DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac
Principal Foundation Engineer,
Materials & Research Section,
Lab. Bldg.,

From: G.C.E. Burkhardt

Bridge Division,

Date: January 23, 1963.

Our File Ref. BA 1562

In reply to

Subject: Township of Windham,
Bridge over Big Creek,
Lot 11, Con III/IV,
County of Norfolk
Structure Site # 21-7

Attached please find one copy of the Foundation Report by William A. Trow & Associates Limited, and one copy of the Final Plans for your comments.

We intend to approve the plans as soon as possible. Therefore we would appreciate it very much, if we could have your comments at your earliest convenience.

G.C.E. Burkhardt

G.C.E. Burkhardt,
for K.L. Kleinsteiber,
Municipal Bridge Liaison Engineer.

GCEB/dm

No comment
advised by phone

[illegible]
23/1/63

WILLIAM A. TROW AND ASSOCIATES LTD. BA-1562

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J798

January 26, 1962

McDowell and Jewitt,
Consulting Engineers,
92 Kent Street South,
Simcoe, Ontario

62-F-304M

Attention: Mr. McDowell

Re: Foundation Conditions
Proposed ~~County~~ Twp Road Bridge Replacement, Big Creek
Between Concessions III and IV, West of Vanessa

Dear Sirs:

We have completed an investigation of the subsoil conditions existing at this site. The field work consisted of 2 borings taken to a maximum depth of 91 feet below the ground surface.

The factual information obtained from this survey and a discussion of the capacity of the soil at various possible foundation levels are presented in detail in following sections of this report. The main observations and conclusions of these sections briefly are as follows:

1) The existing bridge is underlain by a stratum of loose to medium dense sand of fine to medium grading which extends to a depth of about 45 feet below the river bed. Below this sand is a deposit of very stiff to stiff reddish grey clay till of low to medium plasticity. This stratum was penetrated to a depth of 91 feet without encountering any refusal condition. No well drilling records are available in this area for the determination of bedrock levels.

2) Although other foundation schemes could be considered, the support of the bridge structure on simple Class B wood piles driven about 40 feet below the creek bed probably is the most practical proposal. The estimated safe capacity for one pile driven to this depth is 15 tons. It is not anticipated that the piles will encounter refusal at this 40 foot level.

3) The other alternatives include Franki type piles driven to 12 feet below creek level at which depth a safe load of 100 tons can be applied; cylindrical steel piles driven to a depth of 90 feet where a safe load of 35 tons can be developed; abutments supported 2 feet below creek bed level utilizing a safe bearing pressure of 3000 psf.

4) The sand banks and creek bed are quite susceptible to erosion, as evidenced by undermining of some trees that grow adjacent to the river. Because of this, the base of the abutments and tops of the piles must be protected with light steel sheet piling.

Site Description

In this north part of Norfolk County, the flood plain and valley of Big Creek is shallow and wooded. The surrounding farm land is relatively level. The river channel meanders through its flood plain and, at this bridge location, its course is in a southerly direction. Constrictions in the river exist just to the north and south of the bridge. During Spring flooding periods, the flow through these narrow river sections has been described as fast. It is understood that the water flows over the banks when the river is in flood. The banks range from about 1 to 4 feet high. Some of the trees adjacent to the creek have been undermined by erosion.

The existing bridge is a timber trestle structure, having a width of 63 feet between abutments. Photographs of the bridge are presented in the report. The river widens in the immediate vicinity of the bridge and it was about 1 foot deep at the time of the investigation. Just over 2 feet of soft sludge covers the underlying sand in the creek bed.

The earth fill road approaches adjacent to the bridge, are supported with timber sheeting. The road and bridge deck lie about 11 feet above the river surface. Since the ground and river were covered with snow and ice, the condition of the lower levels of the piling and abutments was not readily discernable.

Photographs of the site are included in this report.

Geology

This site lies in the Norfolk sand plain, which is a deltaic deposit emptied into glacial Lakes Whittlesey and Warren by the Grand River during the early stages of the glacial retreat from this part of Ontario. This sand has buried much of the moraine material left by the ice, although some outcrops of glacial debris project above this delta deposit.

Investigation Procedures and Subsoil Description

Conventional wash boring methods were used in this investigation. BX casing was driven to the desired sampling depth, the soil enclosed in the casing was washed out and a sample was obtained. The casing was always kept full of water during the withdrawal of wash rods and sample tubes in order to maintain a balance with the water pressure existing at sampling level.

Samples, for the most part, were recovered in the disturbed state, using a thick-wall two inch O.D. split spoon. It was driven into the soil below the bottom of the casing using an energy of 350 ft. lbs. per blow. One open drive shelby tube sample was recovered in the top of the stiff clay at 52 feet in hole 2 so that a laboratory measurement of the undrained shear strength of the clay could be made. The clay till was found to be too stiff for field vane measurements of shear strength.

Hole 2 was sampled to a depth of 72 feet and then the boring was extended by washing ahead to 91 feet. The purpose of this was to determine if any very hard stratum lay within reasonable reach of end-bearing piles. No refusal condition was encountered.

Hole 1 was terminated in the clay at 61 feet and a cone was driven below this level to 66 feet. Beside each boring some penetration tests were made through the upper sand stratum as well, in order to obtain a check on penetration resistance measurements obtained with the split spoon.

As indicated in the opening paragraphs of this report, the first approximately 45 feet of soil below river bed level consists of fine to medium sand that exists in a loose to medium dense condition. The penetration resistance of the sand was in the order of 20 blows per foot. Cone penetration measurements indicated a denser condition below about 24 feet from the river bed, but this possibly reflects the presence of occasional thin seams of clay interbedded with the sand. The penetration resistance was found to be somewhat higher in hole 2 than in hole 1.

Because the sand does not appear to be dense enough to offer refusal to a displacement pile, it was decided, in hole 2, to perform tests which would give some indication of the friction resistance and end-bearing values that may be developed in it. These tests took two forms. One involved a measurement of the total load required to force a ½ inch diameter rod into the undisturbed sand below the casing. The end-bearing properties of the soil were determined by equating this result to recognized bearing capacity formulae. An indication of the friction force developed between steel and sand was obtained by measuring the force required to withdraw the rod from the sand.

The results of these tests are summarized in Table 1. In one instance, at 15 feet, the sand was too dense to permit penetration of the rod. However, lower down, below about 24 feet, measurements were obtained which seemed reasonable for this sand. A bearing capacity factor $N_q = 50$, approximately, was obtained which is equivalent to an angle of internal friction of 36 degrees. The corresponding friction coefficient between the sand and the rod was computed to be about 0.3. However, the results of this latter testing were somewhat erratic.

In order to obtain a check on these measurements, four attempts were made to obtain undisturbed samples of the sand using a fixed piston sampler. If successful, measurements of in-place density and angle of internal friction could be made on the recovered specimens. Unfortunately, because sand is extremely difficult to sample below the water table, only one good specimen was recovered at a depth of 24 feet. This sample was carefully waxed and then, after transportation to the laboratory, its density was determined. Wet and dry unit weights of 128.5 and 105.2 pcf respectively were determined. The maximum and minimum density of the sand also were determined by laboratory test and with this information the relative density of the sand was computed as 59 percent, which corresponds to a medium dense state.

The sand was found to be too unstable and loose to maintain shape during preparation for triaxial shear measurement. Therefore, samples were prepared artificially at densities as close as possible to the measured state and triaxial tests with pore pressure measurement were performed. The angle of internal friction, from these tests, ranged between 37½ and 41½ degrees for dry densities of 107.2 and 114.8 pcf respectively. The results of this work are presented on Dwg. 4, and the preparation procedure is recorded in the Appendix.

The soil below the sand deposit consists of a deep stratum of reddish grey lean clay till. This material is very stiff at the surface but becomes somewhat less stiff at greater depths. This transition is evidenced by the increase in liquidity index of the clay with depth. The liquidity index is an indicator of the position of the natural moisture content within the plastic range of the soil. The condition noted here suggests that the upper clay was desiccated by surface drying at some time before it was covered by the sand and submerged below the water table.

The contact between the clay and the sand was marked in Hole 2 by a very thin layer of dense gravelly sand. Some thin layers of very fine sand were noted at 72 feet in the clay till. The results of moisture content and plasticity measurements are shown in the borehole logs, Dwgs. 2 to 3 of this report.

As noted in these logs, the clean sand begins near relative El 86 feet, or about 2 feet below the river bed. This was noted by the results of hole 2 and confirmed by three probings into the river bed.

The probings encountered dense conditions after penetrating 2 feet of soft river bed material. It is believed that these observations are indicative of the maximum scour depth in the creek.

The soil above this scour level, in hole 2, consisted of alluvial silt with pieces of wood and other flood plain deposits. The upper soil in hole 1 consisted of more heterogeneous material since this boring was taken through the approach fill.

Discussion of Foundation Requirements

As indicated in earlier comments, no positive refusal level for the support of the proposed bridge replacement was encountered in the subsoil at this site. As a consequence, it is necessary to examine those foundation proposals which, conceivably, could be considered for this project.

The most desirable method of support, from the point of view of ease of construction and economy, is the use of wood piles. It was for this reason that the condition of the sand stratum was examined more closely than might otherwise be necessary.

The ultimate capacity of a wood pile in sand is given by the expression:

$$Q = (0.3\gamma D N_\gamma + \gamma H N_q) A + \tfrac{1}{2}\gamma H^2 P k$$

where: γ is the unit weight of the sand with allowances made for submergence

D & H are the diameter of the pile tip and the depth of the tip below the lowest adjacent ground surface

N_γ & N_q are bearing capacity factors dependent upon the angle of internal friction

A is the area of pile tip

P is the average perimeter of the pile

k is the coefficient of friction

$$= K_o \alpha \tan \phi$$

where: K_o is the earth pressure at rest

α is the percentage of ϕ, expressed as a decimal, generated between the pile and the sand.

ϕ is the angle of internal friction of the sand

For a Class B pile, D is approximately equal to 8 inches and P = 31.4 inches. The submerged weight of the sand may be taken = 65 pcf. According to the results of the limited field testing N_γ & N_q = 50 approximately. From recent published information, α, for a wood-sand interface is equal to 0.8 approximately.* The earth pressure at rest, ko, in sand can be conservatively taken = 0.4 and tan ϕ for ϕ = 36 degrees = 0.7. Therefore k = 0.225 and the ultimate capacity of a Class B timber pile is

$$Q = 19.1 H^2 + 1145H + 227$$

For a pile driven 40 feet below river bed level Q is found to be equal to 38 tons.

Applying a factor of safety equal to 2.5 the safe bearing value per pile is computed to be about 15 tons. The use of a factor of safety of this low magnitude is considered reasonable for the following reasons.

1) The National Building Code permits a working load equal to one half the failure load, i.e., F.S. = 2

2) Certain levels of the sand are denser than assumed in this calculation.

3) A slightly denser condition will exist in the sand after all piles have been driven.

On the basis of the results of the dynamic penetration measurements, it is not anticipated that Class B timber piles will encounter refusal in the sand stratum. As indicated previously, the high cone penetration measurements noted in the lower level of this stratum are believed to be caused by the thin layers of clay bedded in the sand. Therefore wood piles will be essentially floating in this stratum.

In the unlikely event that the sand is found to be denser than is indicated by these penetration measurements, a higher load per pile may be applied. If a refusal condition is experienced, - as indicated by a penetration resistance of 8 to 10 blows per inch under 8750 ft. lbs. of energy, - the recommended pile load to apply is 20 tons.

An examination of the above expression for ultimate bearing capacity, shows that the load capacity of a pile can be increased by enlarging the size of its tip. This can be accomplished by bolting lagging to the lower end of the pile or by driving the unit upside down. In this latter instance, some slight reduction can be expected in the friction force generated along the shaft of the pile, since the sand immediately in contact with it will be in a somewhat looser state.

