

#60-F-332 C

CONTRACT 59-330

HOMER BRIDGE OVER

WELLAND CANAL

PILE TEST .

COPY

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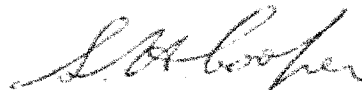
TORONTO, October 3, 1960.

60-F-332 C

HOMER BRIDGE
OVER
THE WELLAND CANAL
PILE TEST CONTRACT 59-530

The enclosed report contains the results of the pile test contract carried out during the early part of this year.

These results show conclusively that piles, similar in weight and size to those tested, driven with the specified energy to a known penetration resistance, are perfectly capable of carrying the design loads of 70 tons under dead load and 100 tons under dead plus live loads.



S. H. Cooper
ed

6 c.c. to Homer Bridge Field Office



REPORT
ON THE
PILE TEST CONTRACT
AT THE
HOMER BRIDGE
OVER
THE WELLAND CANAL

October, 1960.

FOUNDATION OF CANADA ENGINEERING
CORPORATION LIMITED

FENCO

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

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INTRODUCTION

Homer Bridge is the name attached to the new elevated structure carrying the Queen Elizabeth Highway over the Welland Canal near St. Catharines, Ontario. This structure will take the through traffic off the lift bridge on No. 8 highway, and eventually form the link between the projected New York Throughway and Highway 401.

This new structure divides naturally into three distinct sections; namely, the West Approach Spans, the Main Spans, and the East Approach Spans. In the West Approach Spans, the structure is carried on piers supported on spread foundations. Piles are not required in this section for supporting the foundations. However, for the centre and part of the east approach sections, the foundations of the piers are supported on Steel H Piles driven into the underlying till. The length of bridge structure which requires to be carried on piles is some 3,000 feet long.

STRATIGRAPHY

A detailed report on soil conditions by Geocon Ltd is contained in Fenco's report on the Homer Bridge dated February 24, 1960 to the Department of Highways, Ontario. However, for the purpose of this report, a brief description of the underlying strata is given.

STRATIGRAPHY (Continued)

The site of the proposed Queen Elizabeth crossing over the Welland Canal near Homer, Ontario, is located on the lowland between the Niagara Escarpment to the south, and Lake Ontario to the north.

Bedrock beneath this whole lowland area consists of red oxidized sandy Queenston Shale.

The bedrock is overlain by about 120 feet of glacial and post-glacial sediment of complex geological history. A dense reddish sandy glacial till directly overlies the Queenston Shale and consists largely of material derived from it.

The till is overlain by a variable thickness of varved clay. This varved clay grades upwards into a structureless silty clay stratum containing small pebbles and tiny pockets of silt. The upper portion of this stratum has been oxidized to a brown colour and preconsolidated. The shoreline and associated beach deposits of the former Lake Iroquois generally covers the central portion of the site.

The condition of the upper layer of preconsolidated clay varies considerably. In the west and east approaches this clay is relatively thick and is sufficiently strong to support the piers on spread foundations. However, throughout the central section of the bridge, on either side of the Canal, this layer of preconsolidated clay is relatively thin and

STRATIGRAPHY (Continued)

piers throughout this 3,000' length require to be supported on long piles driven through the clay to the till overlying the Queenston Shale. These piles vary in length from 57 feet at Pier E1, to 90 feet at Pier E5.

Consideration was given to the use of displacement piles, but because of the depth and properties of the clay, pre-bored holes would be required to locate these piles and carry them into the till. Steel H Piles however, could be driven directly with a conventional type hammer and studies indicated that these piles would be more economical.

For the purpose of this test, two different types of H-Piles were used at six different locations throughout the centre section, and consisted of:-

14" x 14" Bethlehem piles
weighing 73 lbs. per foot, and

11" x 11" Columeta piles, size 28,
weighing 75.74 lbs. per foot.

The piles were supplied in 35 foot lengths, and plain butt welds were used for splicing. All piles were driven vertical with the tips reinforced.

SUMMARY

A total of nine piles were driven on the site from station 217+10 to station 247+41. The piles were driven in such locations so as not to interfere with future pile

FMSC

SUMMARY (Continued)

driving in the footings. Six of these piles were load tested.

Two types of pile drivers were employed:

A Vulcan "O" with a ram, weighing 7500 lbs. and a rated energy per blow of 24,375 ft. lbs., and a

Vulcan "OH" with a ram, weighing 9300 lbs. and a rated energy per blow of 30,225 ft. lbs.

Loading was accomplished by means of a 400-ton hydraulic jack, reacting against a dead load of 238 tons supported on two cribs as shown in Figure No. 1. The load was increased in increments of 25 tons at a minimum of every two hours or whenever the recorded settlement was less than 0.010 inches per hour. The piles, with one exception, were loaded to a maximum of 200 tons. The maximum load was held on the piles for a minimum of 48 hours, and then reduced in 50-ton increments at a minimum of two hours or whenever the recorded rebound was less than 0.010 inches per hour. Settlements were recorded on Auto Gauges reading directly to 0.001 inch.

Test Pile No. 1 was a 12" Columbia No. 28 @ 13.74 lbs. driven with the Vulcan "O" pile driver at station 231-96 on centerline. The length driven was 90.5 feet with a penetration of 12.0 feet into the till. Driving was comparatively easy in the clay, averaging 5 blows per foot.

SUMMARY (Continued)

The final driving resistance was 28 blows per inch. This pile was loaded to 238 tons with a net settlement of 1.03 inches.

Test Pile No. 2 was a 14 BP 73 driven with the Vulcan "O" at station 232+21 on centreline. The length driven was 90.50 feet with a penetration of 11.0 feet into the till. Driving resistance in the clay averaged 7 blows per foot. Final driving resistance was 30 blows per inch. This pile was loaded to 200 tons with a net settlement of 0.140 inches.

Test Pile No. 3 was a 14 BP 73 driven with the Vulcan "O" at station 244+58 on centreline. The length driven was 75.50 feet with a penetration of 13.85 feet into the till. Driving resistance in the clay averaged 6 blows per foot. Final driving resistance was 31 blows per inch. This pile was loaded to 200 tons with a net settlement of 0.120 inches.

Test Pile No. 4 was a 11" Columeta No. 28 @ 75.74 lbs. driven with the Vulcan "O" at station 247+41 on centreline. The length driven was 100.83 feet with a penetration of 25.73 feet into the till. Driving resistance in the clay averaged 7 blows per foot. This pile was driven to a final resistance of 96 blows per inch in an attempt to penetrate through the till to bedrock. This was unsuccessful.

SUMMARY (Continued)

ful with the top of bedrock 5.17 feet below the tip of the pile. This pile was not load tested.

Test Pile No. 5 was a 11" Columeta No. 28 @ 75.74 lbs. driven at station 237+00 on centreline, with the Vulcan "O". The length driven was 100.16 feet, with a penetration of 21.22 feet into the till. Driving resistance in the clay averaged 3 blows per foot. Final driving resistance was 29 blows per inch.

Test Pile No. 6 was a Columeta driven four feet to the left of Test Pile No. 5 with the Vulcan "OR". The length driven was 104.63 feet. Driving resistance in the clay differed little from No. 5, averaging 7 blows per foot. This pile was driven to a final resistance of 36 blows per inch. The nearest borehole, a distance of 164 feet away, indicates bedrock two feet below the tip of the pile. However, it is possible that this pile may have penetrated to bedrock. Test Pile No. 5 was loaded to 200 tons with net settlement of 0.106 inches. Test Pile No. 6 was not loaded.

Test Pile No. 7 was a 14 BP 73 driven at station 217+10, four feet right, with the Vulcan "OR". The length driven was 98.50 feet with a penetration of

SUMMARY (Continued)

14.0 feet into the till. Driving resistance in the clay averaged 7-1/2 blows per foot. Final driving resistance was 74 blows per inch. Borehole data indicates that bedrock was 10.0 feet below the tip of the pile.

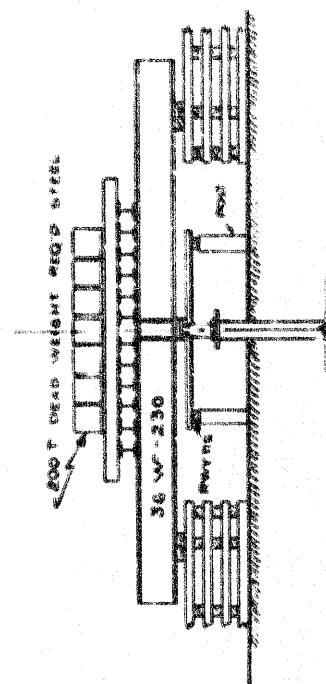
Test Pile No. 8, a Columeta, was driven at station 217+10 on centreline with the Vulcan "O". The length driven was 97.0 feet with a penetration of 12.5 feet into the till. Driving resistance in the clay averaged 6 blows per foot. Final driving resistance was 36 blows per inch. Test Pile No. 7 was not load tested. Test Pile No. 8 was loaded to 200 tons with a net settlement of 0.092 inches.

Test Pile No. 9, a Columeta, was driven at station 219+90 on centreline with the Vulcan "O". Total penetration was 89.58 feet with a penetration of 8.58 feet into the till. Driving resistance in the clay averaged 9 blows per foot. Final driving resistance was 30 blows per inch. This pile was loaded to 200 tons with a net settlement of 0.11 inches.

It is worth noting that during the driving of the piles there was no damage to the flange or web of either sections.

CONCLUSIONS

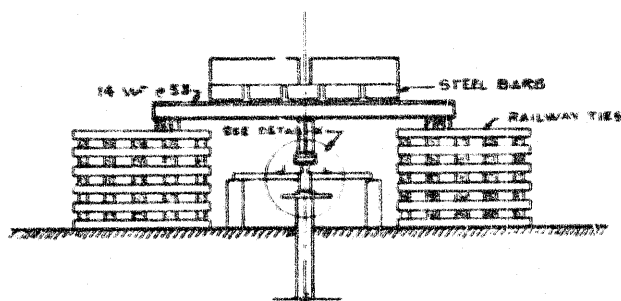
It may safely be concluded that if Steel H Piles similar to those tested are driven to a final resistance of 20 blows per inch with equipment of ram weighing not less than 7500 lbs. and a minimum rated energy per blow of 24,000 ft. lbs. there is no danger of failure from loads up to 200 tons.



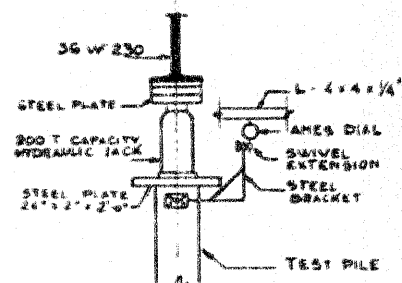
SECTION A-A

PLAN

SCALE



SECTION B-B



TWO DEFLECTORS USED - ONE AS SHOWN
ON ONE OPPOSITE HAND ON FAR SIDE OF FILE

ARRANGEMENT
FOR MEASURING SETTLEMENT.

