Assessment of Subgrade Soils for Pavement Design for Highway 407, East Extension Pickering to Oshawa, Ontario

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Abstract: Assessment of Subgrade Soil for Pavement Design for Highway 407, East Extension Pickering to Oshawa, Ontario

Although a variety of different input is required for pavement design, subsoil characteristics are one of the key inputs.

In Phase 1 of the extension of Highway 407 eastbound (Pickering-Whitby-Oshawa) and the connection to Highway 401 (West Durham Link), the subgrade soil has been investigated with nearly 700 test pits and boreholes, evenly spaced along the route, over nearly 30 km. The subgrade soil resilient modulus was measured using a light weight deflectometer (LWD) in some test pits.

The mechanical characteristics of the subgrade soil, mainly ranging from silty sand to sandy silt in the northern section and from silty clay to clayey silt in the southern section, were determined using CBR and resilient modulus testing in the laboratory.

The thermal characteristics of the subgrade soil were determined using soil segregation potential tests (SP_0) and standard geotechnical tests (sieve and hydrometer analysis, Atterberg limits and methylene blue values).

Various correlations were established between the different results obtained to achieve the most accurate possible characterization of the materials while avoiding long and costly testing. The methodological approach used to adequately assess these subgrade soils will be presented. While pavement's structural design is pretty much detailed in AASHTO 1993 design procedures, a new approach for thermal design (frost protection) will also be presented.

1.   Introduction

Pavement design of a new highway should address many parameters, where some of them are fixed by contractual constraints and where some of them have to be established by testing or correlation with other parameters. The present paper will address the methodology that has been used to determine the subgrade resilient modulus and to assess the frost considerations in the pavement design of Highway 407, east extension between Pickering and Oshawa. These two parameters are the two main elements that may influence the pavement design, especially in Canada.

While the pavement's structural design was pretty much restricted to the AASHTO 1993 design guide, a new approach for thermal design (frost protection) will be presented.

2.   Project Description

The Phase 1 of the east extension of Highway 407 from Pickering to Oshawa includes a 20 km extension of Highway 407 (HWY 407), a 10 km of West Durham Link (WDL) between HWY 407 and HWY 401 and a 5 km of realigned HWY 401, as shown on Figure 1. HWY 407 will have 6 lanes between Brock Road and WDL and 4 lanes between WDL and Harmony Road. WDL will have 4 lanes on the entire length, and re-aligned HWY 401 will have 6 lanes as existing. The WDL is now called Highway 412.
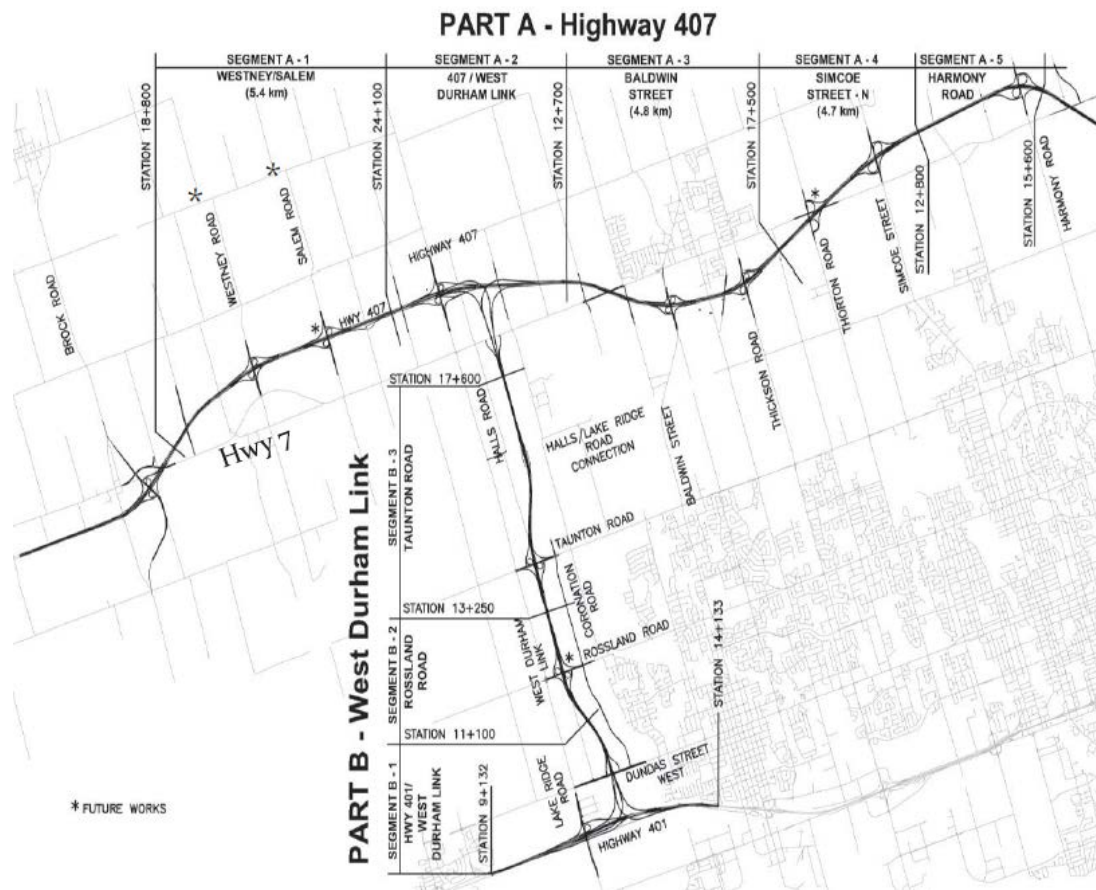