* "Some Problems in the Design of Rigid Retaining Walls" - Meyerhof
Annual Soil Mechanics Conference - Montreal, November, 1961

Possible alternatives to the use of timber are Franki type compaction piles, cylindrical steel piles driven into the underlying clay, and simple footings founded just below maximum scour level. The sand at upper levels of this site is sufficiently permeable to permit its compaction during the formation of the "Franki" base. Therefore, provided that the base is formed a safe distance below anticipated scour level, a satisfactory, high capacity support for the bridge can be obtained. Using the bearing capacity formula and soil properties indicated above, the ultimate theoretical load capacity for a 20 inch pile with 4 foot diameter base formed at a depth of 10 feet below the present scour level is about 1230 tons. This estimate is quite conservative, since no allowance is made for the compaction of the sand around the base of the pile. The recommended safe load to apply at this level is 100 tons.

Cylindrical steel piles would be required to penetrate a considerable distance into the clay underlying the sand before sufficient resistance could be generated. Since no refusal depth was encountered, these piles would be essentially floating in the clay.

The ultimate capacity of a cylindrical pile is given by the expression:

$$Q_u = 9CA + C_s P H_1 + \tfrac{1}{2}\gamma H^2 P K$$

where: the first term indicates the capacity developed at the pile tip, the second and third terms represent the resistance generated along the shaft in the clay and sand respectively

C is the undrained shear strength of the clay

A is the tip area of the pile

C_s is the skin friction developed along the shaft in the clay over a length H_1

P is the perimeter of the pile

γ = 65 pcf is the buoyed weight of the sand

and K is the friction coefficient developed between the sand and steel. On the basis of the aforementioned information published by Meyerhof, a value of k = 0.16 is suggested.

The undrained shear strength of the top of the clay stratum in hole 2 was found to be about 3100 psf. Below this level, satisfactory samples for test were not recovered because of the interbeds of silt in the soil. However, the liquidity index measurements for the lower samples

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT are suggestive of a slightly lower strength. The clay was found to be too stiff for the vane at 87 feet and accordingly the shear strength at this level must be in excess of 2000 psf, the capacity of this instrument. Therefore, a value of C = 2000 psf and a skin friction force, C_s, of 1000 psf are reasonable, although probably conservative values to apply in the above expression.

Inserting these values, the ultimate capacity of a 12 inch cylindrical pile driven to a depth of 90 feet in this clay, or 85 feet below river bed level, is computed to be about 87 tons. Applying a factor of safety of 2.5, the safe load per pile is determined as 35 tons. In view of this low ratio of capacity to pile length, it would seem that cylindrical steel piles are not an economical means of support.

The other foundation alternative, involves the use of simple footings bearing at maximum scour level or about 2 feet below the river bed. This is not a particularly desirable scheme, since it will require work to be carried out below the water table in sand. On the basis of widely used empirical relationships between penetration resistance measurements and safe bearing values for building footings on sand, the estimated safe bearing value to apply for this loading and soil condition is about 2000 psf. The limiting settlement following the application of this pressure is 1 inch. However, recent studies by the Bureau of Reclamation indicate that this empirical method of designing for footings on sand is unduly conservative. In addition, a rigid pier is more resistant to settlement than a building footing and as well a simple span bridge probably can tolerate more movement than is permissible in a building, particularly since this movement will take place as load is applied. In view of these factors, it is recommended that a pressure of 3000 psf be used if this bridge support scheme is adopted.

It should be possible to reduce the excavation problems in the submerged sand if light interlocking sheet piling is driven around each footing location before excavation work is begun. Sheet piling will be required, in any event, for all schemes as a protection against scour or abrasion. The piling must be driven 5 feet below the foundation level or to a depth at least equal to the distance between the river surface and the foundation level, whichever is the greater. By doing this, the seepage path will be extended and the hydraulic gradients in the sand will be reduced safely below the values at which piping occurs. Sumps, lined with pit-run gravel could be installed, for pumping purposes in the extreme ends of each footing excavation. Internal bracing of the piling will be required.

The ring of interlocking sheet piling around the footings will greatly increase the ultimate capacity of the footings since the effective bearing depth will be transferred to the bottom level of this piling.

We hope that the information enclosed in this report assists you in the preparation of a foundation design for this structure. If you have any queries on the subject, please do not hesitate to contact us.

Yours very truly,

W. Trow

WAT/gc

William A. Trow (P.Eng.)

APPENDIX

Preparation of Samples of Sand for Determination of Drained Strength

One relatively undisturbed sample of sand was recovered from a depth of 23-24½ feet in Hole 2, using a fixed piston. The sample recovery was about 85 percent. The total wet weight of the sample was measured and attempts were made to prepare a specimen for test. Unfortunately, because the sand was wet and loose, it would not maintain shape. Accordingly, the sample was dried, the dry unit weight computed and arrangements were made to prepare artificial specimens at this density.

To do this, the total dry weight required for a cylindrical sample, 2 inches in diameter and 4 inches long was computed. This weight was divided into 4 equal parts, and each part was boiled in water to remove air. Then, in successive intervals, each quarter part was placed in the sample mould on the triaxial apparatus. After depositing each quarter section, the side of the mould was tapped until the sand grains settled to a layer thickness of 1 inch.

With the sample built up in this manner to a height of 4 inches, a negative pressure of 0.35 p.s.i. was applied through a connecting burette. This was necessary in order to maintain the sample in a cylindrical form while the mould was being removed and the remainder of the test cell was assembled. Some consolidation of the sample took place during this stage.

A consolidating all around cell pressure of 10 p.s.i. then was applied to the sample and the amount of water squeezed out was measured in the burette. A change of dry density to 112.5 p.c.f. occurred under this consolidation. The drainage tap was then closed, a back pressure was applied to the pore water in the sample and an undrained triaxial test with pore pressure measurement was performed. The results were recorded on a "Henkel type" plot so that the test could be discontinued just at the point of maximum capacity. The rate of strain in this test was 0.07 percent per minute.

When the maximum capacity had been reached, straining was discontinued, the sample was reconsolidated under 25 p.s.i. and the undrained test was repeated. The dry unit weight of the sample during this stage was 114.8 p.c.f.

Since these densities were greater than the estimated in situ density another specimen was prepared in which no consolidating pressure other than the initial negative pressure of 0.35 p.s.i. was applied. The results of these three tests are recorded in the form of Mohr envelope plots on Dwg. 4.

At the conclusion of the tests, the weights and volumes of the samples were carefully measured. The dry unit weights, computed in this manner were checked by density determinations calculated from known values of specific gravity, dry weight and assuming a sample 100 percent saturated.

TABLE NO. 1

SUMMARY OF STATIC PENETRATION TEST RESULTS

Depth of Casing (1)	Depth of test (2)	Total load on Rod (3)	Net Uplift force on Withdrawal (4)	Net Bearing load 3-4 (5)	Nq (6)	Coeff. of Friction (7)
14	14	Unable to insert				
18	18	"				
19	20	794	178	616	243	.756
23	25½*	310	166	144	48	.296
28	31*	failed under approx 111 lbs.				
	32.7*	336	147	189	53	.15

* Test done after sample recovered or attempted.

Test performed on ½" diam. rod 30 inches long.

Vanessa Bridge From the North

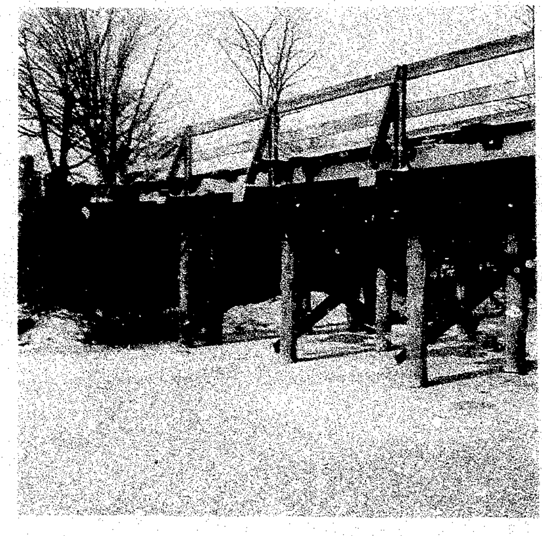
Vanessa Bridge From the South

Vanessa Bridge, View From the East
(Near BH 2)

Vanessa Bridge From the North

Vanessa Bridge From the South

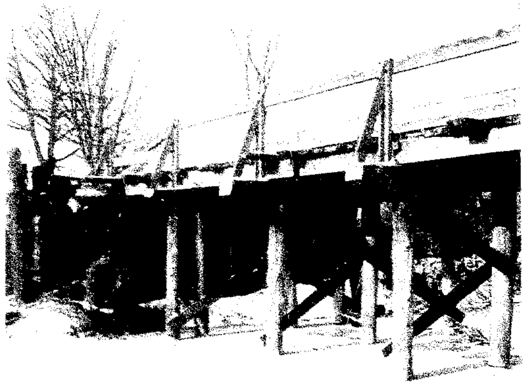
Vanessa Bridge, View From the East
(Near BH 2)



Vanessa Bridge, From the Northwest
Showing The East Abutment

Vanessa Bridge From the West

Vanessa Bridge, Looking West



Vanessa Bridge, From the Northwest
Showing The East Abutment

Vanessa Bridge From the West

Vanessa Bridge, Looking West

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 2

PROJECT NO. J798

LEGEND

BOREHOLE NO. 1

PROJECT Proposed County Rd. Bridge Replacement

LOCATION Big Creek, Between Concessions III & IV, West of Vanessa

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 100.0 ft.

DATUM See Dwg. 1.

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 3" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S)

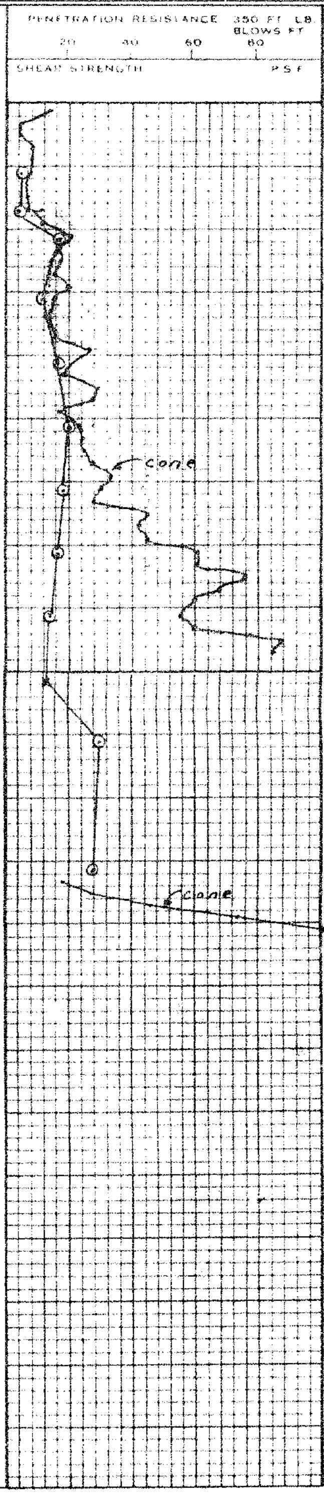
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

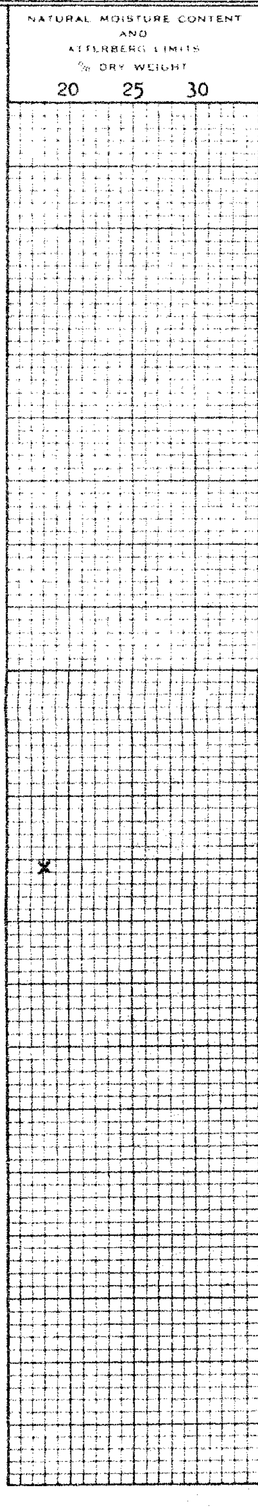
ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No
Ground Surface	100.0	0	
FILL-Dry sandy silty topsoil, broken brick, pcs. of gravel. Sand and gravel increases with depth.	92.3		SS1
SAND-loose to med. dense, brown, med. to coarse with fine gravel; some pcs. of organic material and wood 10 - 11½ft.; sand becomes grey and finer below 13 ft. then coarser again below about 15½ ft.;		10	SS2, SS3, SS4
Some thin silt seams about 20 ft.;		20, 30	SS5, SS6, SS7, SS8
layers of silty clay below 40 ft.,		40	SS9
sand becomes reddish grey and silty below about 45 ft.	42	50	TW10, SS11
CLAY-very stiff, reddish grey, low plasticity, some gravel sizes, some fine sand partings.	38.5	60	SS12
End of Bore			

Notes:
1) Boring cased with BX pipe to full depth. Casing kept full of water during withdrawal of rods in order to avoid suction effect on sand.
2) Cone driven 5 ft. west of boring after it was completed; cone also driven at bottom of hole below 62 ft.





