 1.2 m high. This minimum fill height has been established to ensure that the rock fill can be properly placed and chinked. Can granular material be substituted for the rock fill at this site? Granular 'B' Type II will be used on this project. If granular material can be used, what slope should it be placed to? Rock Protection will probably be needed to protect the granular slope.

Traffic is to be maintained on the existing causeway and bridge during the new construction. Excavation of the existing rock fill material at the bridge site has been recommended to facilitate the driving of the new piles. There is approximately 2.0 m of granular material over the rock fill at the site. The rock fill will be encountered at or below the water level of the lake. The depth of the rock fill layer is not known. The amount of roadway protection work that will be needed at this site is probably much greater than has been assumed based on old borehole information.

Another issue that has been raised in the past concerns blasting. It is proposed that rock for this project be obtained by widening an existing rock cut approximately 500 m south of the bridge. Is blasting in this cut area likely to cause a failure at the causeway? A commercial quarry is located about 500 m west of Highway 64, and 400 m south of the bridge. This quarry was last used in 1976, and will be listed as a source of material for this project. Past history at the quarry, and at the existing 4.0 m \pm high rock cut suggests that blasting in the cut area south of the structure can be safely be carried out, but we feel that your section should comment on this operation.

Thank you for your attention in these matters.



Evan Clinch
Pavement Design and
Evaluation Officer

cc: P. Furst
Head, P & D
File

**WP 89-86-00, HIGHWAY 64,
NORTHWEST BAY BRIDGE,
TOWNSHIP OF MacPHERSON.**

21+450 4.90 Rt C/L

0	-	060	Asph
060	-	340	Cr Gr
340	-	1.10	F-M Sa W Gr
1.10	-	1.60	F-M Sa W Gr & Si Wet
		1.60	NFP BR

21+500 2.80 Lt C/L

0	-	050	Asph
050	-	450	Cr Gr
450	-	1.40	F-M Sa W Gr
		1.40	NFP RF
			10% Extra Material Needed To Fill Hole

21+500 5.00 Rt C/L

0	-	060	Asph
060	-	300	Cr Gr
300	-	800	F-M Sa W Gr
		800	NFP RF

21+550 3.80 Lt C/L

0	-	100	Asph
100	-	740	Cr Gr
740	-	2.30	F-M Sa W Gr
		2.30	NFP RF

21+550 5.20 Rt C/L

0	-	150	Asph
150	-	750	Cr Gr
750	-	2.30	F-M Sa W Gr
			Fr Wat @ 1.30
		2.30	NFP RF
			30% Extra Material Needed To Fill Hole

21+650 3.20 Lt C/L

0	-	080	Asph
080	-	470	Cr Gr
470	-	2.20	F-M Sa Tr Gr
			Fr Wat @ 1.30
		2.20	NFP RF

21+650 4.90 Rt C/L

0	-	060	Asph
060	-	650	Cr Gr
650	-	2.10	F-M Sa W Gr
			Wet @ 1.20
		2.10	NFP RF

21+700 2.70 Lt C/L

0	-	050	Asph
050	-	870	Cr Gr
			Fr Wat @ 900
870	-	1.80	F-M Sa Tr Gr
		1.80	NFP RF

21+700 4.90 Rt C/L

0	-	070	Asph
070	-	800	Cr Gr
800	-	1.80	F-M Sa W Gr
			Fr Wat @ 800
		1.80	NFP RF

21+750 4.90 Rt C/L

0	-	070	Asph
070	-	820	Cr Gr
820	-	1.80	F-M Sa W Gr
			Wet @ 1.30
		1.80	NFP RF

**WP 89-86-00, HIGHWAY 64,
NORTHWEST BAY BRIDGE,
TOWNSHIP OF MacPHERSON.**

21+750 3.20 Lt C/L

0	-	050	Asph
050	-	570	Cr Gr
570	-	2.00	F-M Sa W Gr
			Occ Cob
		2.00	NFP RF

21+800 3.70 Lt C/L

0	-	070	Asph
070	-	470	Cr Gr
			Wat Seepage @ 300
470	-	3.00	F-M Sa Tr Gr & Si
3.00	-	6.00	Si W Cl &
			Org Mix Wet Firm

21+800 4.40 Rt C/L

0	-	070	Asph
070	-	720	Cr Gr
720	-	1.80	F-M Sa W Gr
1.80	-	4.00	F-M Sa W Gr
			Tr Org Wet Firm

