APPENDIX A: GEOTECHNICHAL REPORT

GENIVAR

BRIEF GEOTECHNICAL REPORT

TO:	**SCOTT MINTY**	DATE:	September 21, 2010

COMPANY:	GENIVAR C/O CITY OF WINNIPEG	TELEPHONE:	

PROJECT #:	WE 091-15184-00 WE	☐ THE ORIGINAL WILL FOLLOW	FAX:

SUBJECT: **PROPOSED NAVIN DRAIN CULVERT CROSSING FOR LAGIMODIERE MULTI-USE PATH**	NUMBER OF PAGES: (INCLUDING COVER SHEET)

One testhole was drilled on September 16, 2010 for the proposed Navin Drain (2-1800mm Culvert) Crossing at Lagimodiere Multi- Use Path. The purpose of these testpits was to assess the general subsurface conditions. It was requested that foundation recommendation for the proposed culvert crossing be provided. The testhole drilled to 12.2m depths revealed a general soil profile consisted of 0.6m fill over a thick clay layer, which extended to the depth explored. Detailed description of the soil profile is shown in the attached testhole log. Note that there is about 0.6m of water on top of the based of the drain.

FOUNDATION RECOMMENDATIONS

The foundation recommendations are made on our understanding that the proposed structure is 2-1800mm precast concrete culvert connected with two concrete wing-walls located at the northwest and southeast of the culvert over a concrete apron.

CAST-IN-PLACE FRICTION PILE (CIP)

The preferred foundation is the used of cast-in-place friction pile. Using pile lengths of 7.6m (25 ft), 9.1m (30 ft) and 12.2m (40 ft) below grade, an allowable shaft adhesion value of 15.0 kPa(314 psf), 14.6 kPa(304 psf) and 12.1 kPa(258 psf) applied to the pile circumference within the native clay may be used for the pile design, respectively. A maximum pile length of 9.1m (30 ft) below grade should be used due to considerable low shear strength of the clay below 9.1m depths.

Considering potential long-term soil shrinkage away from the pile face, the upper 1.5m (5 ft) of the piles should be neglected in determining the capacity of all interior piles. For the exterior piles, the upper 3.0m (10 ft) of the piles should be ignored.

Pile spacing should be at least three pile diameters, centre to centre. *Seepage and sloughing will be expected from the base of the drain thus, temporary steel sleeves should be on hand and used during pile installation.* To minimize pile construction difficulties, the total number of pile holes left open at any

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given time should not be more than four and the pile holes should be poured with concrete as soon as they are drilled to the design diameters and depths.

Piles located in unheated areas should be provided with full-length reinforcements, a minimum pile length of 7.62m (25 ft) and the top 2.1m (7 ft) of the pile should be cast using with greased sono tube forms to reduce the potential for frost jacking.

Pile installation may be adversely affected by loose backfill and the possible presence of existing concrete slabs. Thus, contract documents should properly cover these potential obstacles during pile installation.

Pile inspection by qualified geotechnical personnel should be employed to ensure a satisfactory foundation installation.

RAFT SLAB ON COMPACTED GRANULAR FILL

Alternatively, the entire structure may be supported on 600mm of compacted granular fill over non-woven geotextile over stiff clay. This system maybe used provided that immediate and long term foundation settlement of less than 25mm is acceptable and frost protection is provided.

To reduce differential foundation settlement, the compacted fill should be prepared as follows:

- Within the proposed structure area and at least 1.2 m beyond the structure perimeter, remove all heterogeneous fill, organic material, peat moss, softened soil and ponded water to expose the underlying clay layer. Depth of site stripping is expected to be 600mm below existing grade. Any additional fill should comprise of either the same material (20mm) or subbase material (50mm down granular fill). Since the anticipated floor is granular fill, the depth of site stripping should cover the entire structure.

- Call for subgrade inspection. The exposed subgrade should be compacted with vibratory roller equivalent to 95% Standard Proctor density. *If the exposed subgrade can not be compacted due to saturation, the need for permanent subdrains placed underneath the subgrade should be enforced to attain the required compaction.*

- Once the subgrade is approved, place a 300 mm thickness of 50 to 75mm down granular fill (C-base or subbase) followed by a 300 mm thickness of 20 mm down granular fill (A-base or base course material) across the entire width of the structure area and 1.2 m beyond. Both of the subbase and base course materials should be placed and uniformly compacted with a heavy vibratory roller to at least 98% Standard Proctor density (ASTM D698)

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- Call for compaction testing of all the lifts (150m mm thickness) by qualified geotechnical personnel.

- If required, recompact and retest any area of low percent compaction to meet the specified 98% compaction.

For frost protection, raft slab should be provided with a minimum granular soil cover of 1.8m or an equivalent insulation thickness of R20 rigid polystyrene skirt around the perimeter to a depth of 600mm below grade prior to backfilling and extending diagonally outward over the weeping tile a distance of 1.2m.

Final site grading should ensure that all surface runoff is directed away from the any built structure using minimum gradients of 1% and 2% for paved and landscaped areas, respectively.

For winter construction, all bearing surfaces and the newly poured slab should be protected by hoarding and heating to avoid frozen concrete and heaving due to frost penetration below the slab.

To avoid the saturation of the bearing soil and scouring during the spring season, the exposed granular fill beyond the perimeter of the slab should be capped with impermeable clay compacted to at least 95% Standard Proctor density.

Based on the groundwater conditions, the foundation system should be designed to resist an up-lift pressure of about 15 kPa.

For the wing-wall foundation system, the footing may also be designed for an allowable bearing pressure of 72 kPa. A cohesion value of 37.5 kPa with a safety factor of 1.5 against sliding should be used.

The new fill surcharge should be limited to not more than 750 mm above the average ground surface elevation. Raising the existing grade more than 750 mm should not be attempted, without a detailed geotechnical evaluation. On the foregoing basis and contingent upon a minimum depth of 600 mm for the granular pad and a maximum footing width of 750mm(30 in.) is used, an allowable soil bearing pressure of 71.8 kPa(1500 psf) may be used. The associated total soil settlement is estimated to be 25 to 50 mm. Differential settlements of the granular pad are not expected to be significant.

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LATERAL PRESSURE OF THE WING WALL

The degree of compaction of backfill against the wall, backfill soil type and subdrainage conditions will greatly affect the magnitude and shape of the lateral earth pressure on the structured walls.

