DESCRIPTION OF A FIELD TEST FACILITY TO EVALUATE FWD INTERPRETATION METHODS

Simon GRENIER, Ph.D. Candidate, Département de génie civil, Faculté des Sciences Appliquées, Université Laval, Québec, Qc, Canada, G1K 7P4
Denis LEBOEUF, Associate Professor, Département de génie civil, Faculté des Sciences Appliquées, Université Laval, Québec, Qc, Canada, G1K 7P4, Denis.leboeuf@gci.ulaval.ca
Jean-Marie KONRAD, Associate Professor, Département de génie civil, Faculté des Sciences Appliquées, Université Laval, Québec, Qc, Canada, G1K 7P4, Jean-Marie.Konrad@gci.ulaval.ca

ABSTRACT
Deflections from falling weight deflectometer (FWD) are used to backcalculate the stiffness of the pavement structure and its subgrade. Because loading is transient, different dynamic forward methods and inverse algorithms have been developed to solve this problem. However, limited use of these tools is observed in the current practice. One of the reasons for their low spreading among road engineers is the lack in field comparison between FWD measured deflections and results form theoretical dynamics methods.

This paper present an extensive geotechnical investigation carried out on two flexible pavements of three layers built on an experimental site near Quebec City. The investigation of the sand deposit includes SPT and cross-hole tests. Also, laboratory tests were performed for physical and mechanical characterisation of soil and pavement materials. FWD deflections from two different devices and deflections from plate-loading tests provide a better understanding of the influence of loading on pavement response.

RÉSUMÉ
Les déflexions du déflectomètre à masse tombante (FWD) sont utilisés pour déterminer la rigidité de la structure de la chaussée et de son sol d'infrastructure. Différentes méthodes d'analyse dynamique et algorithmes d'inversion ont été développés pour résoudre ce problème. Cependant, ces outils font l'objet d'une utilisation limitée dans la pratique. Une des raisons pour leur lente diffusion chez les ingénieurs routiers est le manque de comparaison entre les déflexions mesurées par le FWD et les résultats théoriques donnés par les méthodes d'analyse.

Cet article présente une investigation géotechnique d'envergure sur deux chaussées flexibles à trois couches construites sur un site expérimental à proximité de la Ville de Québec. L'investigation du dépôt de sable comprend des essais SPT et des essais cross-hole. Des essais en laboratoire ont également été réalisés pour la caractérisation physique et mécanique des sols et des matériaux routiers. Les déflexions de deux différents FWD et celles tirées des essais de plaque fournissent une meilleure compréhension de l'influence du chargement sur le comportement structural des chaussées.

1. INTRODUCTION

The falling weight deflectometer (FWD) is increasingly used for structural non-destructive testing of roads. In a FWD test, a mass is dropped from a specified height and the resulting deflection basin is used to backcalculate the apparent stiffness of the pavement structure and its subgrade. Thus the residual life and the required reinforcement asphalt concrete thickness can be calculated to assess needs and design maintenance and rehabilitation interventions. This structural analysis of pavement is mainly done with the use of the multilayered linear-elastic theory. For backcalculation of stiffness moduli, CROW (1998) suggests to use the same theory in order the ensure consistency with the structural analysis. Thus, Burmister's theory is currently used as forward models in most pavement backcalculation methods (Ullidtz & Coetzee, 1995). This model considers that the load is statically applied over the loaded surface.

However, the dynamic nature of FWD test has been recognized by a number of researchers (Roesset & Shao, 1985; Magnuson & al., 1991) and limitations of the current practice have been noted by Stolle & Parvini (2001). Therefore, dynamic forward methods (Foinquinos Mera, 1995; Al-Khoury & al., 2001; Uddin, 2002) and inverse algorithms (Uzan, 1994) have been developed during the last twenty years to solve the dynamic problem and, hopefully, to get a better understanding of the pavement behaviour under dynamic loading conditions. However, most of the published results of dynamic analysis of FWD tests are for synthetic pavements with theoretical FWD loading, and little is found on real pavement structures where geotechnical soil properties and the boundary conditions are well known. Thus, there is a lack in field comparison between FWD measured deflections and results from theoretical dynamics methods.

The present paper aims at describing a test facility built to evaluate field FWD measurements. An extensive field and laboratory investigation program has been undertaken to characterize soil and pavement materials. It also presents deflection time histories and deflection basins measured by two different FWD equipments and by plates-loading tests showing the importance of the loading conditions on pavement structural response.

2. REVIEW OF LITERATURE

2.1 FWD device

The falling weight deflectometer is used to simulate a truck wheel in terms of load and duration. A mass is dropped from a specific height on a circular plate that produces a dynamic load at the surface of the pavement structure and the resulting deflections are collected with geophones. A schematic view of the FWD is shown in figure 1. Load intensity and duration depend on the device, the mass weight, the drop height, the loading system (plate, buffers) and the pavement rigidity.

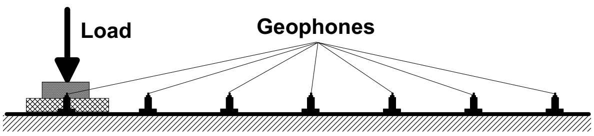

Figure 1: Schematic principle of FWD.