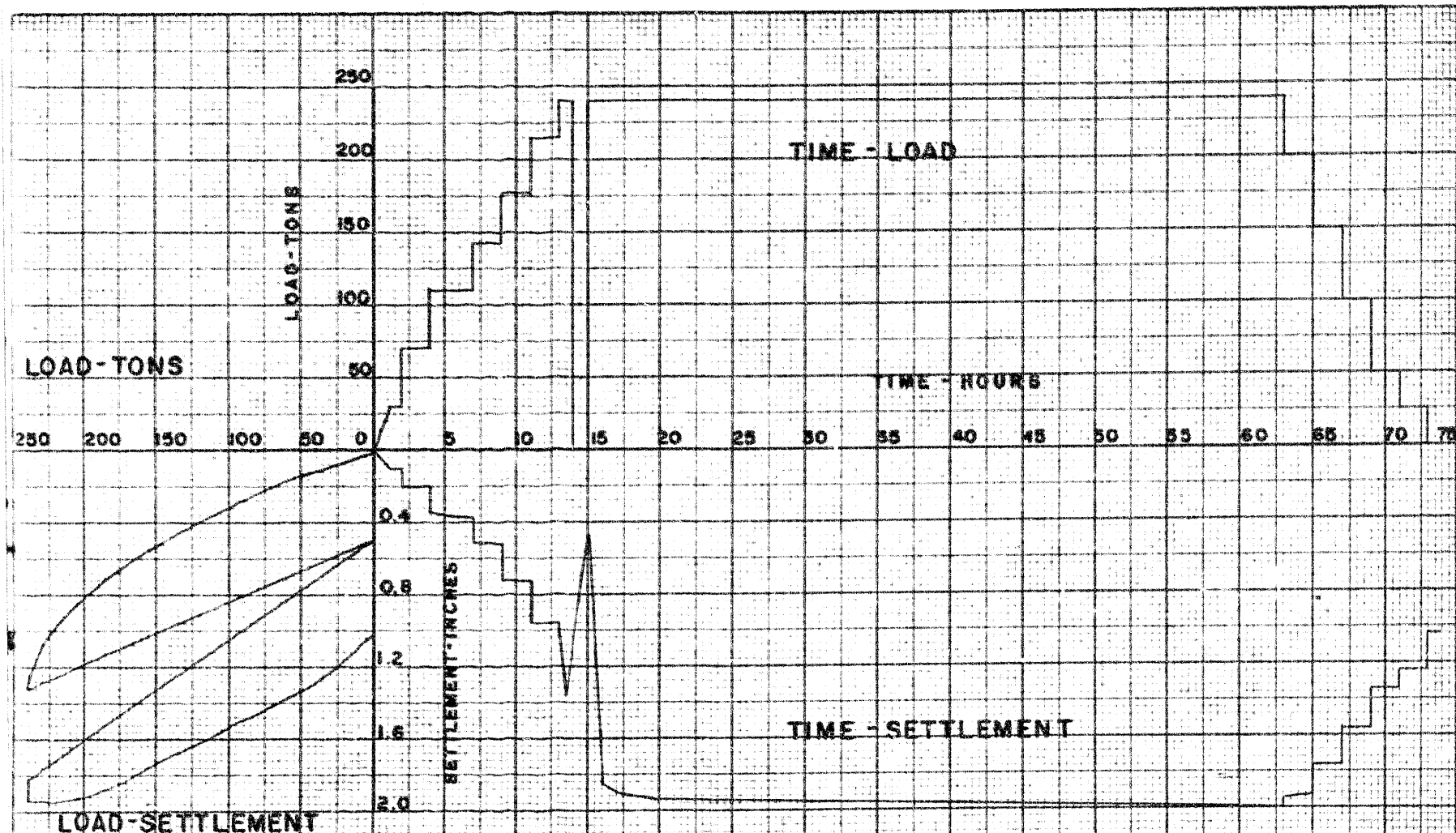
DETAIL "X"

HOMER BRIDGE - PILE TEST CONTRACT

TEST PILE	# 1	# 2	# 3	# 4	# 5	# 6	# 7	# 8	# 9
TYPE	COLUMETA " 28/75.74	14 BP 73	14 BP 73	COLUMETA " 28/75.74	COLUMETA " 28/75.74	COLUMETA " 28/75.74	14 BP 73	COLUMETA " 28/75.74	COLUMETA " 28/75.74
LOCATION	231 + 96 ON C	232 + 21 ON C	244 + 58 ON C	247 + 41 ON C	237 + 00 ON C	237 + 00 - 4'L	217 + 10 - 4'R	217 + 10 ON C	219 + 90 ON C
HAMMER	VULCAN'O'	VULCAN'O'	VULCAN'O'	VULCAN'O'	VULCAN'O'	VULCAN'OR'	VULCAN'OR'	VULCAN'O'	VULCAN'O'
GROUND ELEV.	351.38	352.40	342.75	366.30	355.64	355.64	343.90	343.90	340.50
TILL ELEV. (FROM BOREHOLES)	272.80 78.58'	272.80 79.60'	281.10 61.65'	291.20 75.10'	276.70 78.94'	276.70 78.94'	259.40 84.50'	259.40 84.50'	259.50 81.00'
TIP ELEV.	260.88	261.90	267.25	265.47	255.48	250.81	245.40	246.90	250.92
TOTAL PENETRATION	90.50'	90.50'	75.50'	100.85'	100.16'	104.83'	98.50'	97.00'	89.58'
TOTAL NO. BLOWS	1121	1403	1582	3244	2192	2267	2032	1623	1814
BLOWS LAST INCH.	28	30	31	96	20	86	74	36	30

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P. 10



TEST PILE No. 1

LOCATION: STA. 231+96 & BRIDGE

PILE TYPE: COLUMETA SECTION 28 11x11 75.74' / FT.

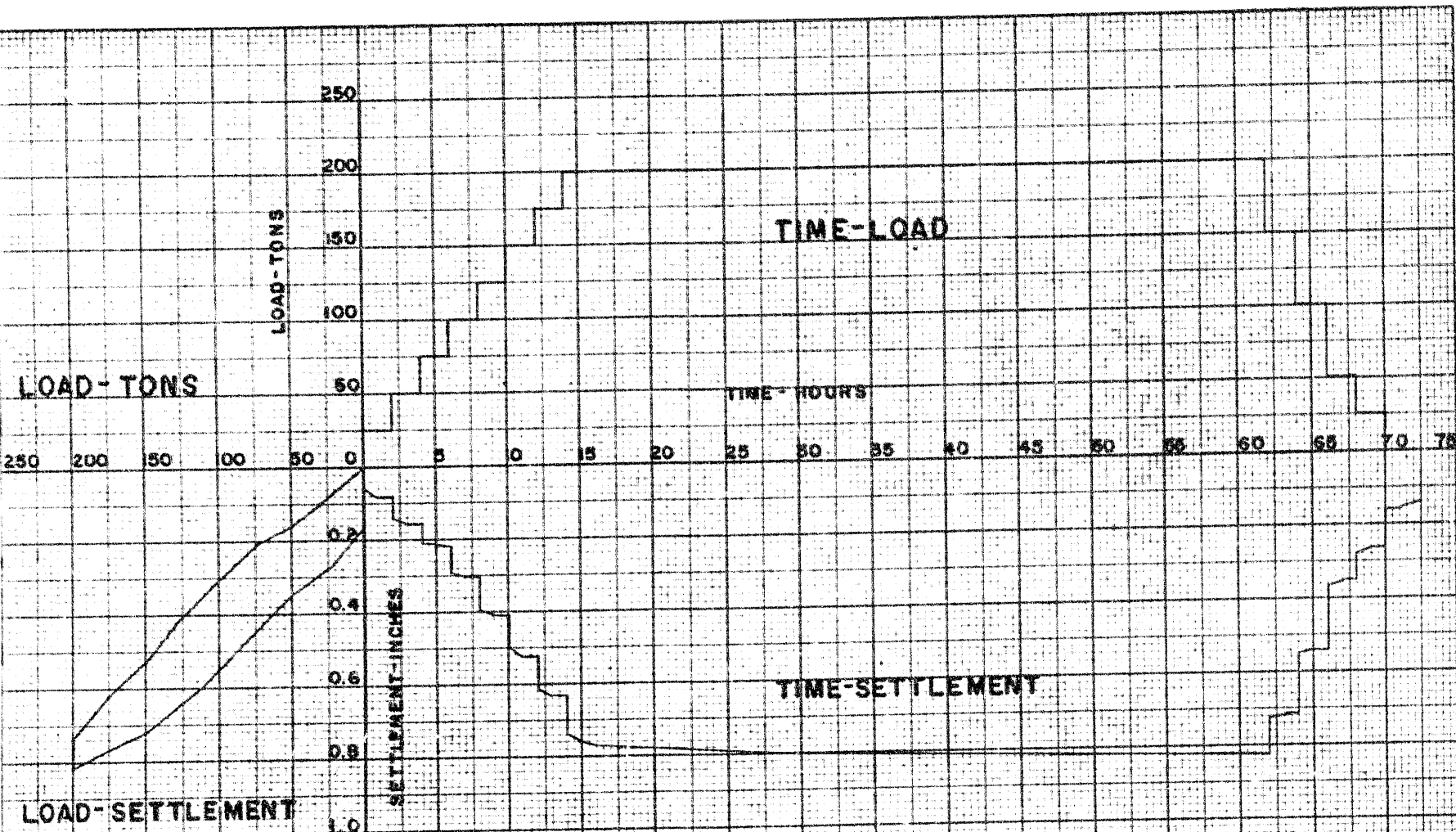
PILE LOADING TEST

D.W.O. CONTRACT 59-350

FENCO

HOMER BRIDGE

OVER
THE WELLAND CANAL



TEST PILE No. 2

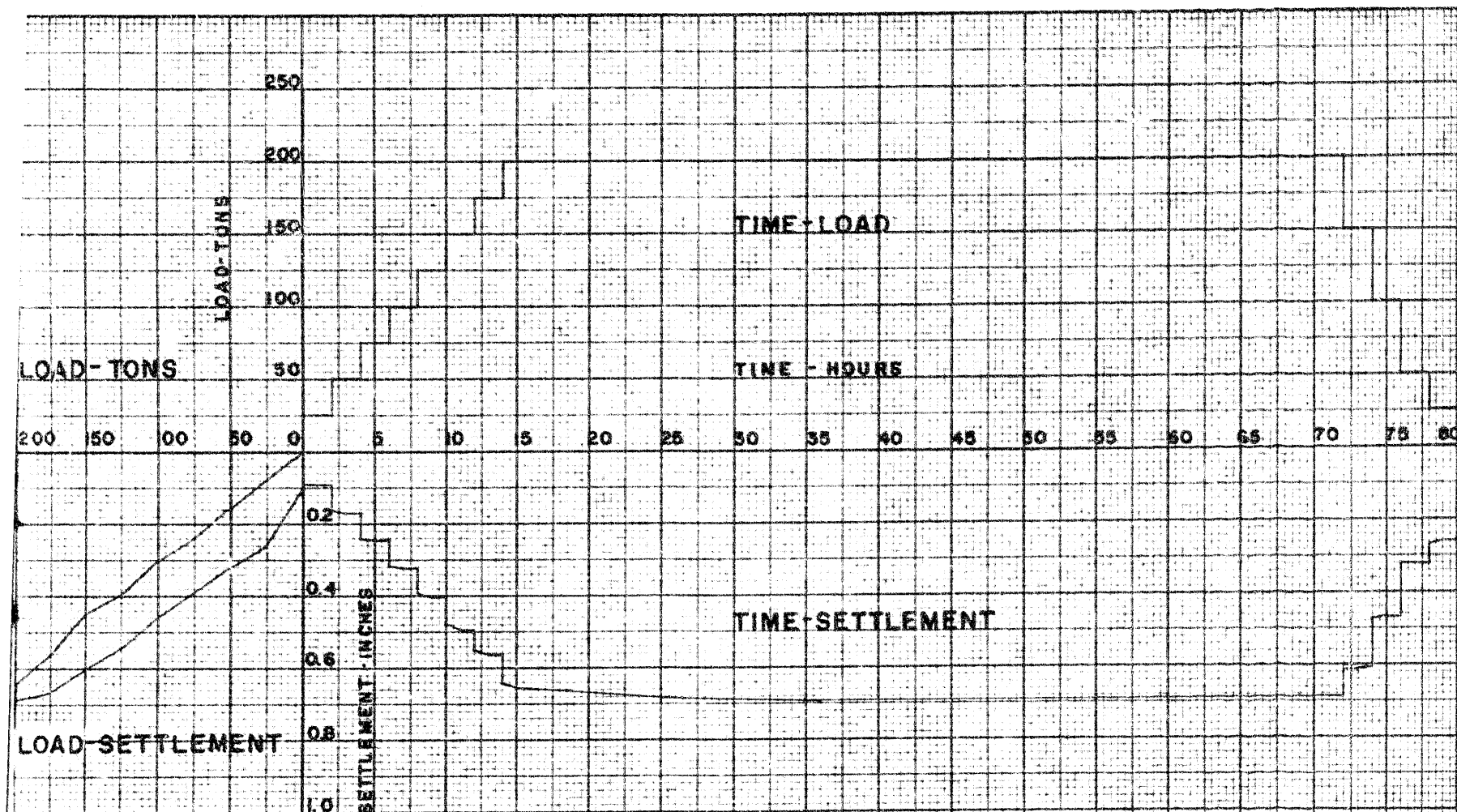
LOCATION: STA. 232+21.5 BRIDGE

PILE TYPE: 14 BP 73

PILE LOADING TEST
D.H.O. CONTRACT 59-330

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HOMER BRIDGE
OVER
THE WELLAND CANAL