NATURAL UNIT WEIGHT P.C.F.

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SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

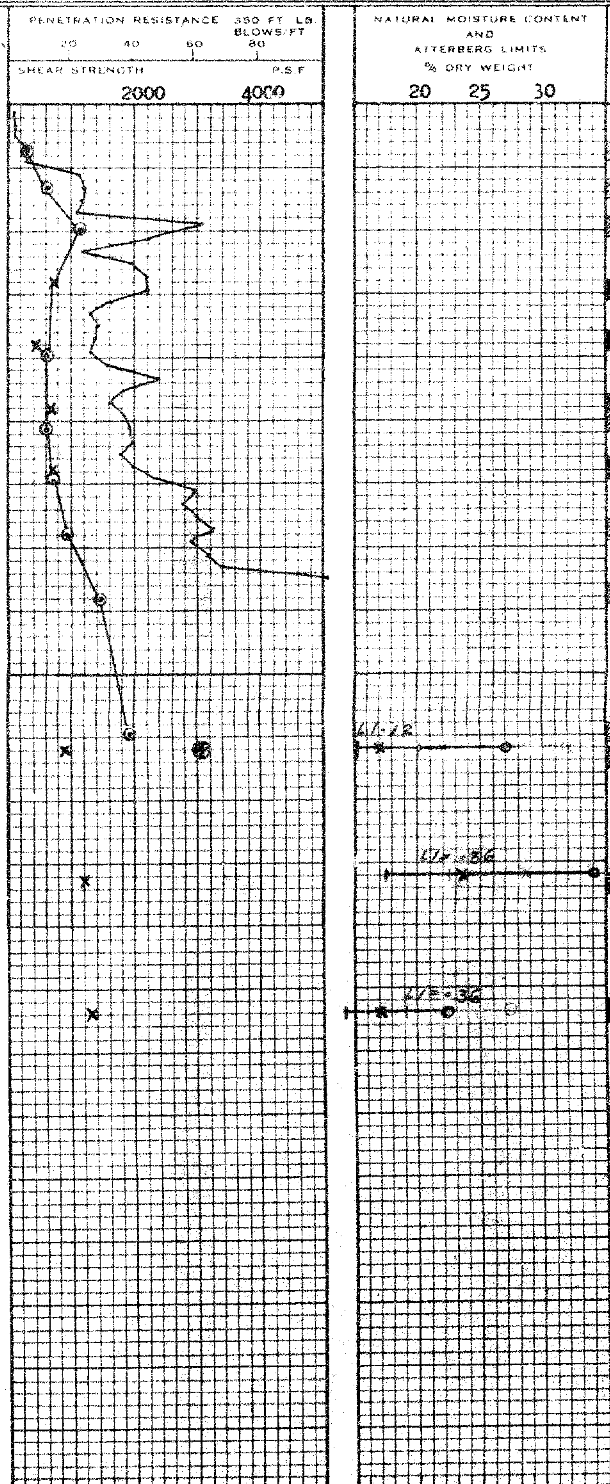
DRAWING NO. 3
PROJECT NO. J798

LEGEND

PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X LI
2" O.D. SPLIT TUBE	—o—o—o—	ATTERBERG LIMITS	
2" I.D. SHELBY TUBE	—x—x—x—	LIQUID LIMIT	—o
2" DIA. CONE	———	PLASTIC LIMIT	|—
SHEAR STRENGTH		SAMPLE TYPE	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊙	2" O.D. SPLIT TUBE	
UNCONFINED COMPRESSION	⊗	2" I.D. SHELBY TUBE	
VANE TEST AND SENSITIVITY (S)	+S	3" O.D. SHELBY TUBE	

BOREHOLE NO. 2
PROJECT Proposed County Rd. Bridge Replacement
LOCATION Big Creek, Between Concessions III & IV, West of Vanessa
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 94.0 ft.
DATUM See Dwg. 1.

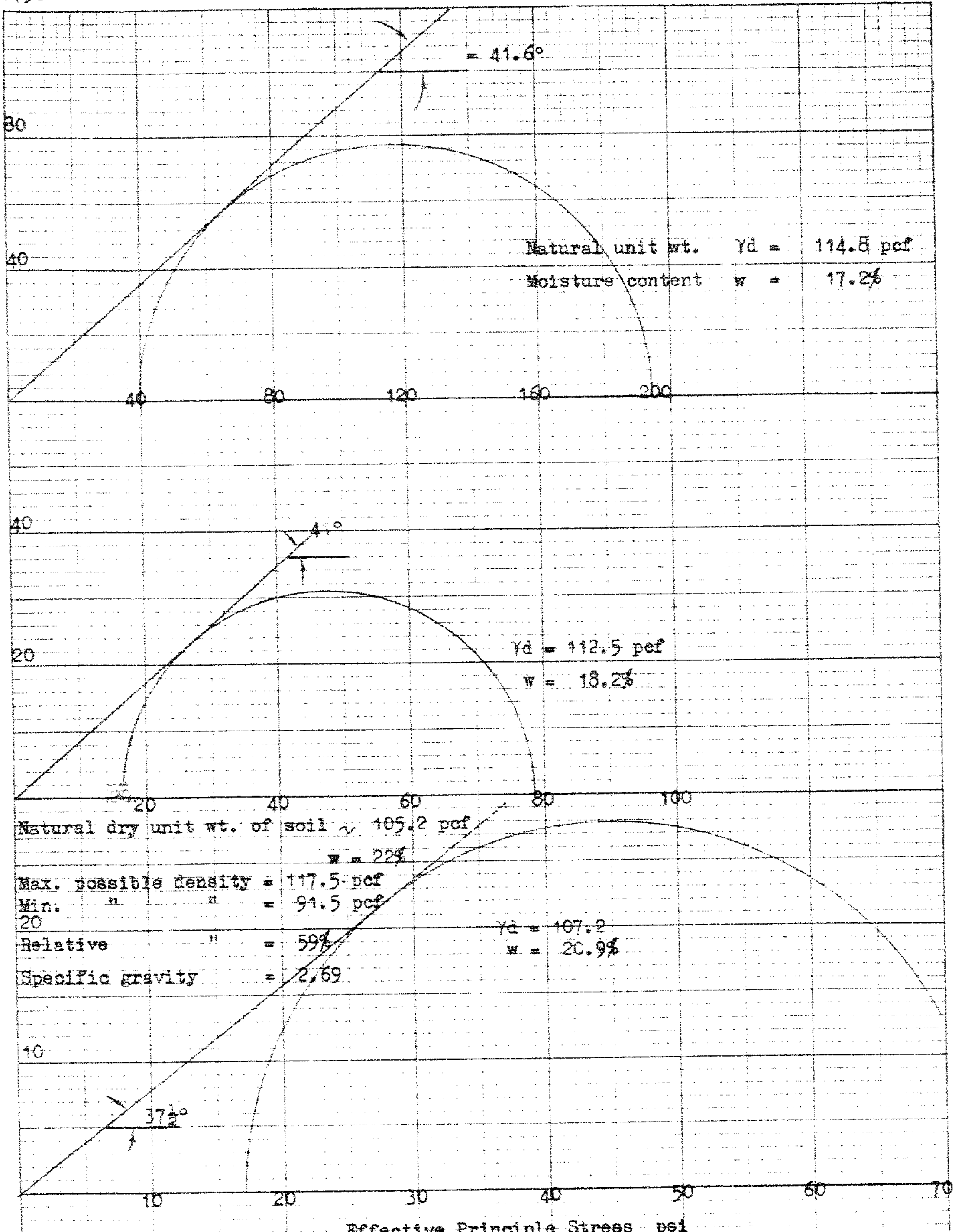
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	Ground Surface-(Some snow cover)		0
	Organic Silt-loose, fine, pcs. of wood.	94.0	
		90.0	
	SAND-loose, med. to coarse, fine to med. rounded gravel; some organic material above 8 ft.; soil denser below 8½ ft.;		10
			20
	sand becomes fine below about 23 ft.,		
	some widely spaced very thin (1/8") reddish clay clayers about 25 ft. and below 33 ft.; 7 inch layer of reddish grey clay in 1/4" seams with fine sand at 38 ft.		30
			40
	Hammer bouncing.		
	Very dense sandy silt with med. to coarse gravel.	44.9	50
	Clay-very stiff, reddish grey, low plasticity, some fine gravel sizes; becomes less stiff with depth; 2 inch layers of very fine sand about 72 ft.	42.1	
	Washed ahead to 91 ft. in same material - no hard soil. Attempted vane test at 87 ft.; unable to insert into the soil- (clay adhered to the lower end of vane.)		60
			70
			80
			90
	End of Bore		
			100
			110



SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
SS1	
SS2	
SS3	
TW4	
SS5	
TW6	
SS7	
TW8	
SS9	
TW 10	
SS11	
SS12	
SS13	
SS14	
TW15	137
TW16	
TW17	

Notes:
1) As in hole 1.
2) Cone driven 4 ft. to east of hole before boring made.
3) Static penetration tests made between 15 and 32 ft. Results recorded in report.
4) Fixed piston sampling attempted in order to obtain measurement of sand density. Only one good sample recovered.
5) Weather extremely cold during investigation.