2.2 Theoretical FWD dynamic aspects

When an impact is generated at the surface of a pavement, like the load pulse from FWD tests, body waves and surface waves will propagate in the medium and at its surface. The Cauchy-Navier equation of motion describes waves in homogenous isotropic linear-elastic solids. The displacement field resulting from external stress follows elasticity law for given shear modulus (G) and first Lamé parameter (λ) (related to Young modulus by elasticity relationships) and it depends on solid density through inertial force. Damping can also be introduced by linear hysteretic damping (Foinquinos Mera, 1995). This equation of motion can be solved for layered systems such as pavement with adequate interface and boundary conditions (Al-Khoury & al., 2001). Real pavements are quite complex systems especially when materials are found to be stress sensitive (Stolle & Parvini, 2001). This stress sensitivity is usually accounted in the equation of motion by using secant modulus at corresponding loaded truck conditions simulated by the FWD avoiding more complex constitutive laws. Therefore, the important dynamic parameters to consider in FWD tests are: 1) material mechanical properties such as secant shear modulus, secant Young modulus, density and damping; 2) pavement structure such as layer thicknesses and boundary conditions; and 3) loading conditions such as load intensity and duration.

A research was undertaken to study the dynamic interpretation methods of FWD tests where these dynamic parameters are considered for real pavement structures. As part of this research, a field test facility has been built.

3. SITE DESCRIPTION

The selected site for FWD dynamic evaluation is located 50 km north of Quebec City (Canada) at Lac-St-Charles in a sand pit. This unique environment offers advantageous site conditions such as uniform soil deposit with a deep water table and bedrock. Because the site is not located on the road network, it avoids traffic control. Also, it gives opportunity for construction of road test sections because of on site material availability.

3.1 General site description

A site was selected within the sand pit to build the test facility. It covers an area of 80 m by 50 m on which two road sections were constructed as shown in figure 2. The section 1 is 20 by 15 m wide. The pavement structure is an average of 100 mm asphalt concrete surface layer with an average of 600 mm granular base layer resting on the subgrade sand deposit. Section 2 is 15 by 5 m wide with a lighter pavement structure consisting of 95 mm of asphalt concrete surface layer and a 100 mm granular base layer placed over the subgrade sand deposit. The small base layer was required for asphalting purposes. Because sections are embedded in the soil deposit, lateral discontinuities at section perimeter are thought to be insignificant. Between the two sections, a zone was delimited for excavation to measure the in situ soil density through 3.3 m depth.

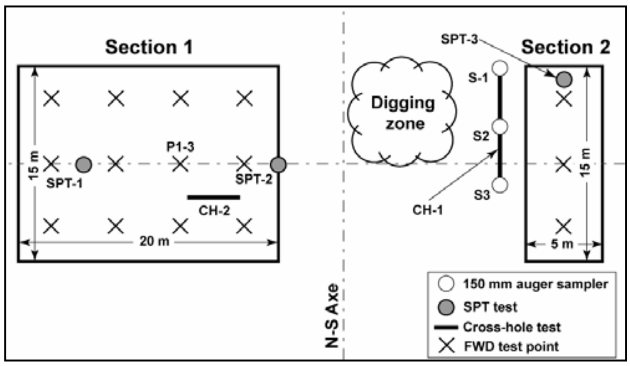

Figure 2: Field test facility configuration.

3.2 Geotechnical in situ tests results

An extensive field investigation program has been carried out at the test site, including three SPT tests, seismic refraction tests, sampling with a 150 mm auger sampler, cross-hole tests and density measurement with a nuclear moisture-density gauge. The resulting geotechnical profile is shown in figure 3. The following geotechnical features are included: soil description, USCS soil classification, in situ mass density (ρ), natural water content (ω_n), corrected N_1 values of SPT tests and shear wave velocity (V_s) from cross-hole tests. The sand deposit is covered by a 0.4 m layer of sand and silt with traces of gravel and organic soil, a 1.1 m layer of fine to medium sand with some gravel and traces of small boulders, a 2.0 m layer of fine to medium