Figure 1: Phase I of Highway 407 East Extension Project

As it can be seen on Figure 1, the project was divided and designed in 8 segments (A-1 to A-5 and B-1 to B-3) plus the realignment of HWY 401. For the purpose of this paper, the methodology is discussed for the whole project while the results will mainly be presented for Segment A-1, to simplify the presentation.

Generally, the highway profile produced balanced cut and fill where about 50 % was in cut, the remainder in fill.

The project has been won by 407 East Development Group where 407 East Construction General Partnership (composed of SNC-Lavalin Construction and Ferrovial Agroman) has been retained for design and construction. In this project, SNC-Lavalin, Environment and Geoscience has been in charge of geotechnical engineering and pavement engineering. The main milestones of the project were:

› June 2010:   Request for Qualification (RFQ)
› March 2011:   3 Pre-qualified firms announced
› April 2011:   Request for proposal (RFP) with preliminary design
› May 2012:   Announcement of winner
                  Start of design and construction
› Summer 2016: Open to traffic

During the RFP phase, a preliminary design has been established for the geometry and also for the pavement design. This paper will show how the methodology has optimized the pavement design during detailed design phase.

3.   Contractual Parameters for Pavement Design

3.1   Pavement Design Parameters

For the pavement design, the following reference documents had to be considered by the project agreement:

› 1993 AASHTO *Guide for the Design of Pavement Structures*;
› MTO Materials Information Report MI-183 *Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions*, where the following parameters are fixed:
  o Z_R: Standard Normal Deviation (R: Design reliability)
  o S_0:   Overall Standard Deviation
  o ΔPSI: Variation of Serviceability Index
  o a: Structural Coefficient
  o m: Drainage Coefficient
› MTO Special Provisions;
› Ontario Provincial Standard Specification;
› SDO-90-01, *Pavement Design and Rehabilitation Manual* (MTO);
› MTO Directive PHM-C-001 *The Use of Surface Course Types on Provincial Highways*;

Design of all new pavement structures has been done in accordance with the 1993 AASHTO *Guide for the Design of Pavement Structures*. Except for subgrade resilient modulus, AASHTO

Pavement design parameters have been selected as described in the MTO Materials Information Report MI-183 *Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions*. Frost protection considerations have been done in accordance with *Cold Regions Pavement Engineering* manual.

3.2   Pavement Performance Parameters

Since it is a private-public partnership, certain performance criteria are determined in the Project Agreement. For the pavement, these are:

› Roughness (IRI)
  o "Roadway sections"   IRI < 1,8 m/km (for flexible pavement)
  o 50 m   IRI < 2,5 m/km (for flexible pavement)
› Pavement Condition Index (PCI) et Distress Manifestation Index (DMI)
  o "Roadway sections"   PCI > 65
  o 50 m   DMI > 7
› Rutting
  o "Roadway sections"   Depth < 13 mm
  o 50 m   Depth < 20 mm
› Skid Resistance
  o 500 m   Friction value > 30

It might be noticed that the values for IRI and rutting are quite stringent and these could have an influence in the design, especially for the frost consideration which is known to influence the IRI annual evolution.

We will see in Section 6 how frost considerations were taken into account in the design for the pavement design.

3.3   Traffic

Pavement structures should also be designed according to future traffic. In order to design pavement structures, traffic data supplied by the Concessionaire were used. Table 1 presents design traffic data.

Table 1: Concessionaire Traffic Volume

	Freeway	Segment	Design ESAL
407	East limit to Lake Ridge Rd	A-1, A-2	23 555 185
	Lake Ridge Rd to Baldwin St.	A-2, A-3	19 499 554
	Baldwin St. to Simcoe St	A-3, A-4	16 683 799
	Simcoe St. to Harmony Rd	A-4, A-5	13 550 426
WDL	South limit to North limit	B-1/2/3,A-2	10 817 128
401	West limit to East limit	B-1	53 217 881

4.   Subgrade Soils

For the final pavement design, 450 test pits along the Highway have been completed between November 25, 2012 and March 13, 2013 with a backhoe. Test pit depth varied between 1 and 4 m. Shallow ones were completed in fill areas, and deeper ones were completed in cut areas.

Site investigation for the project also included boreholes for structures, boreholes in high fill or deep cut areas and test pits in proposed ponds.

Most of the gathered samples have been subject to water content determinations, Atterberg's limits or standard sieve and hydrometer analysis.