Since the walls of the structure will be unyielding (i.e. rigid), the "at-rest" (K_0) earth pressures will be applicable for all cases. Where the backfill behind the wall will be required to provide subgrade support for slabs, pavements, etc., the backfill will need to be compacted. The lateral earth pressures induced by compaction and surcharge loadings will need to be accounted for in design. Where the subgrade support will not be required, the backfill need not be well compacted, and the conventional triangular earth pressure distribution will be appropriate.

As the clay layer is considered impermeable for design purposes, it is anticipated that full hydrostatic pressures will be realized and it is recommended that a water level co-incident with the ground surface be utilized for design. Preferably, a subdrainage system, consisting of filter protected perforated pipes installed at the wall base, would be installed, thus eliminating the requirement to design for full hydrostatic pressures.

LIGHT TO MODERATELY COMPACTED BACKFILL

Light to moderately compacted backfill is estimated to correspond to compaction levels of 90 to 95% of standard Proctor. For such degrees of compaction, settlement is expected to be in the range of 2 to 4 percent of the fill height,

Where subdrainage will be provided at the base of the wall, the lateral earth pressures should be determined by:

$$P_h = K_o \bullet H$$

Where: P_h = lateral earth pressure at any depth(kpa)

Ko = earth pressure coefficient

$\bullet$ = total unit weight of the soil(kg/m^3)

H = height of the wall (m)

In the case where subdrainage will not be provided, the design lateral earth pressure acting on the wall may be determined by :

$$P_h = K_o \bullet' H + \bullet_w H$$

Where: P_h = lateral earth pressure at any depth(kpa)

K_o = earth pressure coefficient

$\bullet'$ = buoyant unit weight of the soil(kg/m^3)

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H = height of the wall (m)

$\bullet_w$ = unit weight of water

The recommended soil parameters are presented below in Table 1. Due to the frost susceptibility of a clay material, clay backfill is not recommended for the ramp.

TABLE 1
RECOMMENDED EARTH PRESSURES COEFFICIENTS AND UNIT WEIGHTS
FOR LIGHT TO MODERATELY COMPACTED BACKFILL

SOIL TYPE	K_o	TOTAL UNIT WEIGHT(kg/m^3)
Light to Medium Compacted Granular Fill (Pit-run Material: 75 mm down)	0.50	2150

WELL COMPACTED BACKFILL

Where the backfill adjacent to the structured walls will be well compacted, there will be a build-up of lateral compaction stresses and the design earth pressure envelope will be as illustrated in Figure 2. For this case, the design earth pressure for the upper portions of the walls will be dependent on loads imposed by the compactor and includes the centrifugal force in the case of a vibratory compactor. The combined dead weight and centrifugal force for typical compactors is given in Figure 3. It may be necessary to provide temporary bracing during compaction to resist the lateral pressures generated by the compaction equipment.

For the well compacted case, compaction to 98 to 100% of standard Proctor is required and settlement of the fill is expected to be in the range of 0.5 to 1 percent of the fill height. Obviously, the greater degree of compaction achieved, the less settlement will occur. The recommended soil parameters are presented below in Table 2.

TABLE 2
RECOMMENDED EARTH PRESSURES COEFFICIENTS AND UNIT WEIGHTS
FOR WELL COMPACTED BACKFILL

SOIL TYPE	K_o	TOTAL UNIT WEIGHT(kg/m^3)
Well Compacted Granular Fill (Pit-run Material: 75 mm down)	0.40	2275

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SURCHARGE

Lateral pressures induced by surcharge loads will need to be included for in the design of some or all of the walls. In the case of uniformly distributed surcharge loads, the lateral pressures should be determined by multiplying the surcharge load intensity by the appropriate K_0 value. For live point or line surcharge loads, the lateral pressures should be determined as shown on Figure 4.

ADDITIONAL CONSIDERATIONS

Concrete should be manufactured with sulphate-resistant (Type 50) cement and air content between 4% and 7%. Any concrete subject to cycles of freezing and thawing should be air entrained in accordance with the latest edition of CSA A23.1, Concrete Materials and Methods of Concrete Construction.

The granular base course and subbase materials should include organic-free, non-frozen, aggregate conforming to the City of Winnipeg gradation limits.

Where soft spots are encountered at the slab level, construction traffic should be restricted. Soft spots should be excavated with a large backhoe fitted with a smooth bucket, to at least 300 mm below the underside of the subbase and replaced with a 300 mm thick layer of 150 mm crushed limestone.

Sieve analysis and compaction testing of the granular base and subgrade materials should be conducted by qualified geotechnical personnel to ensure that the materials supplied and percent compactions are in accordance with design specifications.

CLOSURE

The findings and foundation recommendations provided in this report were prepared in accordance with generally accepted professional engineering principles and practices. The recommendations are based on the results of field investigations. If conditions encountered during construction appear to be different than those shown by the testpit at this site, this office should be notified immediately in order that the recommendations can be reviewed.

The findings and recommendations provided in this report were prepared by GENIVAR (the Consultant) in accordance with generally accepted professional engineering principles and practices. The recommendations are based on the results of field and laboratory investigations and are reflective only of the actual testhole(s) and/or excavation(s) examined. If conditions encountered during construction appear to be different than those shown by the testhole(s) and/or excavation(s) at this site, the Consultant

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should be notified immediately in order that the recommendations can be reviewed and modified as necessary to address actual site conditions.
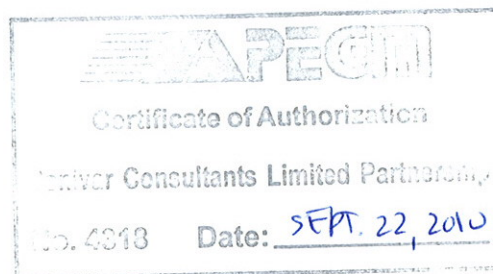
This report is limited in scope to only those items that are specifically referenced in this report. There may be existing conditions that were not recorded in this report. Such conditions were not apparent to the Consultant due to the limitations imposed by the scope of work. The Consultant, therefore, accepts no liability for any costs incurred by the Client for subsequent discovery, manifestation or rectification of such conditions.

This report is intended solely for the Client named as a general indication of the visible or reported physical condition of the items addressed in the report at the time of the geotechnical investigation. The material in this report reflects the Consultant's best judgment in light of the information available to it at the time of preparation.

This report and the information and data contained herein are to be treated as confidential and may be used only by the Client and its officers and employees in relation to the specific project that it was prepared for. Any use a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. The Consultant accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

The report has been written to be read in its entirety, do not use any part of this report as a separate entity.

All files, notes, source data, test results and master files are retained by the Consultant and remain the property of the Consultant.