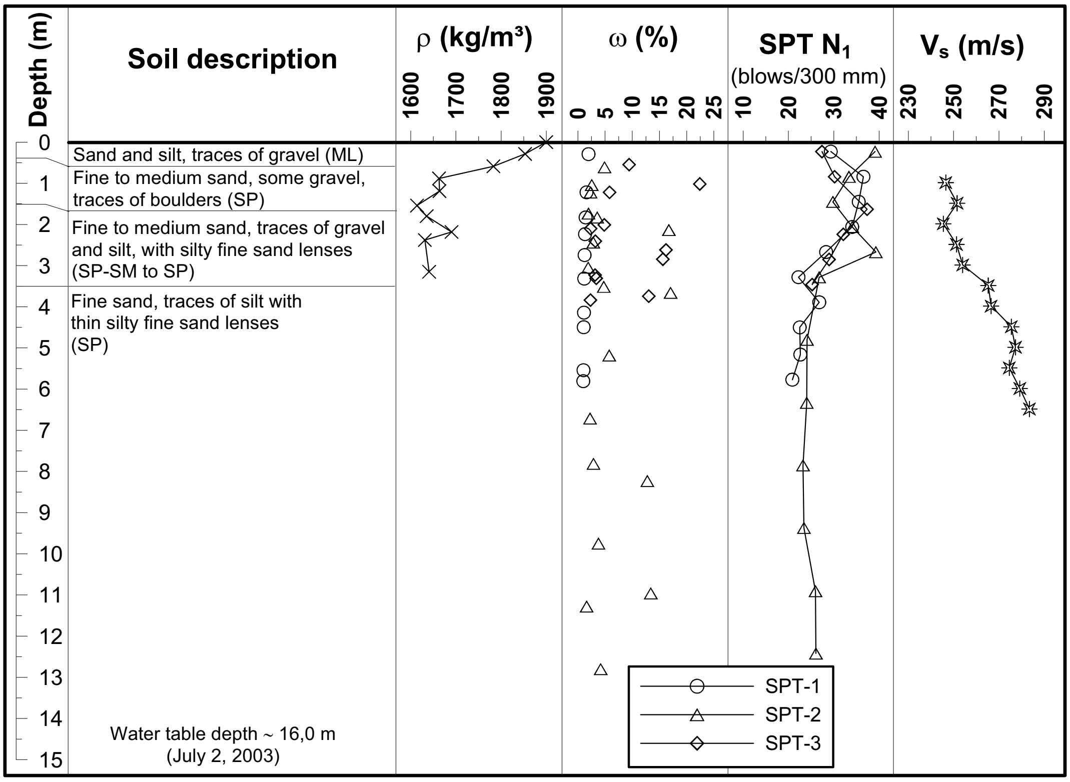

Figure 3: Test site geotechnical properties.

sand with traces of gravel and silt, and few silty fine sand lenses of 1 to 10 cm. Within the first 1.5 m of soil, the average ρ is 1 746 kg/m³, ω_n is generally between 1 to 5 %, the average N_1 SPT value equals 33 for a relative density of 74 % and the average V_s is 249 m/s.Between 1.5 to 3.5 m, the average ρ is 1 642 kg/m³, ω_n is generally between 2 to 5 % except for some silty sand lenses where it is 13 to 17 %, the average N_1 SPT value equals 31 for a relative density of 72 %, and the average V_s is 254 m/s. A thick layer of fine sand with traces of silt and thin silty fine sand lenses is found below 3.5 m. Within this layer, ω_n equals 1.5 to 6 % except in the lenses, N_1 is fairly constant to a value of 24 for a relative density of 64 % and V_s gradually increase from 265 m/s at 3.5 m to 283 m/s at 6.5 m. The water table is at a depth about 16 m as obtained from seismic refraction tests (measured compression wave velocity, $V_{p,}$ is around 2 200 m/s) and confirmed by a sampler auger test. The maximum depth reach during sampling was 22.5 m and the bedrock was not encountered.

3.3 Pavement construction

The natural soil was first excavated to a depth of 650 mm in section 1, and then compacted by a vibrator compactor to a average density of 1 785 kg/m³ (ω_n = 5.4 %). Subsequently, the base layer of crushed gravel mixed sand MG-20 (according to specifications of Quebec Ministry of Transportation) was placed and compacted in two sub-layers for this section. Finally, an asphalt concrete EB-14 mixture with PG-52-28 bitumen (according to specifications of Quebec Ministry of Transportation) was laid in place and compacted in two layers glued with a bender asphalt emulsion. For section 2, 220 mm of top soil was removed, the surface soil was then compacted by a vibrator compactor to an average density of 2 005 kg/m³ (ω_n = 9.1 %) and the same asphalt concrete was also placed in two layers over a thin MG-20 base layer. Table 1 presents the pavement structure for the two sections.

Table 1. Pavement structure.

Layer	Material	Average thickness (mm)	
		Section 1	Section 2
Surface	Asphalt concrete	100	95
Base	Crushed gravel 0-20 mm	600	100
Subgrade	Sand		

3.4 Geotechnical laboratory test results

Standard laboratory tests were performed on the base material and on the natural subgrade sand soil. The geotechnical characteristics of the first 300 mm tested subgrade soil in section 1 and base material are presented in table 2. Laboratory tests were also done on the asphalt concrete EB-14 mixture and its standard properties are listed in table 3.

Moreover, repeated flexural bending tests were carried out in the Quebec Ministry of Transportation Laboratory (MTQ) in order to determine the master curve of the asphalt concrete EB-14 mix. Two rectangular beams were cut in laboratory from a specimen of the surface layer collected in place on section 1. Tests were performed on each beam at temperatures of 0, 5, 10, 15, 20 and 25°C and frequencies between 0.01 to 10 Hz according to the MTQ procedure. A description of the apparatus can be found in AASHTO (1996). The obtained master curves at 5 and 10 °C are shown in figure 4. The dynamic flexural modulus ($|E_f^*|$) is obtained from the tension stress and strain at the bottom of the beam in its middle part. For a loading frequency of 33 Hz, which could correspond to a 30 ms FWD loading pulse, the dynamic asphalt concrete moduli are: $|E_f^*|$ = 10 300 MPa at 5 °C and $|E_f^*|$ = 7 450 MPa at 10 °C.

Table 2. Properties of tested soil and base material.