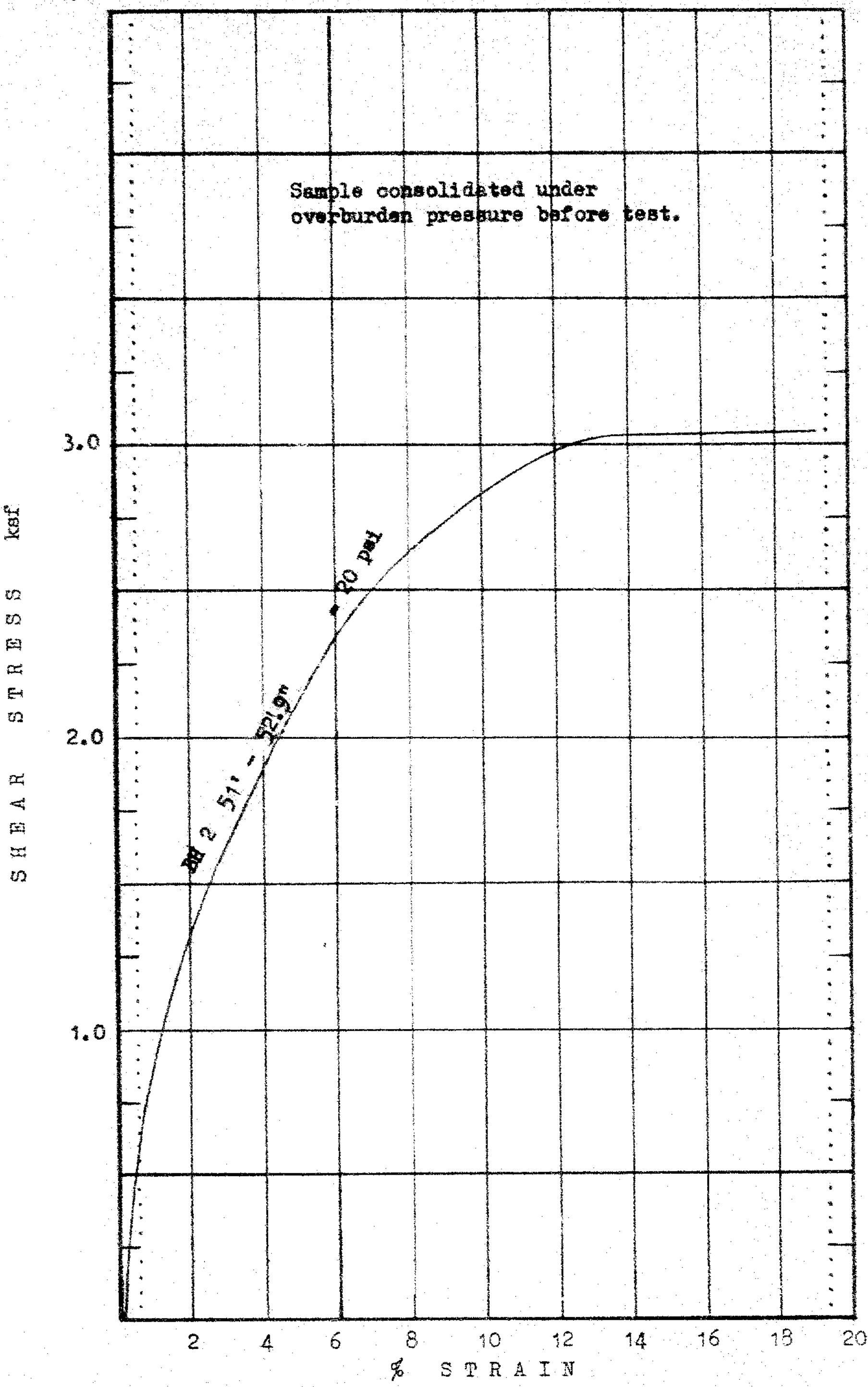
DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



RESULTS OF TRIAXIAL TESTS
TO DETERMINE DRAINED ANGLE OF INTERNAL FRICTION OF SAND

PROJECT NO. J798

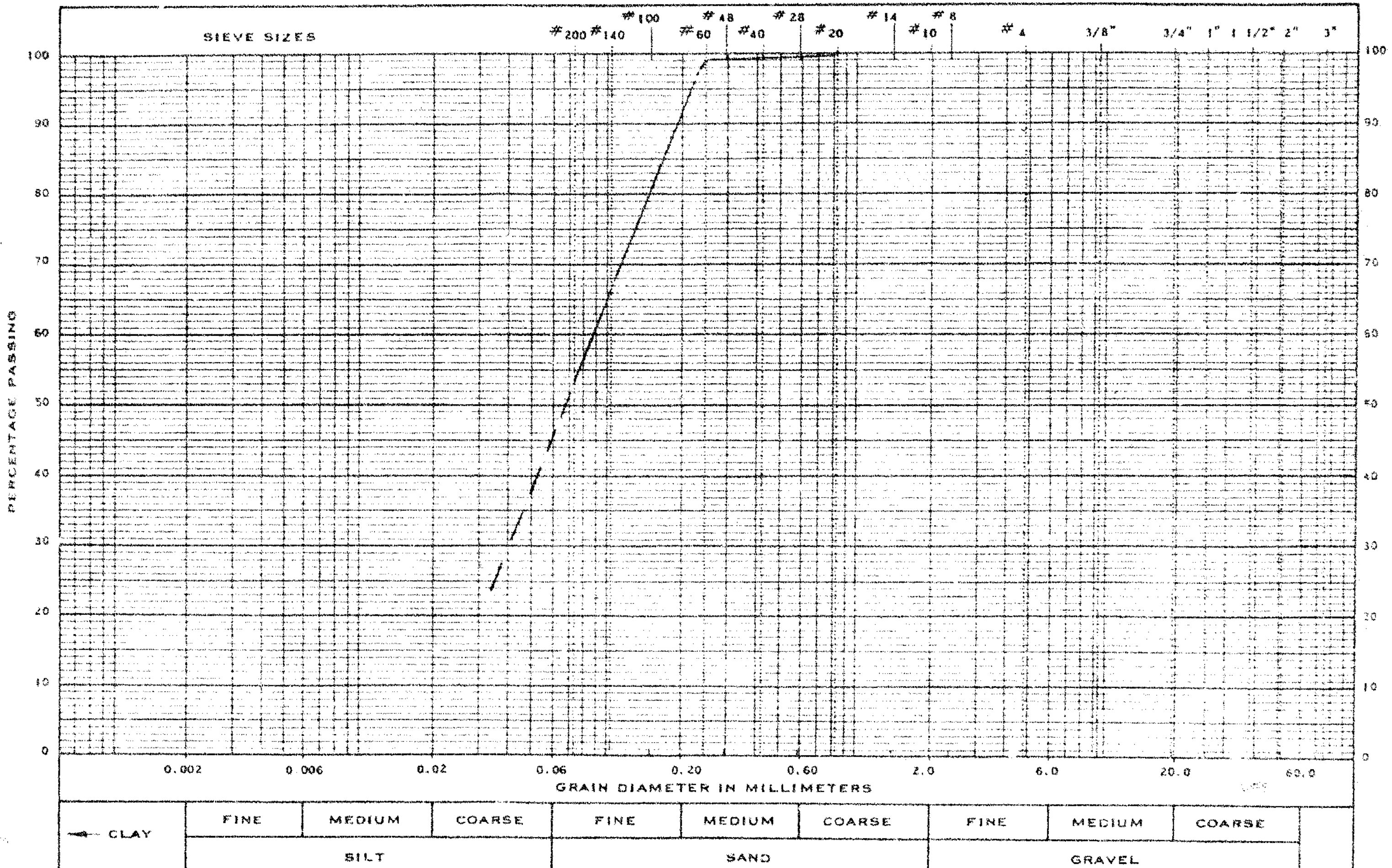
DRAWING NO. 5



STRESS-STRAIN CURVE - UNDRAINED TRIAXIAL TEST

WILLIAM A. TROW AND ASSOCIATES

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

GRAIN SIZE DISTRIBUTION OF SAND ABOVE NO. 200 SIEVE, HOLE 2 - 24 FEET

WILLIAM A. TROW AND ASSOCIATES LTD.