Certificate of Authorization

Genivar Consultants Limited Partnership

No. 4818   Date: SEPT. 22, 2010

S. S URBANO
SEPT. 22, 2010

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02.2007

Project No: WE-091-15184-00-WE

TH1

Project: Lagimodiere Multi-Use Path

Client: City of Winnipeg

Enclosure:

Location: 2-1800mm Culvert Crossing

Engineer: SSU

GENIVAR

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation, m	Field Vane(m-kg)	Blows/0.3m	Pocket Penetrometer Test kPa	Water Content %

Pocket Penetrometer Test kPa: 50 150 250 350

Water Content %: ● Wp |------O------| Wl ● 20 40 60 80

Ground Surface		100	
FILL		99.5	
50mm sandy topsoil fill with rootlets over 550mm of CLAY FILL, mixed brown and black, trace of fine gravel			

CLAY
stiff, brown, fissured; SILTY at 1.8 to 1.95m, firm, tan-brown; stiff below 1.95m, trace of sulphate inclusions down to 4.6m; grey at 10m, firm to soft. TESTHOLE WAS DRY AFTER COMPLETION OF DRILLING

Elevation 87.9

End of Testhole

Pocket Penetrometer values (kPa):
150, 100, 100, 90, 90, 80, 40, 40

$$\text{FOR } Z_c \le Z \le d$$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}$$

$$\text{FOR } Z > d$$

$$\bar{\sigma}_h = K_0 \cdot \gamma \cdot Z$$

$$Z_c = K_0 \sqrt{\frac{2P}{\pi\gamma}}$$

$$d = \frac{1}{K_0} \sqrt{\frac{2P}{\pi\gamma}}$$

$$\text{P (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

L : LENGTH OF ROLLER

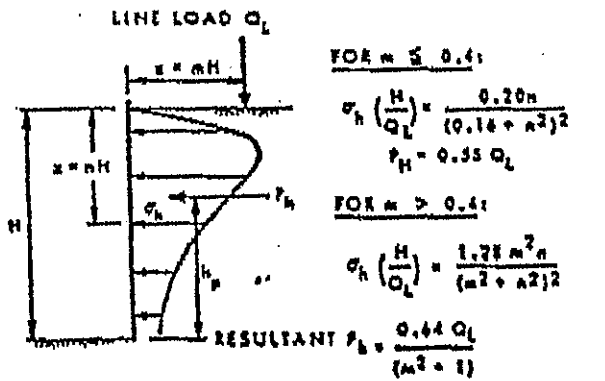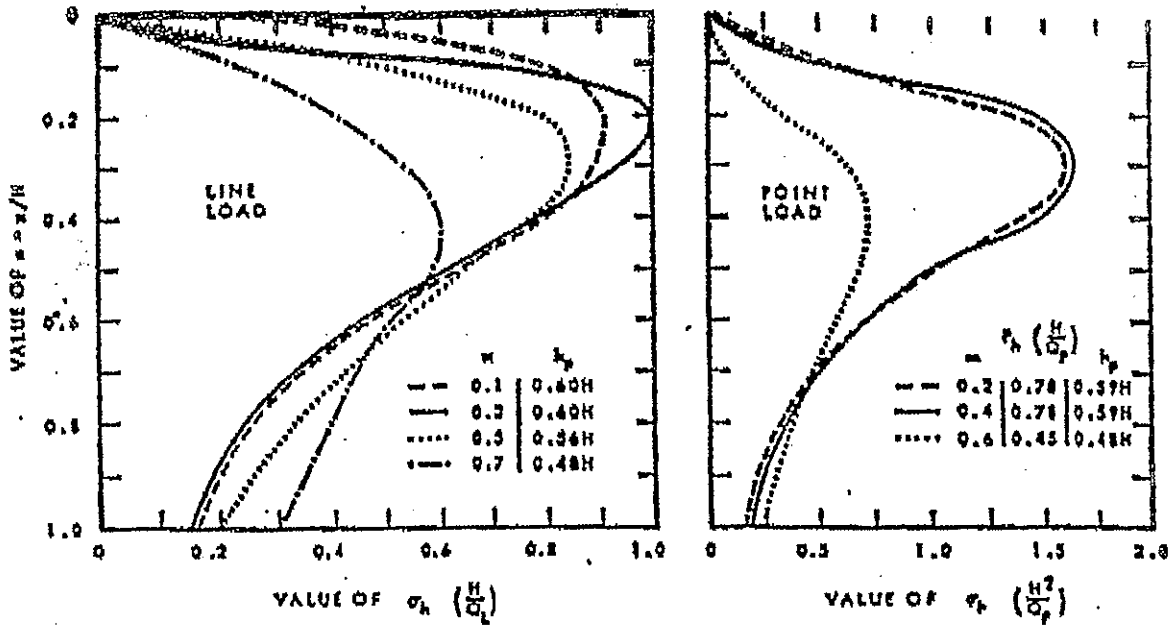**SOIL PARAMETERS ARE GIVEN IN THE TEXT OF REPORT**

REF: INGOLD (1980), INTERNATIONAL CONFERENCE ON COMPACTION

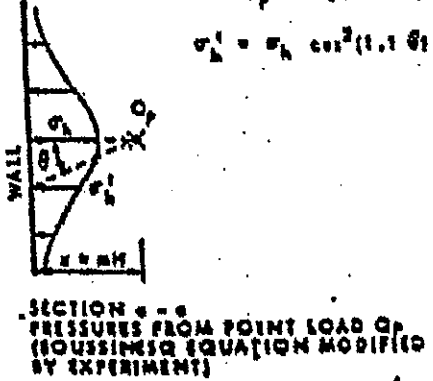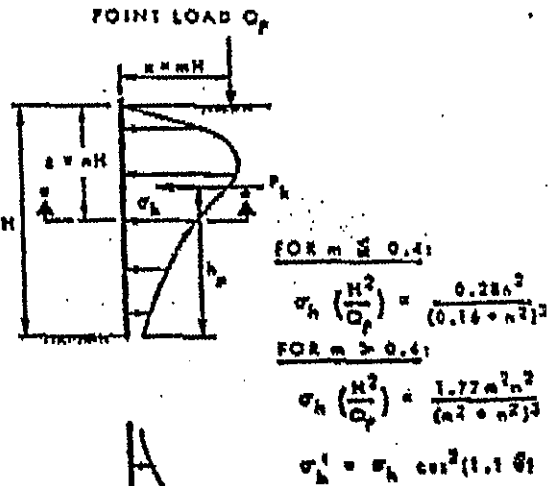| DRAWN BY :		CLIENT :	
PROJECT No :		PROJECT :	
DATE :		DESCRIPTION :	HORIZONTAL PRESS. ON WALLS DUE TO COMPACTION EFFORT
SCALE :	NOT TO SCALE	DRAWING :	FIGURE 2