21+850 3.80 Rt C/L

0	-	070	Asph
070	-	580	Cr Gr
580	-	800	F-M Sa W Gr
800	-	3.00	F-M Sa W Gr
			Si & Org Wet
			Fr Wat @ 1.10

21+850 3.80 Lt C/L

0	-	050	Asph
050	-	270	Cr Gr
270	-	570	M Sa W Gr
570	-	6.00	F-M Sa Tr Gr
			& Si Wet Firm

21+900 3.80 Lt C/L

0	-	050	Asph
050	-	640	Cr Gr
640	-	1.50	F-M Sa W Gr
1.50	-	6.00	F-M Sa W Gr Si
			& Org Mix Wet Firm

21+900 3.80 Rt C/L

0	-	050	Asph
050	-	430	Cr Gr
430	-	1.00	F-M Sa W Gr
		1.00	NFP RF

21+925 3.50 Lt C/L

0	-	050	Asph
050	-	300	Cr Gr
300	-	1.60	F-M Sa W Gr
			Fr Wat @ 1.10
		1.60	NFP RF

21+925 3.80 Rt C/L

0	-	050	Asph
050	-	400	Cr Gr
400	-	1.00	F-M Sa W Gr
1.00	-	1.30	Blk Org
			Wat Seepage @ 1.00
1.30	-	6.00	Si W Cl Wet, Firm
			Soft @ 3.50

21+942 3.80 Rt C/L

0	-	070	Asph
070	-	250	Cr Gr
250	-	700	F-M Sa W Gr
700	-	1.00	Si W Cl Moist Firm
1.00	-	1.20	Cord
			Wat Seepage @ 1.00
1.20	-	5.10	Si W Cl Wet Firm
		5.10	NFP BR

memorandum



To: M. Devata
Chief Foundation Engineer
3 rd Floor, Central Building
1201 Wilson Avenue
Downsview, Ontario

Date: 91 06 20

Phone 1-705-497-5478

Ext. 6322

From: Geotechnical Section
Northern Region

Re: W.P. 89-86-00
Northwest Bay Bridge at Lavigne (Site 43-057)
District 13, North Bay

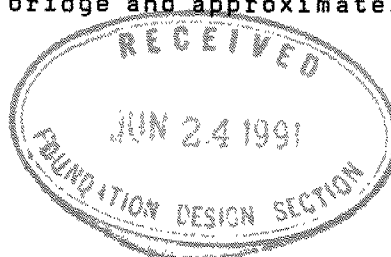
As part of the bridge construction work the existing causeway will be widened at this location. Rock fill/protection will be required for this work and in order to generate this rock the Planning and Design Section wish to extend the South Limit of this project by approximately 300 m to include an existing 4.0 m high rock cut area. It is there intention to widen this rock cut to provide rock borrow for the bridge approach work.

In the past there has been some concern about allowing blasting to take place near this location for fear that vibrations set up by the blast could cause a base failure at the causeway site.

Construction work was carried out in this area in 1976/77, under Contract 76-126. A rockcut operation was carried out at that time at the north end of the causeway at Station 21+970 \pm . The existing bridge is at Station 21+600 \pm and the north end of the causeway is at Station 21+925 \pm . A no blasting restriction was put into Contract 76-126 applied to the rock cut at Station 21+970 \pm .

Mr. H. Pattenden was the Project Supervisor on Contract 76-126. He stated that blasting of the rock was allowed after the start construction. An attempt was made to chip the rock out, but progress was very slow. Blasting at this location was approved by the Ministry and monitored by a Consultant hired by the Contractor. Possibly you have some record of the blast monitoring at this site.

As well as the blasting that was carried out in the cut at the north end of the causeway a rock quarry operation was used to supply materials for Contract 76-126. This quarry is located on private land 400 m south of the bridge and approximately 500 m west of Highway 64.



Past History suggests that blasting in the cut area south of the bridge can be safely carried out, but we feel that your section is better able to make a recommendation regarding this operation.

Please advise us on this issued as to:

- a) Can blasting safely be carried out in the rock cut area between Station 20+800 and Station 21+150?
- b) If blasting is allowed what restrictions, if any, will be required in controlling the size of the charge used?

We will pass on your recommendation to the Planning and Design Section so that they can set the project limits and if necessary find an alternate source of rock borrow. I assume that if blasting is not allowed on the roadway cut then the quarry near this site is also not to be used to provide materials for this project.

