EVALUATION OF EXISTING
MAIN HOWSON DAM
BRIDGE STRUCTURE
OVER MAITLAND RIVER ON WATER STREET
IN THE TOWN OF WINGHAM

"Office Copy" BR-476

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B. M. ROSS AND ASSOCIATES LIMITED
CONSULTING ENGINEERS

62 NORTH STREET
GODERICH, ONTARIO
TELEPHONE (519) 524-2641
N7A 2T4

B.M. ROSS, P.ENG.,
K.G.DUNN, P.ENG.
S.D. BURNS, P.ENG.
B.W. POTTER P.ENG.

OUR FILE NO. BR-476

December 14, 1984

Mr. Byron Adams
Clerk
Town of Wingham
Box 90
WINGHAM, Ontario
N0G 2W0

Dear Sir:

Re - Evaluation of Existing Main Howson Dam and Bridge Structure Over Maitland River on Water Street

In accordance with Council's authorization, we are pleased to submit the following Report on the evaluation of the Main Howson Dam and Bridge Structure.

We thank you for the opportunity to be of service on this project, and we would be pleased to discuss any aspect of the Report at your convenience.

Yours very truly,

B. M. ROSS AND ASSOCIATES LIMITED

KGD:jj

Per ______________________
K. G. Dunn, P. Eng.

TOWN OF WINGHAM

REPORT ON EVALUATION

OF EXISTING MAIN HOWSON DAM AND BRIDGE

OVER MAITLAND RIVER ON WATER STREET

INTRODUCTION

The purpose of the requested evaluation and of this Report is to examine the existing Main Howson Dam and Bridge Structure on Water Street in the Town of Wingham and to provide an assessment of the condition of the existing structure, and an evaluation of the bridge super-structure load carrying capacity.

GENERAL SITE CONDITIONS AND BACKGROUND INFORMATION

The Main Howson Dam and Bridge Structure is constructed on Water Street over the Maitland River immediately downstream from the Hanna Bridge on Josephine Street or the Highway No. 4 Connecting Link. The Water Street approaches run in a northerly-southerly direction with the main structure constructed near the southerly part of the Maitland River channel. We understand that the main structure was constructed in the early 1920's. Sometime in the late 1940's or early 50's, a steel sheet piling cut-off wall was driven along the downstream edge of the concrete dam spillway apron. The concrete headwall railings along the side of the roadway were constructed around 1965. There is considerable evidence where the spalled surfaces of the concrete along the faces of the piers and abutments, the spillways and the soffit of

GENERAL SITE CONDITIONS AND BACKGROUND INFORMATION (cont'd)

the deck have been patched and refaced with concrete repairs.

We understand that in 1965, a three-span emergency spillway relief structure was constructed to the north of the Main Howson Dam and along the north side of the Maitland River channel. Repairs to undermining of the floor slab of this emergency relief structure were completed in the fall of 1983. The top surface of the roadway on Water Street over the relief emergency structure were completed in the fall of 1983. The Water Street approach to the north of the relief structure rises gently. The southerly Water Street approach to the Main Howson Dam falls to a lower elevation with relief provided over the south road approach. It was reported that in periods of extreme flooding in 1948, the flood waters used this relief.

The purpose of this Report is to comment on the surface deterioration and the integrity of the concrete in the Main Howson Dam and Bridge Structure, and to assess the load carrying capacity of the roadway supporting the bridge super-structure. This Report does not deal with any aspect of the emergency spillway structure, nor does it deal with the stability of the Main Howson Dam or the hydrological capacity of the combined dam and emergency spillway.

GENERAL SITE CONDITIONS AND BACKGROUND INFORMATION (cont'd)

The main structure consists of a four-span poured in place concrete combination dam and bridge structure. We were provided with one sheet of plans dated June, 1920, and prepared by Fred B. James from Walkerton, Ontario. The plans show the length of the two end spans at the underside of the deck at a clear distance of 10.06 m (33 feet) and the centre two spans at 10.67 m (35 feet). The width of the roadway is shown as 5.49 m (18 feet) with a 1.52 m clear sidewalk along the east side and 0.40 m wide concrete headwall handrails on either side to form the full deck width. The concrete deck is supported on four reinforced concrete beams. The present clear spans as measured by our firm on October 26, 1983, consist of 9.9 m - 9.7 m - 10.0 m - 9.8 m, indicating that if the original spans were constructed as shown on the plan, refacings have been added subsequently.