TYPICAL COMPACTOR LOADS (P)

Compactor	Load (P) kN/m
Bowmag TSE	31.3
Bowmag 60S	31.8
Bowmag 65S	22.7
Bowmag 75S	32.5
Bowmag 90S	39.3
Bowmag 75AD	19.6
Bowmag 100AD	20.3
Bowmag 120AD	33.5
Bowmag 130AD	36.1
Bowmag BW122D	30.4
Bowmag BW122PD	35.8
Dynapac LR100	41.7
Dynapac 2100V	92.9
Bowmag 142PDB	46.6
Bowmag 172PDB	92.9
Dynapac CA121D	53.2
Dynapac CA121PD	63.7
Dynapac CA151	79.9
Dynapac CA151D	80.2
Dynapac CA151PD	96.2

VALUE OF $n = z/H$

LINE LOAD

POINT LOAD

m	k_p
0.1	0.60H
0.3	0.40H
0.5	0.36H
0.7	0.48H

m	$\sigma_h \left(\frac{H}{Q_p}\right)$	h_p
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H

VALUE OF $\sigma_h \left(\frac{H}{Q_L}\right)$

VALUE OF $\sigma_h \left(\frac{H^2}{Q_p}\right)$

LINE LOAD Q_L

$x = mH$

$z = nH$

FOR $m \leq 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

$$P_H = 0.55 \, Q_L$$

FOR $m > 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{1.28 \, m^2 n}{(m^2 + n^2)^2}$$

RESULTANT $P_h = \frac{0.64 \, Q_L}{(m^2 + 1)}$

PRESSURES FROM LINE LOAD Q_L
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

POINT LOAD Q_p

$x = mH$

$z = nH$

FOR $m \leq 0.4$:

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$:

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77 \, m^2 n^2}{(m^2 + n^2)^3}$$

$$\sigma_h' = \sigma_h \cos^2(1.1 \, \theta)$$

WALL

SECTION a-a
PRESSURES FROM POINT LOAD Q_p
(BOUSSINESQ EQUATION MODIFIED
BY EXPERIMENT)

REFERENCE:
CANADIAN FOUNDATION
ENGINEERING MANUAL,
2nd EDITION, 1985

DRAWN BY :	CLIENT :
PROJECT No :	PROJECT :
DATE :	DESCRIPTION : LATERAL PRESSURE ON WALLS DUE TO POINT & LINE LOADS
SCALE : NOT TO SCALE	DRAWING : FIGURE 4

APPENDIX B: HYDRAULIC REPORT

Bruce Harding Consulting Ltd

To	Scott Minty, P.Eng. Manager – Transportation Group	**Date**	November 30, 2010
From	Bruce Harding, P.Eng.	**File**	GR2
Subject	Navin Drain Pedestrian Trail Crossing Hydraulic Assessment for Proposed Crossing Rev 2	**cc**	

This memorandum summarizes the results of our hydrologic analysis and hydraulic assessment for a proposed pedestrian trail crossing of Navin Drain immediately east of Lagimodiere Boulevard. The location of the site is indicated on Figure 1. Photos of the crossing and drain are appended for reference. The total contributing drainage area to the crossing has been estimated to be 18.4 km^2 with a corresponding 10% and 3% discharge of 4.7 m^3/s and 7.1 m^3/s respectively.

Other pertinent features of the site are as follows:

- Municipality   - City of Winnipeg
- Watercourse   - Navin Drain
- Stream Order   - 3rd order drain
- Flow Direction   - west
- Designation of Drain Map   - No. 9
- Total Drainage Area   - 18.4 km^2
- UTM Coordinates   - 640310E, 5522830N (Zone 14)

This reach of the Navin Drain has been classified by Fisheries and Oceans Canada[1] as Type E – ephemeral with indirect simple habitat. The typical fish passage requirements, as per the Manitoba Stream Crossing Guidelines[2] including maximum permissible velocity and embedment, will not be required for the proposed crossing.

An assessment on whether the Navin Drain is considered navigable has not been undertaken by Transport Canada; however it is unlikely that the waterway would be deemed navigable. On that basis it has been assumed that no provisions under the Navigable Waters Act are required for this proposed crossing.

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1 "Fish Habitat Classification for Manitoba Agricultural Watersheds", Map 062H14, March 2008, Fisheries and Oceans Canada.
2   "Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat", Manitoba Natural Resources - Fisheries Department and the Canadian Department of Fisheries and Oceans, May 1996.

The proposed replacement crossing would consist of a single 1.8m high by 2.4m wide by 5 m long reinforced concrete box culvert complete with headwalls. Additional details with respect to the hydrologic assessment, the hydraulic sizing and layout of the crossing are summarized in the following sections.

1   Flood Hydrology

The total contributing drainage area to the crossing has been estimated to be 18.4 km^2 as delineated on Figure 1. The hydrology for Navin Drain at the proposed crossing site was developed using transitional techniques, utilizing rational and regional discharge coefficients. The transitional approach utilizes direct interpolation on the basis of drainage area between the rational method estimate for a 13 km^2 drainage area and the regional method estimate for a 39 km^2 drainage area.

Rational Estimates

Runoff events, and not snowmelt typically govern flood hydrology for watersheds with small drainage areas. Manitoba Stewardship, Water Branch has developed standards[3], based on the application of rational analysis techniques, for estimating discharge for small rural watersheds. Unit area runoff values applicable to a specific frequency of occurrence are given, with correction factors to account for land use, soil type, slope and rainfall intensity. The drainage area is predominantly cropped, flat with tight clay. The following table summarizes the rational estimates for this site.