TEST PILE No. 3

LOCATION: STA. 244+58 ON C BRIDGE

TYPE: 14 BP 73

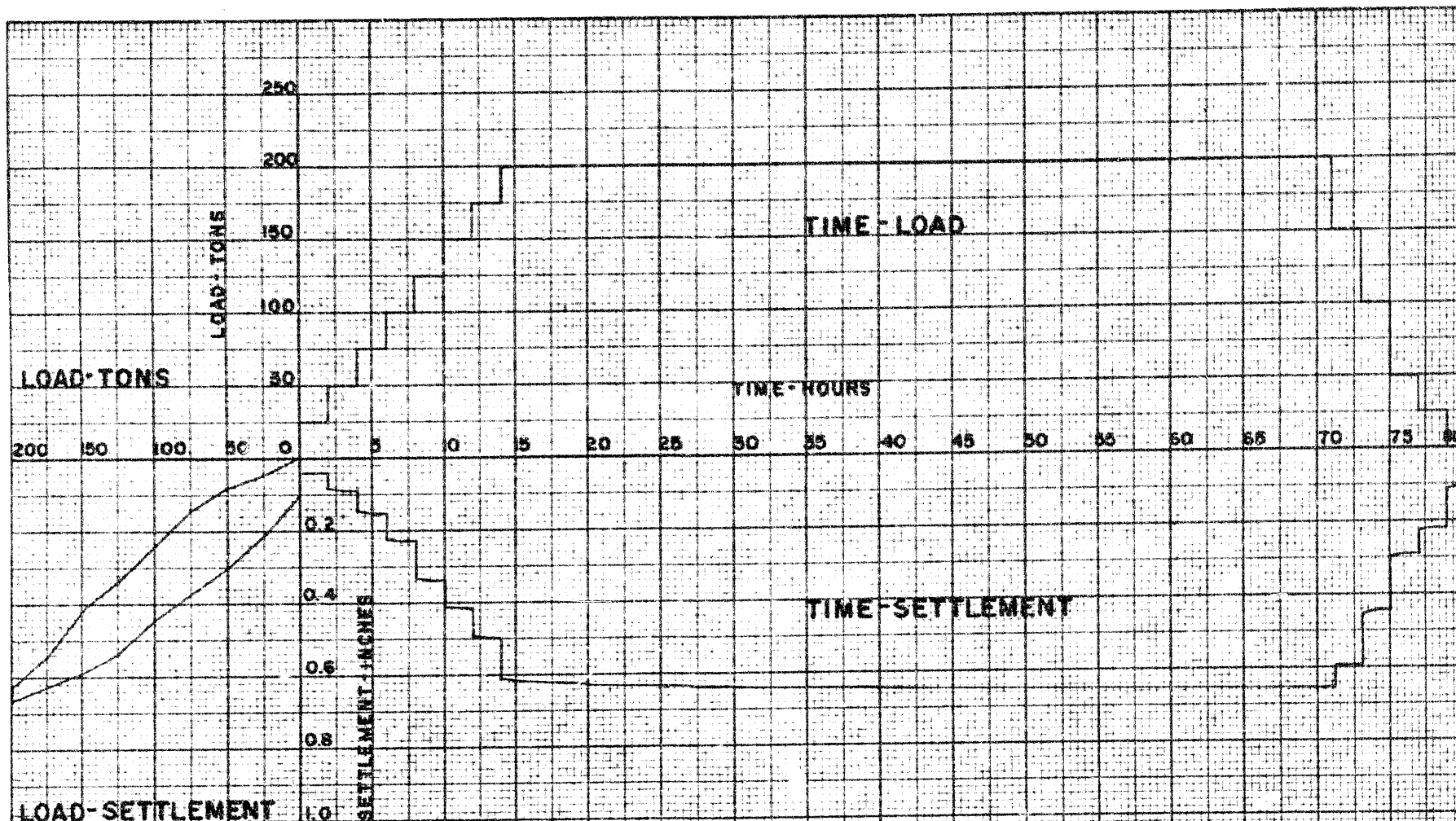
PILE LOADING TEST

DMD. CONTRACT #59-380

FENGO

HOMER BRIDGE

OVER
THE WELAND CANAL



TEST PILE No. - 5

LOCATION: STA 237+00 ON C BRIDGE

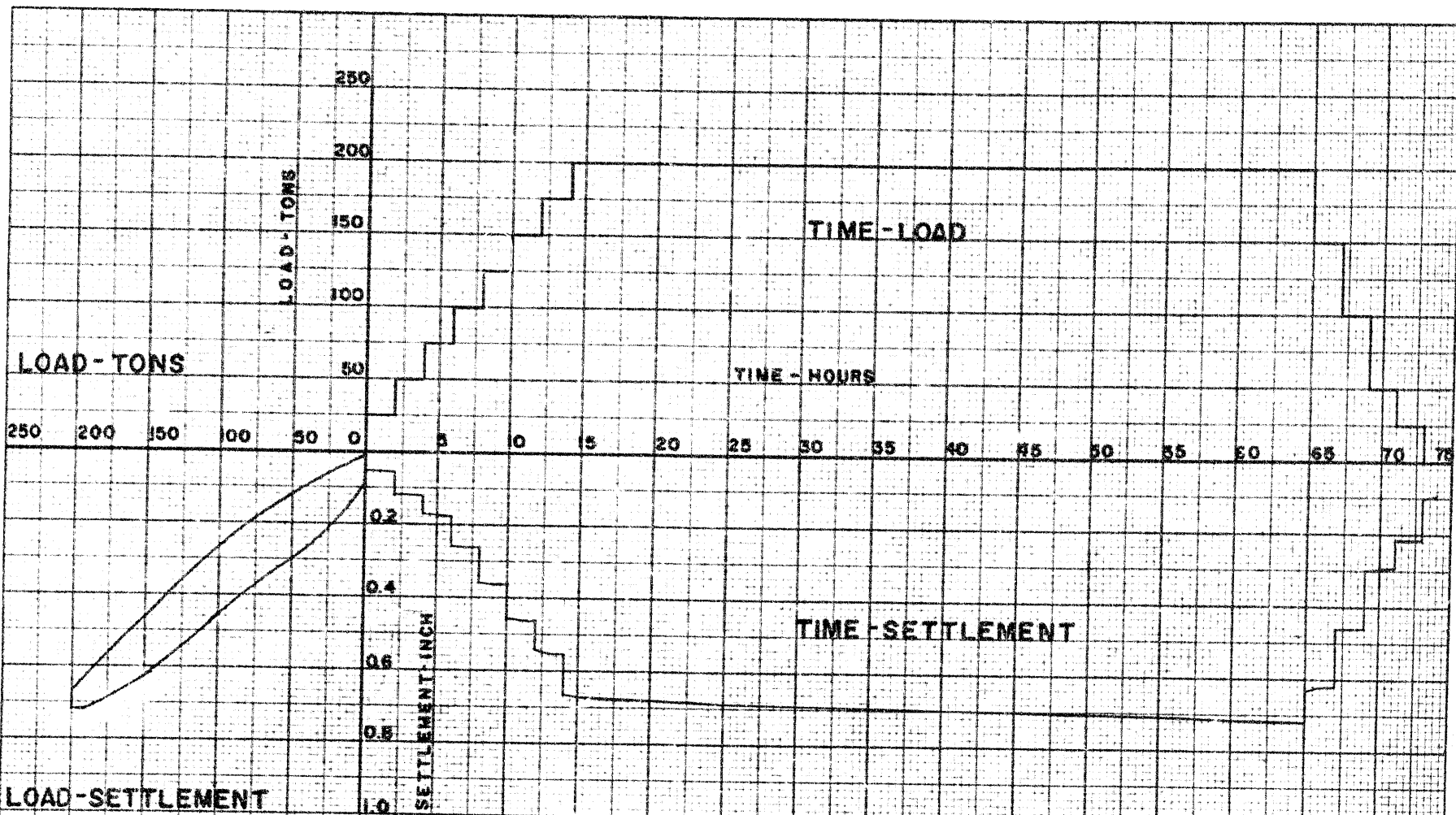
PILE TYPE: 11" COLUMETRA #28/75 ZC

PILE LOADING TEST

DMD. CONTRACT #59-330

FENCO

HOMER BRIDGE
OVER
THE WELLAND CANAL



TEST PILE NO. 8

LOCATION: STA. 217+10 ON G BRIDGE

PILE TYPE: 11" COLUMETA #28/75.74

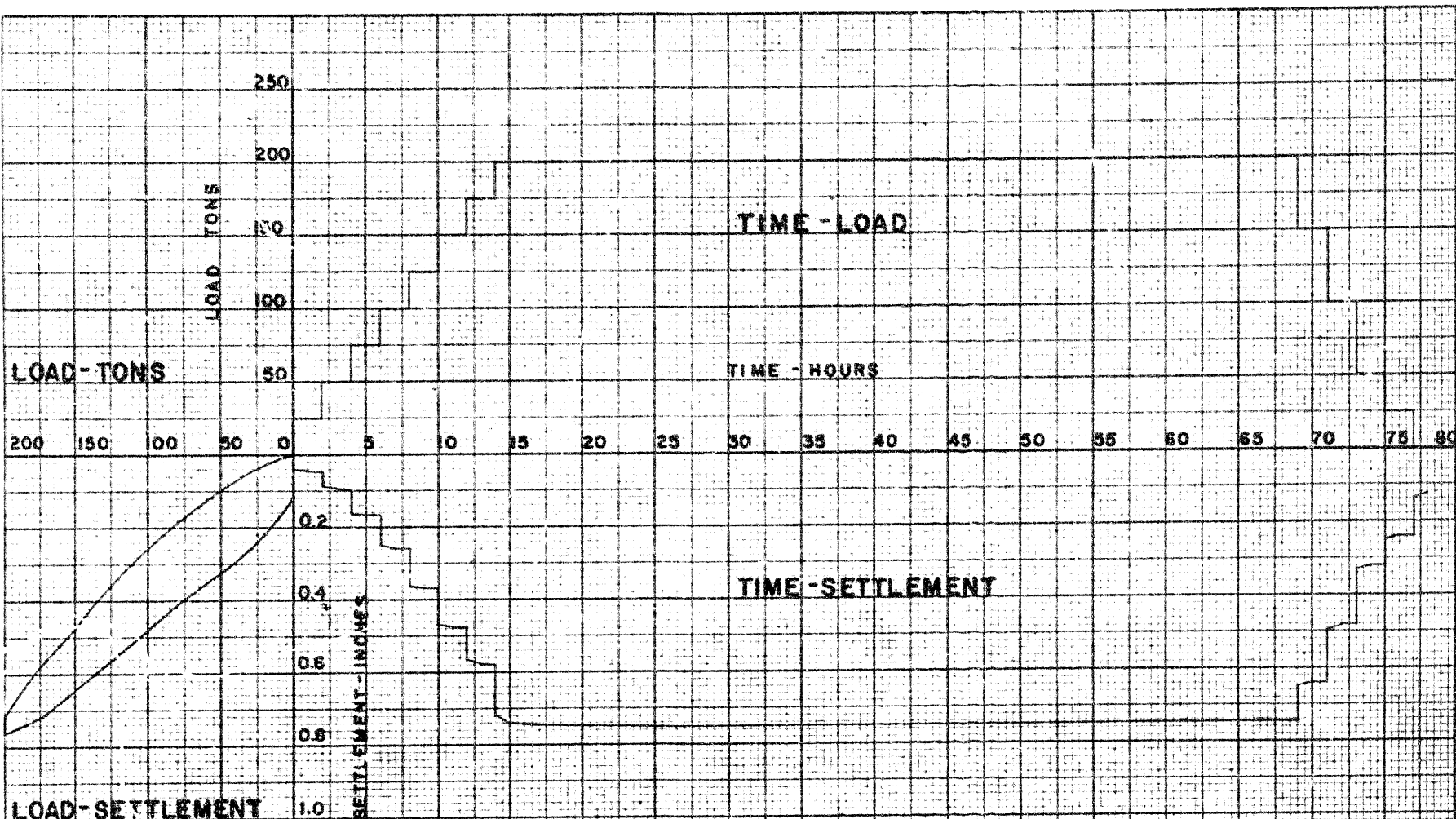
PILE LOADING TEST

D.H.G. CONTRACT #59-330

FENCO

HOMER BRIDGE

OVER THE WELLAND CANAL



TEST PILE NO. 9

LOCATION STA 219+90 ON G. BRIDGE

PILE TYPE: 11" COLUMETA # 28/75.74

PILE LOADING TEST

D.H.O. CONTRACT # 59-330

F E N C O

HOMER BRIDGE

OVER
THE WELLAND CANAL

P10

Planning
Engineering
Project Management

FENCO

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Telex: 02 2514

November 18, 1971

Mr. H.W. Adcock, P.Eng.
Assistant Deputy Minister
Engineering and Operations
Department of Transportation and
Communications
Downsview 464, Ontario

Dear Mr. Adcock,

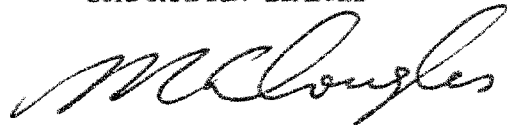
GARDEN CITY SKYWAY
1971 INSPECTION

We are pleased to submit herewith the result of our 1971 visual inspection of the Garden City Skyway, together with supplementary settlement data for the west approach span piers.

In general the structure was found to be in sound structural condition throughout, although continued observations should be programmed for the piers in the vicinity of W10 where an abnormal settlement pattern has occurred. The other significant deficiencies noted in our previous report have been taken care of and no longer pose a problem.