Characteristics	Subgrade	Base
Gravel (%)	0	55
Sand (%)	99	39.5
Fine < 80 μm (%)	1	5.5
$\rho_{d\ opt}$ (kg/m³)	1 695	2 212
w_{opt} (%)	9.0	4.7
C_u	3.2	67
C_c	1	0.7
Classification USCS	SP	GP

Table 3. Characteristics of asphalt concrete surface layer.

Characteristics	Asphalt concrete EB-14
Gravel (%)	50.0
Sand (%)	45.1
Fine < 80 μm (%)	4.9
Density (kg/m³)	2 465
Asphalt content (%)	4.95
Effective asphalt content (%)	3.99
Total specific surface of aggregate (m²/kg)	5.54
Air voids (%)	2.1

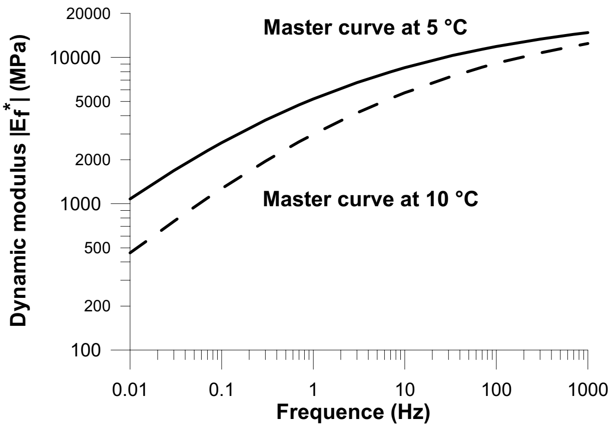

Figure 4: Master curves of asphalt concrete EB-14 mixture.

3.5 Additional cross-hole tests

After completion of sections 1 and 2, additional cross-hole tests were carried out in section 1 at test location CH-2 (figure 2). Tests were done in the base layer and in the subgrade until 3.0 m depth. Figure 5 shows these results in comparison with those obtained at test location CH-1. The pavement structure of section 1 is also given on the figure. A good agreement was found between 1.0 to 3.0 m depth. In base layer, the average shear wave velocity equals 210 m/s. The maximum shear modulus (G_{max}) can be calculated from cross-hole tests by mean of equation 1:

$$G_{max} = V_s^2 \rho \qquad [1]$$

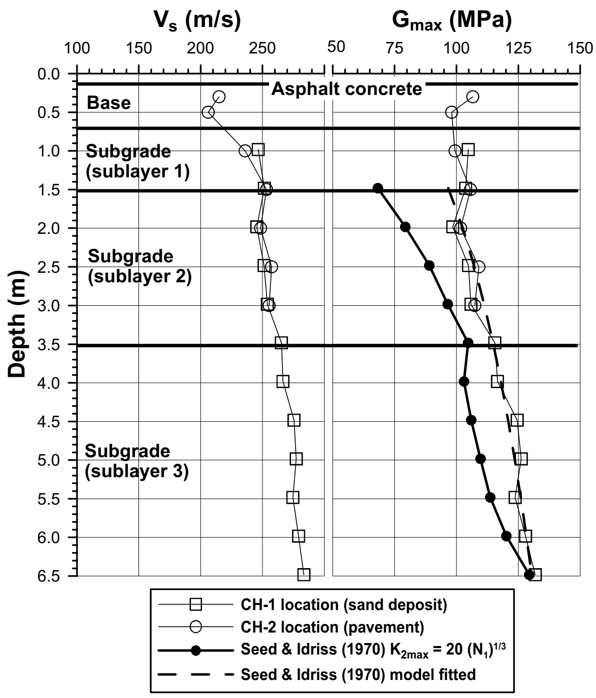

Figure 5: Shear wave velocities profile from cross-hole tests.

The shear modulus profile is also shown on figure 5 for cross-hole tests performed at CH-1 and CH-2 locations. G_{max} values are near 105 MPa in the base layer and vary gradually from 100 MPa at 1.0 m to 135 MPa at 6.5 m depth in the subgrade. The shear modulus can also be estimated from the relation established by Seed & Idriss (1970) which can be expressed by the following equation in metric units:

$$G_{max} = 21.7 p_a K_{2max} \left(\frac{\sigma_m'}{p_a} \right)^{0.5} \qquad [2]$$

where p_a is the reference atmospheric pressure (100 kPa), K_{2max} is a constant and σ_m' is the effective mean principal stress. The average effective stress is obtained assuming an earth resting factor K_o (σ'_{ho} / σ'_{vo}) equal to 1.0. From Seed & al. (1986), the K_{2max} constant (at low strain) is calculated by $K_{2\,max} = 20\sqrt[3]{N_1}$ which gives an average K_{2max} value of 61.1. The G_{max} values from equation 2 are plotted in figure 5. The trend of the measured values from cross-hole is capture by equation 2 but results are 16 % lower. Thus, an attempt was done to fit equation 2 to measured data by changing the K_{2max} and the exponent (n). The result is also shown in figure 5 for K_{2max} = 60.7 and n = 0.21. It is interesting to note that K_{2max} values are quite the same but the exponent is two times less than the value of 0.5 in the equation 2.