Table 1
Navin Drain Flood Hydrology
Rational Discharge Coefficients

Event	Mean Daily Discharge Coefficient (m^3/s/km^2)	Correction Factor for Land Use, Slope and Soil Type *	Correction Factor for Rainfall Intensity	Kelln Runoff Adjustment Factor	Adjusted Mean Daily Discharge Coefficient (m^3/s/km^2)
50% Flood	0.29	0.75	1.0	0.57	0.12
20% Flood	0.42	0.90	1.0	0.61	0.23
10% Flood	0.51	1.0	1.0	0.64	0.33
3% Flood	0.66	1.0	1.0	0.71	0.47

* - flat, cropped with tight clay soils

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3   "Runoff from Small Rural Watersheds", Province of Manitoba, Water Stewardship

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Regional Estimates

A streamflow gauge had been operated by Water Survey of Canada on Omand's Creek (Omand's Creek near Metro Route 90 – WSC 05MJ007) for the period from 1978 to 1993. The flood hydrology derived for the Omand's Creek streamflow gauge was selected as the index gauge for Navin Drain. The flood hydrology for the Omand's Creek gauge 05MJ007 was developed by Manitoba Water Stewardship (MWS) utilizing recorded and correlated data for Omand's Creek. The flood hydrology and computed regional discharge coefficients for the Omand's Creek gauge are summarized in Table 2.

Table 2
Omand's Creek at Metro Route 90 – WSC Gauge 05MJ007
Flood Hydrology – Regional Coefficients and Flood Estimates

Flood Event	Flood Estimate Omand's Creek near Metro Route 90 Gauge 05MJ007 Drainage Area = 72.3 km^2 (m^3/s)	Regional Discharge Coefficient
50% Flood	2.7	0.102
20% Flood	6.8	0.257
10% Flood	10.4	0.393
3% Flood	17.8	0.673
Q3D10	7.2	0.273

* - from Regional Flood Formulae Tables, Zone 3, Manitoba Water Stewardship, August 21, 2009, n=0.765

Flood Estimates

Table 3 summarizes the transitional estimates for Navin Drain at the proposed pedestrian trail crossing.

Table 3
Navin Drain Pedestrian Trail Crossing
Flood Hydrology Estimates

Flood Event	Flood Estimate Navin Drain - Drainage Area = 18.4 km^2 (m^3/s)
50% Flood	1.6
20% Flood	3.3
10% Flood	4.7
3% Flood	7.1
Q3D10	3.3

A 3% discharge will be selected as the proposed design discharge for the pedestrian trail crossing. It was noted that the downstream Lakewood Boulevard box culvert crossing of Navin Drain was designed for a discharge of 7.1 m³/s, which is equivalent to the design discharge proposed for the pedestrian trail crossing.

3   Hydraulic Sizing of Replacement Crossing

A steady-state backwater model of the Navin Drain within the study reach was developed using the US Army Corps of Engineers River Analysis System HEC-RAS model. The HEC-RAS model is a one-dimensional backwater model, which is considered to be the universal standard for computing steady-state water surface profiles. The backwater model was assembled from cross-sections, channel profiles and details of the crossings surveyed by Genivar, the City of Winnipeg and the R.M. of Springfield.

The backwater model has been developed to the level of detail required to estimate the relative effect of the proposed crossing. The model has not been calibrated to observed water levels during periods of high flow, and hydraulic parameters such as channel roughness have been selected based on observations, judgement and experience gained from similar projects.

The hydraulic design criterion selected for the replacement crossing is as follows:
- Design discharge – 3% Flood
- Maximum headloss of 0.3 m during the passage of the design discharge
- Culvert soffit to remain free of water surface by a minimum of 0.2 m during passage of design discharge.

The crossing would not be subject to the specific velocity requirements due to the habitat classification; but the assessment will be presented. The culvert length as proposed is less than 25 m; therefore a maximum permissible fish passage velocity of 1.0 m/s during the passage of the 3 day delay –10% fish passage discharge (Q3D10) would typically be required.

The details of the proposed crossing are summarized as follows:
- Single 1.8m high by 2.4m wide by 5 m long reinforced concrete box culvert complete with headwalls.

- Culvert set level with no embedment and not backfilled with rock. Upstream and downstream culvert inverts set at elevation 231.10.
- Culvert set with no skew.
- Refer to appended sketches of the proposed crossing for additional details.

The upstream and downstream aprons should be armoured to minimize erosion and to ensure long term function. The crossing aprons would have the following geometry:

- Base width = 3.0 m
- Armoured apron length at proposed base width = 3.0 m upstream and downstream of culvert ends
- Channel invert at upstream and downstream ends of culverts = 231.10
- Rock armouring to be Class 450 rock 0.65 m thick. Rock armouring to extend to elevation 233.0
- Channel side slopes at 3H:1V
- A transition from the upstream apron to the existing channel is required due to the discontinuity in the channel grade. The transition should be over a 4 m length using Class 450 rock.
- The base of the drainage channel should be excavated to elevation 231.10 between the proposed pedestrian crossing and the Lagimodiere box culvert crossing.

The estimated water surface profiles with the proposed crossing are shown on Figure 2. Table 4 summarizes the hydraulic assessment for the proposed crossing.

Table 4
Navin Drain Pedestrian Trail Crossing
Hydraulic Summary for Proposed Crossing
Proposed 1.8 m high by 2.4 m wide by 5 m long Reinforced Concrete Box Culvert

Flood Event	Discharge	Water Level Downstream of Crossing	Headloss	Soffit Clearance	Average Culvert Velocity
	(m^3/s)	(m)	(m)	(m)	(m/s)
50% Flood	1.6	231.82	<0.05	1.1 clear	0.95
20% Flood	3.3	232.12	0.07	0.8 clear	1.4
10% Flood	4.7	232.32	0.10	0.6 clear	1.7
3% Flood	7.1	232.62	0.14	0.35 clear	2.1
Q3D10	3.3	232.12	0.07	0.8 clear	1.4

It was noted that the upstream Navin Drain culvert crossings located at Plessis, Dawson and

Bruce Harding Consulting Ltd

Symington Roads are two 1500mm diameter corrugated steel culverts, which would be consistent with what is proposed at the pedestrian crossing when the additional drainage area is taken into consideration.

5   Other Considerations

Best Management Practices for working near waterways including the appropriate implementation of sediment and erosion control measures should be followed. Exposed slopes not covered with riprap should be seeded with native flood resistant species and covered with erosion control blanket. Construction activities within the drain shall not take place between April 1 and June 15 of any given year.

Water management during construction can be an important aspect of any project and may influence the cost and scheduling for crossing replacement. The largest flows within the drain are expected to occur during the spring runoff period and following a heavy summer rainfall event. Construction should take place in the fall and winter period when the potential for runoff is reduced thereby minimizing water management requirements. It is anticipated that the drain would not be flowing throughout the fall and winter.

Habitat compensation shouldn't be required to offset the loss of aquatic habitat due to the proposed crossing, as this reach is classified as Type E habitat.