We trust the enclosed fulfills your requirements at this time, however we would be pleased to further discuss any related matter at your convenience, if deemed warranted.

Yours very truly,
FOUNDATION OF CANADA ENGINEERING
CORPORATION LIMITED



MCD/dlm
3569

M.C. Douglas, P.Eng.
CHIEF ENGINEER, TRANSPORTATION

REPORT
ON
1971 INSPECTION
OF
GARDEN CITY SKYWAY

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

FENCO
3569

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INTRODUCTION

In March 1971 FENCO was authorized by the Department of Transportation and Communications to carry out a complete visual inspection of the Garden City Skyway over the Welland Canal. Subsequent discussions and correspondence further authorized a preliminary review of conditions existing in the vicinity of pier W10; a settlement survey on the west approach from pier W6 to the west abutment; and the establishing of plumbing reference datums for future observations to be made on piers W8 - 9 - 10 and 11.

The above work was performed under the direction of FENCO's Mr. A. Uesson assisted in part by the Department's Mr. J. Shorrocks. The preliminary inspection of the structure adjacent to pier W10 was carried out in May while the full scale inspection, settlement survey and plumbing reference work was completed in October.

Inspection was performed on the following elements:

1. Substructure - Foundations, columns, pier caps and four protection cribs in the canal.
2. Superstructure - Structural steel, bridge seats and bearings.
3. Deck - Finished concrete roadway, sidewalks, curbs, median, handrails, joints and drainage.
4. Electrical - Lighting systems.
5. Mechanical - Traveller systems.
6. Ancillary Work - Lighting standards, signs and sign structures, emergency telephones, access manholes and ladders, and fencing.

The results of the inspection are discussed in the body of the report; locations referred to in the text may be found on drawing No. D 4346-101, in the Appendix. Photographs, charts and sketches related to items discussed are also included in the Appendix.

1. SUBSTRUCTURE

a) General

Concrete in all pier footings, columns, pier caps and protective canal cribs was found to be in excellent condition. All footings were checked for settlement, and water depths in the cellular foundations recorded. The settlement pin elevations recorded are shown in Table V in the Appendix.

b) Drainage

Considerable regrading has been recently carried out within the right-of-way from Canal Road to the west abutment to relieve the severe ponding previously reported. The area, now appears to be well graded throughout, fill having been placed adjacent to the piers and swales formed along the right-of-way.

The site was observed to be dry throughout. Since no rain fell during the inspection period it was not possible to observe the runoff pattern in the ditches and swales. Any minor deficiencies in this part of the work could however be readily detected after a heavy rainfall and adjusted accordingly.

All of the spread footings were inspected for water depth inside of the cells and very little was found except at pier W1. The following Table I shows the results of these observations.

TABLE I
DEPTH OF WATER IN FOOTINGS

PIER	DEPTH	REMARKS	PIER	DEPTH	REMARKS
W1	13'	1	W22	5"	3
W6	2"	2	W23	5"	0
W7	4"	1	W24	2"	3
W8	6"	1	W25	1"	3
W9	3"	3	W26	2"	3
W10	3"	2	W27	12"	3
W11	4"	1	W28	1'-4"	3
W12	1"	1	W29	1"	3
W13	9"	2	W30	6" Dec '71	No obser. pipe
W14	2"	2	E1	4'	1
W15	7"	0	E12	2"	1
W16	2"	0	E13	7"	1
W17	5"	3	E14	8"	1
W18	10"	0	E15	1"	1
W19	1'-3"	0	E16	4"	3
W20	3"	0	E17	0	1
W21	5"	3			

*6" Pipe installed
Dec '71*

- 1 - denotes no apparent pressure on removal of vent cap.
- 2 - denotes negative pressure in cell on removal of vent cap.
- 3 - denotes positive pressure in cell on removal of vent cap.

c) Piers W1 and E1

The timber glancing structures (bumpers) were observed to be in good condition. The massive steel plates and chains are rusting and will require a coat of paint in the future. The neoprene cushions are like new.

The water depth in the pier foundations W1 and E1 was found to be 13 feet and 4 feet respectively. When the canal is drained in the winter it is recommended that the water be pumped from the footings.

The water depth in the cells should be observed on a regular basis, and if more than 1 foot in depth and rising, should be pumped dry.

Observations on the various superstructure elements indicated no settlement of the footings at piers W1 and E1. Although no elevations were recorded at these locations during this inspection the following table is included for future reference.

TABLE II
ELEVATIONS PIERS W1 AND E1

PIER	1969	1971
W1 NE Corner	345.027	-
NW "	344.996	-
SE "	345.005	-
SW "	345.024	345.032
E1 NE Corner	345.125	-
NW "	345.130	-
SE "	345.091	345.095
SW "	345.125	-

d) East and West Abutments

Elevations on the west abutment were taken and it was found that the settlement over the past two years amounted to $1\frac{1}{2}$ inches and $1\frac{3}{4}$ inches for the south and north sides respectively. This compares to the $2\frac{7}{8}$ inches recorded during the five year period between 1964 and 1969.

Slight forward movement of the west abutment was observed, as evidenced by the gap in the finger plates and position of the rockers on pier W30.

Settlement at this time does not appear critical but should be kept under observation to detect the magnitude of further movement.

No apparent movement of the East abutment was observed. The remedial measures carried out since 1969 appear to have remedied the problems being experienced at that time. The position of the rocker bearing and the finger plate at pier E17 confirms this observation. The repairs to the ballast wall are in good condition.

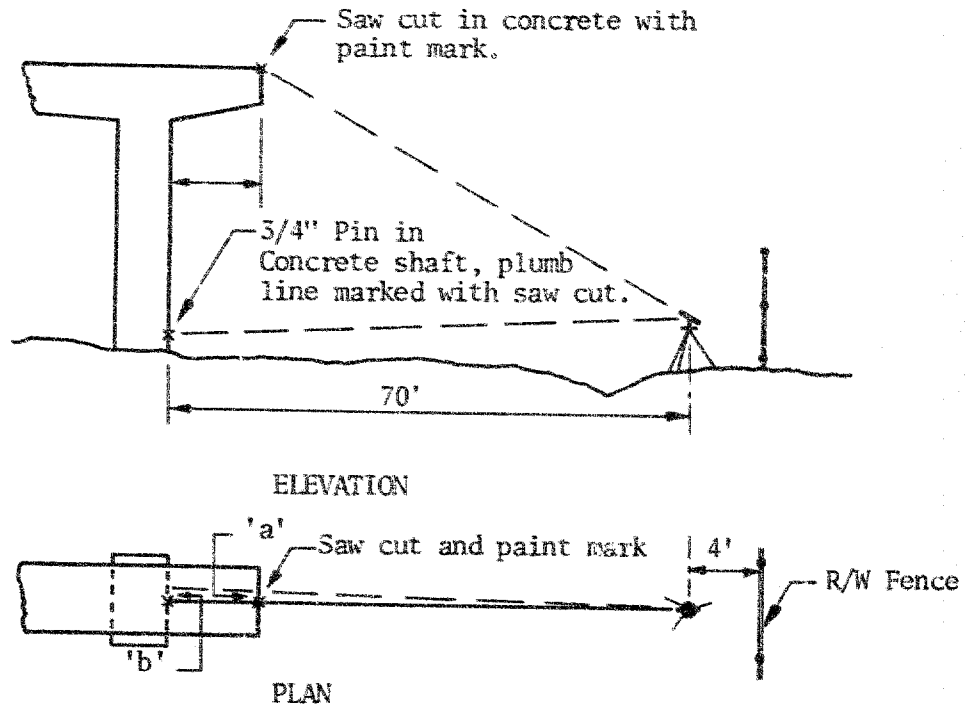
e) Pier W2

It was noted that the metal door, provided for future cable entry into the hollow pier shaft, has been broken off from its hinges, and apparently lost. Water inside the shaft should be pumped out and the opening closed with a plate bolted to the frame. Hinges are not required.

f) Pier W10 and Settlement Survey

The 1969 inspection indicated that pier W10 was leaning approximately 2 inches to the east. Our preliminary inspection in May of this year indicated that this tilting had increased slightly by $\frac{1}{4}$ to $\frac{1}{2}$ inch approximately. At that time we again recommended regrading of the area under the structure to eliminate surface water ponding as noted in our letter of June 8, 1971, appended to this report. Subsequent to this data the site on the west side of the structure has been regraded resulting in some of the reference points used for settlement readings being buried.

A survey was carried out in October to 1) establish new reference points and 2) to determine the effect of the additional fill load on the spread footings from piers W6 to the west abutment. The results of this survey are recorded on Table V in the Appendix. In addition reference marks were set up on the pier caps and shafts of piers W8, W9, W10 and W11 as noted on Figure I in order to check on any further tilting of these piers in the future.



NOTE: Read out on face of shaft, use formula $\text{Lean 'a'} = \frac{6}{7} \text{'b'}$

FIGURE I

SUPERSTRUCTURE

a) Bridge Seats

A number of the concrete bearing pedestals were originally fractured as a result of ice expansion in the anchor bolt holes during construction. This was particularly prevalent on piers W13 to W30 where high pads were required for the super-elevated deck. These cracks were previously repaired with epoxy cement, which now appears to have deteriorated permitting the cracks to open. Listed below are the pedestals which require repair or should be kept under surveillance.

The badly cracked bridge seats on piers W17, W18 and W19 should be patched with concrete and sealed against further intrusion of moisture. There appears to be sufficient bearing area available at this time to preclude the need of constructing new seats at this time. Jacking will not be required to carry out these repairs.

All bearing seats that have cracked should be sealed to prevent water seeping into the loosely packed anchor bolt holes. Sealant should also be applied on top of the plate adjacent to the anchor bolt.

TABLE III
CONDITION OF BRIDGE SEATS

PIER	PEDESTALS	COMMENTS
WA	South #1 Pad	1 small crack - no attention
W24	South #5 Pad	NE corner cracked - attention
W20	South #1 to #6	Orig. repair cracked - attention and repairs needed
W19	South #1 to #6	-do-
W18	South #1 to #6	-do-
W17	South #1 to #7	Orig. repair cracked - repair
W15	South #1 to #4	-do-
W13	South #3 to #5	Small crack N. side - no attention
*		All pads should be sealed regardless of concrete patching.

b) East Abutment Seats

The new laminated steel bearing seats were observed to be in good condition, although the outer exposed surfaces displayed more rust than anticipated for the rust resistant steel used. There appeared to be no rusting between the individual plates.

c) Rocker and Fixed Bearing Assemblies

All rocker and fixed bearing assemblies were observed to be structurally sound and free from dirt and sand as a result of good maintenance procedures. However, in areas where the joint steel in the deck has been damaged, or alternatively at the finger plates, considerable paint damage was noted and repainting is recommended.

Table VI in the Appendix shows the position of the rockers as recorded during this inspection. These measurements may be used as a guide for future observations, in respect to additional tilting of the piers.

d) Box Girders (W5 - E5)

The box girders were visually inspected both outside and from within, and found to be structurally sound.

In consideration of the recent problems encountered in certain types of box girder designs, constructed elsewhere, particular attention was paid to the inspection of the diaphragms web and stiffeners throughout the centre span and at the main bearing supports. No structural defects nor signs of distress were observed.

The problems referred to above are currently being investigated in the United Kingdom and are the subject of an interim report by Merrison entitled "Inquiry into the Basis of Design and Methods of Erection of Steel Box Girder Bridges". We expect to obtain a copy of this report in the near future and will comment on the findings as they relate to the Garden City Skyway at that time.

The floor of the open end of the girders, the exterior surfaces at their extremities and their bearing assemblies have accumulated considerable rust and require attention.

The epoxy tar paint applied to the inside of the girders was found to be in generally good condition except for a floor area of 3 to 4 square feet in the north girder at pier W1 and in isolated locations where bolts or nuts were partially rusted.

Paint applied to the exterior surface of the girders is generally in good condition however, the finish coat was observed to be peeling in several places, particularly on the underside. The light grey field undercoat, where exposed, was in good condition.

It is recommended that 1) a baffle be installed to prevent sand and dirt from accumulating between the diaphragm and the end of the girder and 2) the paint be repaired, particularly under the finger plates.

e) Plate Girders (W5 to West Abutment and E5 to East Abutment)

All plate girders and cross member connections were observed to be in structurally good condition.

Rusting was in evidence on the ends of the girders and on the bearing plate assemblies under the finger plates and most of the fixed joints. Although the remainder of the painted areas exhibited some peeling of the finish coat, the light grey field undercoat was noted to be intact. More than usual paint deterioration was observed between piers E5 to E10. It is suggested that the extremities of the girders should be repainted at this time but the balance of the painting could be deferred until a complete repainting programme is initiated.