Project J798

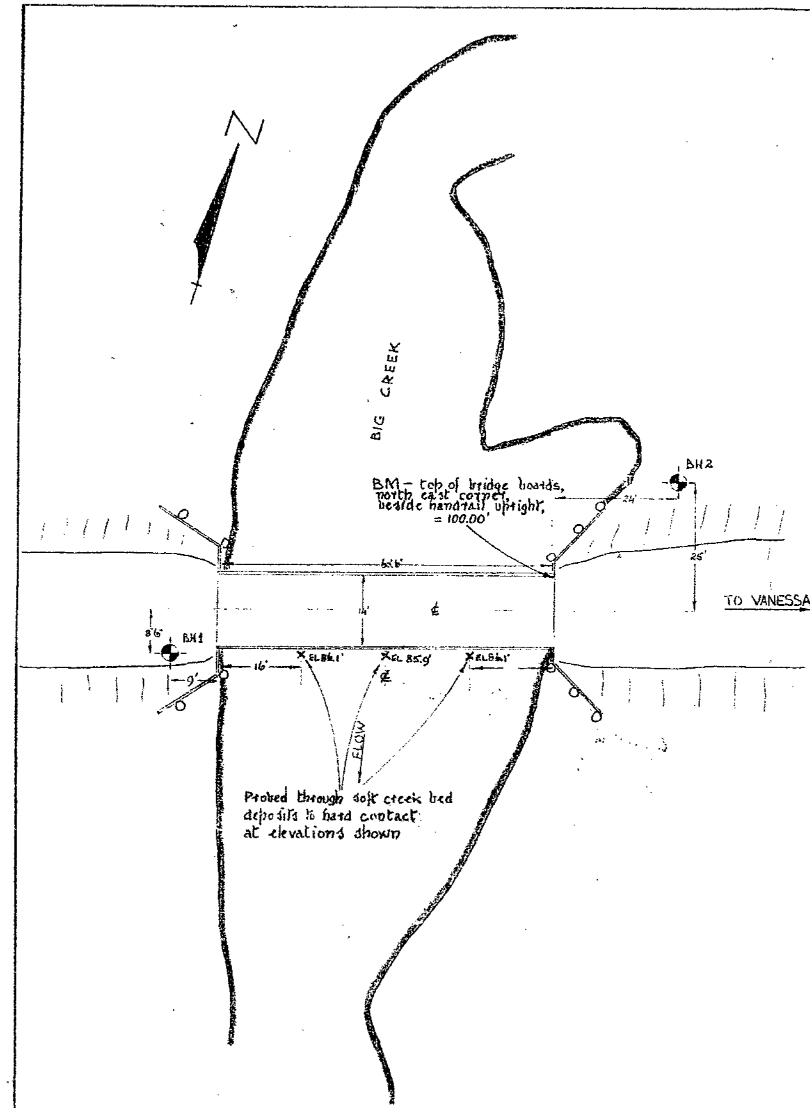
Dwg. 6

#62-F-304 M

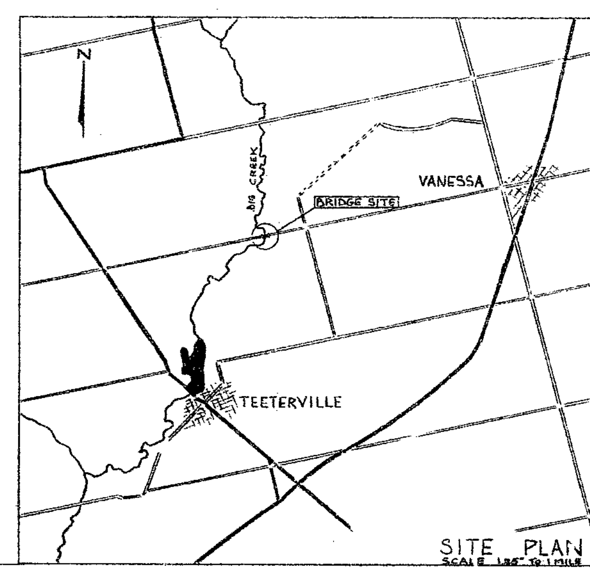
TWP. RD. BRIDGE

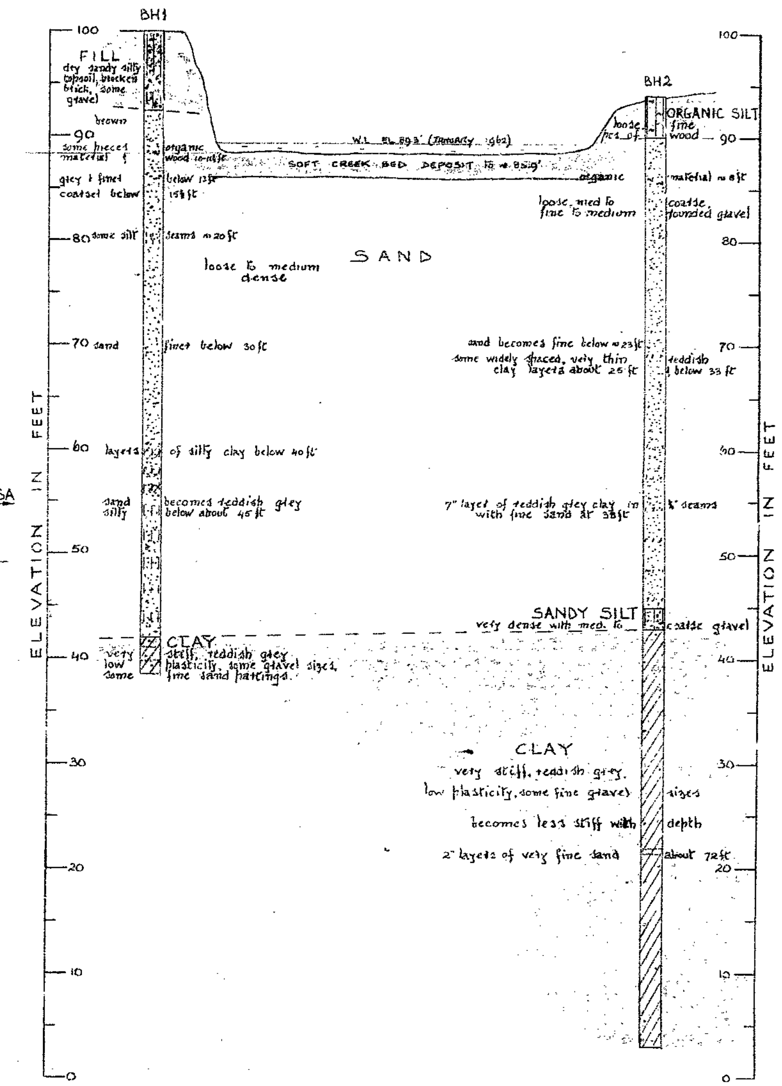
BIG CREEK

LOT 11, CON. III/IV



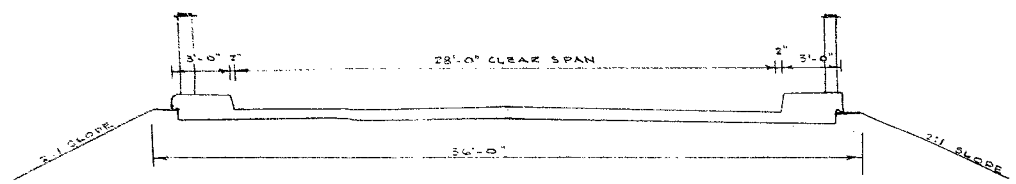
BOREHOLE LOCATION PLAN
SCALE 1" = 20'



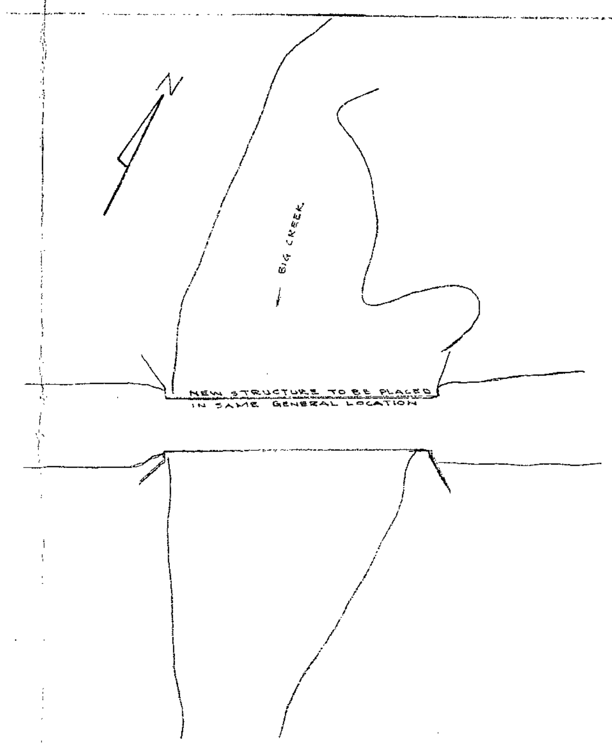


ESTIMATED SUBSOIL STRATIGRAPHY
SCALES: HOR. 1" = 20' VER. 1" = 10'

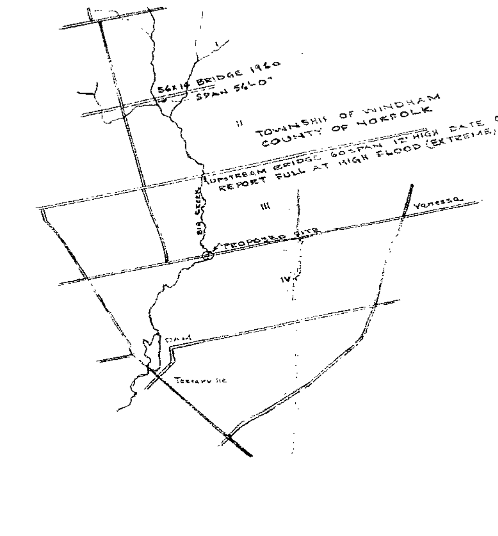
FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD BRIDGE
REPLACEMENT
VANESSA, ONTARIO
WILLIAM A. TROW & ASSOCIATES LTD
PROJECT Nº 798 DRAWING Nº 1



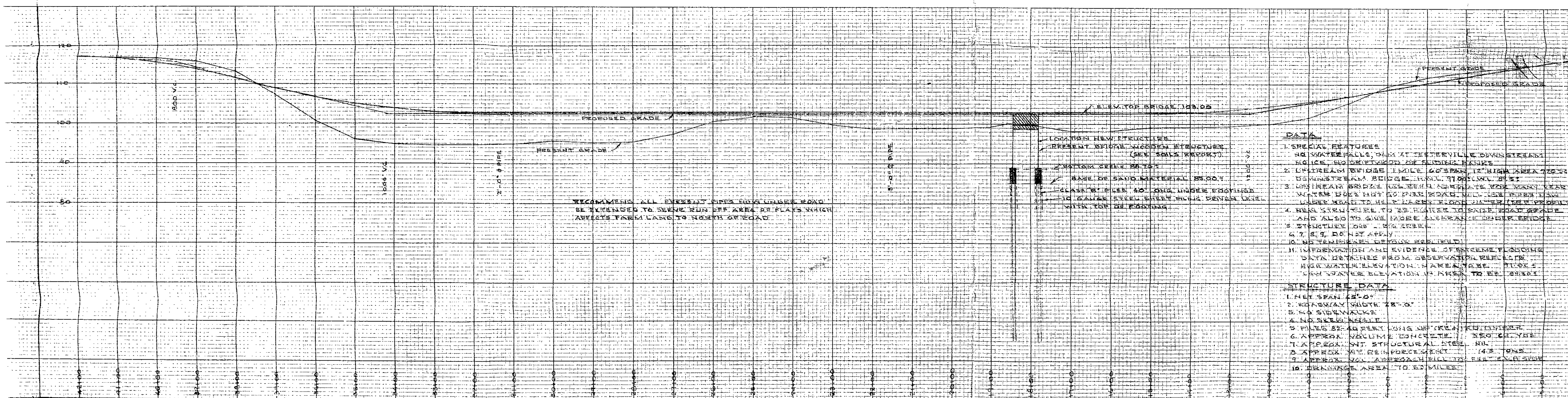
TYPICAL ROADWAY SECTION

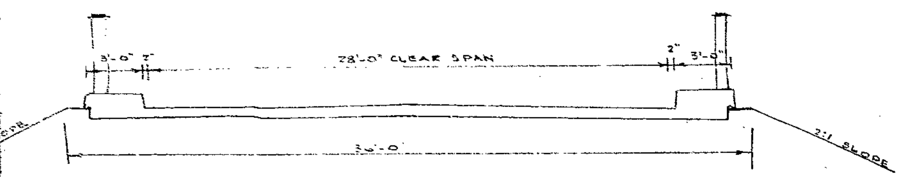


SCALE 1" = 30'

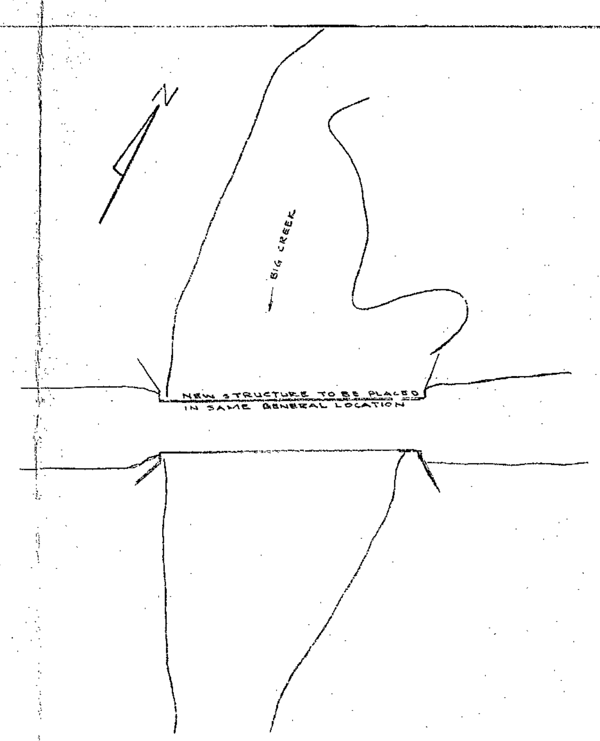


SITE PLAN
Scale 1" = 1.25 miles

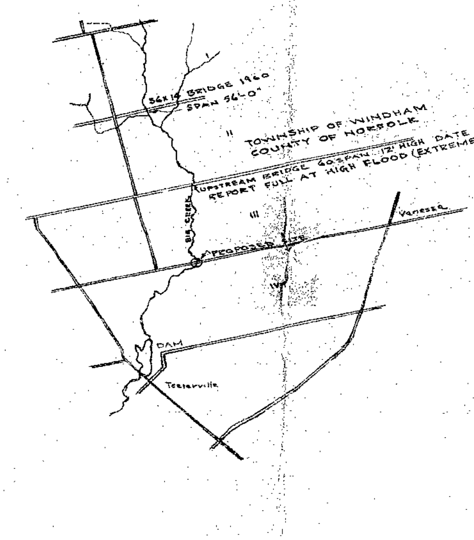




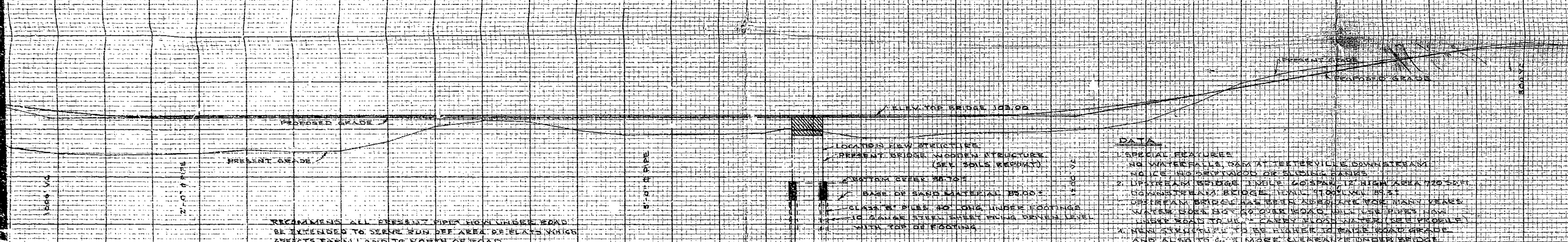
TYPICAL ROADWAY SECTION



SCALE 1" = 20'



SITE PLAN
Scale 1" = 1.25 mile



RECOMMEND ALL PRESENT PIPES NOW UNDER ROAD BE EXTENDED TO SERVE RUN OFF AREA OF FLATS WHICH AFFECTS FARM LAND TO NORTH OF ROAD.