Bruce Harding, P.Eng.
Senior Hydraulic Engineer

NAVIN DRAIN PEDESTRIAN TRAIL CROSSING
D.A. = 18.4 km²
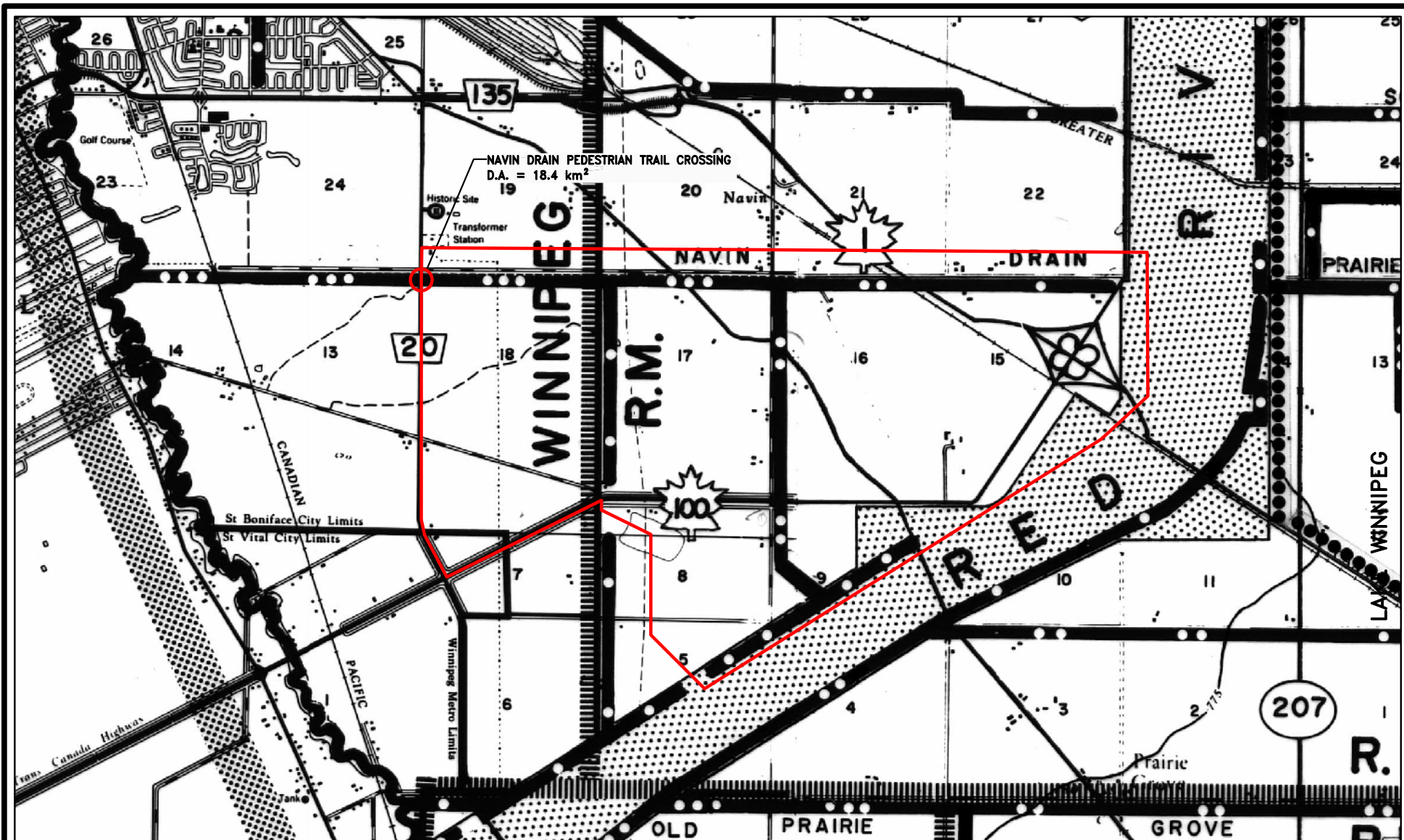
WATERSHED MAP 9

NAVIN DRAIN PEDESTRIAN TRAIL CROSSING
LOCATION PLAN
FIGURE 1

SCALE 1:50000 (METRES)
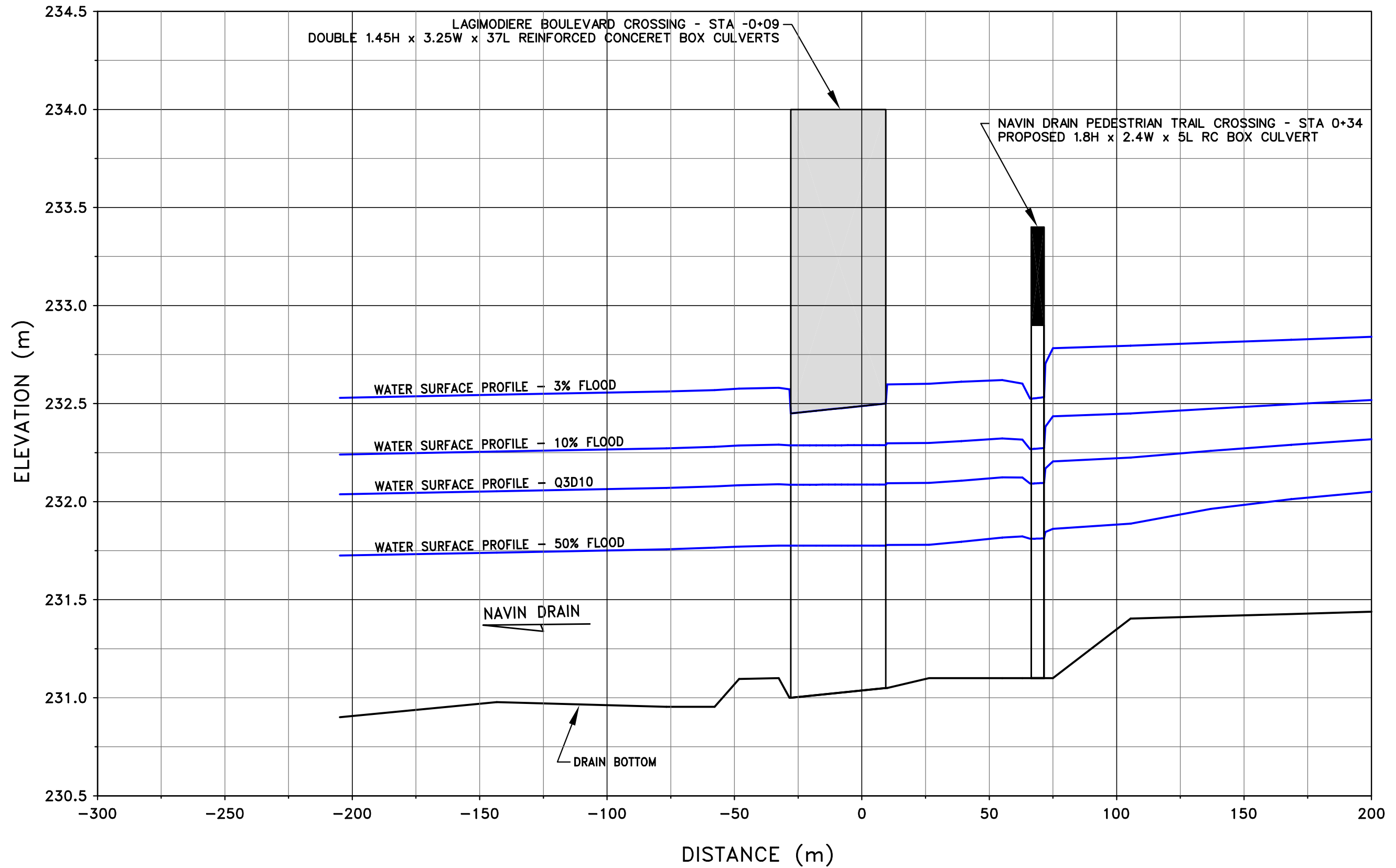
1250   0   1250   2500