4. LOADING TESTS

Loading conditions considered in multilayer linear-elastic theory and in dynamic elasticity theory differ from each other: the former being static, the latter dynamic. Static loading conditions correspond to those of plate-loading test while FWD tests apply dynamic loading conditions. These two kinds of tests were performed on section 1 of the field test facility in order to demonstrate the fundamental distinctions between pavement responses from these two modes of loading. The influence of load intensity and duration for FWD tests are also addressed.

4.1 Static plate-loading tests

Static plate-loading tests were carried out directly on the asphalt concrete layer using the circular plate of 300 mm in diameter which has been removed temporarily from the FWD equipment. The surface deflection basin is measured by six high precision LVDT placed at 0, 300, 600, 900, 1200 and 1500 mm from the center of the plate. The LVDTs are fixed near the middle of a 6.0 m beam keeping the referential supports out of the deflection area. The load was applied by mean of a hydraulic jack with a pump where the pressure is controlled manually. A loaded float truck was used as reaction. A load cell placed between the truck frame and the hydraulic jack measures the applied load.

Tests have been conducted according to the following procedure: three repeated loading-unloading cycles were applied for each selected load level beginning with the lower level (10 or 20 kN) to the higher level (75 kN), for each cycle, loading was done in about 30 s, the load was sustained for 3 min, after unloading was done in about 15 s, and a resting period of 3 min was observed before the next cycle was initiated. Figure 6 shows typical load-deformation curves for three repeated loading-unloading cycles at a 45 kN load level. The loading curve is almost linear until the 45 kN load is reach.

The total plastic settlement is noted as d^p_{total} in figure 6 and the final elastic vertical displacement at the end of the test is noted d^e_{final}. The latter corresponds to the static elastic deflection at each LVDT used to obtain the deflection basin. Quasi-static deflection basin can also be obtained from the loading curve for intermediate load. For example, in figure 6, the total deflection (elastic and plastic) at 40 kN is near 0.6 mm at the third cycle for LTVD 1 in the middle of the load plate (the elastic deflection was 0.493 mm).

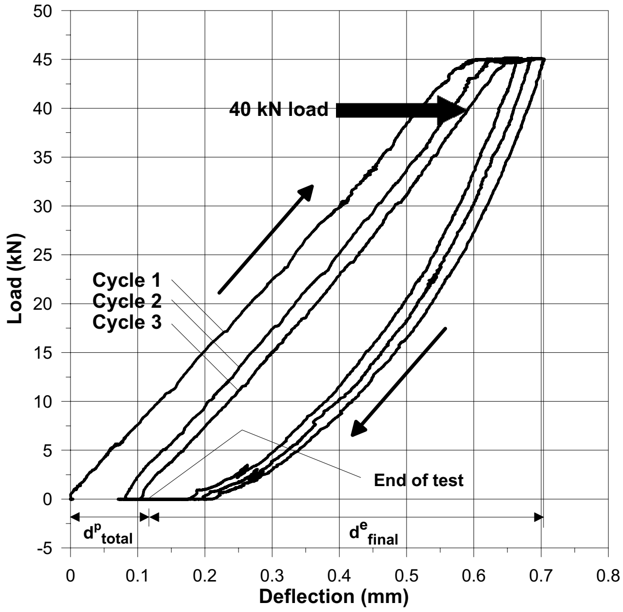

Figure 6: Typical load-deformation curve at LVDT 1 from plate-loading test.

4.2 FWD tests

An extensive FWD test program was carried out on the recently built sections. Twelve test points were located on section 1 and three test points on section 2 as shown in figure 2. On each point, different loading conditions were applied by varying the height, the weight and the type of buffers of the FWD loading system. Moreover, two kinds of FWD were used for testing: the PRI 2100 model built by Carl Bro Pavement Consultants, Inc owned by Laval University, referred as FWD UL, and the FWD 8002E model built by Dynatest owned by Quebec Ministry of Transportation, referred as FWD MTQ. The nine geophones were placed at 0, 200, 300, 450, 600, 750, 900, 1200 and 1500 mm from the middle of the load plate.

During testing, asphalt concrete temperatures were measured with three thermistors installed at specific depths in a hole in the surface layer. From several FWD tests performed at different temperatures on the same test point, linear deflection-temperature relationships were found at the load level of 40 kN: peak deflection at geophone 1 (0 mm) is influenced at a rate of 5.1 μm/°C which decreases linearly to 0.56 μm/°C at geophone 4 (450 mm). For geophones at 600 to 1500 mm, temperature has no significant influence. The humidity was monitor of the base layer and within the first 0.5 m of the subgrade by mean of three TDR. It is found that the humidity in the material was fairly constant during the testing period.

4.3 Influence of loading conditions on deflection

FWD tests and plate-loading tests performed at point P1-3 on section 1 (figure 2) are used for closer examination of the influence of loading conditions on deflection. Table 4 in connection with figure 5 gives a complete description of the pavement structure at this test point with mechanical properties of materials and soils. After coring, the thickness of the asphalt concrete layer at this test point was found to be 20 % thicker than the average value.

Table 4. Pavement structure at P1-3 on section 1.