DATA

1. SPECIAL FEATURES
   NO WATERFALLS, DAM AT TEETERVILLE DOWNSTREAM
   NO ICE, NO DRIFTWOOD OR SLIDING BANKS
2. UPSTREAM BRIDGE 1 MILE 60' SPAN, 12' HIGH AREA 720 SQ. FT.
   DOWNSTREAM BRIDGE, H.W.L. 97.00, L.W.L. 89.3±
   UPSTREAM BRIDGE HAS BEEN ADEQUATE FOR MANY YEARS
   WATER DOES NOT GO OVER ROAD, WILL USE PIPES NOW UNDER ROAD TO HELP CARRY FLOOD WATER (SEE PROFILE)
4. NEW STRUCTURE TO BE HIGHER TO RAISE ROAD GRADE AND ALSO TO GIVE MORE CLEARANCE UNDER BRIDGE
5. STRUCTURE OVER BIG CREEK
   6, 7, 8, 9, DO NOT APPLY
10. NO TEMPORARY DETOUR REQUIRED
11. INFORMATION AND EVIDENCE OF EXTREME FLOODING DATA OBTAINED FROM OBSERVATION REFLECTS HIGH WATER ELEVATION IN AREA TO BE 91.00±
    LOW WATER ELEVATION IN AREA TO BE 89.30±

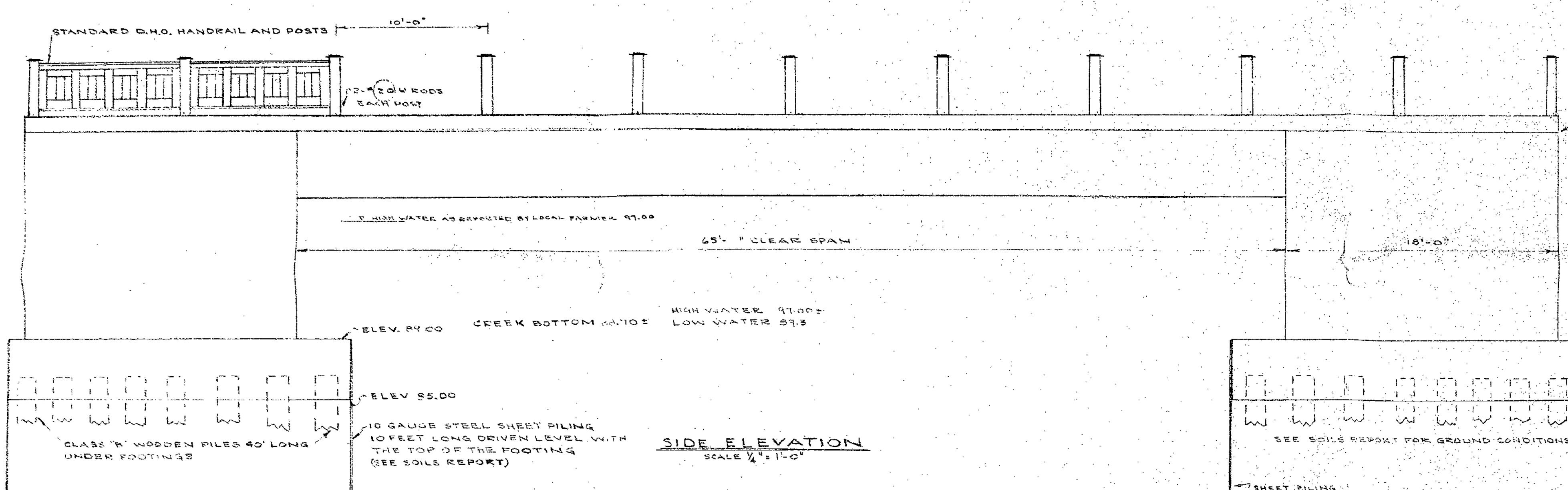
STRUCTURE DATA

1. NET SPAN 45'-0"
2. ROADWAY WIDTH 28'-0"
3. NO SIDEWALKS
4. NO SKEW ANGLE
5. PILES 32-40 FEET LONG UNTREATED TIMBER
6. APPROX. VOLUME CONCRETE 150 CU. YDS.
7. APPROX. WT. STRUCTURAL STEEL NIL
8. APPROX. WT. REINFORCEMENT 14.5 TONS.
9. APPROX. VOL. APPROACH FILL 100 FEET EACH SIDE
10. DRAINAGE AREA 10 SQ. MILES.

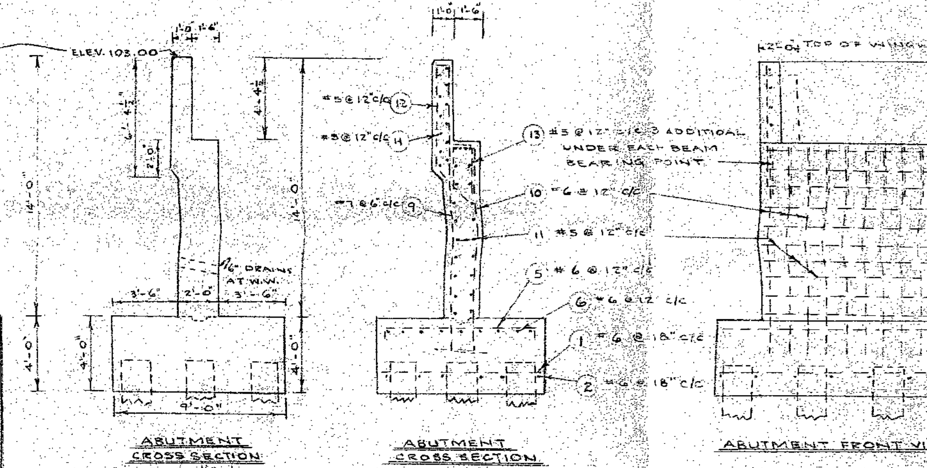
McDOWELL & JEWITT
Engineering and Surveying
92 KENT ST. S. SIMCOE

PROPOSED BRIDGE OVER BIG CREEK
OWNER TOWNSHIP OF WINDHAM
COUNTY OF NORFOLK
TOWNSHIP OF WINDHAM CON. III-IV LOT 11
DATE DEC 12 1962
SCALE PROFILE HORIZ. 100' = 1" VERT. 10' = 1"
LOADING H20-S16
DESIGN ENGINEER
DWG. NO.

STRUCTURE SITE No. 21 —>



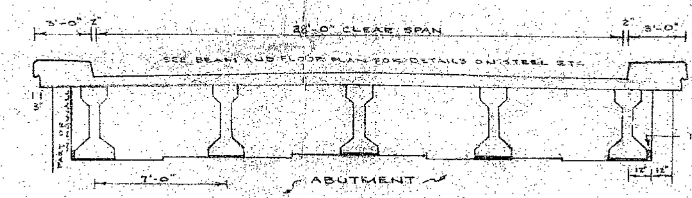
SIDE ELEVATION
SCALE 1/4" = 1'-0"



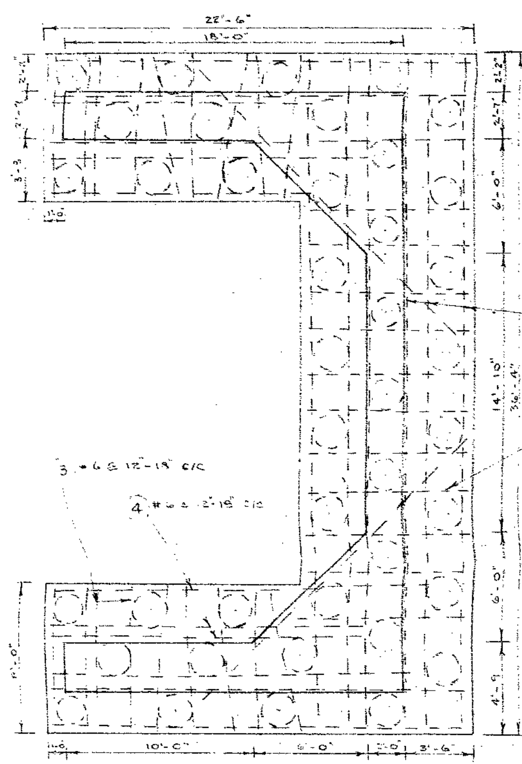
ABUTMENT CROSS SECTION
SCALE 1/4"=1'

ABUTMENT CROSS SECTION
SCALE 1/4"=1'

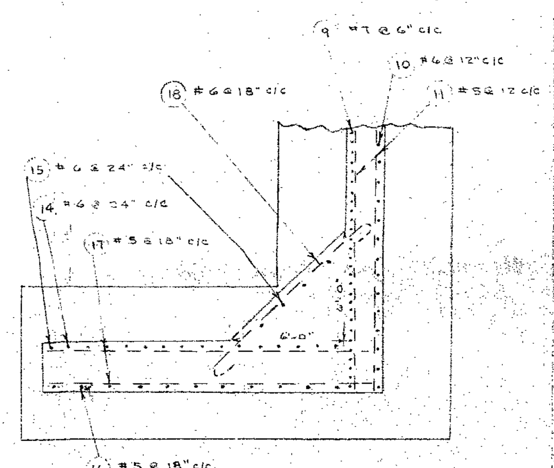
ABUTMENT FRONT VIEW



SECTION THRU' DECK
SCALE 1/4"=1'-0"

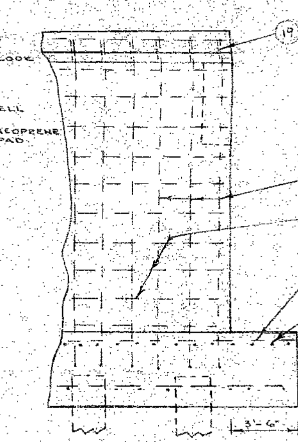


FOOTING PLAN
SCALE 1/8"=1'-0"

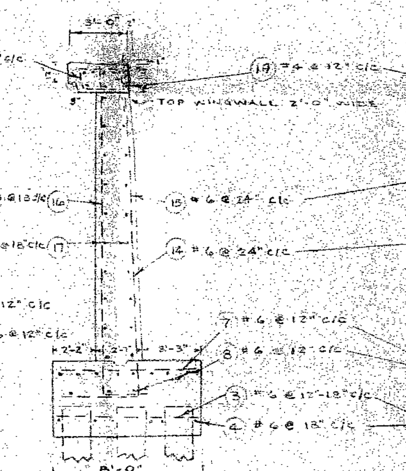


SECTION OF WINGWALL AND ABUTMENT

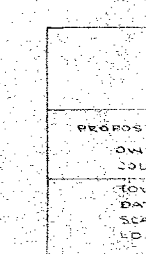
BILL OF MATERIAL					
BAR NO	SIZE	LENGTH	NO REQUIRED	LOCATION	TYPE
1.	#6	8'-6"	24	FOOTING	STRAIGHT
2.	#6	35'-10"	12	FOOTING	STRAIGHT
3.	#6	7'-6"	36	FOOTING W.W.	STRAIGHT
4.	#6	22'-0"	24	FOOTING W.W.	STRAIGHT
5.	#6	10'-4"	44	FOOTING	1'-0" 8'-4" 1'-0"
6.	#6	35'-10"	18	FOOTING	STRAIGHT
7.	#6	7'-6"	56	FOOTING W.W.	STRAIGHT
8.	#6	22'-0"	48	FOOTING W.W.	STRAIGHT
9.	#7	17'-6"	144	ABUTMENT	2'-0" 15'-6"
10.	#6	13'-0"	74	ABUTMENT	2'-0" 11'-0"
11.	#5	35'-10"	68	ABUTMENT	STRAIGHT
12.	#5	9'-1"	74	ABUTMENT	7" 15 5'-6" 45° 2'-0"
13.	#5	5'-6"	90	ABUTMENT	1'-6" 2'-0"
14.	#6	19'-3"	32	WINGWALL	4'-3" 15'-0"
15	#6	20'-5"	44	WINGWALL	4'-3" 16'-5"
16.	#5	18'-11"	44	WINGWALL	2'-6" 16'-5"
17.	#5	17'-3"	108	WINGWALL	STRAIGHT
18.	#6	13'-8	44	WINGWALL	11'-0" STD. HOOK 6" ON END HOOK
19.	#4	8'-5"	72	WINGWALL	2'-3" 1'-5" 1'-9" 3'-0"
20.	#6	6'-6"	44	POST	5" 3'-0"
21	#5	20'-0"	10	EXTRA BARS FOR FOOTINGS AT CORNERS STRAIGHT (CUT IN FIELD)	