NAVIN DRAIN PEDESTRIAN TRAIL CROSSING
WATER SURFACE PROFILES WITH PROPOSED CULVERT CROSSING
FIGURE 2

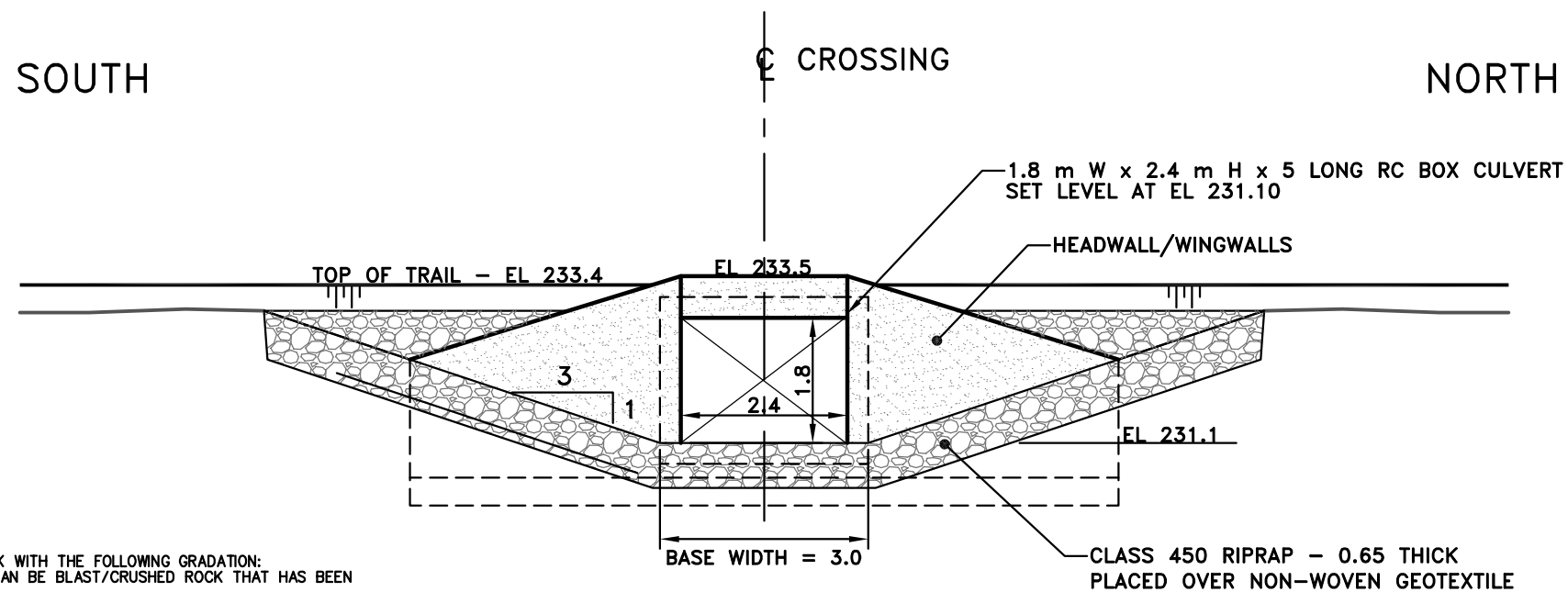WEST   EAST

| DOWNSTREAM APRON = 3 m | 3.9 | 5.0 | 3.9 | UPSTREAM APRON = 3 m | TRANSITION LENGTH = 4 m |

TOP OF RIPRAP EL 233.0

₵ TRAIL
EL 233.4

EXISTING DRAIN BOTTOM

FLOW   1.8

EL 231.1

EXISTING DRAIN BOTTOM

EL 231.3 +/-

CLASS 450 RIPRAP — 0.65 THICK
PLACED OVER NON-WOVEN GEOTEXTILE

CLASS 450 RIPRAP — 0.65 THICK
PLACED OVER NON-WOVEN GEOTEXTILE
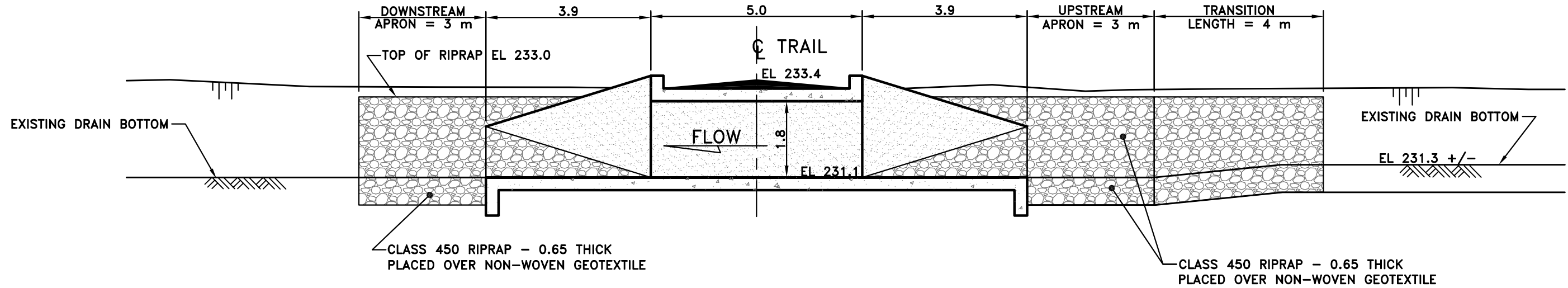
PROFILE THROUGH CULVERT CROSSING
SCALE 1:100

