FALLING WEIGHT DEFLECTOMETER MEASUREMENTS FOR EVALUATION OF THE BEARING CAPACITY OF GRANULAR UNBOUND LAYERS

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ABSTRACT

The proper use of natural resources or recycled materials offers considerable advantages, both from an environmental and economic point of view. Thus in recent years interest has focused on the performance of these materials when used in road construction to build granular unbound layers.

The experimental research described in this study was undertaken to acquire more precise data on the most important parameters characterizing actual performance once granular materials are laid in place. A further aim was to evaluate the reliability of quality controls performed using the Falling Weight Deflectometer (FWD) directly on finished unbound layers. It is well known that while evaluation of elastic moduli by back-analysis methods is common practice for in-service roads, analysis of deflection basins measured directly on unbound layers can present notable difficulties.

Finally, the study proposes to contribute to the establishment of standard practice for correct evaluation of data obtained from deflection tests performed on unbound layers.

The results underline that the elaboration of deflection data measured on unbound layers provides bearing capacity values strictly related to the mathematical model used to represent the behavior of the material.
Keywords: Falling Weight Deflectometer, backcalculation, unbound layers, bearing capacity, elastic moduli

INTRODUCTION

The pavement design process consists of an integrated set of models which allow prediction of the progression of deterioration for each mode of distress over time, as a function of pavement structure, materials, traffic loading and climate effects. These calculations require a theoretical model (e.g. elasticity theory) in order to determine stresses and strains induced in the pavement layers by traffic loads. The model adopted is dependent upon the constitutive laws (e.g. Hooke's law) assumed for the materials in the different layers, including the subgrade. Hooke's law has two elastic parameters, the elastic modulus and Poisson's ratio.

The Falling Weight Deflectometer (FWD) is one of the meaningful tools available to describe and understand pavement structural behavior. This device is commonly used to measure the response, in the form of a deflection basin, exhibited by a test surface following application of a load that simulates the passage of a heavy load vehicle. These measured deflection basins are also used once the pavement structure is known, to determine the elastic moduli of the component pavement layers by backcalculation.

In the design of a pavement, stiffness are assumed for the different materials, such as the subgrade, subbase, road base and asphalt layers. During road construction, the degree of compaction is checked but the (assumed) stiffness of the different pavement layers are rarely measured. Generally the static load plate bearing test is used. As this test method is time-consuming, a number of road authorities have started to use FWD for measuring granular layer stiffness as well.

It is known that for completed pavement structures, FWD is a useful tool. It is likewise well known that far less experience has been gained with the use of this device directly on unbound layers, although it would be very helpful to collect field data in terms of layer stiffness during the process of road building. In Quality Control and Quality Assurance (QC/QA) approaches these results could be useful for validation of the quality of work, allowing a greater number of inspection stations and direct evaluation of layer stiffness. Field assessment of elastic moduli for pavement layers and for the subgrade is fundamental as a means of assessing the work executed and its conformity to the design assumptions [Thurner (2001), Stubstad (2002), Nazzal (2002)]. Premature structural failure may be brought about by poor construction practice but if regular inspections are carried out, timely action can be taken.

However, utilization of this tool is dependent on availability of reliable calculation models that make it possible to simulate pavement and subgrade mechanical behavior. It is important to underline that backcalculation procedures, based on the multilayer elastic model, are commonly utilized to evaluate the mechanical characteristics of materials by conducting deflection tests on finished pavements. In contrast, when tests are carried out directly on the subbase layers, on the subgrade or on embankments, interpretation of the results can be rather more difficult [Losa et Al. (2003)]. Almost all of the available backcalculation programs were developed for analysis of data obtained from tests conducted on a tensile resistant surface layer, which is generally far more rigid than the underlying layers. If tests are conducted directly on unbound layers, absence of the surface layer may lead to difficult (and in many cases erroneous) interpretation of FWD measurements [Hildebrand (2002)].

Finally, for correct interpretation of measurements the generally non-linear behavior of unbound materials should be taken into account, allowing for this behavior as part of the theoretical calculation model adopted [Ullidtz (1998)].

The present experimental study was designed to provide further information that can be incorporated into guidelines for interpretation of FWD data acquired by tests carried out directly on unbound layers.