The dam portion of the structure consists of a raised concrete weir at the upstream face of the bridge transforming into a sloped spillway down onto a concrete apron with cut-off wall. The original plans show 1" diameter pipe sockets along the top of the concrete weir which were to be used to hold splash boards in position to raise the level of the upstream pond above that of the surface of the concrete weir. The construction has been altered and the present usage consists of 200 mm square stop log held in place by steel "H" beam sections at the third points of each span with the top of the intermediate column support resting against the side of the bridge deck, and the bottom fitted into a recess

GENERAL SITE CONDITIONS AND BACKGROUND INFORMATION (cont'd)

in the top of the concrete weir. The abutment and pier ends of the stop log are restrained by either a vertical chase cast in the concrete or a steel channel section lagged to the concrete. We understand it is the intention in the operation of the dam that all stop log and intermediate vertical supporting beams be removed during times of any major flooding.

The configuration of the stop logs, their supports, the upstream weir and spillway is shown in Pictures (i), (ii) and (iii) of the Appendix.

DETERIORATION OF EXISTING STRUCTURE

The deterioration of both the dam and bridge structure was inspected by both our firm and Mr. P. H. Davies, P. Eng., of Atkinson Davies Inc., Consulting Soils and Materials Engineers. Mr. Davies was retained by the Ministry of Natural Resources and the Maitland Valley Conservation Authority to perform a visual inspection along with a thorough investigation by ultrasonic testing and core samples drilled from the concrete to discover the strength of the concrete and confirm if the original concrete would form a suitable base for repair work. The location and extent of the testing was also co-ordinated with the requirements of our firm to provide the needed information on concrete strengths for a proper structural evaluation.

DETERIORATION OF EXISTING STRUCTURE (cont'd)

A visual inspection of the site was completed by Mr. Davies on September 20, 1983. The results of this inspection and Mr. Davies' recommendation are contained in his letter dated September 30, 1983, to the Ministry of Natural Resources, London Regional Office. A copy of this letter report is provided in Pages 1 - 3 of the Appendix. From this initial inspection, Mr. Davies recommended that their firm complete a program of ultrasonic testing and the taking of concrete compressive core samples to be tested in the laboratory to determine the soundness and strength of the original concrete.

A further investigation by Mr. Davies was authorized by the Maitland Valley Conservation Authority on December 20, 1983. Because of the severe winter conditions, the field work was completed on May 8, 1984. The results of this further testing are contained in a letter from Mr. Davies to the Maitland Valley Conservation Authority dated May 10, 1984, and found in Appendix Pages 4 - 8 at the back of this Report. As originally planned, this investigation was to consist of drilling a number of cores from the concrete with additional information being obtained from an ultrasonic survey. All areas of the dam and bridge including the weir, spillway, piers, abutments, deck and beams were to be included. The location of the cores in the deck super-structure was outlined by our firm and was to consist of six cores taken through the deck of the structure and one core to be taken through the centre of one of the supporting beams. With

DETERIORATION OF EXISTING STRUCTURE (cont'd)

the six tests through the deck, we wished four to be used for compressive strength purposes and two to be used to determine the chloride content of the concrete.

As reported in Mr. Davies' May 10, 1984, letter, nine concrete compressive core samples were taken and it was impossible to obtain any solid original concrete core samples suitable for testing. The samples consisted of lenses of concrete separated by dirt seams, and Mr. Davies reports that the result is that the concrete has a compressive strength which is so low that it is effectively zero. This condition applied to both the weir, spill-way, pier and abutment sub-structure and the deck super-structure. Because of the large number of voids and discontinuities in the concrete, ultrasonic testing was not attempted. Any readings obtained would have been invalid.

A detailed visual inspection was made of all the components of the bridge by the undersigned on May 8, May 11 and June 6, 1984. The soundness of all concrete was checked by tapping with a chipping hammer. The underside of the beams and deck were visually inspected with the use of a ladder. The upper surface of the deck is covered with 75 mm of asphalt and was not visually inspected. Based on the poor results obtained by Mr. Davies testing, we recommended to Committee of Council on Friday, May 11, 1984, that the Town immediately take steps to post a 3 tonne live load limit on the structure, and that our firm consult with the Structural

DETERIORATION OF EXISTING STRUCTURE (cont'd)

Office of the Ministry of Transportation and Communications as to further testing and calculations which could be completed to more accurately assign a load carrying capacity. Confirmation of our recommendation is contained in our letter to the Town dated May 15, 1984, and shown as Appendix 9 - 10. The results of this further investigation were that there was not any additional testing which could be used to accurately assess the concrete compressive strength. The one other method of assessing the load carrying capacity would be to have the Structural Office perform an official and relatively complicated load testing program at the site. Mr. Kleinsteiber of the Structural Office of the M.T.C. wished us to proceed with a structural evaluation to obtain the carrying capacity of the structure assuming the concrete strength was satisfactory. After this information was available, we would have more background information to assess the carrying capacity of the structure under its present condition. The results of this further investigation with the M.T.C. is contained in our June 8, 1984, letter to the Town, a copy of which is in Appendix 11 and 12.