SOUTH   ₵ CROSSING   NORTH

1.8 m W x 2.4 m H x 5 LONG RC BOX CULVERT
SET LEVEL AT EL 231.10

TOP OF TRAIL — EL 233.4

EL 233.5

HEADWALL/WINGWALLS

3
1

1.8

2.4

EL 231.1

BASE WIDTH = 3.0

CLASS 450 RIPRAP — 0.65 THICK
PLACED OVER NON-WOVEN GEOTEXTILE

UPSTREAM ELEVATION
SCALE 1:100

NOTES

1) CLASS 450 RIPRAP TO BE WELL GRADED, ROUNDED, CLEAN, SOUND ROCK WITH THE FOLLOWING GRADATION:
    100%< 450 mm, 15-50% < 350 mm AND 0-15% < 100 mm. THE ROCK CAN BE BLAST/CRUSHED ROCK THAT HAS BEEN
    PROCESSED TO THE SPECIFIED GRADATION.
2) ALL EXPOSED EARTH SLOPES NOT COVERED WITH RIPRAP WILL BE SEEDED AND COVERED WITH EROSION
    CONTROL BLANKET.
3) CULVERT INSTALLATION TO FOLLOW STANDARD PRACTICES FOR BEDDING AND BACKFILL

SCALE 1:100 (METRES)
2.5   0   2.5   5

NAVIN DRAIN PEDESTRIAN TRAIL CROSSING
PROPOSED CROSSING DETAILS

Photo No. 1   Navin Drain west of Lagimodiere Boulevard (May 2005)

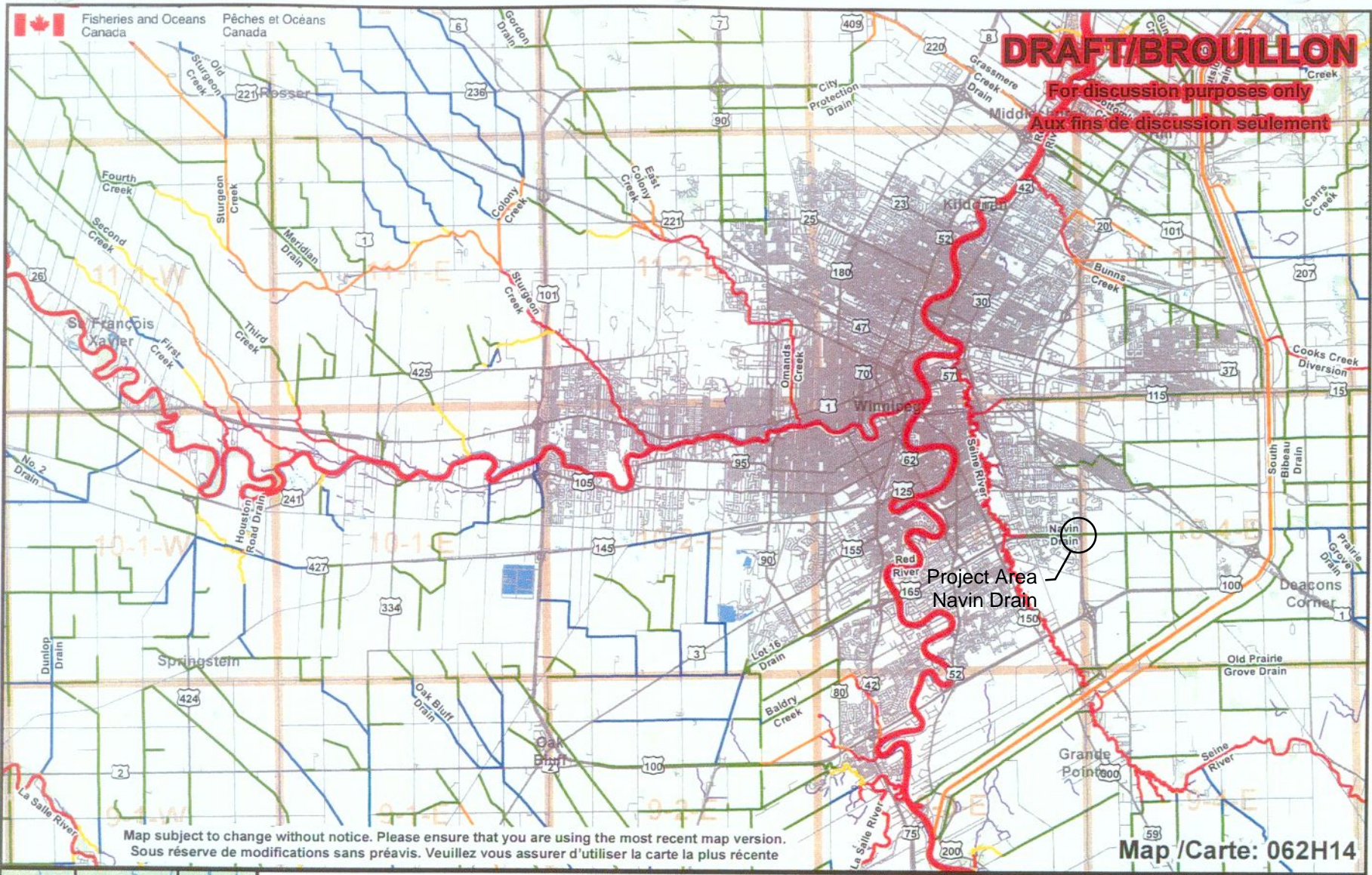**Photo No. 2**   West side of Lagimodiere Boulevard Crossing (May 2005)

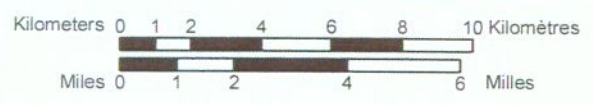**Photo No. 3** East side of Lagimodiere Boulevard Crossing (May 2005)

Photo No. 4 Navin Drain east of Lagimodiere Boulevard near proposed crossing (May 2005)