Layer	H (m)	ρ (kg/m³)	G_{max} (MPa)
Surface (EB-14)	0.120	2394	
Base (MG-20)	0.596	2308	105
Subgrade 1 (sand)	0.784	1785	109
Subgrade 2 (sand)	2.0	1647	106
Subgrade 3 (sand)	12.5	1641	135
Subgrade 4 (saturated sand)	∞	2030	893

Figure 7 shows time histories for load levels equals to 27, 40 and 50 kN where the increase in deflection obviously follows the increase in load level. It should be noted on deflection time histories that peak deflection does not occur at the same moment at each geophone. This can be explained by the dynamic nature of the loading where generated body and surface waves travel to reach each geophone position and combine to give the measured surface deflections.

The influence of FWD loading system configuration and the type of FWD device should be investigated. Depending on the loading system characteristics, different loading pulses will be obtained for the same load level thus leading to different deflection time histories. This is also true for different type of FWD devices. Figures 8a) and 8b) show deflection time histories respectively for 265 kg and 420 kg mass weight loading system setups on the FWD UL. For the same load level of 40 kN, the pulse time was 29 ms with 265 kg mass weight and passed to 38 ms with 420 kg mass weight resulting in reduction of the measured deflections. Figure 8c) shows the measured time histories from the FWD MTQ at 40 kN load level. The pulse time is now equal to 27 ms with slight differences in deflection time histories compared to those of FWD UL as shown in figure 8a).

Static and dynamic loading conditions are compared in figure 9. It presents two deflection basins from plate-loading tests: the static deflection basin for a 40 kN load level and the quasi-static deflection basin at 40 kN from the 45 kN load level test as obtained from figure 6 for LVDT 1. In figure 9, an increase in deflections from quasi-static to static deflection basins is noted and the bowl shape is preserved between these two loading conditions. On the same figure, the three dynamic FWD deflection basins from time histories shown in figure 8, are presented. Deflections have been corrected for temperature effect to the same asphalt concrete temperature during plate-loading tests. The dynamic deflection basins are similar with a little reduction in deflection intensity when the time pulse increases, as obtained from 420 kg mass weight compare to the 265 kg mass weight setup of FWD UL. This reduction which follows a different trend compared with static deflection basins, is not already explained and requires dynamic analysis of the FWD tests. However, dynamic deflection basins exhibits a clear difference shape compare to the static ones. On one hand, distinctive responses in the vicinity of the load plate (< 600 mm) could be explained partly by the load-rate dependency of asphalt concrete where an increase in stiffness reduces deflections intensity: from the master curve (figure 4) a factor of 5 to 10 is found between dynamic modulus of quasi-static plate-loading test (loading in 30 s – 0.033 Hz) and dynamic modulus of FWD test. On the other hand, there is a systematic increase in deflections at radial distances larger than 600 mm. This discrepancy is caused by dynamic effects associated with wave propagation which can't be accounted by static analysis.

4.4 Field reference deflection data

Table 5 presents a compilation of all measured deflection basins shown in figure 9 for FWD and plate-loading tests on point P1-3, with their corresponding maximum applied load and asphalt concrete temperature.

Table 5. Reference deflection data at P1-3 on section 1.

Position (mm)	Deflection basins (μm)				
	FWD UL		FWD MTQ	Plate-loading test	
	265 kg	420 kg		Static	Quasi-static
0	367	358	356	533	493
200	302	290	299		
300	264	251	262	255	244
450	214	204	209		
600	169	160	168	120	108
750	141	133	135		
900	115	108	111	68	58
1200	81	77	79	42	34
1500	62	58	60	26	21
Q (kN)	40.02	40.17	40.40	40.00	40.00
T (°C)	9.9	10.0	7.3	6.3	6.3