Our visual inspection and checking of the soundness of the concrete with a chipping hammer confirmed the results obtained from Mr. Davies testing. The concrete in the weir, spillway, piers, abutments and the beams contains many areas of delaminated hollow sounding concrete. In many areas, the concrete is a combination of a collection of loose aggregate stones and small particles of concrete separated by dirt lenses. This also applies

DETERIORATION OF EXISTING STRUCTURE (cont'd)

to sections of the deck supporting beams as well as the sub-structure. Pictures (i) and (ii) in the back of the Appendix are indicative of the upstream spalled concrete surfaces of the sub-structure. Pictures (iv) and (v) show the spalling of the outside downstream beam of spans two and three from the north abutment.

There is evidence of considerable leaching of chloride through the porous concrete of the deck, beams and sub-structure. While this is shown in many of the photographs enclosed, it is particularly evident in photos (iii), (vi) and (vii).

There are severe signs of deterioration with spalling and cracking in the super-structure. There is a pattern of vertical cracking in the concrete headwall handrails. The majority of these cracks are located either at the one-third point of the spans or at the mid-span. Normally such cracking occurs over the piers and is caused by tension as a result of settlement at the centreline of the span. The location of these cracks would normally be caused by tension from uplift of ice on the underside of the deck. One other possible explanation for the location of these cracks may be due to the fact that the top of the beams supporting the stop log at the one-third point is now held by the side of the deck. If in the past the stop log and supporting beams were left in under severe flooding conditions, this could have resulted in the cracking of the handrail. An example of the typical cracking is shown in Picture (viii) and is located in the west handrail at the two-thirds point of the first span from the north abutment.

The underside of the concrete deck is in fair condition with areas of surface spalling and delamination. There was also considerable evidence of leaching of chlorides through the deck. It was impossible to inspect the upper surface of the concrete deck with the presence of the 75 mm thickness of asphalt. From the core samples that were taken through the roadway section of the deck, the thickness is approximately 280 mm (11 inches) and not the 200 mm (8 inches) shown on the original plans. One of the poorer areas showing the spalling of the underside of the deck and the exposed reinforcing is shown in Picture (ix) and is between the first beam east of the centreline of the road and the outside east beam of the second span from the north abutment.

There is serious spalling, delamination and cracking in the beams of the superstructure. There is also considerable evidence of leaching from chlorides. There are four beams in the super-structure of each of the spans. The two outside beams are directly under the concrete headwall handrails. The spalling on the bottom of the downstream side of the westerly outside girder is quite deep as shown in Pictures (iv) and (v). Much of the remaining concrete is delaminated and can be readily chipped away. The inside face of the outside west beam of the second span from the north abutment is shown in Picture (x). The bottom 200 mm of the beam is badly cracked and

DETERIORATION OF EXISTING STRUCTURE (cont'd)

delaminated and could be removed through chipping. Picture (xi) is also of the mid-span of the same beam and shows a serious horizontal stress crack located approximately 150 mm from the underside of the concrete deck slab. There were also places in the other beams where there is spalling, delamination and cracking. Picture (xii) is of the bottom of the first beam west of centreline in the second span from the north abutment. Picture (xiii) shows a serious delamination on the bottom of the east outside girder of the first span from the north abutment while Picture (xiv) shows the bottom of the first beam east of centreline in the first span. In both of the last two mentioned locations, the remaining concrete consisted of loose aggregate which could be readily chipped away.

From our deterioration examination of both the sub-structure and the super-structure, we concur with Atkinson Davies Inc.'s assessment that the existing concrete is not sound enough to act as a base for satisfactory long term repair work, and that the only course open is to remove and replace the dam and bridge structure.

STRUCTURE EVALUATION OF DECK SUPER-STRUCTURE

As requested by Mr. K. L. Kleinsteiber of the Structural Office of the M.T.C., we have proceeded with an evaluation of the deck super-structure to obtain the carrying capacity of the structure assuming that the concrete strength was satisfactory. It was intended that the dimensions and reinforcing steel shown on the original drawing confirmed by dimensions taken in the field would be used for this analysis. In reviewing the existing plan, we found that there was considerable important information missing, and many unconfirmed assumptions had been made. This applies particularly to the reinforcing steel, number of bars, their length and layout. There was also a scarcity of concrete dimensions on the plan and the dimensions that are given, especially for the depth of the beams, do not match the scale on the drawing and are not confirmed by the depth of the beams in the field. The depth of the central part of the deck shown on the plans is given as 8" and scales as 11".