There is also the issue of the transportation of the rock from the south end of the project, across the Bailey Bridge that will carry the traffic during construction, to place it on the north bridge approach. Load restrictions on the Bailey bridge will be requested from the Structural Section.

Thank you for your attention in this matter.



Evan Clinch
Pavement Design
and Evaluation
Officer

EC/wa

cc: D. Armatage
G. Todd
File (2)

memorandum



To: P. Furst
Head, Structural Section
Northern Region

Attn: P. Stuart

From: Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Causeway Realignment and
Structure Rehabilitation at
Lavigne, Hwy. 64
W.P. 89-86-01, Site 43-57
District 13, North Bay

Date: 1991 03 26

This memo accompanies the report on a recent foundation investigation carried out at the subject site by our Consultant, B.P. Walker Associates Ltd. Only factual data obtained during the above investigation are included in their report. Interpretative comments regarding the proposed construction at this site are discussed in this memo.

BACKGROUND

In addition to the B.P. Walker Associates Ltd. report, the following documents were reviewed in the preparation of this memo. Reference should be made to these documents for a detailed discussion of the items discussed below.

1. "Soil Investigation and Engineering Study, Proposed Realignment - Highway 64, Lavigne, Ontario" Geocon Ltd. report dated January 15, 1959.
2. "Project Appraisal Report - Causeway Realignment and Structure Rehabilitation at Lavigne, Highway 64". Report prepared by the Planning and Design Section, Northern Region dated April, 1990.

The initial construction of Hwy. 64 across the Northwest Bay of Lake Nipissing at Lavigne was carried out in the early 1900's. In 1952 the original bridge structure was replaced by the existing wooden structure.

The present structure consists of rockfill approach embankments from both banks and a 63.2 m long by a 10 m wide timber bridge structure. The bridge structure is founded on timber pile bents. During the construction of approach embankments adjacent to the bridge, horizontal movements of several pile bents took place. To prevent further movement of the piles, rockfill was placed between the pile bents and the approach embankments.

Based on previous investigations carried out at the site, it is considered that the highway embankment consists of rockfill resting on a blanket of sand and gravel founded on a corduroy pad about 4.3 m (14 ft.) in width.

.../2

PROPOSED CONSTRUCTION

Several options were considered for the upgrading of the present structure. These are discussed in detail in the Project Appraisal Report, referred to in the previous section. Based on a detailed office study, it was concluded that Alternative 1 discussed in the above report was selected. The construction aspects are as follows:

- The new structure would be constructed at roughly the same site as the existing one.
- The horizontal alignment would be shifted 1 m to the west.
- The grade will be raised 0.5 m to 1.0 m throughout the length of the structure, including 200 m of the causeway.
- The construction is to be carried out in stages, while maintaining traffic across Lake Nipissing, preferably along existing causeway and bridge structure.

SUMMARIZED SUBSURFACE CONDITIONS

The subsurface soil conditions at the site consist of a thin deposit of organic silt underlain by a stratum of soft to firm silty clay, becoming stiff from 15 to 20 m depth below the lake level. The bottom of the silty clay deposit extends to depths of about 12 to 24 m below the surficial organic layer. Embedded within the silty clay deposit were a clayey silt layer and one or two layers of a silty sand deposit. The silty clay stratum is underlain by a layer of silty sand and gravel which in turn rests on bedrock. Even though not investigated during this investigation, the composition of the highway embankment fill is considered to consist of rockfill overlying a sand and gravel bed which in turn is underlain by a corduroy pad.

The main soil strata which would influence the proposed construction at the site are the existing rockfill and the silty clay deposit.

The variation of undrained shear strength versus elevation (plotted as depth below water level) is shown on Figure 1. The vane shear strength values obtained during the recent investigation are shown on this figure. Also included in the figure is the range of shear strength values obtained from the previous investigation included in the Geocon Report. No well defined trend could be seen in the change in shear strength over the last 30 years or so. Some increase in shear strength appears to have occurred in the area of boreholes 5 and 7, whereas in boreholes 1, 3 and 8, the average shear strength at shallow depths remained practically the same.

The consolidation tests carried out during the present investigation indicated that the silty clay at shallow depths is normally consolidated with respect to the existing effective overburden pressure.

RECOMMENDATIONS

Recommendations are given in this section regarding the design and construction of the proposed embankment and the rehabilitation of the bridge structure.