3. DECK

a) Roadway

The concrete deck was visually inspected from the roadway surface, the top of the piers and the ground beneath.

The wearing surface was observed to be in excellent condition with the exception of some pitting or surface wear on the east bound lanes between Piers W5 and W4. This wear has not progressed to the extent where it has any significant effect on the structural strength of the slab, or the roadway surface.

b) Sidewalks, Curbs and Medians

All sidewalks, curbs and medians were found to be in excellent condition.

c) Bridge Railing

All railing posts and panels were found to be in good condition. Some intermittent rusting was noted, particularly on the posts.

d) Deck Jointsi) Finger Plates

All finger plates were found to be in good condition and free to expand or contract as intended except at pier W9 which was closed at a temperature of 60°F. At this location the concrete was in contact at the north sidewalk and at the median.

The gap at each finger plate assembly was measured at four locations and recorded in Table IV.

ii) Fixed and Construction Joints

The Rodofix joint filler in the fixed joints and the construction joints of the continuous spans at W5-W2 and E5-E2 has tended to work loose, and in spite of the waterstops in the deck, has permitted leakage to occur. No further broken joint edges were observed however, at piers W12, W18, W20 and W26.

The grooved joint faces in the deck are still in good condition thus permitting replacing of the Rodofix filler in the future. Any broken edges that might appear in the future should be repaired as soon as possible to prevent progressive deterioration since traffic tends to completely remove the spalled concrete from the joint.

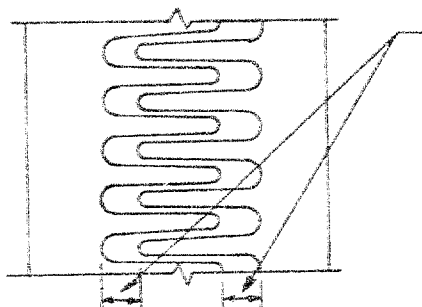
e) Drainage

All catchbasins, down pipes and troughs were found to be in good working order.

The catchbasins immediately east of Cushman Road (W5) were previously plugged to avoid brine and water from falling on the nearby homes. Drainage normally handled by the plugged system is now flowing through the finger plates at pier W5.

TABLE IV
FINGER PLATE GAPS

EAST BOUND LANES			WEST BOUND LANES		Temp.
Pier No.	Near Sidewalk	Near Median	Near Median	Near Sidewalk	
W30	2 1/2"	2 5/8"	2 1/2"	2 3/4"	60°F
W29	2 5/8"	2 5/8"	2 5/4"	2 5/8"	60°F
W27	3"	3"	2 5/8"	2 3/4"	60°F
W25	2 3/4"	2 5/8"	2 5/8"	2 3/4"	60°F
W23	2 3/4"	2 3/4"	2 5/8"	2 5/8"	60°F
W21	2 1/2"	2 1/2"	2 5/8"	2 1/2"	62°F
W19	2 3/4"	2 3/4"	2 5/8"	2 7/8"	62°F
W17	2 3/4"	2 3/4"	2 3/4"	2 3/4"	62°F
W15	2 1/8"	2 1/4"	2 1/4"	2 1/4"	62°F
W13	2 3/8"	2 1/2"	2 1/8"	2 3/8"	64°F
W11	5 1/4"	5 3/8"	5 1/4"	5 1/8"	64°F
W9	1"	1" Med. Conc. Closed	5/8"	3/4" S.W. Closed	64°F
W7	2 3/4"	2 3/4"	2 3/4"	3"	68°F
W5	5"	3 3/8"	3 3/4"	4 1/8"	68°F
W2	5 3/4"	6 1/8"	6 1/8"	5 5/8"	68°F
E2	6 1/4"	5 3/8"	5 3/4"	5 1/4"	72°F
E4	2"	2 1/8"	2"	2 1/2"	72°F
E6	2 3/4"	2 7/8"	2 7/8"	2 7/8"	72°F
E8	3 1/8"	3 1/8"	3 1/4"	3"	72°F
E10	2 3/8"	2 5/8"	2 5/8"	2 5/8"	72°F
E12	4"	3 1/4"	3 1/8"	3 1/8"	72°F
E14	2 3/8"	2 1/4"	2 1/8"	2 1/8"	70°F
E16	3 1/2"	3 1/8"	2"	2 3/4"	68°F
E17	2 1/2"	2 1/2"	2 1/4"	2 5/16"	68°F



Average gap measurement
taken near curb.

4. ELECTRICAL

The luminaires were observed at night and found to be in satisfactory working order.

It was noted that birds continue to enter the substations located under the deck at piers W13 and E8. The presence of combustible material brought in by them is a potential fire hazard and it is therefore suggested that all possible entry points be sealed with metal screening.

The navigation lights on the W1 and E1 pier footings and on the deck at mid span were observed to be in good working order.

The ground wires at piers E-2, 4, 8, 12 and 16 and W-2, 5, 9, 13, 17, 21, 25 and 29 should be tested and repaired if necessary.

5. MECHANICAL

The track, hanger rods and rod connections for the three travellers were observed from the tops of the piers where access was readily available. They were found to be in satisfactory condition although some rusting was apparent at the expansion joints.

The travellers have been parked at pier W5 and as a result exposed to the drainage passing through the finger plate in the deck above. This has resulted in a general deterioration of the motor-generator, control and power panel. Continued exposure in the same environment will probably lead to a complete loss of the power and drive system very shortly.

During this inspection only the centre traveller was barely useable after some initial repairs. The straight forward and reverse drive switches were working but the jogging switches were out of order. The motor could not be readily started and it could only be shut off by manually stopping the gas flow to the pumps.

It is recommended that the centre traveller be put in useable order with a power drive system and used exclusively for major repainting programs. Access to the underside of the deck for inspection purposes etc. could be accomplished by a series of existing and new manholes. The following should be carried out to meet these requirements.

- a) Immediately remove the motor-generator set and power and drive panel and overhaul in the shop.
- b) Repair the lateral movement drive and carriages. It was not possible to move laterally, with the hand chain drive, since it appears the frame is bent.
- c) Modify power panel and drive panel assembly so that panels can be easily enclosed in a moisture proof wrapping when not in use.
- d) Extend exhaust pipe downwards so that exhaust gases are not discharged into the face of the operator.
- e) Leave traveller at either pier W4 or pier E5 where it will be protected from deck drainage.
- f) Remove entire power drive equipment from the side travellers to reduce weight and permit relatively simple manual operation by hand cranking. The cranks on these carriages should be modified by synchronizing them on a shaft positioned at a more suitable height for hand operation.
- g) Access to the tops of the piers can be provided by:
 - i) W5, W2, E2 and E5 - already existing.
 - ii) W4, W3, E3 and E4 - toe bars need to be bolted to pier caps to restrain portable extension ladder that would be used to gain access through existing median manholes.
 - iii) W1 and E1 - cut median and install manhole frames and covers, and bolt toe bars to pier caps to restrain portable extension ladders to be used for access.

Installation of these manholes at W1 and E1 will necessitate cutting two of the twelve unused cable ducts. If cables are ever installed they can be readily protected by steel covers in the manhole.

6. ANCILLARY WORKSa) Lighting Standards

All of the lighting standards were inspected and found to be in satisfactory condition.

b) Signs and Sign Structures

All signs and sign structures were found to be in satisfactory condition.

c) Emergency Telephones

The emergency telephones were used and found to be good working order.

d) Fencing

The right-of-way fencing was found to be in satisfactory condition throughout.

e) Approach Roadway

Some dishing of the pavement surface, located on fill near the west abutment, was noted and will ultimately need to be corrected.

The new 4 inch neoprene expansion joint installed in the pavement some 70 feet east of the East abutment appears to have worked loose permitting the slabs to move vertically. Cores should be taken through the slab to determine the extent of the void and subsequent corrective repairs made by mud jacking or pressure grouting.

SUMMARY OF RECOMMENDATIONS

The following summarizes recommendations and suggestions brought forward in the body of this report.

1. Continue to observe water depth in footing cells and pump if more than 1 foot accumulates. Since the footing at pier W1 is prone to leaking it is suggested that it need not be pumped except prior to winter when the canal is about to be drained.
2. Continue to observe settlement at east and west abutments.
3. Replace metal door at pier W2.
4. Repair bearing pedestals.
5. Clean and repaint bearing assemblies as noted.
6. Paint structural members as noted.
7. Consider installation of baffle in open ends of box girders to keep out dirt accumulation.
8. Continue to observe finger plate openings and coordinate observations with plumbing data on the pertinent piers using new reference marks.
9. Repair the joint filler in the fixed and construction joints.
10. Continue to remove all ice and snow from entire finger plate assemblies during winter ploughing operations.
11. Repair and modify travellers.
12. Provide for access to pier caps from deck for routine inspection etc.

In general it is recommended that a program for regular inspection, similar in scope to the present one be implemented in order to minimize possible costly remedial measures in the future.

APPENDIX

TABLE V
SETTLEMENT PIN ELEVATIONS
GARDEN CITY SKYWAY

PIER	SOUTH COLUMN				REMARKS
	1964 El.	1969 El.	1971 El.	Diff. '64-'71	
W 6	343.233	3.290	343.312	+.079	
W 7	342.841	2.855	344.717	-	New pin in '71
W 8	342.620	2.629	342.622	+.002	
W 9	342.318	2.318	344.901	-	New pin in '71
W10	342.102	2.159	342.166	+.064	
W11	341.798	1.806	341.804	+.006	
W12	341.407	1.408	341.402	-.005	
W13	339.944	9.947	339.942	-.002	
W14	339.226	9.234	339.226	0	
W15	339.071	9.075	339.073	+.002	
W16	-	342.161	342.160	-.001	Diff. '69-'71
W17	341.391	1.400	341.398	+.007	
W18	342.087	2.093	342.090	+.003	
W19	340.186	0.194	340.860	-	New pin in '71
W20	341.462	1.469	341.463	+.001	
W21	336.545	6.518	342.808	-	New pin in '71
W22	337.056	7.052	341.300	-	New pin in '71
W23	337.851	7.850	337.843	-.008	
W24	337.497	7.493	337.489	-.008	
W25	336.535	6.529	338.933	-	New pin in '71
W26	336.584	6.530	339.537	-	New pin in '71
W27	337.012	7.014	336.976	-.036	
W28	336.243	6.256	338.376	-	New pin in '71
W29	336.713	6.705	339.050	-	New pin in '71
W30	335.840	5.835	337.487	-	New pin in '71
Abut	352.717	2.481	352.469	-.248	

Reference B.M. Elev. 343.750 - S.E. corner of Test Pile on ϵ between Piers
W3 and W4.

TABLE V
SETTLEMENT PIN ELEVATIONS
GARDEN CITY SKYWAY

PIER	NORTH COLUMN				REMARKS
	1964 E1.	1969 E1.	1971 E1.	Diff. '64-'71	
W 6	343.057	3.070	343.078	+.021	
W 7	342.857	2.857	342.856	-.001	
W 8	342.566	2.564	342.550	-.006	
W 9	342.246	2.245	342.245	-.001	
W10	341.982	342.017	342.019	+.037	
W11	341.758	1.764	341.761	+.003	
W12	341.425	1.425	341.419	-.006	
W13	339.948	9.955	339.951	+.003	
W14	339.333	9.344	339.340	+.007	
W15	339.056	9.063	339.061	+.005	
W16	342.083	2.088	342.084	+.001	
W17	341.434	1.445	341.439	+.005	
W18	342.027	2.031	342.024	-.003	
W19	340.198	0.208	340.197	-.001	
W20	-	341.479	341.470	-.003	Diff. '69-'71
W21	336.570	6.533	340.873	-	New pin in '71
W22	336.970	6.968	341.437	-	New pin in '71
W23	338.016	8.017	338.010	-.006	
W24	337.536	7.537	337.528	-.008	
W25	336.575	6.563	339.048	-	New pin in '71
W26	336.571	6.566	339.683	-	New pin in '71
W27	337.321	7.325	337.310	-.011	
W28	336.228	6.226	338.904	-	New pin in '71
W29	336.606	6.603	338.037	-	New pin in '71
W30	335.966	5.960	338.643	-	New pin in '71
Abut	352.440	2.261	352.247	-.193	

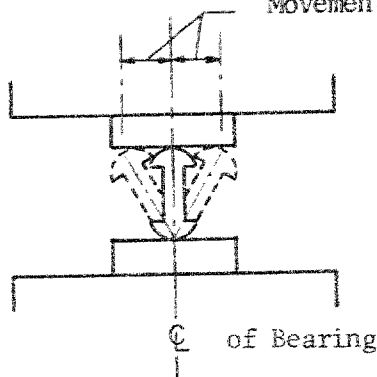
Reference B.M. Elev. 343.750 - S.E. corner of Test Pile on E between Piers W3 and W4.