The plans do not provide sufficient detail on the vertical steel from the beams into the deck to determine if the structure is acting as a "T" beam. As a first trial in making various assumptions, we considered the beams not acting as a "T" section and found their capacity to be 7.1 tonnes using the assumption of 20 MPa compressive concrete. A similar analysis assuming sufficient steel and full "T" beam action and using the depth of the beams as constructed in the field with 20 MPa compressive concrete resulted in the beams being capable of carrying the full design truck loads.

STRUCTURE EVALUATION OF DECK SUPER-STRUCTURE (cont'd)

We also evaluated the load carrying capacity of the deck slab and again making assumptions on the detailing and bending of the transverse reinforcing steel and 20 MPa compressive concrete, we found the steel in the bottom of the slab to be capable of carrying full design truck loading while the top transverse steel over the beams is capable of carrying a design loading of 15.2 tonnes.

After completing the detailed site inspection and the various calculations, we recommend that under its present condition, the 3 tonne live load limit remain on the deck super-structure. We are well aware that loads well in excess of this havebeen using the structure in the past. Our judgment is based on the many assumptions that have to be made and the lack of material given on the plans, the inability to obtain concrete compressive core samples, and most importantly, the cracking, delamination, leaching and absence of cement bond around the aggregate of the concrete in the supporting beams.

SUMMARY

The enclosed Report comments on the deteriorated condition of the Main Howson Dam and Bridge and provides a recommended live load carrying capacity for the bridge super-structure. We wish to summarize the salient features of our Report as follows:

SUMMARY (cont'd)

(a) Atkinson Davies Inc. conducted a visual inspection and attempted to take concrete core samples from both the dam and bridge sub-structure and the bridge super-structure. The results of their testing was that the concrete has a compressive strength which is so low that it is effectively zero, and that the concrete in their opinion will not act as a base for suitable repairs and rehabilitation.

(b) Our firm completed a detailed visual inspection and an evaluation of the deck super-structure using information provided on the original plan. We concur with Atkinson Davies Inc. that the concrete in the dam and bridge sub-structure and the deck super-structure is not sufficiently sound to warrant making satisfactory repairs and rehabilitation of the existing structure, and it is our opinion that the only course open is to remove and replace the dam and bridge structure.

(c) We recommend that based on the condition of the concrete, that the Town continue to post this structure with a maximum 3 tonne live load limit, and that the structure while in use be inspected on a yearly basis to detect further deterioration until it can be replaced.

All of the above is respectfully submitted for your consideration.

B. M. ROSS AND ASSOCIATES LIMITED
Consulting Engineers
62 North Street
Goderich, Ontario
N7A 2T4

December 14, 1984

Per ______________________________
K. G. Dunn, P. Eng.

APPENDIX A-1 - 3	--	Atkinson Davies Inc., letter dated September 30, 1983 to M.N.R.
APPENDIX A-4 - 8	--	Atkinson Davies Inc., letter dated May 10, 1984 to M.V.C.A.
APPENDIX A-9 -10	--	Our letter dated May 15, 1984 to the Town of Wingham
APPENDIX A-11-12	--	Our letter dated June 8, 1984 to the Town of Wingham
APPENDIX A-13-19	--	BR-476 (i) to (xiv) - Pictures

ATKINSON, DAVIES INC. *CONSULTING SOILS AND MATERIALS ENGINEERS*
69 BESSEMER ROAD, UNIT 36, LONDON, ONTARIO N6E 2V6 (519) 685-6400

September 30, 1983

Ref: 3-0198

Ministry of Natural Resources,
1106 Dearness Drive,
LONDON, Ontario
N6E 1N9

Attention: Mr. Jeff Jilek

Gentlemen:

Re: Howson Dam
Wingham, Ontario

As instructed by you, the undersigned visited the Howson Dam, Wingham, on September 20, 1983.

The object of the visit was to visually examine the condition of the concrete in the structure and to make recommendations on further investigative work which is required in order to ascertain the condition of the existing structure.

We understand that the original structure was built in 1920. The concrete appears to have been made using pit-run aggregate. Considerable repairs have been made at certain times during the history of the dam. These repairs consisted of fresh concrete cast against the existing concrete in most instances, although some repairs appear to have been made using shotcrete, an instance of the latter being the stop log support on the north side of the original relief spillway. The downstream faces of the piers also appear to have been repaired in this way.