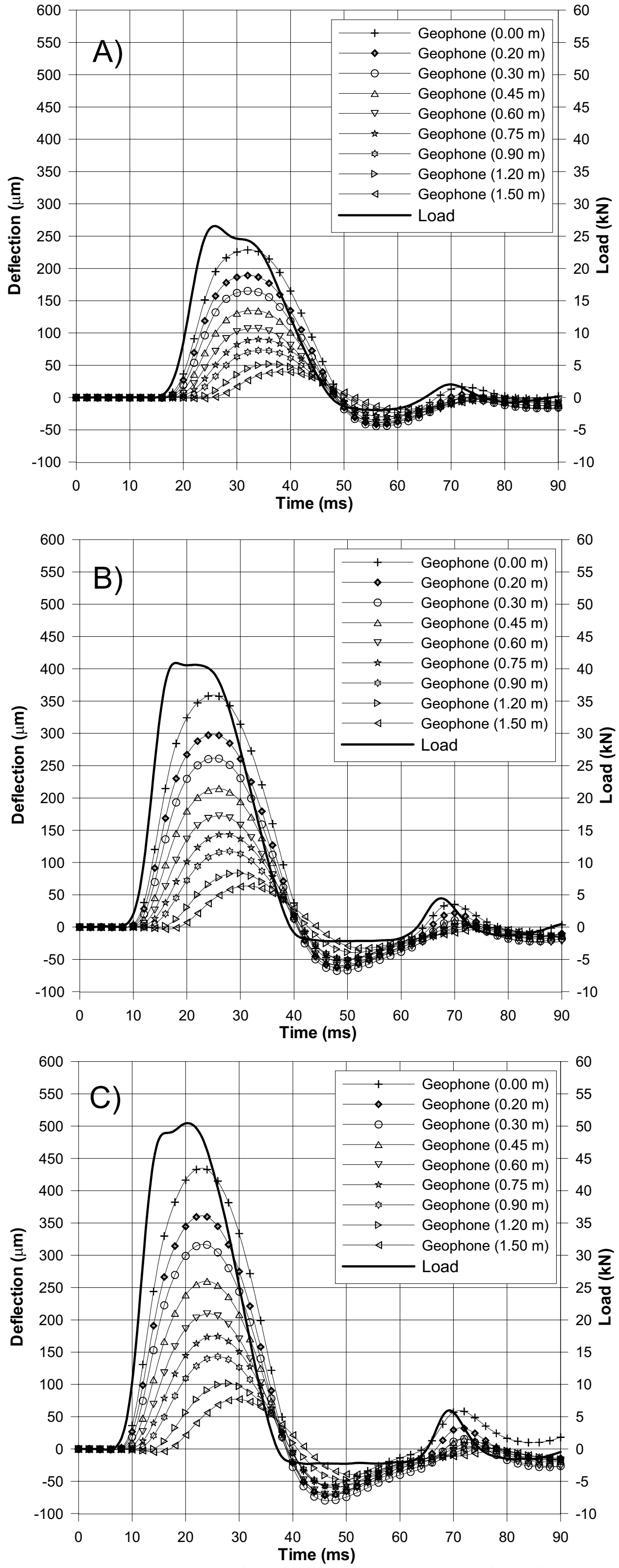

Figure 7: Time histories of FWD tests at point P1-3 for different load levels: A) 27 kN, B) 40 kN and C) 50 kN.

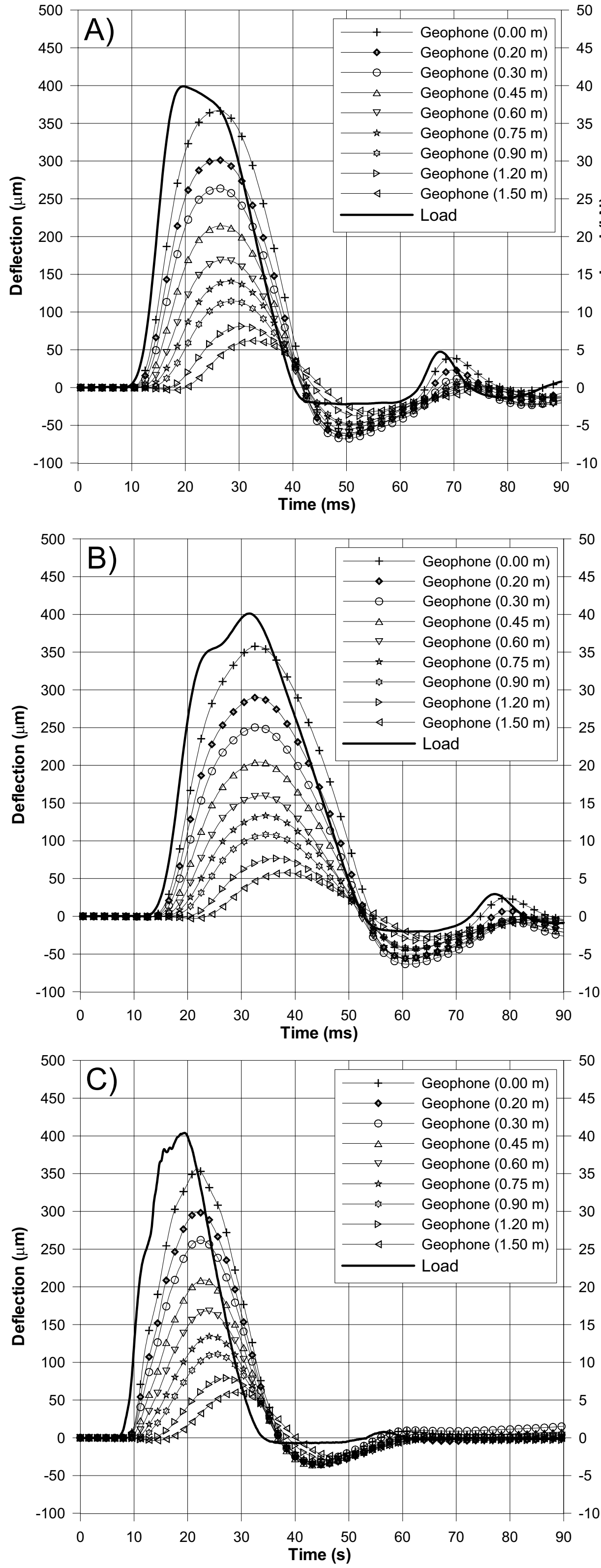

Figure 8: Time histories of FWD tests at point P1-3 for various loading system configurations: A) FWD UL with 265 kg setup, B) FWD UL with 420 kg setup, C) FWD MTQ.