1. DESCRIPTION OF THE EXPERIMENT

1.1 Determination of the test practice

To date, experiments performed by numerous Authors have not led to univocal criteria for conducting high performance deflectometric tests directly on unbound subgrade and subbase layers or on road pavement bases, and for interpretation of results obtained. It is also important to note that since granular materials are generally characterized by non-linear behavior, choice of test pressures may in itself constitute a parameter that markedly affects test materials and results. Thus FWD test practice should always stress the materials with different load levels in order to assess test material behavior.

With regard to the type of load plate, in addition to the traditional 30 cm diameter segmented loading plate the rigid 45 cm diameter load plate is also often used, as its broader area of contact allows more effective simulation of the stress diffusion of into the deeper layers. But it should also be kept in mind that use of a wider diameter load plate results in the application of generally lower test pressures (to simulate the phenomenon whereby the more rigid upper layers attenuate surface pressure). Consequently, reduced deflection values could be obtained, leading to less reliable and less significant data.

Some studies [George (2003)] have demonstrated that when tests are conducted directly on unbound materials using a very low load level, a good correlation between on-site moduli and moduli obtained in the laboratory with dynamic triaxial devices is observed. Other experiments have been designed to establish correlations between the laboratory-determined resilient modulus and the dynamic modulus obtained by FWD [Flintsch et Al. (2003), Gaspard et Al. (2003)], or to compare different on site test procedures [Pidwerbesky (1997), Rahimzadeh et Al. (2004)], or to measure the actual stress acting within the pavement [Hildebrand (2002)]. In these experiments, however, a completely different test procedure was adopted, using only the 30 cm diameter segmented load plate and applying appreciably higher peak loads. With regard to use of the 45 cm diameter rigid load plate, the discordant results of specific studies reported in the technical literature [COST 339 (1999), Losa (2003)] also highlight the marked differentiation of findings according to the types and conditions of materials analyzed.

In the experiment described in this paper we decided, in the light of the different test procedures and experiences reported in the literature and based also on the Authors' experiences, not to establish a specific test setup to be adopted throughout all the investigations. Therefore a number of variations were introduced concerning the system of falling masses utilized, the type of loading plate, and also the peak load applied. The purpose of such variations was to ascertain potential variations in the results as a function of the different types of setup utilized. The tests were conducted using both types of loading plate and adopting a system of masses ranging from 50 to 250 kg. For each point investigated, a load produced by the fall of masses from four predefined heights was applied. In addition, for each pressure level (which was a function of the system of masses adopted, falling height and load plate type) showed in Table 2, at least 10 load repetitions were performed.

With regard to geophone positioning, the experiments reported in the literature adopt positions consistent with those generally utilized for measurements performed directly on finished pavements. Such positions involve a number of geophones ranging from 7 to 9, placed at a radial distance up to 1500 or 1800 mm. The positioning adopted, which was maintained throughout the deflectometric investigations conducted in the present experiment, involved 9 geophones placed at the radial distances from the loading center plate indicated in Table 1.

Table 1 Positioning of geophones for deflection measurements

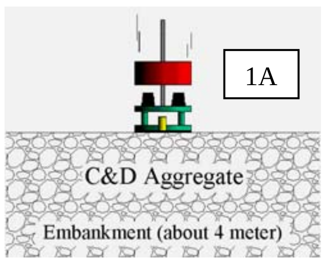
Geophone	**D1**	**D2**	**D3**	**D4**	**D5**	**D6**	**D7**	**D8**	**D9**
Distance from loading center plate (mm)	0	200	300	450	600	750	900	1200	1500

1.2 Test sites

The survey designed to acquire deflectometric data by means of tests performed directly on unbound layers was carried out at three distinct test sites located on newly constructed roads of the region of Tuscany.

Test site nr. 1 was located at a motorway interchange. Deflectometric tests were performed on two different road embankments constructed with aggregate materials recycled from Construction and Demolition waste (C&D).