The crests of all the weirs have been raised by additional concrete. The report from Mr. Stan McClellan on his under-water inspection reveals that the slabs downstream of the weirs have also had additional concrete placed on them and that he was able to see a gap between new and old concrete at one point. This confirms quite general hollowness found by tapping the existing top surface of this slab.

An additional and fairly recent addition to the bridge part of the structure is a solid concrete parapet on each side of the structure. This appears to have been constructed so that it forms a part of each outside beam, effectively deepening these beams. The parapets were apparently not designed to do this and consequently are cracking badly. The structural aspect of this wall should be investigated by a structural engineer.

The general state of the concrete in the structure is badly deteriorated. On the upstream side the repairs have broken away from the original structure. This has occurred at all piers and both abutments. The deterioration is worse at the north abutment and at the first pier from the north end, but is well advanced at the other piers and in progress at the south abutment.

The additional concrete on the weirs appears to have separated from the base concrete. The letter is badly deteriorated.

The least deterioration appears to have occurred in the shotcreted sections, but even there considerable cracking has occurred and many hollow areas found.

The concrete in the bridge beams is not badly deteriorated except on the underside of the downstream beam where much spalling of the concrete has occurred, exposing the steel reinforcement. Also, the hauches of the beams seem to have been deepened after the original construction. This additional concrete at the haunch is spalling severely.

In the deck concrete, viewed from the underside only due to the asphalt paving, the cover to the steel appears to have been inadequate. The reinforcement is seen on the underside in several areas, particularly in the second span from the north end. Grazing of the surface has occurred generally and deposited salts can be seen on the underside of the concrete. The salts are also seen on the sides of the beams and probably originate from road salt seeping through pores and cracks in the deck concrete.

As stated earlier, we were unable to see the upper surface of the deck, but, in view of the cracking and salt deposits seen on the underside, the state of the upper surface must be suspect. If arrangements could be made at the time of our visit to remove 4 or 5 areas of asphalt, say 2 feet by 2 feet, we would be able to form some opinions as to the state of the deck.

The basic cause of the deterioration is, in our opinion, the lack of resistance to cycles of freezing and thawing by the original concrete, and also insufficient care taken in the repair work to ensure that the new concrete was fully bonded to the old in such a way that seepage of moisture between the new and old could not occur.

At the time the original concrete was placed in 1920, nothing was known about the addition of air entrainment to resist freeze-thaw forces. Thus it is essential to protect this concrete completely wherever it is subject to cycles of freezing and thawing. None of the repair work has done this.

We propose that the next stage of the investigation should consist in determining whether or not the original concrete is adequate to form a base for repairs. Since none of the previous repairs are properly bonded, it is clear that all such concrete must be removed and replaced, with the possible exception of the downstream slabs where grouting the gap between the original slab andthe later addition might be filled by grouting.

In this same area, the condition under the original slab should be investigated to ascertain whether the voids which existed prior to the driving of the steel sheet piling are filled.

Because of the apparent general lack of bond between the old and new concrete, we do not think that a thorough investigation by ultra sonic testing is justifiable. We propose that a series of 12 cores be drilled from the concrete in the weirs, piers and beams to discover the strength of the concrete in those areas. Also we propose to do a thorough but not exhaustive program of ultra-sonic testing to confirm that the original concrete is sound. The drilling machine used for coring would not be suitable for drilling down through the slabs to investigate voids under the original slab. A larger drill such as is used for soil investigation would be required for this because the work must be done in the wet.

We think that one day will be sufficient for the investigation as outlined above, barring unforseen delays.

As discussed previously, we have assumed that scaffolding will be provided to enable us to gain access to the upper part of the piers and the beams, probably two sets to avoid stand-by time, and that one man will be available from the Town of Wingham to assist by holding one of the ultra-sonic transducers in place as required.

Our estimate of the upset figure for the cost of the work as outlined above, plus a report and laboratory testing to be $3,200.00, exclusive of the cost of any labour supplied by the Town and the local rental of scaffold.

We would be very happy to discuss with you further the amount of work involved.

Yours very truly,

ATKINSON, DAVIES INC.

P.H. Davies, P.Eng.