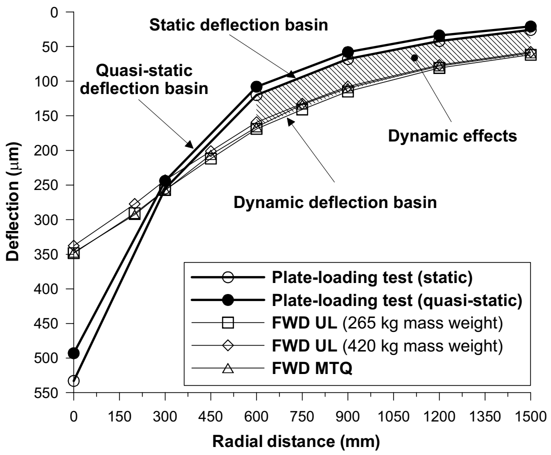

Figure 9: Comparison between static and dynamic static loading deflection basins.

5. CONCLUSION

In this paper, a field test facility for evaluation of FWD interpretation methods is described. Before any attempt is made to assess theoretical models for interpretation of FWD test results, soil and pavement materials have to be characterized. Thus, an extensive field and laboratory investigation program was carried out. Results from SPT tests and auger sampler have furnished a detail layering of the sand deposit, density measurements have revealed a picture of the density profile in the first 3.3 m depth and cross-hole tests have allowed maximum shear modulus profile evaluation. Standard and more sophisticated laboratory tests have be carried out on sandy soil, granular base material and asphalt concrete mixture.

FWD and plate-loading tests have been carried out on experimental flexible pavement sections. Results at test point P1-3 on section 1 with a thick base layer includes deflection time histories of FWD tests where the influences of load intensity and load system configuration are shown.

Comparison between static and dynamic deflection basins demonstrates two distinctive pavement responses where wave propagation during FWD tests has to be recognized for adequate interpretation of deflection results, at least from a fundamental point of view.

Reference deflection data with complete pavement structure description are provided for test point P1-3. This valuable information will be used for further analysis in a comprehensive research undertaken to study the dynamic interpretation methods of FWD tests.

6. ACKNOWLEDGEMENTS

This research work was funded through the NSERC industrial chair and its 22 partners (Quebec Ministry of Transportation; Muni cipalities of Sainte-Foy, Quebec, Charlesbourg, Saint-Augustin, Vanier, Saint-Georges; Consulting companies: Roche, SNC-Lavalin, BPR, CIMA, TECSULT, GLD; Contractors: Les Entreprises P.E.B., Talon Sebeq, Sintra, ACRGTQ, EBC; Laboratories and manufacturers: LEQ, LMQ, Techmat, Soleno TT and Université Laval) and the FQRNT – Equipe.

7. REFERENCES

AASHTO, 1996. Standard Test Method for Determining the Fatigue Life of Compacted Hot Mix Asphalt (HMA) Subjected to Repeated Flexural Bending. AASHTO Designation TP8-94, American Association of State Highway and Transportation Officials, Washington, D.C.

Al-Khoury, R., Scarpas, A., Kastergen, C. & Blaauwendraad, J., 2001. Spectral Element Technique for Efficient Parameter Identification of Layered Media, I. Forward Calculation. International Journal of Solids and Structures, Vol. 38, No 9, pp. 1605-1623.

CROW, 1998. Deflection profile – not a pitfall anymore. CROW Record 17, The Netherlands.

Foinquinos Mera, R., 1995. Dynamic Nondestructive Testing of Pavements. Geotechnical Engineering Report GR95-4, Geotechnical Engineering Center, University of Texas at Austin, Austin, Texas.

Magnuson, A.H., Lytton R.L. & Briggs, R., 1991. Comparaison of Computer Predictions and Field Data for Dynamic Analysis of Falling Weight Deflectometer Data. Transportation Research Record 1293, TRB, National Research Council, Washington, D.C., pp. 61-71.

Roesset, J.M. & Shao, K-Y., 1985. Dynamic Interpratation of Dynaflect and Falling Weight Deflectometer Tests, Transportation Research Record 1022, TRB, National Research Council, Washington, D.C., pp. 7-16.

Seed, H.B. & Idriss, I.M., 1970. Soil Moduli and Damping Factors for Dynamic Response Analyses. Earthquake Engineering Research Center, Report No. EERC 70-10, University of California, Berkeley, California.

Stolle, D.F.E. & Parvini M., 2001. A Look at Modelling Errors for Falling Weight Deflectometer Backcaluclation. Transportation Research Board, 80th Annual Meeting, January 7-11, Washington, DC. 16 p.

Uddin, W., 2002. Finite Element Modeling and Simulation of FWD Dynamic Load Tests. FWD/Backcalculation Worckshop 3, 6th International Conference on Bearing Capacity of Roads, Railways and Airfields, 24-26 June, 2002, Lisbon, Portugal.

Ullidtz, P. & Coetzee, N.F., 1995., Analytical Procedures in Nondestructive Testing Pavement Evaluation, Transportation Research Record 1482, TRB, National Research Council, Washington, D.C., pp. 61-66.

Uzan, J., 1994. Advanced Backcalculation Techniques. Nondestructive Testing of Pavements and Backcalculation of Moduli, Second Volume, ASTM STP 1198, Harold, L. Von Quintus, Albert J. Bush III, and Gilbert Y. Baladi, Eds., ASTM, Philadeliphia, pp. 3-37.

Seed,. H.B, Wong, R.Y., Idriss, I.M. & Tokimatsu, K., 1986. Moduli and Damping Factors for Dynamic Analyses of Cohesionless soils. Journal of Geotechnical Engineering, Vol. 112, No 11, pp. 1016-1032.