Proposed Embankment

As mentioned earlier in this memo, it is understood that the centre line of the highway embankment would be a maximum of 5 m above the lake bed and that it would be shifted 1 m to the west of the present location. The proposed construction details are shown on the attached sketch. (See Figure 2)

The details and sequence of construction and expected differential settlement between the new and old portions of the embankment etc. would be discussed in the proposed discussion meeting on this project.

Proposed Bridge Structure

It is recommended that the new bridge structure should be supported on steel H piles, driven to the bedrock surface. To facilitate driving of the piles through the silty sand and gravel layer encountered immediately above the bedrock surface, the piles should be provided with the standard MTO tip reinforcement. From consideration of the expected lengths of piles and the properties of the silty clay strata encountered at the site, it is recommended that the piles should be at least 310 X 110 size. Since the bedrock surface in the area of construction is quite variable, and the thickness of the silty sand layer overlying bedrock is very thin, it is recommended that the piles should be equipped with rock points, in addition to the standard MTO tip reinforcement.

In the area where pile driving is to be carried out, any existing rockfill should be excavated and replaced by sand and gravel fill. The pile installation should be deferred until after the approach embankments have been constructed, at least in the vicinity of the abutments. This way, structural damage to the piles could be avoided during fill placement.

The piles would be subjected to some downdrag forces due to the settlement of the silty clay layer. For design purposes, it is recommended that a factored ULS and SLS capacities of 1350 and 980 kN be used.

Rock point in addition to steel MTO tip reinforcement.

Rock point

CLOSURE

The foundation investigation at the site was carried out by B.P. Walker Associates Ltd. Several meetings were held between the consultant and our Section during the field investigation phase and the preparation of their report. Our comments have been incorporated in their report, copies of which are attached to this memo.

Interpretative comments regarding the construction of the highway embankment and rehabilitation of the existing bridge are given in this memo. Please contact this office if you require further elaboration on any aspect of this memo or the attached report by B.P. Walker Associates Ltd.

This memo was prepared by Dr. Balu Iyer, P. Eng. and reviewed by M.S. Devata, P. Eng.



Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

for

M.S. Devata, P. Eng.
Chief Foundation Engineer

MSD/BI/jb

cc: J. McDougall
G. Todd
S. Wilson (2)
K.G. Bassi
S.J. Dunham
E.A. Joseph

UNDRAINED SHEAR STRENGTH vs ELEVATION

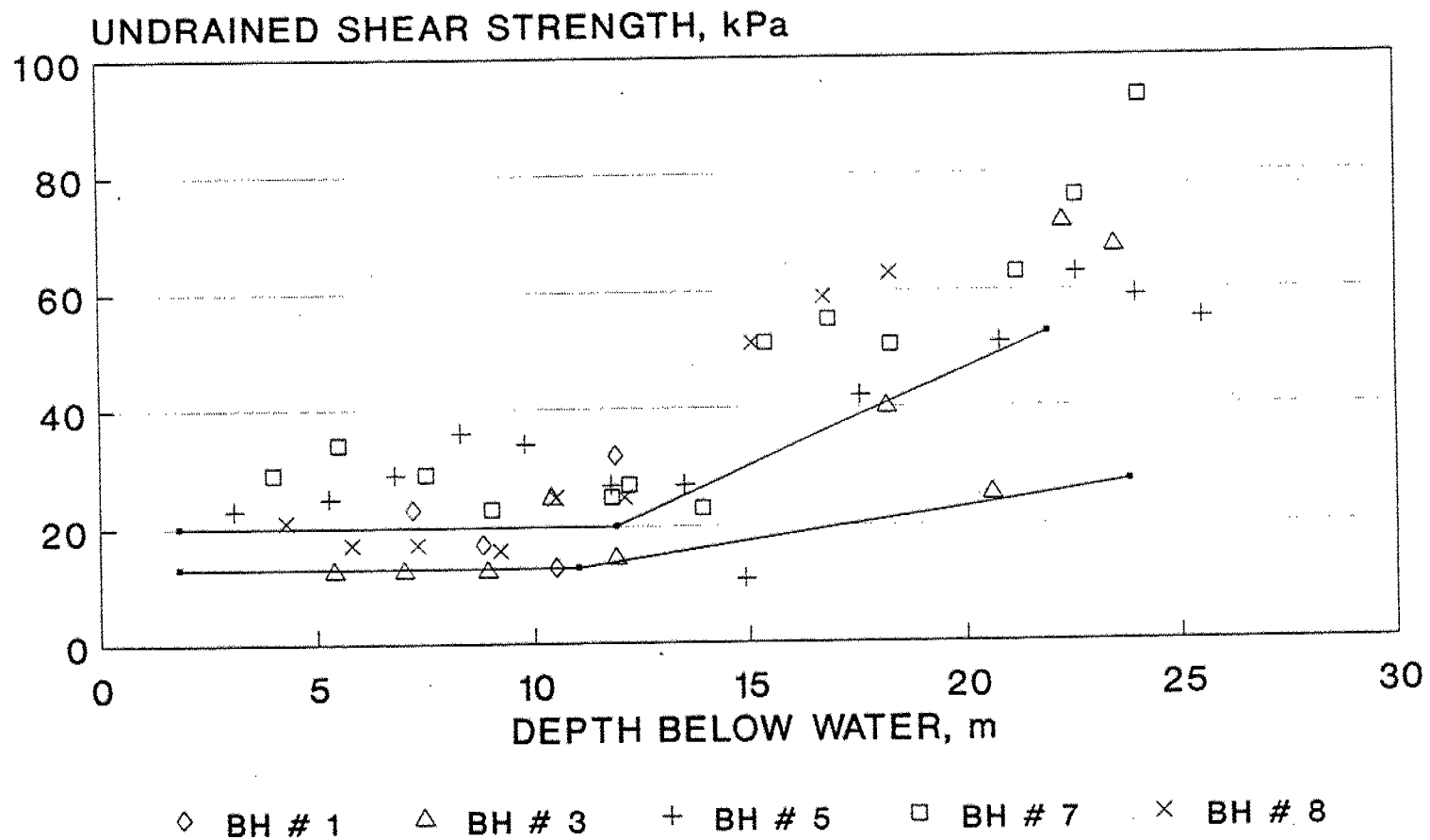
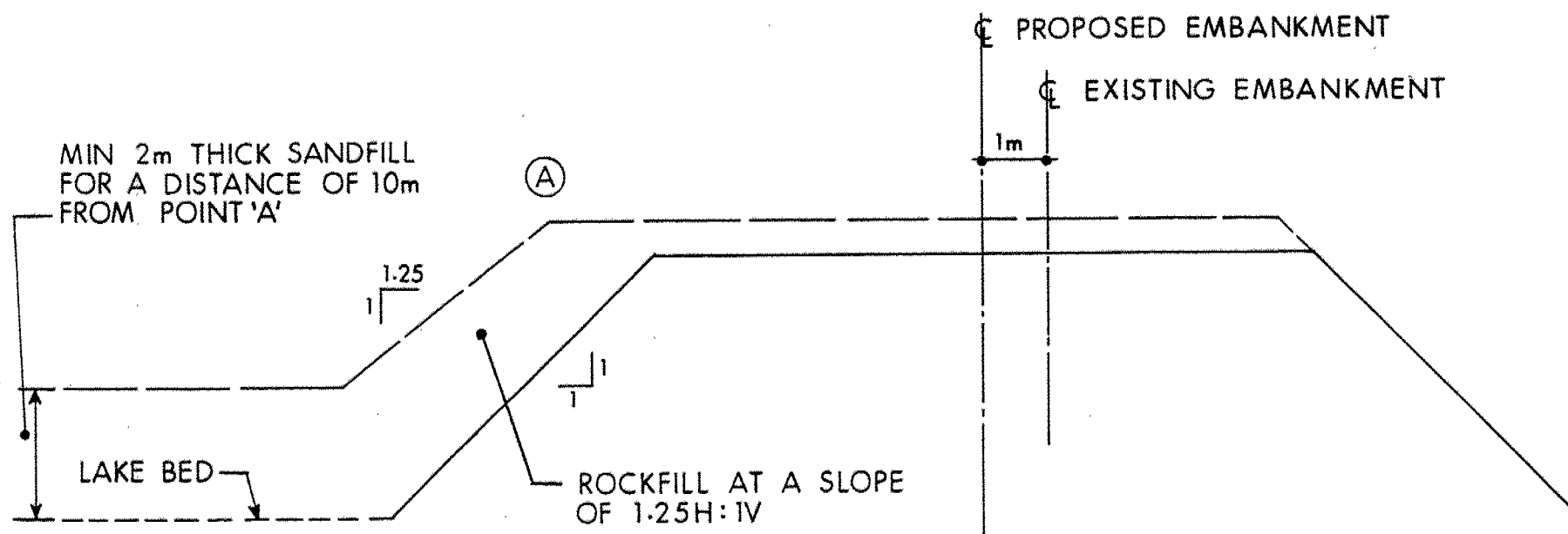


FIG -1
WP 89-86-01



PROPOSED EMBANKMENT WIDENING
TYPICAL DETAIL

memorandum



To: G. Todd, Head,
Planning and Design Section,
Northern Region.