As shown in Figure 2, in-place soil resilient moduli have been measured with a PRIMA100 light weight deflectometre (LWD) in 60 test pits where the cuts were shallow. It should be noted that in-place resilient moduli should be multiplied by an adjustment factor of no more than 0.33 prior to be used in AASHTO model. An example of results for Segment A-1 is presented in Table 2.

Figure 2: LWD Testing in Test Pits

Table 2: Example of Results for Segment A-1

TP	Depth	Description	Dynamic Resilient Modulus [1] (MPa)
TP-A1-05	0.6 m	sand, some silt, trace gravel and clay (SM)	25
	1.5 m		102
TP-A1-06	1.1 m	silt and sand, some clay, trace gravel (ML)	143
	2.0 m		280
TP-A1-08	1.0 m	silt, trace sand, clay and gravel (ML)	73
	2.0 m		263
		Average	**148**

(1) For a stress level of ± 37 kPa.

To complete mechanical soil characteristics determination, 42 samples have been compacted in laboratory and subjected to CBR measurements (ASTM D 1883 - Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils).

Sixteen (16) of these samples have also been subjected to laboratory resilient modulus measurements according to AASHTO T 307-99 - Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials. How those results have been used will be discussed in Section 5.

In general, the subgrade soils were mostly composed of a sandy silt to a silty sand for segments A-1 to A-5 and B-3. For Segments B-1 and B-2, they were composed mostly of clayey soils (silty clay or clayey silt) showing softer behavior.

5.   Methodology for Assessment of Subgrade Modulus for Pavement Structural Design

During the RFP stage, the preliminary design of the pavement has been carried out by using a subgrade resilient modulus (M_R) of 25 MPa. This prudent value has been selected for all the segments from Materials Information Report MI-183 *Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions*, as shown in Table 3.

Table 3: Recommended M$_R$ Values for the AASHTO–Ontario Pavement Design Model

Brief Description	Category No.	MTO Classification (MTO, 1980)	Drainage Characteristics	Susceptibility to Frost Action	Resilient Modulus (M$_R$) for Typical Subgrade Conditions, MPa		
					Good	Fair	Poor
Rock, rock fill, shattered rock, boulders/cobbles	1	Boulders/ cobbles	Excellent	None	90	80	70
Well graded gravels and sands suitable as granular borrow	2	GW, SW	Excellent	Negligible	80	70	50
Poorly graded gravels and sands	3	GP, SP	Excellent to fair	Negligible to slight	70	50	35
Silty gravels and sands	4	GM, SM	Fair to semi-impervious	Slight to moderate	50	35	30
Clayey gravels and sands	5	GC, SC	Practically impervious	Negligible to slight	40	30	25
Silts and sandy silts	6	ML, MI	Typically poor	Severe	30	25	18
Low plasticity clays and compressible silts	7	CL, MH	Practically impervious	Slight to severe	35	20	15
Medium to high plasticity clays	8	CI; CH	Semi-impervious to impervious	Negligible to severe	30	20	15

Many relationships exist that relates CBR and resilient modulus (M$_R$). To optimize the design, we looked at establishing our own relationship, because our test results on samples did not fit with AASHTO relationship, presented in Figure 3 based on work by Heukelom and Klomp, between CBR and resilient modulus (M$_R$).

$$M_R \ (psi) = 1500 \ CBR \ (for \ CBR<10)$$

However, AASHTO mentions that constant varies between 750 and 3 000 times CBR. For tested samples, coefficient between 2,000 and 2,500 seems to be more appropriate for Segment A-1.
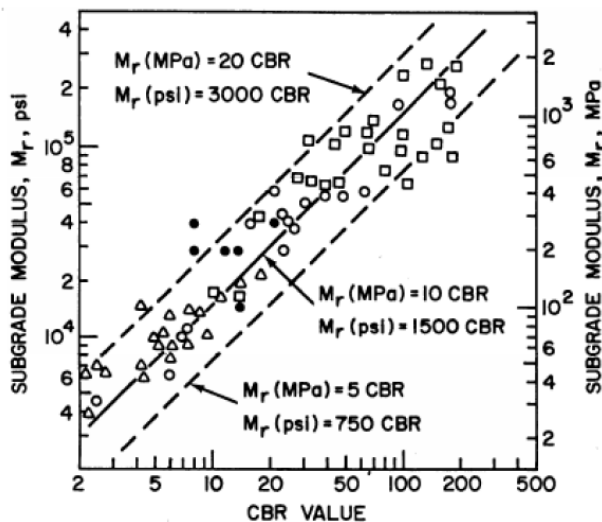

Figure 3: AASHTO Relationship between M$_R$ and CBR

Others relationships between CBR and resilient modulus (M_R) were available in literature, like one included in NCHRP 1-37A (new AASHTO pavement design guide, level 2).

$$M_R \ (MPa) = 17.6(CBR)^{0.64}$$

This relationship was better but still not accurate to our data.