Test site 1A	**Test site 1B**
C&D recycled aggregate	C&D recycled aggregate
(AASHTO mod. comp.) γ_{dmax} = 2.025 Mg/m^3 w_{ott} = 9.5% CBR = 109%	(AASHTO mod. comp.) γ_{dmax} = 1.875 Mg/m^3 w_{ott} = 11.4% CBR = 67%
(Gyratory compaction) γ_{dmax} =2.030 Mg/m^3 w_{ott} = 8.7% CBR = 133%	(Gyratory compaction) γ_{dmax} =1.925 Mg/m^3 w_{ott} = 10.1% CBR = 81%
γ_{sito} = 1.981 Mg/m^3 $G = \gamma_{sito}/\gamma_{dmax} \cdot 100$=97.8% M_d = 147.5 N/mm^2 M_d' = 191.5 N/mm^2 M_d / M_d' = 0.77	γ_{sito} = 1.730 Mg/m^3 $G = \gamma_{sito}/\gamma_{dmax} \cdot 100$=92.3% M_d = 103.5 N/mm^2 M_d' = 209.3 N/mm^2 M_d / M_d' = 0.49



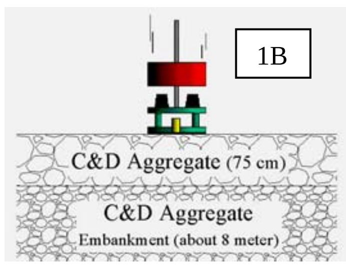


Figure 1 Test sites 1A and 1B

Test site nr. 2 was composed of three different stations along a regional highway, where earthworks had been carried out (embankments, subgrade layers or substitution layers of unsuitable natural subgrade). These works had likewise been carried out using recycled C&D aggregate. The stations examined at this test site presented different characteristics as regards both the type of unbound materials utilized and, above all, their structural response.

Test site 2I	**Test site 2II – 2III**
Embankment: C&D recycle. aggr. A_{1a} γ_{dmax} = 1.94 Mg/m^3 w_{ott} = 11.0% CBR = 62% M_d = 115.4 N/mm^2 M_d' = 182.2 N/mm^2 M_d / M_d' = 0.63	Substitution layer: C&D recycled aggregate A_{1b} – A_{2-4} γ_{dmax} = 1.87 Mg/m^3 w_{ott} = 10.4% CBR = 24% M_d = 47.6 N/mm^2 w = 8.1 %
Natural soil: A_6 GI=10 γ_{dmax} = 1.83 Mg/m^3 w_{ott} = 12.4% CBR = 5% M_d = 27.7 N/mm^2 w = 8.3 %	Natural soil: A_{7-6} GI=12 γ_{dmax} = 1.67 Mg/m^3 w_{ott} = 16.2% CBR = 1% M_d = 2.3 N/mm^2 w = 19.8 %

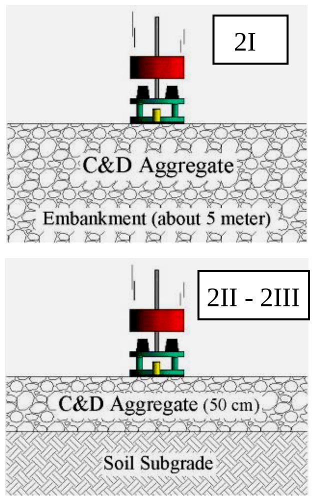


Figure 2 Test site 2A and 2B

Test site nr. 3 was located along a newly built stretch of a secondary road. The materials utilized for the unbound layers were of a traditional type and it was possible to perform deflectometric tests both directly on the subbase layers and also on the asphalt concrete layer.

Subbase Layer: A_{1-a} GI=0 γ_{dmax} = 2.27 Mg/m^3 w_{ott} = 4.7% - CBR= 140% M_d = 187÷250 N/mm^2 L.A. = 38%	Embankment: A_{2-4} GI=0 γ_{dmax} = 1.99 Mg/m^3 w_{ott} = 10.2% CBR = 28% M_d = 86÷113 N/mm^2	Natural soil: A_6 GI=11 γ_{dmax} = 1.93 Mg/m^3 w_{ott} = 11.6% CBR = 4% M_d = 41 N/mm^2