PHD/kih

ATKINSON, DAVIES INC. CONSULTING SOILS AND MATERIALS ENGINEERS
69 BESSEMER ROAD, UNIT 35, LONDON, ONTARIO N6E 2V6 (519) 685-6400

May 10, 1984

Ref: 3-0198

Maitland Valley Conservation Authority,
P.O. Box 5,
Wroxeter, Ontario.
NOG 2X0

Attention: Ms. Jane Taylor, Water Resources Technologist

Dear Ms. Taylor:

As instructed by your letter of 20 December, 1983, we have carried out an investigation of the condition of the concrete in the existing Howson Dam.

The field work was done 8 May, 1984, the delay being caused by the severe winter.

As originally planned, the investigation was to consist of drilling a number of cores from the concrete with additional information being obtained from an ultra-sonic survey. This program, which is a fairly standard type of investigation was planned based on the visual observation of the state of the concrete made by the undersigned on 20 September, 1983. From that, it appeared that very severe deterioration had occurred on the surface, but it was anticipated that once the surface deterioration had been penetrated by the cores, a soild concrete would be found, which might act as a base on which to perform an extensive repair program.

When the coring was done, it was found that the dirt which had been observed embedded in the cracks of the deteriorated concrete had not come from debris deposited by the river in flood but had in fact always been a part of the concrete. The cores were found to consist of comparatively small pieces of solid concrete separated by dirt seams.

COLIN J. W. ATKINSON M.Sc. P.Eng. PETER HOWELL DAVIES B.Sc. M.I.C.E. P.Eng.

The list of core locations and comments about each is attached. Only one short length of solid core was obtained, Core No. 4, from the north face of the north pier, above the shotcrete repair. This sheared from the remaining concrete during the drilling.

To investigate the condition of the concrete behind the shotcrete repair, Core No. 5 was drilled in such an area of the south face of the north abutment. This core yielded a thin solid disc of shotcrete from the surface, then part of the core from the original concrete emerged as a shattered mixture of stones and mortar, and one part, about 4 inches long, as a highly fissured series of poorly connected discs of concrete which would easily shatter if dropped.

Since virtually no solid concrete was located in the north abutment, north pier and north spillway and since the visual appearance of the remainder of the lower part of the structure was, if anything, worse than the north section, it was decided to terminate the coring of the lower structure at that point.

The next part of the investigation was to drill cores vertically down through the deck in two lines, (1) at the mid point between the two centre beams and (2) at the mid point between the west centre beam and the west outside beam, in each case drilling close to mid-span. The thickness of the asphalt, approximately 3 inches, plus the cover to the top reinforcing steel prevented us from locating any top reinforcing steel accurately. Consequently, the cores were placed where the drawing indicated that no top steel existed.

It was not possible to locate the bottom steel, but only one of the three cores drilled out this.

The purpose of the deck coring program was to obtain samples for compression testing and also for testing for the presence of chloride ions.

Once the three inches of asphalt had been drilled through, only shattered pieces of concrete and separated discs were recovered from the core barrel. At Core No. 7, one 5/8 inch square bar was cut. This bar had little or no bond to the concrete and was virtually uncorroded. The operator of the coring drill observed that while he normally expected to drill quickly through the asphalt and slow down when he came to the concrete deck, here his drilling rate was speeding up when he encountered the concrete. The thickness of the deck was found to be 11¼ inches, not 8 inches as stated on the drawing. It had also been planned to take a horizontal core through one of the centre beams, at about mid depth. Since it had been found impossible to obtain any cores from the first three deck locations drilled, it was decided, in consultation with Mr. Ken Dunn, P.Eng., B.M. Ross & Associates Ltd., to abandon the bridge deck coring program.

Because of the large number of voids and discontinuities in the concrete, ultra sonic testing was not attempted. Any readings obtained would have been invalid.

Comparing the aggregate in the concrete with the gravel in the river bed, it seems to be a reasonable supposition that the concrete was made from the gravel on site, including all dirt present. Mixing was obviously not thorough, with the result that the concrete in the dam consists of lenses of passable concrete separated by dirt seams. The result is that the concrete has a compressive strength which is so low that it is effectively zero. The severe deterioration is due to freezing and thawing of water which is able to enter the concrete through the dirt passages. It seems surprising that the deterioration is not more severe. This may be due to water being able to flow out of the concrete with the same ease with which it enters.

There is very little corrosion of reinforcing steel to be seen so that the bridge structure remains reinforced as designed.

In view of the condition of the concrete, we are of the opinion that it will not act as a base for repair work and that the only course open is to remove and replace the dam and bridge structure.

Yours very truly,

ATKINSON, DAVIES INC.