AASHTO *Guide for the Design of Pavement Structures* (Part I, article 1.5) indicates that the design of the pavement structure should be based on average resilient modulus. Low value, based on some minimum or conservative criteria, should not be selected as this will introduce increased conservatism in design beyond that provided by the reliability factor.

To establish this average resilient modulus, two steps have been followed. First, based on 16 resilient modulus laboratory tests completed on samples gathered along HWY 407 and West Durham Link (WDL), a new relationship between CBR and resilient modulus (M_R) has been established as shown in Figure 4.
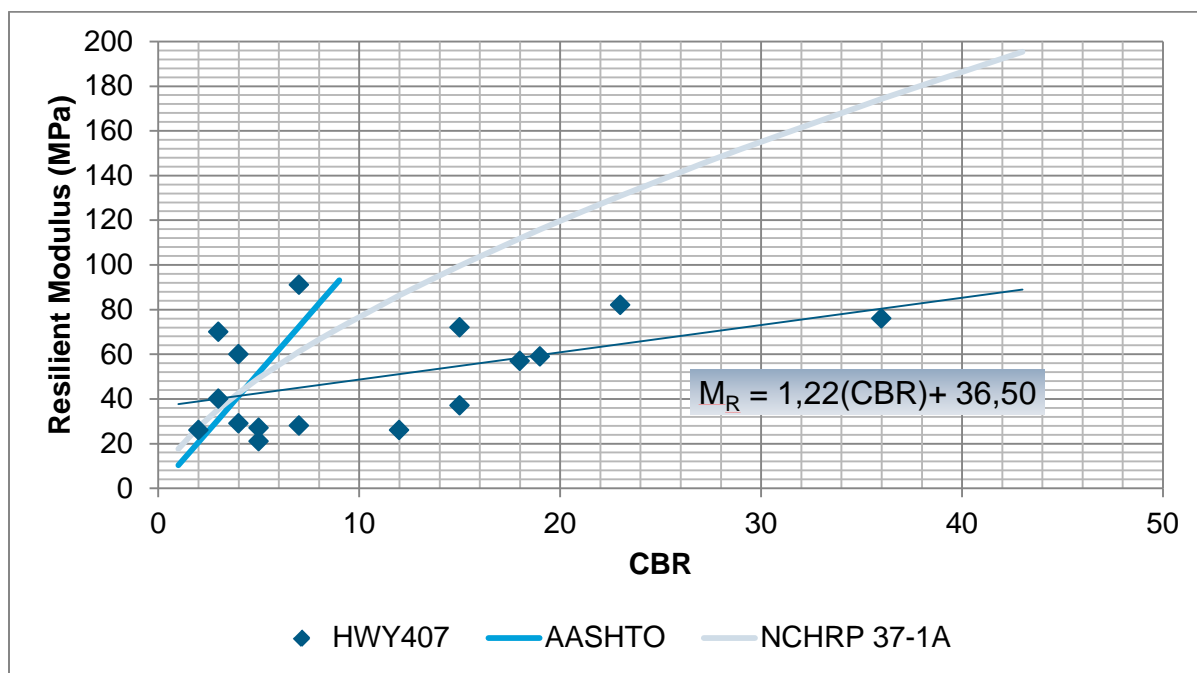

Figure 4: Relationships between CBR and Resilient Modulus (M_R)

Based on tests completed on 16 samples gathered on all segments, resilient modulus can be estimated from CBR according to the following equation.

$$M_R \ (MPa) = 1.22CBR + 36.50$$

Then this relationship has been applied to all CBR results measured on samples gathered in each segment. Finally, the average resilient modulus (M_R) has been calculated for each segment. An example is shown in Table 4 for Segment A-1.

Table 4: Average Subgrade Resilient Modulus (M$_R$) for Segment A-1

Road	TP	Sample	Measured CBR	Resilient Modulus (MPa)		Average
				Measured	Estimated from CBR[1]	
HWY 407	TP-A1-01	GS4	4	60	60	
HWY 407	TP-A1-06	GS6	2		39	
HWY 407	TP-A1-08	GS5	3	40	40	
HWY 407	TP-A1-22	GS8	3		40	47
HWY 407	TP-A1-31	GS3	2		39	
HWY 407	TP-A2-03	GS5	23	82	82	
LakeRidge	TP-A2-06	GS5	7	28	28	

(1) M$_R$ = 1.22CBR + 36.50

For each segment, the average resilient modulus (M$_R$) has been checked to be similar to the LWD test results. As mentioned, average LWD results should be corrected by 0.33 to get an AASHTO equivalent resilient modulus. For example, on site measurements gave an average of 49 MPa (148 MPa x 0.33) for LWD test compared to 47 MPa retained M$_R$ for Segment A-1.

Finally an effective subgrade soil resilient modulus has been calculated from the average value, taking into consideration seasonal material variation. AASHTO procedure (Part II article 2.3.1) was followed. As recommended 138 MPa (20 000 psi) has been considered for frozen conditions. For spring-thaw conditions, the retained modulus has been 30% of the normal summer modulus, i.e. the average resilient modulus calculated above, for one month and 50% of the normal summer modulus for one month. AASHTO procedure is summarized in Table 5. For example, the design effective roadbed soil resilient modulus (M$_R$) for pavements of Segment A-1 has been determined to be 32 MPa.