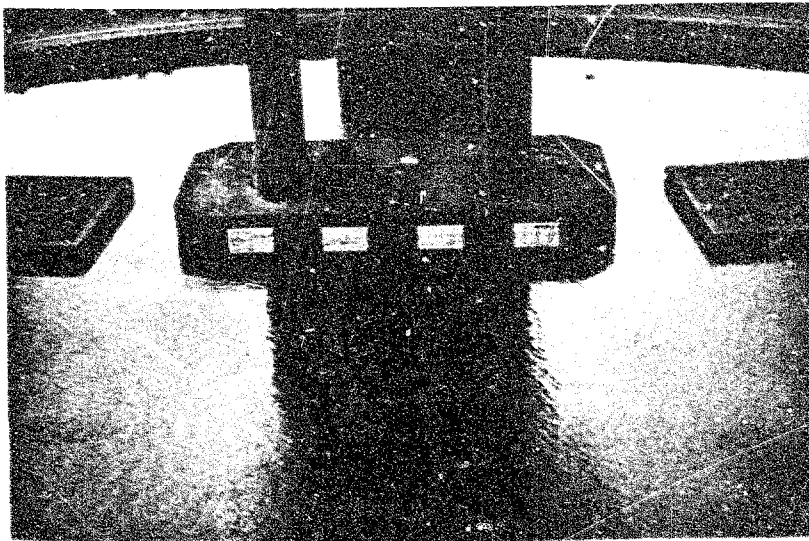
TABLE VI
POSITION OF ROCKER BEARINGS
October 1971

(Bearings measured are adjacent to centre line of bridge)

PIER NO.	NE BEARING	SE BEARING	NW BEARING	SW BEARING	TEMP.
W30	F	F	7/8" East	7/8" East	60°F
W29	1/4" West	3/8" East	3/8" West	3/8" East	60°F
W27	0	0	1/8" West	1/8" West	60°F
W25	1/4" East	1/8" West	1/8" West	1/4" East	60°F
W23	1/4" West	0	1/8" East	1/8" East	60°F
W21	1" West	1" West	1/8" West	1/2" West	62°F
W19	1/8" West	1/8" East	1/2" East	1/4" East	62°F
W17	1/8" East	1/4" East	3/4" East	3/4" East	62°F
W15	3/8" West	3/8" West	1/8" East	1/8" West	62°F
W13	1-1/8" West	1-1/8" West	1/4" West	5/8" West	64°F
W11	2" East	2-1/8" East	1/4" East	1/8" West	64°F
W 9	1/4" East	1/8" West	2-5/8" East	2" East	64°F
W 7	1-5/8" East	1-1/2" East	1-1/2" East	1-3/4" East	68°F
W 5	3/4" East	1/8" East	1-1/8" East	1" East	68°F
W 4 cont.span	F	F			
W 3 cont.span	N. 1/4" East	1/4" East	-	-	50°F
W 1 cont.span	F	F	-	-	
E 1	N. 3/4"	S. 1/2" East	-	-	52°F
E 2	0	1/4" East	1/4" East	1/8" East	68°F
E 3	F	F	-		
E 4	3/8" East	0	5/8" West	1/2" West	50°F
E 6	1" West	1" West	1/2" East	1/2" East	72°F
E 8	5/8" West	3/4" East	3/8" East	5/8" East	72°F
E10	7/8" West	1-1/8" West	5/8" East	5/8" East	72°F
E12	1-5/8" East	1-3/8" East	2" East	1-1/2" East	72°F
E14	5/8" West	5/8" West	1-1/16" East	1/8" East	70°F
E16	0	0	1/2" East	3/8" East	68°F
E17	1/2" West	1/2" West	F	F	68°F

Movement East or West

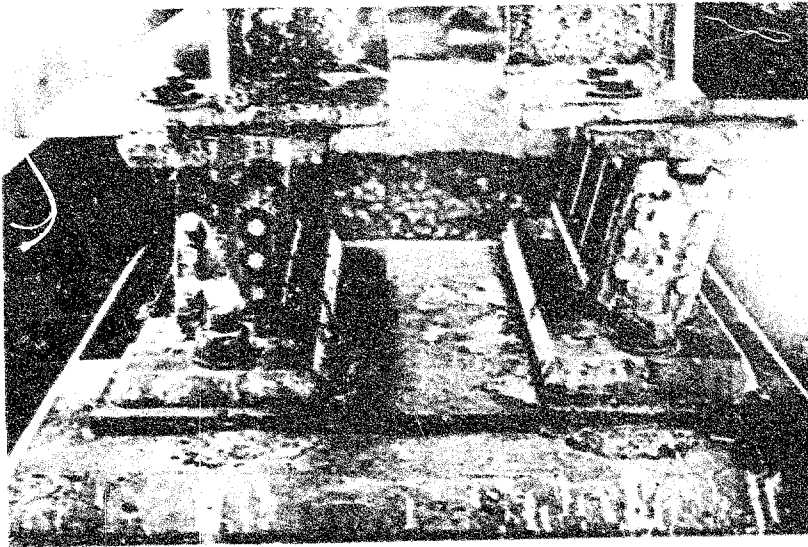




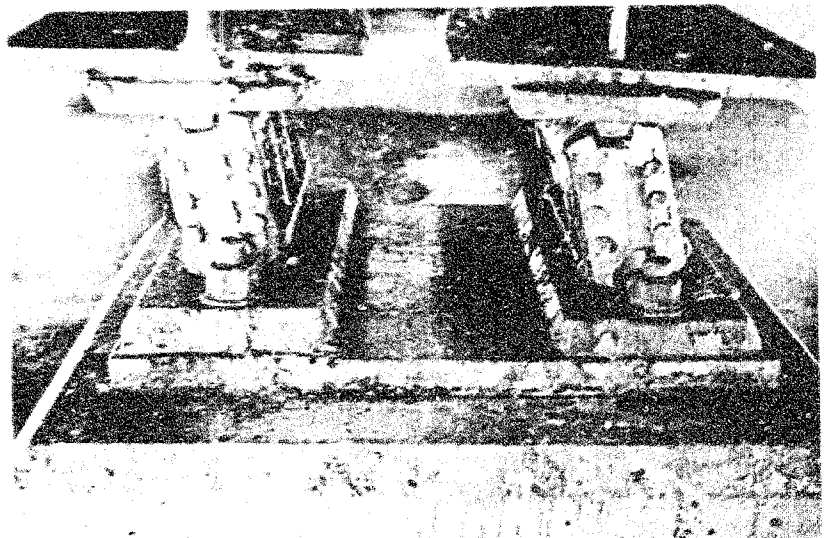
No. 1
CRIBS AND BUMPERS
IN GOOD CONDITION



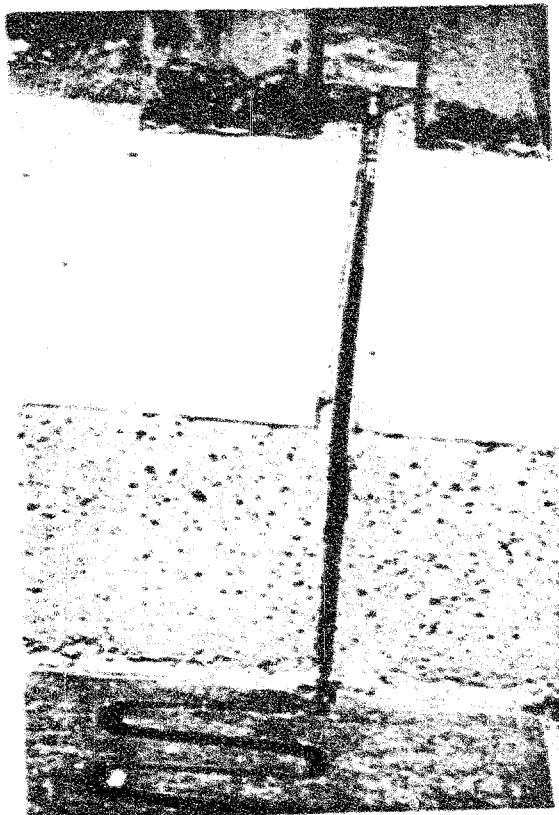
No. 2
BROKEN BRIDGE SEAT
TYPICAL FOR PIERS W-17, 18, 19