Date: 1989 11 17

From: Structural Section.

(705) 472-7900
Extension 330

RE: WEST BAY OF LAKE NIPISSING AT LAVIGNE
W.P. 86-86-01 SITE 43-57
HIGHWAY 64

A preliminary report evaluating options for a new bridge at this site was recently received from the Foundation Section. A copy is attached for your use.

A summary of the options, selected foundation information and my comments follows:

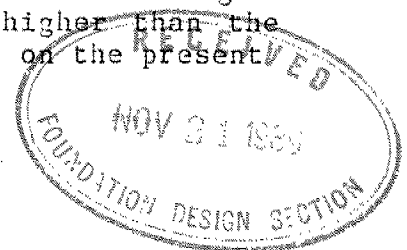
Under option one the grade is raised 0.5 meters and the new centerline shifted one meter from the existing. The new structure would be built half at a time.

Settlement is estimated as 0.15 meters at the structure and 0.35 meters some 15 meters from the structure where the grade raise increases to one meter. This scheme appears acceptable as a reasonable ride would result with the predicted settlement.

Option two considers a grade raise of 1.7 meters with the centerline and staging the same as option one.

Settlement is estimated as much higher than option one. No comment is made on embankment stability or the possibility of berms being required. The higher settlement predicted would probably result in an unacceptable bump at the bridge. Preloading is not possible because of the need to maintain traffic on the existing bridge; and because of the need to stage construction. Moving the abutments back to a point where the fill heights would be lower is impractical because it would require driving piles through the existing rockfill embankment for abutment and pier support.

In option three a new alignment to the north of the existing is considered. The grade is set 0.5 meters higher than the existing. This would allow two way traffic on the present alignment during construction.



Settlement is estimated as up to five meters. It is recommended that a 1.5 meter thick granular mat should be placed on the lake bottom to minimize stability problems. In a telephone discussion with Dr. Iyer of the Foundation Section he indicated that the five meters represents four meters of displacement and settlement that would occur during construction and approximately 1.5 meters of long term settlement. This long term settlement could be reduced by pre-building and surcharging the approach embankments. To reduce the settlement to an acceptable amount it would probably be necessary to lengthen the structure so that the maximum fill height was reduced. When compared with option one the advantages of this scheme (avoiding stage construction) are outweighed by the disadvantages (delay and higher cost).

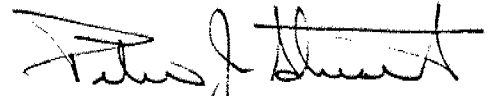
Option four follows the same alignment as option three but the grade is 1.7 meters higher than the existing road.

No comment on this option was made in the Foundation Report. It can however be inferred that very large settlements or displacements would take place. Dr. Iyer estimated that displacement and settlement during construction would be four or five meters and that long term settlement would be approximately two additional meters. I believe this option is only viable if the abutments are moved back to a place where the fill heights are much lower. Even then it would be necessary to prebuild the embankments and allow them to settle. This would result in a significant time delay and a long expensive structure. However, if a clearance significantly higher than the existing was considered essential, this scheme could be pursued further.

In discussions of the options following the existing embankment this Foundation Report refers to an existing five meter road surface and mentions that higher settlements could be expected under widened portions of the embankment. This results from the fact that this report is based on a report prepared by Geocon Ltd. in 1958 which describes the approaches having a 16 foot road surface and one to one side slopes. In fact the approach embankments were raised and widened; and the side slopes flattened in the early sixties. Following settlement at the north approach the grade was padded to a maximum of .35 meters in the mid seventies. As a result, future widening at the existing embankment will be less significant than what is envisioned in this Foundation Report.

- 3 -

It is hoped that this summary will aid in deciding what options should be pursued further.

A handwritten signature in dark ink, appearing to read 'Peter J. Stuart', with a long horizontal stroke extending to the right.

Peter J. Stuart,
Structural Engineer,
Structural Section,
Northern Region.

PJS/jm

c.c.: K. Bassi
B. Iyer
J.I. McDougall
B. Roberts
D. Armatage