Table 5: Effective Subgrade Resilient Modulus for Segment A-1

Month	Resilient Modulus (M_R)		Relative Damage (μ_f) [1]
	MPa	psi	
January	138	20 010	0.01
February	138	20 010	0.01
March	14	2 045	2.46
April	24	3 408	0.75
May	47	6 815	0.15
June	47	6 815	0.15
July	47	6 815	0.15
August	47	6 815	0.15
September	47	6 815	0.15
October	47	6 815	0.15
November	47	6 815	0.15
December	138	20 010	0.01
		Total	4.31
		Average	0.36
		effective M_R	4 689 psi
			32 MPa

(1) $\mu_f = 1.18 \times 10^8 \times M_R^{-2.32}$

Other segments showed values of effective resilient modulus varying from 28 MPa to 43 MPa, as shown in Table 6.

Table 6: Effective Resilient Modulus for All Segments

Segments	Effective Resilient Modulus (MPa)
A-1	32
A-2	43
A-3	40
A-4 and A-5	29
B-1	28
B-2	30
B-3	37

6.   Methodology for Assessment of Frost Susceptibility

First of all, it should be noted that silty subgrade soils of new Highway 407 have a high to very high frost susceptibility according to *AASHTO Chart for Estimating Frost Heave rate for Roadbed Soil*.
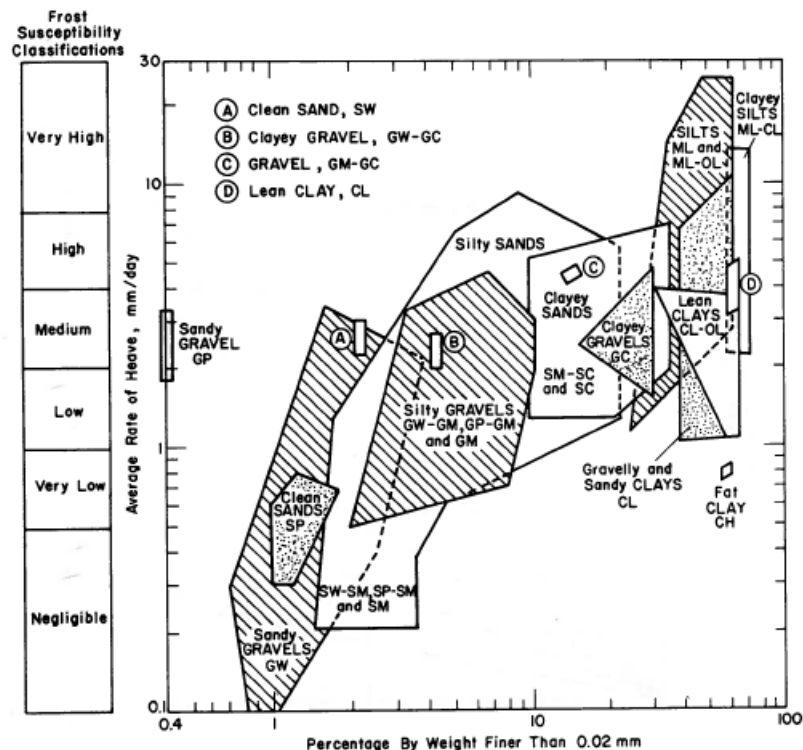

Figure 5: AASHTO Chart for Estimating Frost Heave Rate for Roadbed Soil

Regarding pavement frost design, MTO document SDO-90-01 (*Pavement Design and Rehabilitation Manual*, January 1990) as well as specification OPSS 314 (*Construction Specification for Untreated Granular, Subbase, Base, Surface Shoulder, and Stockpiling*, November 2004) have been reviewed. We also reviewed Highway 407 Project Agreement requirements to see if the project includes specific requirements.

None of the consulted documents presents specific requirements regarding minimum frost protection pavement thickness. SDO-90-01 manual indicates that the depth of frost penetration makes it impractical to provide pavement thickness that would prohibit frost into the subgrade. Such thickness would far exceed the requirements for bearing capacity. This manual recommends uniformization of the subgrade soils as well as designing pavement structure based on reduced strength of the subgrade soil during critical spring thaw period. Good pavement drainage and appropriate transitions between subgrade soils having different properties will be also helpful to prevent bad winter pavement performance. In absence of clear rules, pavement frost design seemed to be engineer judgement.

Pavement frost protection design has been based on the SSR model developed in Finland by Saarelainen which can be found in *Cold Regions Pavement Engineering* by Guy Doré of University Laval (Quebec) and Hannele K. Zubeck of University of Alaska (Anchorage). Depths

of frost in pavement as well as heaving are calculated according to thermal equilibrium at the freezing front. The purpose of this new approach is to limit the heaving of the pavement.