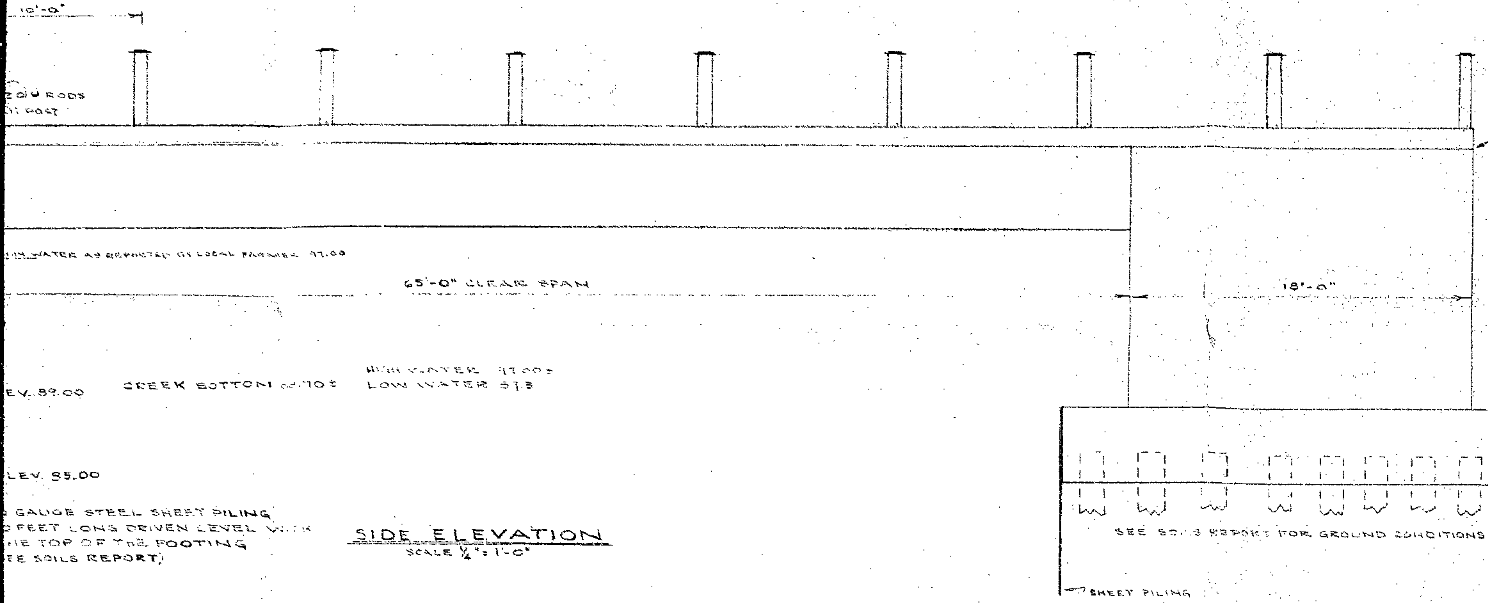


OUTSIDE WINGWALL
SCALE 1/4"=1'

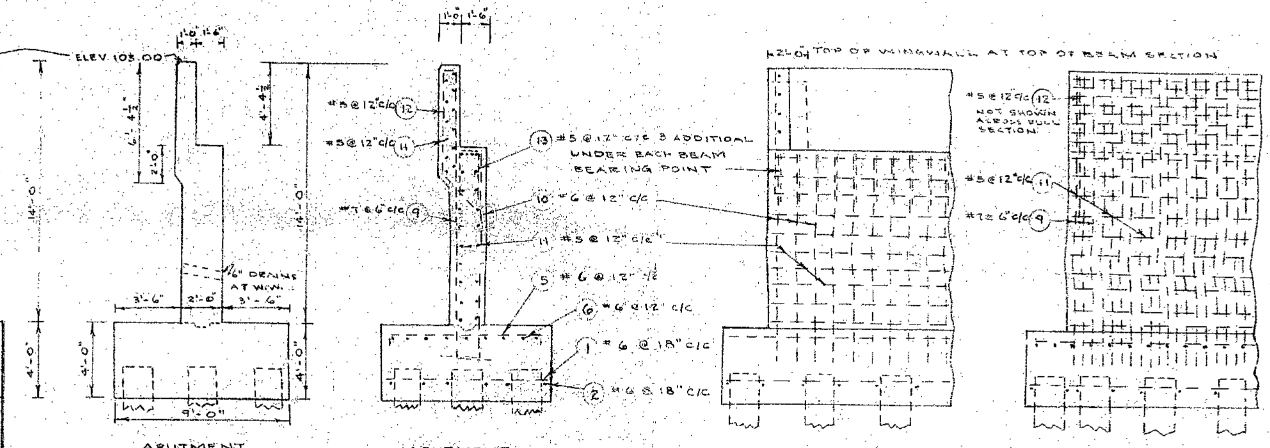


SECTION WINGWALL
SCALE 1/4"=1'





SIDE ELEVATION
SCALE 1/4" = 1'-0"

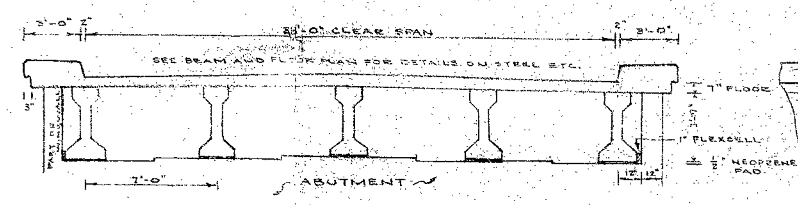


ABUTMENT CROSS SECTION
SCALE 1/4" = 1'

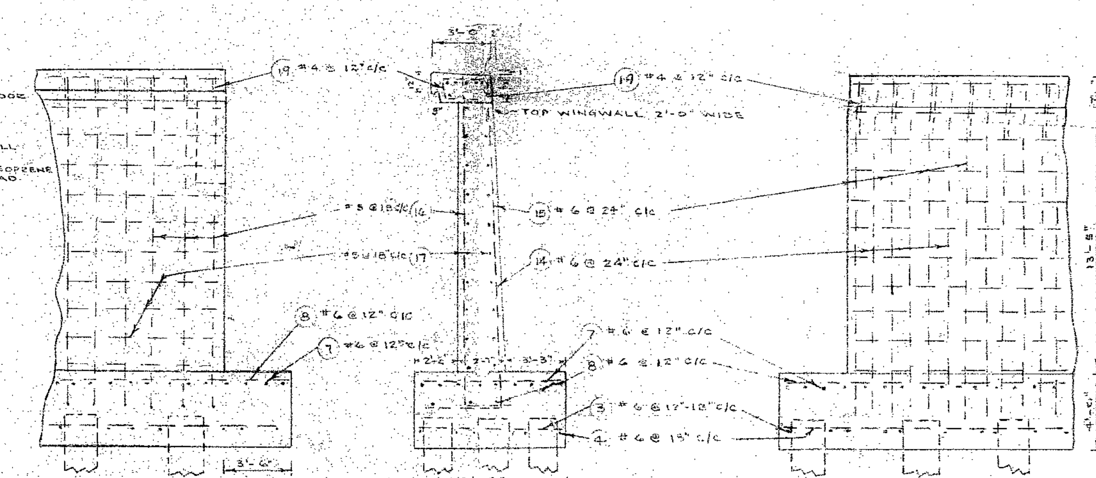
ABUTMENT CROSS SECTION
SCALE 1/2" = 1'

ABUTMENT FRONT VIEW

ABUTMENT REAR VIEW



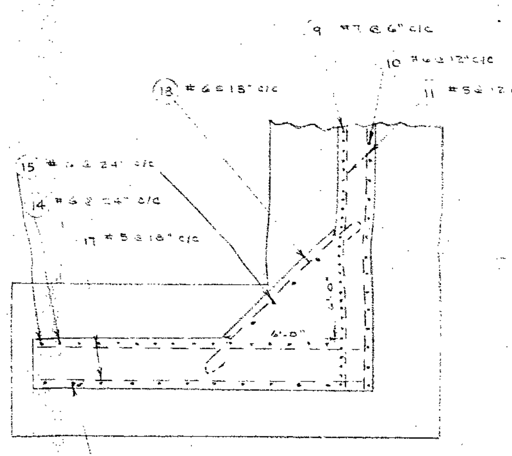
SECTION THRU' DECK
SCALE 1/4" = 1'-0"



OUTSIDE WINGWALL
SCALE 1/4" = 1'

SECTION WINGWALL
SCALE 1/4" = 1'

INSIDE WINGWALL



SECTION OF WINGWALL AND ABUTMENT

BILL OF MATERIAL

BAR NO	SIZE	LENGTH	NO REQUIRED	LOCATION	TYPE
1.	#6	8'-6"	24	FOOTING	STRAIGHT
2.	#6	35'-10"	12	FOOTING	STRAIGHT
3.	#6	7'-6"	36	FOOTING W.W.	STRAIGHT
4.	#6	22'-0"	24	FOOTING W.W.	STRAIGHT
5.	#6	10'-4"	44	FOOTING	1'-0" 8'-4" 1'-0"
6	#6	35'-10"	18	FOOTING	STRAIGHT
7.	#6	7'-6"	56	FOOTING W.W.	STRAIGHT
8	#6	22'-0"	48	FOOTING W.W.	STRAIGHT
9.	#7	17'-6"	146	ABUTMENT	2'-0" 15'-6"
10.	#6	13'-0"	74	ABUTMENT	2'-0" 11'-0"
11.	#5	35'-10"	65	ABUTMENT	STRAIGHT
12.	#5	9'-1"	74	ABUTMENT	7" 5'-6" 45°
13.	#5	5'-6"	90	ABUTMENT	1'-6" 2'-0"
14.	#6	19'-3"	37	WINGWALL	4'-3" 15'-0"
15	#6	20'-5	44	WINGWALL	4'-3" 16'-5"
16.	#5	18'-11"	44	WINGWALL	2'-6" 16'-5"
17.	#5	17'-3"	108	WINGWALL	STRAIGHT
18.	#6	13'-8	44	WINGWALL	11'-0" 6" ON END HOOK STD. HOOK
19.	#4	8'-5"	72	WINGWALL	2'-3" 1'-5" 1'-9" 3'-0"
20	#6	6'-6"	44	POST	5" 3'-0"
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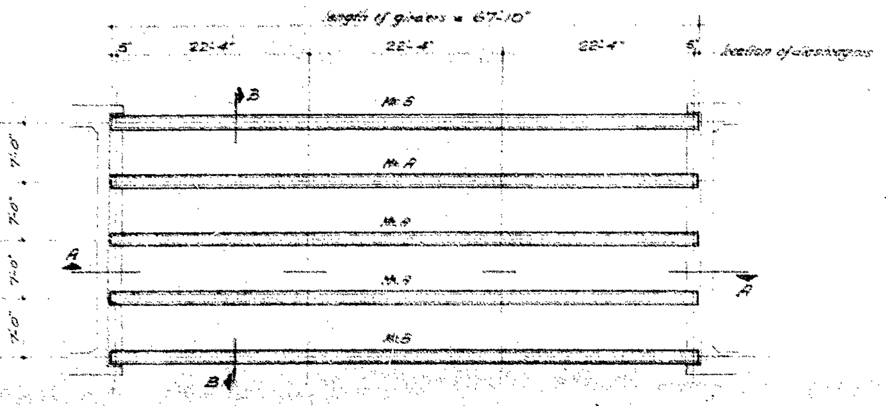
McDOWELL & JEWITT
Engineering and Surveying
92 KENT ST. S. SIMCOE