No. 3
BEARINGS
ON PIER W-II
LOOKING NORTH

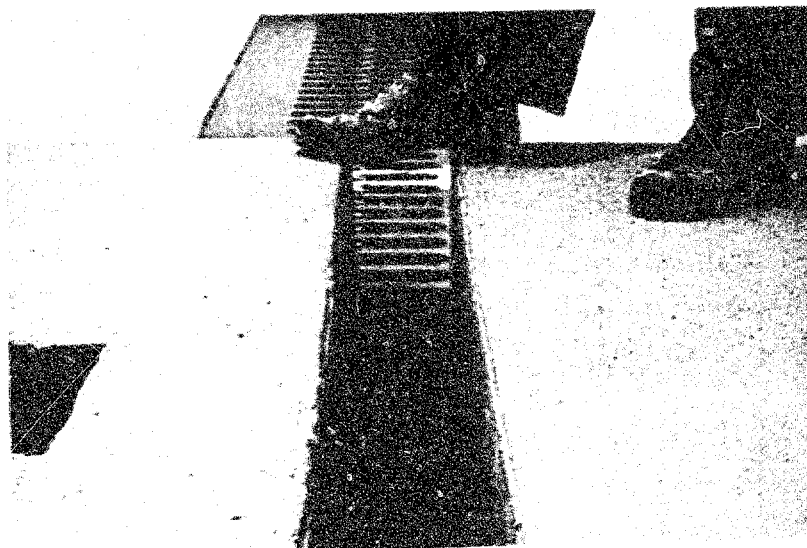


No. 4
BEARINGS
ON PIER W-13



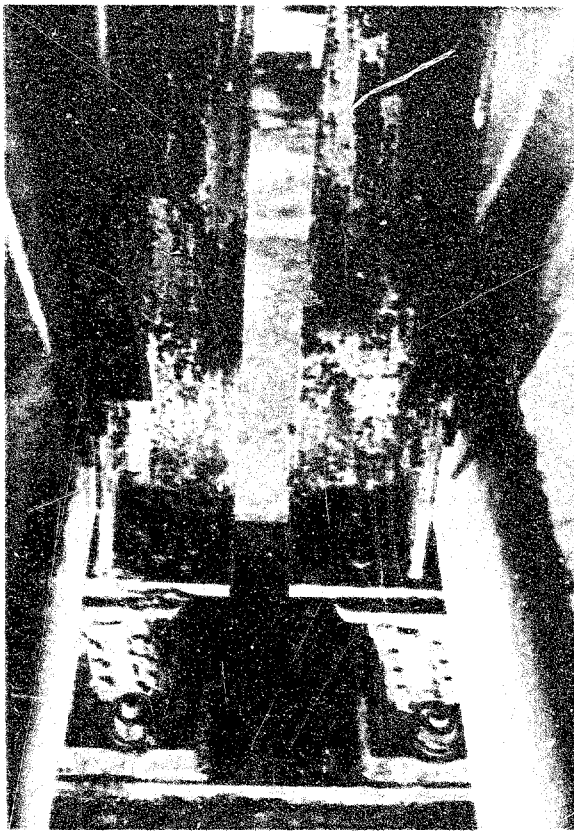
No. 5

NORTH SIDEWALK GAP
CLOSED AT PIER W-9 AT 63°F



No. 6

MEDIAN GAP
AT PIER W-11



No.7

TYPICAL OF RUST
UNDER FINGER PLATES

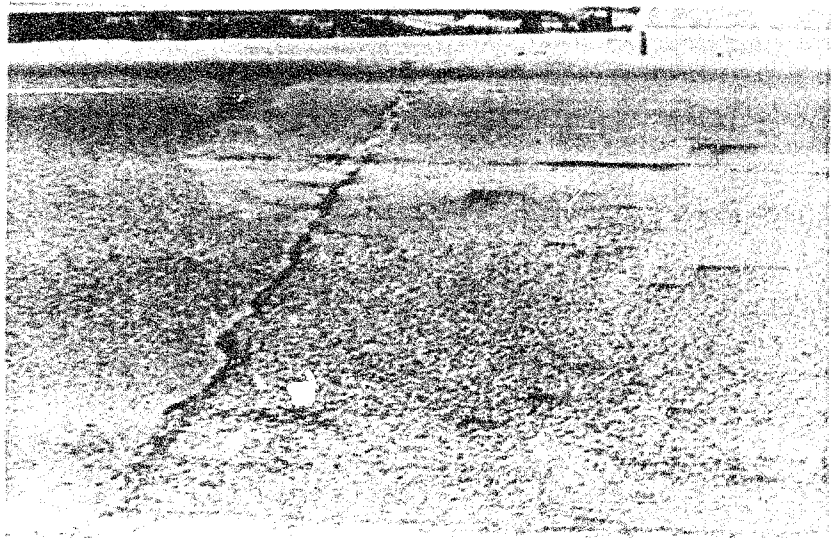
No.8
ORIGINAL SHRINKAGE CRACKS
ON UNDERSIDE OF
CENTRE SPAN DECK
NEARLY SEALED





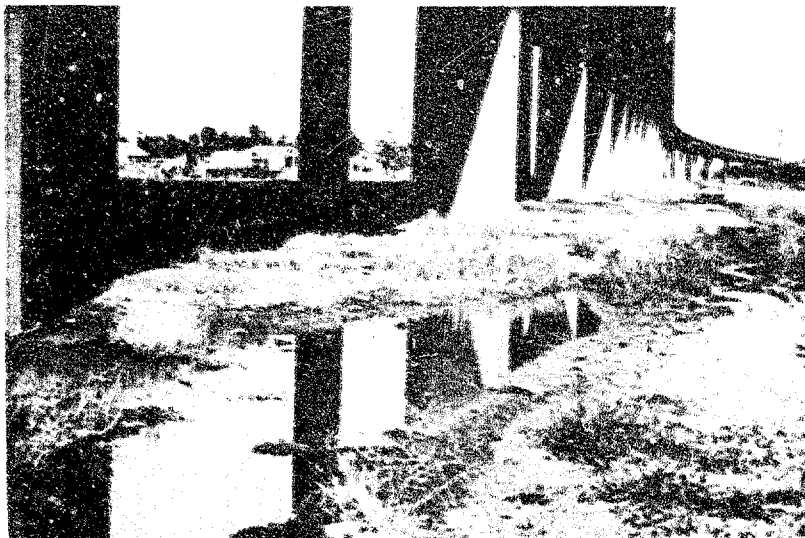
No. 9

LOOSE RODOFIX
IN FIXED JOINT

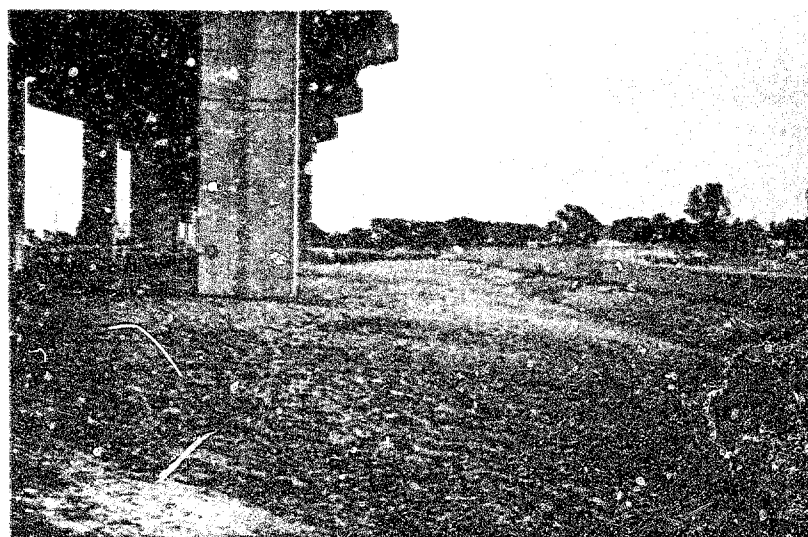


No. 10

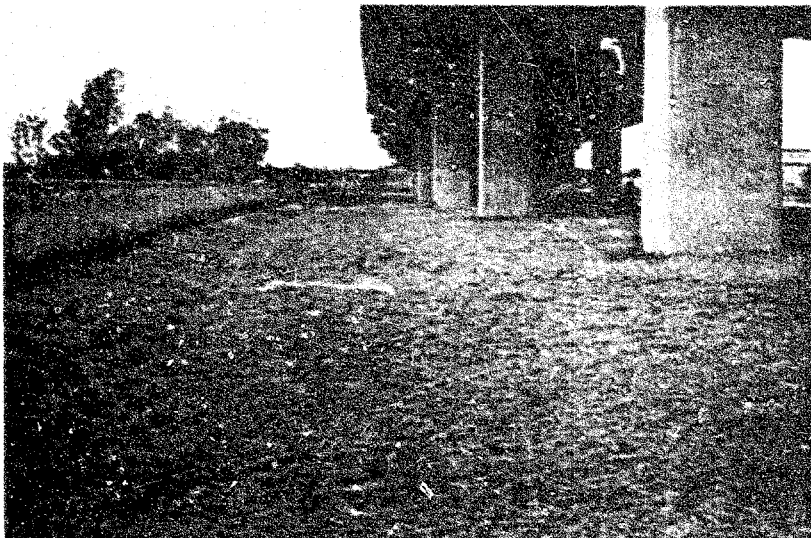
WEST APPROACH SLAB
SETTLED-JOINT CRACKED



No. 11
PIER W-10
BEFORE GRADING

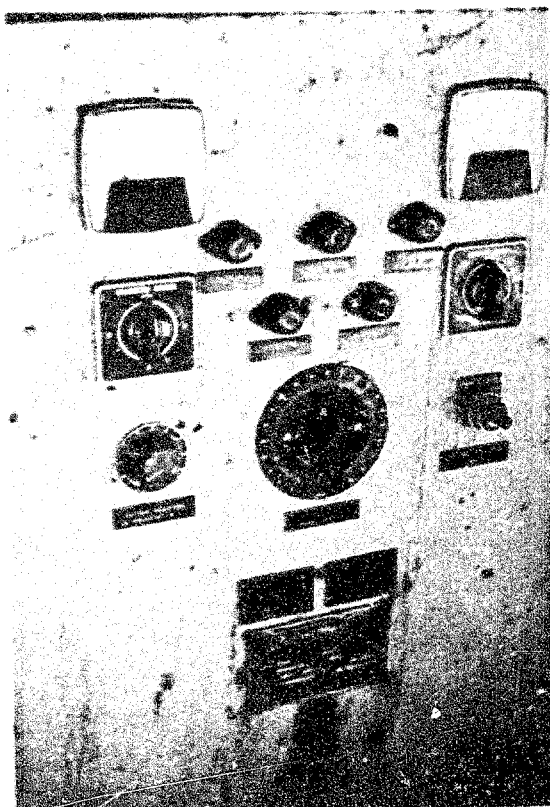


No. 12
RECENTLY GRADED AREA
UNDER STRUCTURE



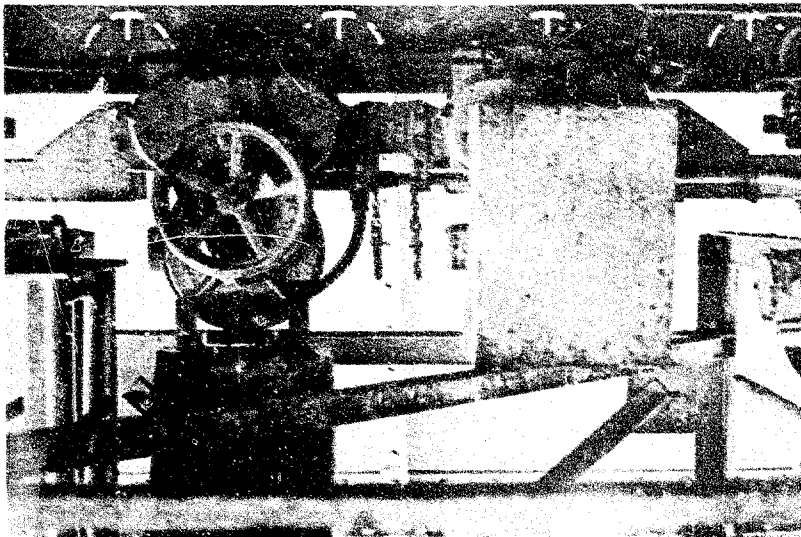
No.13

RECENT GRADING
SOUTH OF STRUCTURE
NEAR WEST ABUTMENT



No.14

TRAVELLER POWER PANEL
RUSTING



No. 15
CENTRE TRAVELLER
DRIVE MOTORS
AND HAND CRANK



No. 16
LATERAL MOVEMENT
CARRIAGE
PRESENTLY INOPERATIVE

FENCO

1 Yonge Street
Toronto Canada
416-361-4722
Cable: Foundationeng
Telex: 02 2814

June 8, 1971

Department of Highways, Ontario
Bridge Maintenance Section
Administration Building
Downsview 464, Ontario

Attention: Mr. W.D. Birch, P.Eng.
Bridge Maintenance Engineer

Dear Sirs,

GARDEN CITY SKYWAY
FIELD INSPECTION PIER W10

In response to your telephone request of May 20, 1971 we have carried out a field inspection of the above structure in the area of Pier W10, with particular reference to the effect of the tilting and settlement of the pier. The position of the expansion joints and expansion bearings at the adjacent Piers W8 to W13 inclusive were observed.

The problem of settlement and tilting of Pier W10 was noted in the Garden City Skyway 1969 Inspection Report carried out by FENCO. At that time, it was observed that the pier had tilted approximately 2" at the top towards the east. A soil investigation was carried out by Geocon Ltd. and it was determined that there had been a rise in the ground water level in the clay underlying the footing. This had resulted in a swelling action causing lifting and tilting of the footing. The effect of the tilt was to move the bridge superstructure to the east, decreasing the gap in the finger plate expansion joint at Pier W9 and increasing the gap at Pier W11. At that time the tilt was such that there was still enough residual gap at the Pier W9 expansion joint to accommodate anticipated temperature movements. The expansion bearings at these locations also indicated the movement by leaning excessively in the direction of shift. The 1969 Report recommended that the pier be left in its tilted condition and the area under the bridge be re-graded and drained to improve the surface drainage, resulting in a lower ground water table and possible reversal of the movements.

../2

Our recent inspection carried out on May 20, 25 and 26 revealed that the tilting action of the pier has continued, resulting in a further lateral movement at the deck level of approximately 1/4 to 1/2 inch. The area under the deck has not been drained and surface water has flooded the area around Pier W10. At an ambient air temperature of 63°F on May 20, the expansion joint at Pier W9 was closed at the north sidewalk and centre median and very slightly open (1/4" approximately) at the south sidewalk. At 50°F on May 26, the expansion joint was approximately 1/4" open at the north sidewalk and median and 1/2" at the south sidewalk. The sidewalks and the median gaps are approximately 1/2" less than the deck finger plate openings and so close before the finger plates bear.

Precise levels were taken at previously established points on the footing of Pier W10 and compared with recorded levels taken in 1969 and earlier. Since 1969, the footing has risen an average of 0.006 feet and the differential vertical movement between the east and west side is 0.006 feet.

The effect of the closing of the expansion joint at Pier W9 at temperatures greater than 63°F will be to exert lateral forces at the fixed bearings of Piers W10 and W8 and the expansion movement can be absorbed by flexing of the pier shafts. The amount of flexure required to accommodate the anticipated temperature movements is such that it will not seriously overstress the reinforced concrete shafts. It is advisable, however, that this action be carefully observed, particularly during periods of extreme high temperatures.

The position of the expansion bearings on the west side of Pier W9 reflects the shift in the deck towards the east. The bearing movement when the deck expansion joint is closed is presently 2-7/8 inches out of a possible 5 inches of travel. The closing of the expansion joint should prevent any further movement of the bearings unless Pier W8 is pushed over by the expansion forces.

In view of the above noted conditions we strongly recommend that immediate measures be taken to improve the drainage of the area under the bridge as recommended in our 1969 Inspection Report. The improvement of the drainage should alleviate the conditions causing the pier to tilt. In addition, marks should be set up on Piers W8, W9 and W10 to check future tilting of these piers by plumbing with a transit. These marks will be set up by FENCO as part of our 1971 inspection to be carried out in the near future.

We trust that this letter adequately answers your request. We would be pleased to offer our services to design the above mentioned remedial

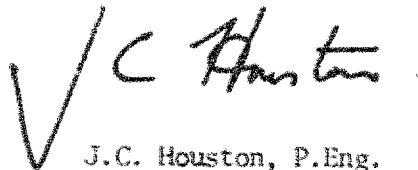
Department of Highways, Ontario

June 8, 1971

Page 3

measures if requested. We would also be pleased to investigate other measures such as jacking of the superstructure to restore the proper gaps in the deck expansion joints and to take up the movement in the expansion bearings.