As established by Doré (1997 and 2002), the IRI annual evolution (ΔIRI/year) is influenced by the pavement's heave that it will suffer during the winter months, as shown in Figure 6.
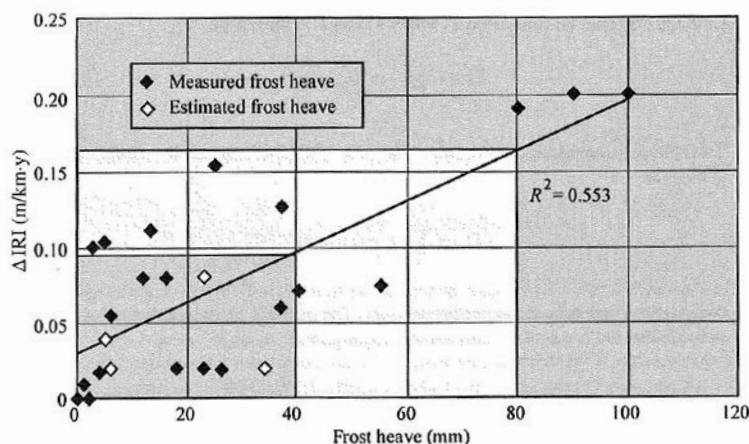

**Figure 6: Relationship between Frost Heave and Roughness Evolution
of Several Road Sections in Quebec Province (Doré 1997, 2002)[1]**

In Finland, Tammirinne considered a heaving less than values varying from 30 to 100 mm is necessary to maintain a good level of service and to limit pavement cracking. For Konrad, from University Laval in Quebec, acceptable heaving should be between 30 and 60 mm. Ministère des Transports du Québec (MTQ) have decided to limit heaving between 50 and 70 mm depending of the classification of the road.

Considering the performance criteria for IRI, it has been recommended to limit the heave to a value below 60 mm for the Highway 407. For an IRI requirement of 2.5 m/km and an initial IRI of 0.9 m/km at the end of construction, mill and overlay should be required after 12-13 years if we use an evolution rate of 0.13 m/km x year, which corresponds to a heave of 60 mm {(2.5 m/km – 0.9 m/km) / 0.13 m/km x year}. Mill and overlay may also be required after these years to correct rutting, for which the typical rate could be around 1 mm/year.

SSR model is based on segregation potential (SP_0) during transient freezing. This parameter (SP_0) can be determined in the laboratory in a thermal chamber according to LC 22-331 - *Détermination du potentiel de ségrégation des sols* (MTQ specifications). At the RFP stage, the preliminary design used an average value of 5.0 mm^2/°C hr for all the segments. For the detailed design, a total of 8 samples have been submitted to this test at Laval University, where SP_0 were determined.

To establish the design value for segregation potential SP_0, two steps have been followed. First, based on gradation, Atterberg limits and methylene blue values, the segregation potential SP_0 can be estimated with the relationship established by Konrad[2] from the different samples gathered in all segments, as shown in Figure 7.
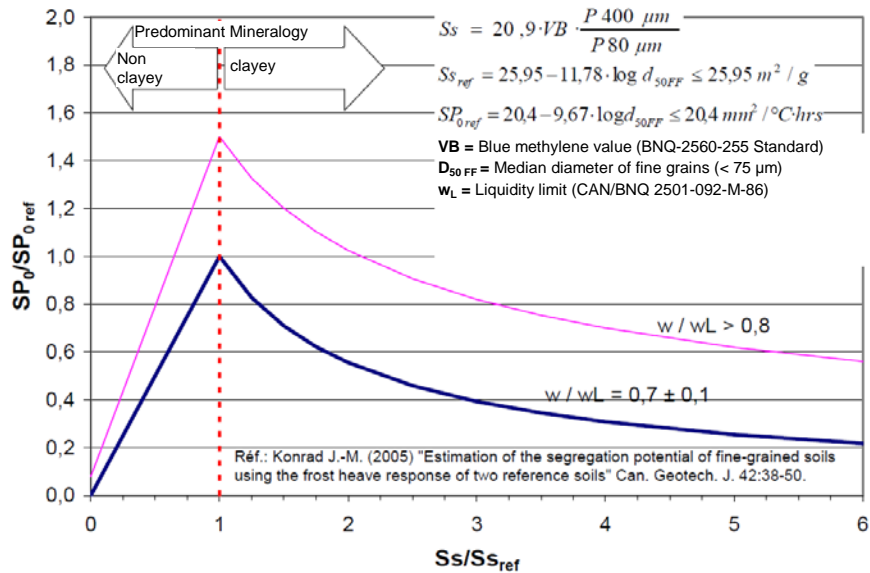
Figure 7: Estimated SP₀ by Konrad (2005)

Then, the lab results have been compared to the estimated values, as shown in Figure 8. With these results, a correlation has been established between the estimated SP₀ obtained by Konrad relationship and the SP₀ values measured in the lab at Laval University. So, estimated SP₀ should be corrected according to the following equation.