P.H. Davies, B.Sc. M.I.C.E. P.Eng.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
P. H. DAVIES
PROVINCE OF ONTARIO

PHD/kih

Enclosures

Copies: Ministry of Natural Resources
Attention: Mr. Jeff Jilek

B.M. Ross & Assoicates Ltd.
Attention: Mr. K.G. Dunn, P.Eng.

ATKINSON, DAVIES INC.,

Enclosure 1
Ref: 3-0198

CONCRETE CORES DRILLED 8 MAY, 1984.

Core No.	Dia.	Location	Remarks
1	4"	North abutment, east side	Core shattered when drilled.
2	6"	" " " "	Core shattered.
3	6"	Upstream face, north spillway.	Core shattered.
4	6"	North face, north pier, above gunite repair.	Solid section 6½" to 8" long recovered, at that point the core sheared
5	6"	South face, north abutment.	One section of 4" long core recovered, highly fissured. Remainder of core shattered.
6	4"	Apron, north spillway.	Core shattered.
7	4"	Deck, east side north span, mid way between two centre beams.	Core broke into irregular discs, also part completely shattered.
8	4"	As 7 but one span to south.	Core shattered.
9	4"	As 7 but two spans to south.	Similar to 7.

BR-476

May 15, 1984

Mr. Byron Adams
Clerk-Treasurer
Town of Wingham
Box 90
WINGHAM, Ontario
N0G 2W0

Dear Sir:

Re: Main Howson Dam Structure
Over Maitland River

We wish to confirm our recommendations to Committee of Council on Friday, May 11th, 1984. This meeting was subsequent to concrete testing of the structure initiated by the Maitland Valley Conservation Authority and performed by Davies Testing on Tuesday, May 8th, 1984.

On Tuesday, May 8th, I met at the site with Peter Davies from Davies Testing, Jane Taylor from the Maitland Valley, Ross Jackson from the Stratford District of the M.T.C. and Jeff Jilek from the Ministry of Natural Resources, to outline the portion of the concrete testing that we required to assist in our structure evaluation to determine the load carrying capacity of the roadway as authorized by your Town. At 11:00 a.m., Davies had completed two test cores in the substructure and had found that the concrete fractured sufficiently that compressive strength cores could not be obtained to test in the laboratory for the concrete strength of the material.

I outlined with Peter Davies that we wished six cores to be taken through the deck of the structure and one core to be taken through the centre of one of the supporting beams. With the six tests through the deck, we wished four to be used for a compressive strength purposes and two to be used to determine the calcium chloride or salt content of the concrete.

At 3:30 p.m. the same afternoon, we received a telephone call from Peter Davies outlining that they had taken three of the deck test cores and found the concrete fractured upon

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coring resulting in not being able to obtain a suitable sample for testing from any of the cores. On the basis of this testing, we did not take the additional three deck cores or the core sample through the face of one of the beams.

Prior to our meeting on Friday, May 11th, I contacted Peter Davies for his recommendations on the strength of the concrete, Ross Jackson from the District Office of the M.T.C. and Ken Kleinsteiber from the Toronto Structural Office of the M.T.C. Based on the testing and our discussions, we propose the following recommendations:

(1) From our initial inspection of the structure and the testing by Davies, we are of the opinion that there is not a sufficiently solid base of sound concrete to repair the existing bridge and dam.

(2) We recommend that the Town immediately take steps to post a 3-tonne liveload limit on the structure.

(3) Over the next two week period, we wish to consult with the research department of the Toronto Structural Office of the M.T.C. as to further testing and calculations which can be completed to more accurately assign a load carrying capacity for the existing structure.

We shall keep you informed of our progress. Should you have any questions on the enclosed, please contact us.

Yours very truly

B. M. ROSS AND ASSOCIATES LIMITED

Per ______________________________

K. G. Dunn, P. Eng.

KGD*jb

c.c. - Ross Jackson
M.T.C., Stratford

- Bryan Howard
M.V.C.A.

BR-476

June 8, 1984

Mr. Byron Adams
Clerk-Treasurer
Town of Wingham
Box 90
WINGHAM, Ontario
NOG 2W0

Dear Sir:

Re - Main Howson Dam Structure Over Maitland River

Subsequent to our letter of May 15, 1984, we have had conversations with Dr. David Manning of the Research Department of the M.T.C., and K. L. Kleinsteiber from the Downsview Structural Office of the M.T.C. Dr. Manning does not feel that there are any additional testing to what we have already taken, which could be used to accurately assess the concrete compressive strength in the existing bridge. The one other method of assessing the low carrying capacity would be to have the Structural Office perform an official and relatively complicated load testing program at the site.