PROPOSED BRIDGE OVER BIG CREEK
OWNER TOWNSHIP OF WINDHAM
COUNTY OF NORFOLK

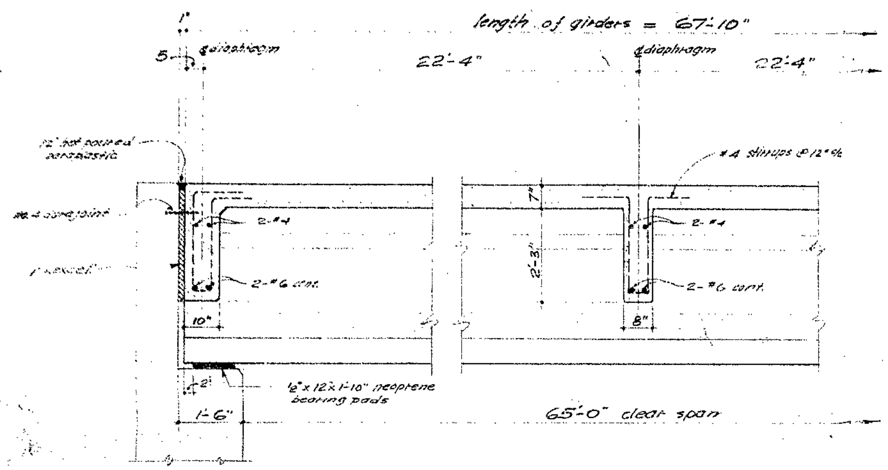
TOWNSHIP OF WINDHAM CON. III-IV LOT 11
DATE DEC. 12 1962
SCALE 1/4" = 1'
LOADING H20S16

M. C. McDowell P.ENG.
DESIGN ENGINEER
DWG. NO.

E3582

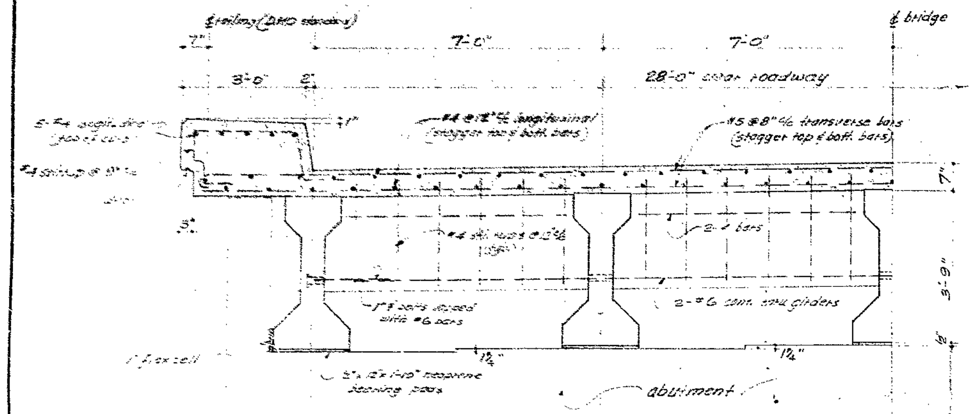


GIRDER LAYOUT PLAN



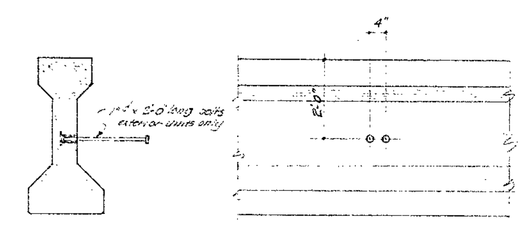
BEARING SECTION A-A

$\frac{1}{2}" = 1'-0"$

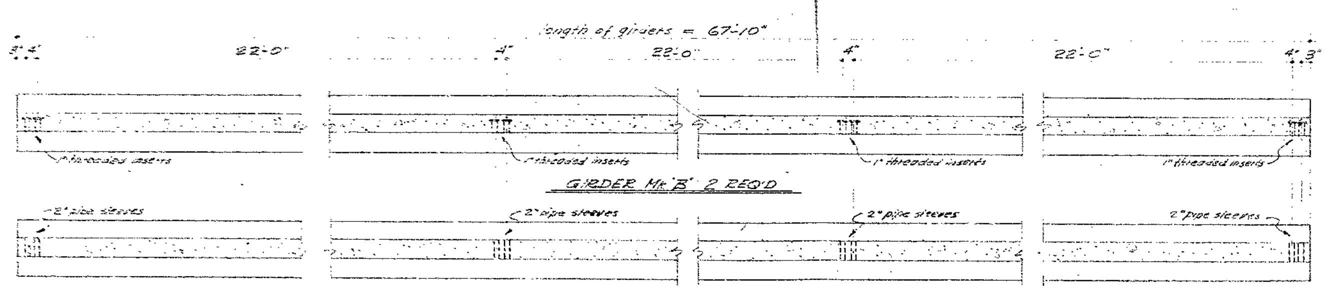


CROSS SECTION B-B

$\frac{1}{2}" = 1'-0"$

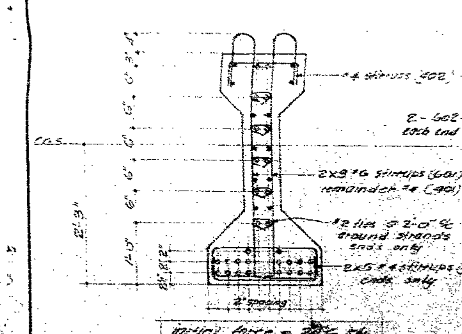


LOCATION OF INSERTS AND PIPE SLEEVES DETAILS

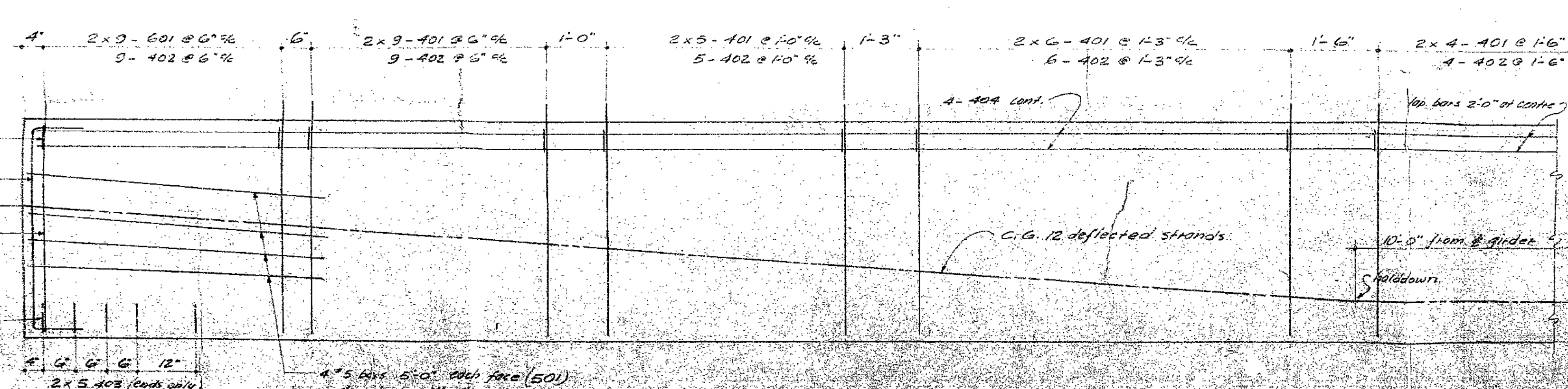


GIRDER Mk 'B' 2 REQ'D

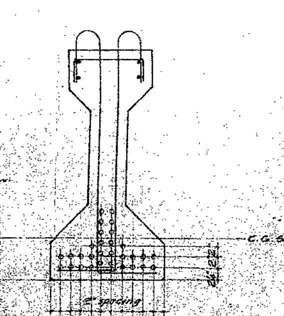
GIRDER Mk 'A' 3 REQ'D



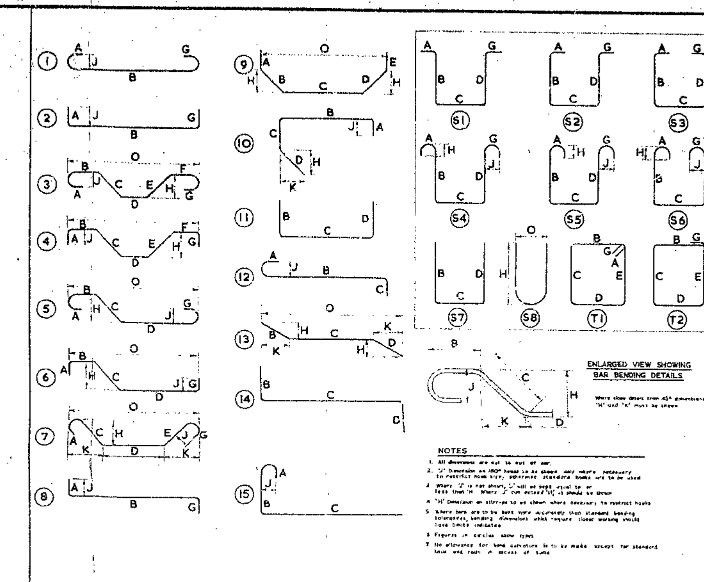
SECTION AT ENDS



REINFORCING ELEVATION



SECTION AT MIDSPAN



MARK	NO. BARS	SIZE	LENGTH	TYPE	A	B	C	D	E	F	G	H	J	K	O	SHAPE	LOCATION
601	180	#6	5'-10¾"	12	11"	47½"	11"						4¾"				stirrups ends
401	480	#4	5'-10¾"	12	11"	47½"	11"						4¾"				stirrups
402	330	#4	1'-11"	11		5¼"	12½"	5¼"									top of beam
403	100	#4	2'-4¼"	11		4¾"	19¼"	4¾"									bott. ends only
404	40	#4	69'-0"	STR.													top of beam
501	100	#5	5'-0"	STR													ends each face
602	20	#6	5'-5½"	11		12"	41½"	12"									ends vert.

DRAWINGS PREPARED FROM

Note:
AASHO type III girders and bearing pads supplied and erected by Schell Industries all other work by gen. contr.

NO.	ITEM	DATE

REVISIONS

SCHELL INDUSTRIES LTD.
PRECAST CONCRETE DIVISION
WOODSTOCK ONTARIO

JOB: PROP. BIG CREEK BRIDGE TOWNSHIP OF WINDHAM

ARCH.:

ENG.: M. Dowell & Jewitt

CONTR.:

DETAIL: BRIDGE DETAILS

DRAWN BY: J. Hoetsisr | DATE: Nov. 23/62 | DRAWING NO.:

CHECKED BY: | SCALE: as noted