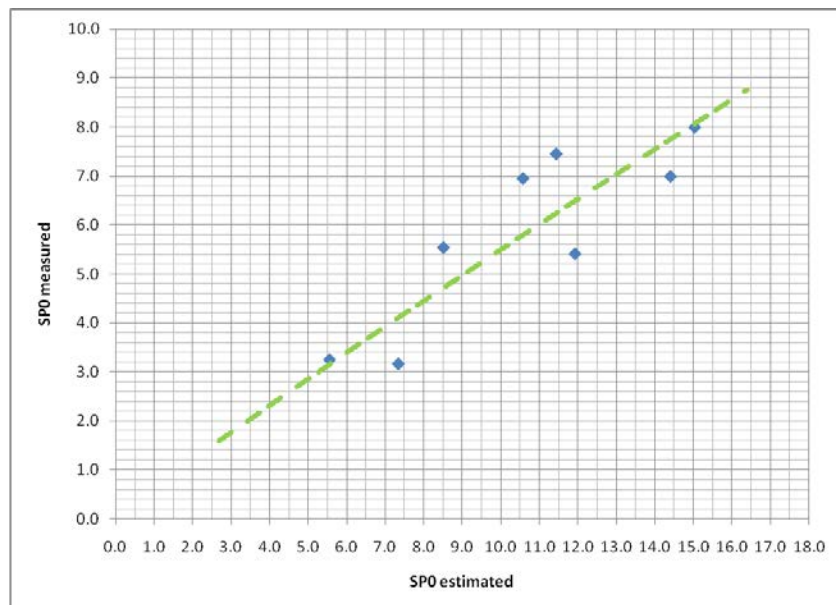$$SP_0 = 0.6263(estimated\ SP_0)^{0.9431}$$

Figure 8: Estimated SP₀ by Konrad (2005) vs Measured SP₀

Finally, this correlation has been used to calculate an average SP₀, where we have similar soils. That is for silty sand to sandy silt at Segments B-3 and A-1 to A-5 and clayey soils at Segments B-1 and B-2.

Table 7 presents corrected estimated SP_0 on samples gathered in segments B-3 and A-1 to A-5. According to the relationship mentioned above, the design SP_0 for Highway 407 has been fixed to 4.6 mm²/°C hr for these segments.

Table 7: Average Segregation Potential for Segments B3 and A1 to A5

Road	TP	Sample	SP_0 (mm²/°C hr)			
			Measured	Estimated (based on Konrad)	Corrected (1)	Average
WDL	TP-B3-04	GS6		12.0	6.5	
Taunton	TP-B3-06	GS5		3.9	2.2	
Taunton	TP-B3-09	GS5		8.9	4.9	
WDL	TP-B3-15	GS5	7.0	10.6	7.0	
Truck LayBy	TP-B3-19	GS4		3.4	2.0	
HWY 407	TP-A1-01	GS4		10.6	5.8	
HWY 407	TP-A1-06	GS6		6.2	3.5	
HWY 407	TP-A1-08	GS5		9.8	5.4	
HWY 407	TP-A1-22	GS8		14.4	7.8	
HWY 407	TP-A1-31	GS3		8.6	4.8	
HWY 407	TP-A2-03	GS5	3.3	5.6	3.3	
LakeRidge	TP-A2-06	GS5	5.5	8.5	5.5	
LakeRidge	TP-A2-10	GS4		4.2	2.4	
Ramp W-S	TP-A2-11	GS4	3.2	7.3	3.2	
HWY 407	TP-A2-27	GS3		8.7	4.8	
HWY 407	TP-A3-02	GS4	5.4	11.9	5.4	4.6
Baldwin	TP-A3-04	GS4		6.8	3.8	
Baldwin	TP-A3-07	GS5		5.6	3.2	
HWY 407	TP-A3-08	GS5		8.7	4.8	
Thickson	TP-A3-17	GS2		12.0	6.5	
HWY 407	TP-A4-01	GS3		5.8	3.3	
HWY 407	TP-A4-04	GS5		5.4	3.1	
HWY 407	TP-A4-05	GS5		8.3	4.6	
HWY 407	TP-A4-10	GS4	7.5	11.4	7.5	
HWY 407	TP-A4-20	GS4		2.7	1.6	
HWY 407	TP-A5-02	GS3		6.5	3.7	
HWY 407	TP-A5-04	GS4	7.0	14.4	7.0	
Harmony	TP-A5-13	GS3		4.5	2.6	
HWY 407	TP-A5-14	GS5		3.1	1.8	
HWY 407	TP-A5-16	GS8		11.5	6.2	
Harmony	TP-A5-37	GS4		13.0	7.0	

(1) $SP_0 = 0.6263(\text{estimated } SP_0)^{0.9431}$

The average SP_0 that has been used for pavement thermal design in Segments A-1 to A-5 and B-3 was 4.6 mm^2/°C hr and 6.4 mm^2/°C hr for segments B-1 and B-2, as shown below in Table 8.