Arrangements for load testing are made through Mr. Kleinsteiber, and are generally used on structures where the materials are sufficiently sound to warrant rehabilitation of the structure.

Prior to load testing, one of the basic steps to be taken is the structural evaluation of the existing structure which you have already authorized our firm to complete. Mr. Kleinsteiber feels that the Town should proceed with the structure evaluation to obtain the carrying capacity of the structure, assuming that the concrete strength was satisfactory. After this information is available, we would have more background information to assess the carrying capacity of the structure under its present condition.

Based on the recommendation of the M.T.C. and as previously authorized by your Council, we are proceeding with the structure evaluation. If there are any questions on the enclosed, please contact us.

.... cont'd

Mr. Byron Adams
Clerk-Treasurer
Town of Wingham
June 8, 1984
Page 2

BR-476

Yours very truly,

B. M. ROSS AND ASSOCIATES LIMITED

Per ______________________
K. G. Dunn, P. Eng.

KGD:jj

c.c. Mr. Ross Jackson
M.O.E. Stratford

Mr. Bryan Howard
M.V.C.A.

TOWN OF WINGHAM

MAIN HOWSON DAM AND BRIDGE

BR-476 - PICTURES

BR-476 - (i) - Upstream East Elevation
Third Span from North

BR-476 - (ii) - Upstream East Elevation
Second Span from North

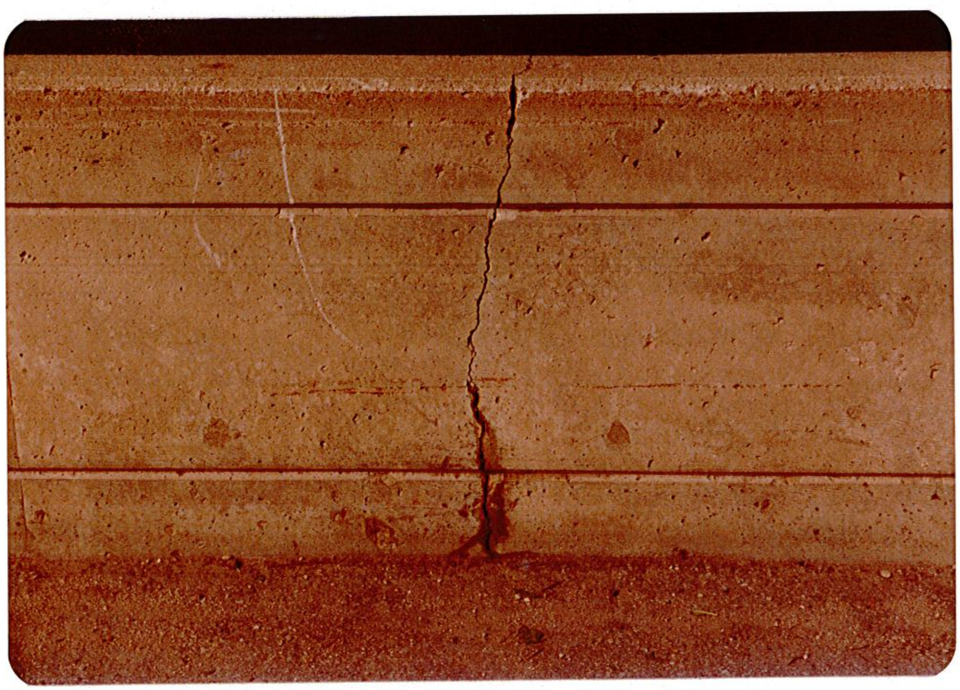
BR-476 - (iii) - North Face of Pier No. 2 from North Abutment

BR-476 - (iv) - West Elevation Second Span from North Abutment

BR-476 - (v) - West Elevation Third Span from North Abutment

BR-476 - (vi) - South Face of Pier No. 1 from North Abutment

BR-476 - (vii) - West Face of First Beam East of Centreline Roadway - Second Span from North Abutment



BR-476 - (viii) - Crack through West Handrail at Two-Thirds Point of First Span from North Abutment

BR-476 - (ix) - Underside of Deck Between First Beam East of Centreline and Outside East Beam - Second Span from North Abutment

BR-476 - (x) - Inside Face of Outside West Beam - Second Span from North Abutment

BR-476 - (xi) - Inside Face of Outside West Beam - Second Span from North Abutment

BR-476 - (xii) - Bottom of First Beam West of Centreline - Second Span from North Abutment

BR-476 - (xiii) - Bottom of Inside Face of East Outside Girder - First Span from North Abutment

BR-476 - (xiv) - Bottom of First Beam East of Centreline - First Span from North Abutment