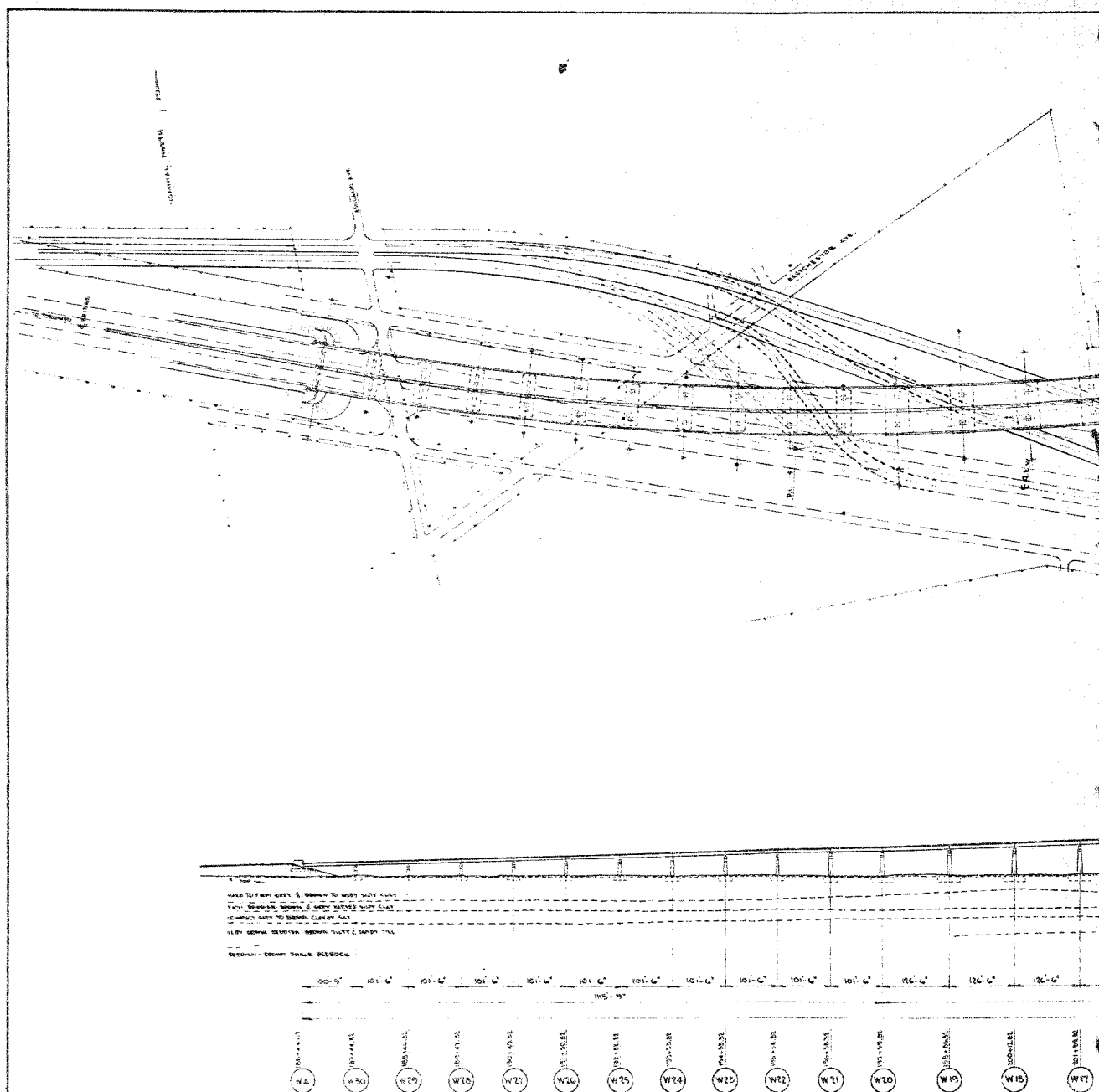
Yours very truly,
FOUNDATION OF CANADA ENGINEERING
CORPORATION LIMITED

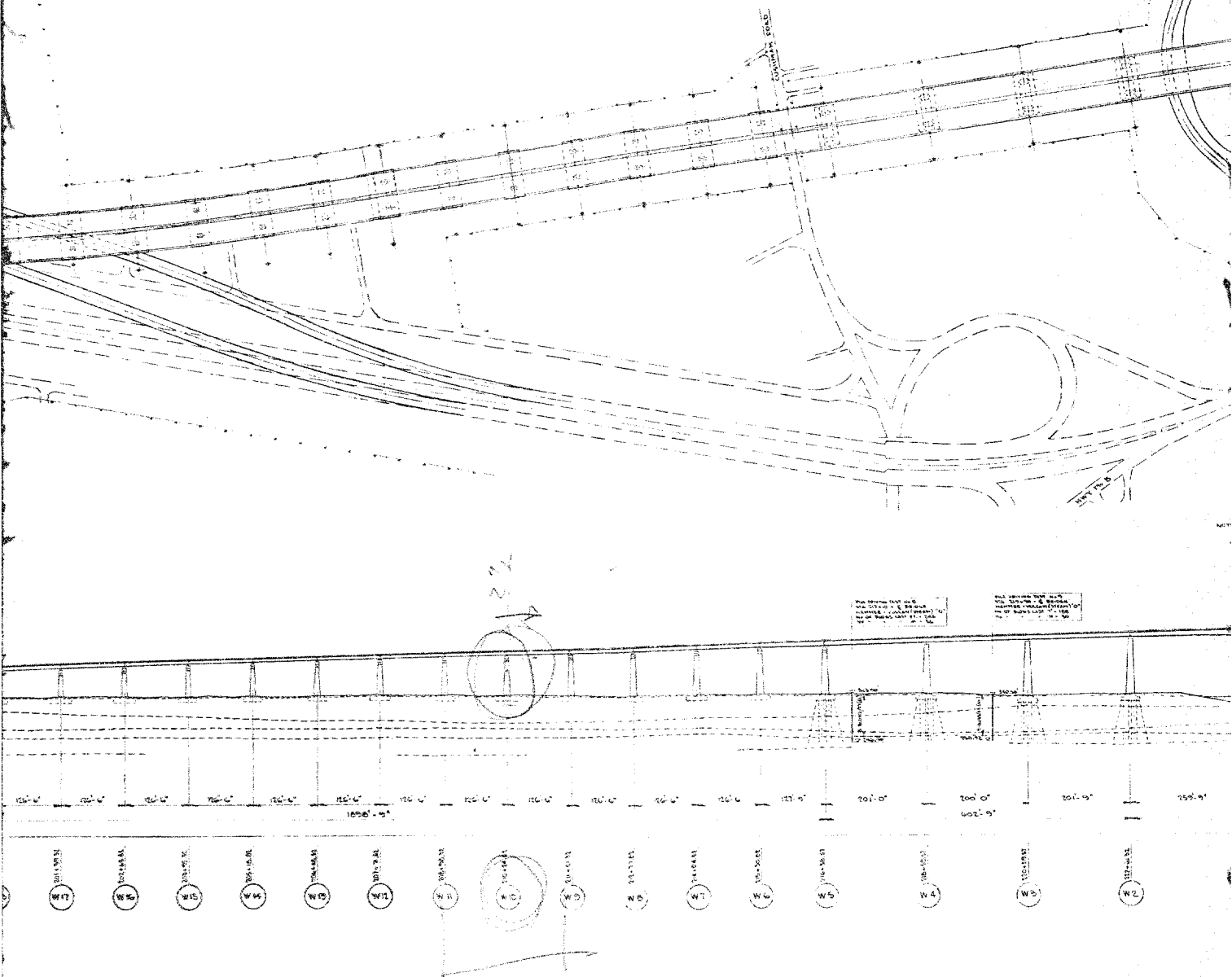
A handwritten signature in dark ink, appearing to read 'J.C. Houston', is written over a large, stylized checkmark.

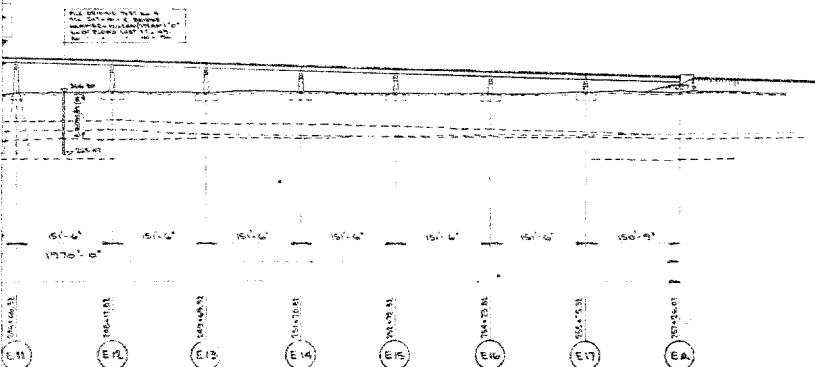
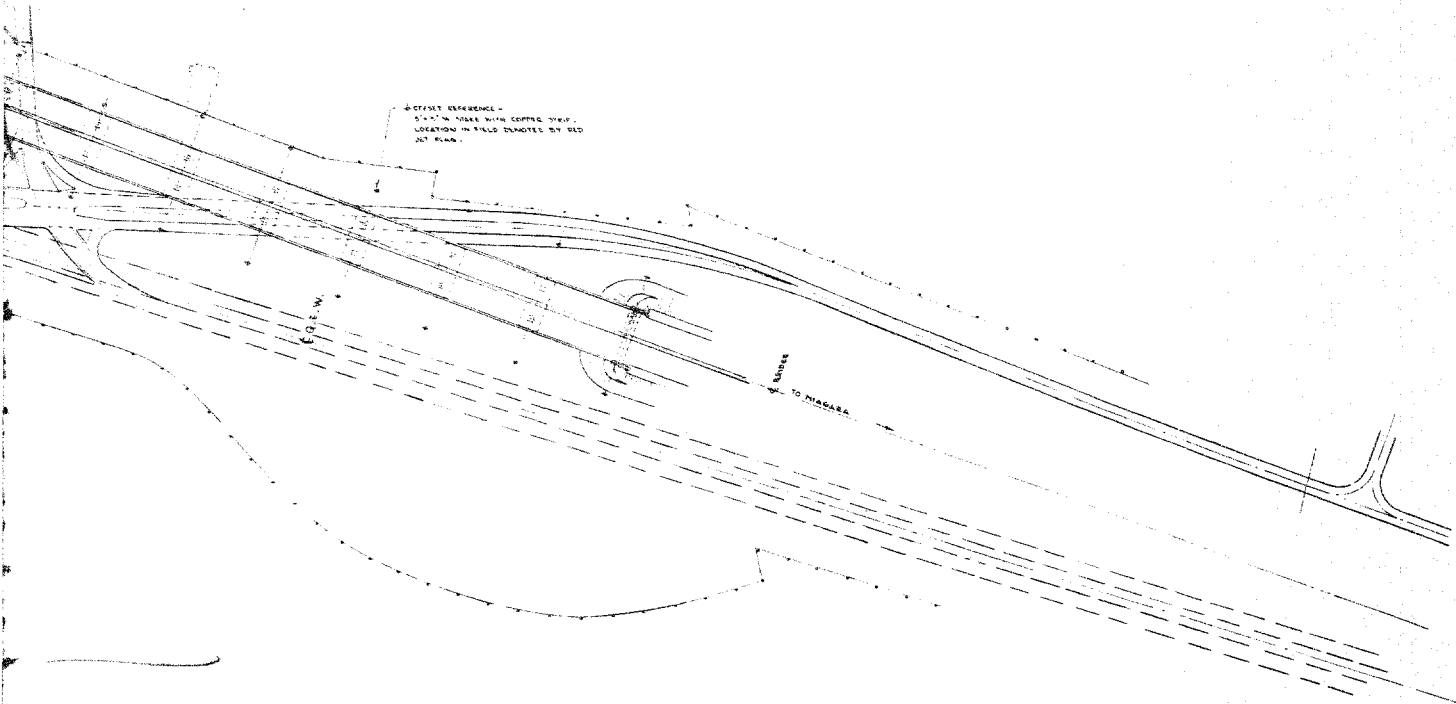
JCH/dlm
3569

J.C. Houston, P.Eng.

30 1000000 1000000
12 1000000 1000000







FOUNDATION OF CANADA ENGINE CORPORATION LIMITED TORONTO			
DEPARTMENT OF HIGHWAYS - ONT. BRIDGE OFFICE - TORONTO			
HOMER BRIDGE OVER THE WELLAND CANAL			
THE GUYEN ELIZABETH WAY C.D. LINCOLN TYPED BY BRANTHAM LOT CO.			
GENERAL PLAN			
APPROVED <i>[Signature]</i> DATE: MARCH 1960			
REFERENCE PLANS			
DESIGN	ENGINEER	CORPORATION	NUMBER
DRAWING	V. W.	ONCE	1500
TRACED	ONCE	1500	1500
DATE	MARCH 1960	1500	1500

DATE	BY	DESCRIPTION