Table 8: Design SP$_0$ for Frost Consideration

Segments	SP_0 (mm² / °C hr)
A-1 to A-5 and B-3	4.6
B-1 and B-2	6.4

7.   Pavement Design

Design of all new pavement structures has been done in accordance with 1993 AASHTO Guide for the Design of Pavement Structures. Except for subgrade resilient modulus, AASHTO design parameters listed in Table 9 were selected according to MTO Materials Information Report MI-183 Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions. Subgrade resilient modulus value was discussed previously.

Table 9: Design Parameters for Flexible Pavement

Parameters	Flexible Pavement [1]
W_{18}: Design ESAL's	See section 3.3
R : Reliability	92.5 %
So : Overall standard deviation	0.49
Po : Original Serviceability index	4.50
Pt : Terminal Serviceability index	2.60
M_R : Subgrade resilient modulus	32 MPa [3]
Layer coefficient for asphalt course	0.42
Layer coefficient for granular base course (Granular A)	0.14 [2]
Drainage coefficient	1.0

(1)   According to MTO Materials Information Report MI-183 *Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions*.
(2)   Resilient modulus of 211 MPa for layered analysis (a_{base} = 0.249 x (log E_{base}) - 0.977)
(3)   For Segment A-1. Other segments are presented in Table 6.

Layered analysis has been completed to determine minimum thicknesses of HMA and base layers. As required, proposed pavement structures have been designed for 20 years traffic (23.6 M ESAL for Segment A-1).

Chaussée2 software (MTQ) has also been used to calculate frost penetration and heaving according to the SSR model and a freezing index of 740°C-d. This design freezing index has been obtained from an average freezing index of 500°C-d (Oshawa WPCP), a coefficient of variation of 37 % and a return period of 10 years, which is half of the 20 years design period. A return period equivalent to one-half of the design period is a common practice.

Proposed pavement structures do not intend to insure a complete subgrade protection against the frost but it tries to limit the frost effect on crack opening and on pavement roughness annual evolution rate and to protect soil layers which lose their bearing strength during thaw periods. Highway minimal pavement thickness has been designed to limit heaving in the order of 60 mm.

Proposed pavement structure for Segment A-1 with a subgrade resilient modulus of 32 MPa is presented in Table 10. Calculated frost penetration and heaving are also shown.

Table 10: Pavement Structure for Segment A-1

Materials	Highway 407 in Segment A1 (32 MPa)	
	Thickness (mm)	Allowable MESALs (Layered Analysis)
HMA	215	25.6
Granular A	550	-
Total pavement thickness	**765**	**25.8**
Frost penetration (m)	1.299	-
Heaving (mm)	54	-

8.   Conclusion

At the end, the pavement has been designed for the different level of traffic (each segment) with 7 resilient modulus (some segment had the same modulus) and 2 segregation potential SP_0.

All the segments had a higher resilient modulus than estimated at the bid phase which concurred to lowering the construction cost.

As for the segregation potential SP_0, only 2 segments (B-1 and B-2) showed value higher than the estimated one at the bid phase.

Based on presented design parameters, structural design controls pavement thicknesses for 3 out of 8 segments, as shown in Table 11.

Table 11: Pavement Thicknesses for All Segments with Controlling Design Parameter (Structural or Thermal)

Section	Traffic M : ESAL	HMA (mm)	Granular A (mm)	M_R (MPa)	Pavement Thickness (mm)	SP_0 (mm^2/°C hr)	Frost Heaving for 740 °C d (mm)	Frost Pen. (m)	Control S: Structural T: Thermal
A-1	23.6	215	550	32	765	4.6	54	1.299	S
A-2	19,5	210	475	43	685	4,6	59	1.289	T
A-3	16.7	205	475	40	680	4.6	60	1.290	T
A-4 and A-5	13.6	200	550	29	750	4.6	55	1.300	S
B-3	10.8	190	500	37	690	4.6	59	1.295	T
B-2	10.8	190	700	30	890	6.4	60	1.185	T
B-1	10.8	190	700	28	890	6.4	60	1.185	T
HWY 401	53.2	250	625	28	875	6.4	60	1.169	S,T

The additional thickness added to the pavement structural design to address the thermal design (frost considerations) is shown in Table 12. It varies between 25 and 75 mm in Segment A-2, A-3 and B-3, but goes up to 150 to 175 mm in Segments B-1 and B-2 where SP_0 is higher in clayey soils.

Table 12: Additional Granular Base Thickness Required to Satisfy Heaving Requirements

Section	Control S: Structural T: Thermal	Additional granular thickness (mm)
A-1	S	-
A-2	T	50
A-3	T	25
A-4 and A-5	S	-
B-3	T	25
B-2	T	175
B-1	T	150
HWY 401	S, T	-

As it can be noticed in this paper, the performance criteria for IRI can have an impact on the pavement design since winter frost heave influences the evolution rate of IRI (ΔIRI/year).

Since the evolution rate of IRI in relation to heave has been measured mostly in the province of Quebec, more research could be needed to have a better understanding of this relationship in other provinces.

Also, frost heave calculation with representative values of SP_0 could be put forward in Ontario where we have similar soils than encountered in 407 area. More experimental data should be gathered to define more precisely the SP_0 value and the calculated heave for other areas.

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