Figure 3 Test site 3

Table 2 Details of FWD tests undertaken

Type of loading plate	Mass (kg)	Pressure (kPa)	Load (kN)	Test site – Stations
Segmented (30 cm)	50	170 – 540	12 - 38	1 – A – 1 / 2 / 3
Rigid (45 cm)	50	70 – 195	11 - 31	1 – A – 14 / 15
Rigid (45 cm)	250	165 – 500	26 - 80	1 – A – 18
Segmented (30 cm)	50	170 - 490	12 - 35	1 – B – 7
Rigid (45 cm)	50	70 - 195	11 - 30	1 – B – 10
Segmented (30 cm)	250	450 - 1300	31 - 93	2 – I - 1
Segmented (30 cm)	150	250 - 770	18 - 54	2 – I - 2 / 3
Segmented (30 cm)	150	245 - 730	17 - 51	2 – II - 1 / 2 / 3
Segmented (30 cm)	150	220 - 760	16 - 54	2 – III - 1 / 2 / 3 / 4
Segmented (30 cm)	250	400 - 1220	28 - 86	3 – 56 H
Segmented (30 cm)	50	170 - 410	12 - 29	3 – 56 L
Segmented (30 cm)	250	170 - 440	12 - 31	3 – 58
Segmented (30 cm)	250	400 - 1260	29 - 89	3 – 56 CB
Segmented (30 cm)	250	440 - 1300	31 - 92	3 – 58 CB

2. DEFLECTOMETRIC DATA ANALYSIS

Data acquired were processed at three different levels. The first investigation level focused on deflection basin analysis. It is known that the shape of the deflection basin allows significant structural analysis of the structure analyzed. Basically, the outer deflections define the stiffness of the subgrade while the basin shape close to the loading plate allows analysis of the stiffness of the near surface layers. A broad basin with little curvature indicates that the upper layers are stiff in relation to the subgrade. A basin with the same maximum deflection but high curvature around the loading plate indicates that the upper layers are weak in relation to the subgrade [COST 336 (1999]. With the principal parts of the structure identified in this manner a preliminary consideration may be outlined.

A second investigation level was performed calculating the surface moduli $E_0(r)$ from the surface deflections (for each distance r from the center of the loading plate) using Boussinesq's equations [Ullidtz (1998)]. The surface modulus can be considered as the "weighted mean modulus" of the half-space equivalent to the structure subjected to investigation. The consistency of the deflection data acquired was checked by plotting "surface modulus" values. The $E_0(r)$ versus r plot also allows immediate determination of whether the behavioral conditions of the tested materials are linear elastic or non-linear.

In the last investigation level, the layer moduli of the each structure analyzed were determined using the optional backcalculation feature FEM/LET/MET for the program ELMOD. This option allows three different calculation models to be used: the equivalent thickness method, or Odemark method (MET), the linear elastic theory (LET) and the finite element method (FEM). FEM and MET analysis suitably accommodates non-linear properties (commonly exhibited by unbound materials) of materials tested.

3. RESULTS ANALYSIS

When deflectometric tests are performed directly on unbound layers, it may be found that the hypothesis of elastic and isotropic multilayer (both linear and non linear) is not verified. In such cases the deflection basins cannot be analyzed by the customary backcalculation procedures.

Cases are known in which the deflection basin is characterized by elevated deflection values near the load plate while values measured at the various radial distances decrease more rapidly than values which can be estimated by the commonly adopted theoretical models [Losa et Al. (2003)]. In such cases, analysis of variation in surface moduli values, calculated using Boussinesq's equations, in relation to variation in distance from the loading center plate, may offer important insight into the actual mechanical behavior of the structure tested.

Figure 4 shows the values of surface moduli of a representative station at investigation site 1A; this site, as regards the stratigraphic composition and the nature of the construction materials utilized, can be schematized as a homogeneous half-space.

The trend of the surface moduli calculated on the basis of the varying distances from the loading center plate showed remarkable characteristics. Theoretically, if the behavior of the mass were effectively linear and isotropic, then modulus variation with varying radial distance should be null and the trend should be horizontal. Alternatively, taking into account that a non-linear stress-hardening behavior may be associated with the granular nature of the materials utilized for the embankment tested, the trend could be expected to show a virtually constant decrease in surface modulus values when variations in distance from the loading center plate are considered.

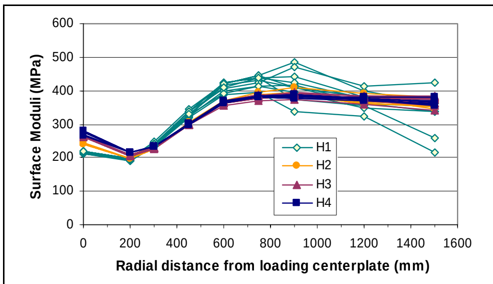


Figure 4
Elastic surface moduli of a representative station at test site nr. 1A

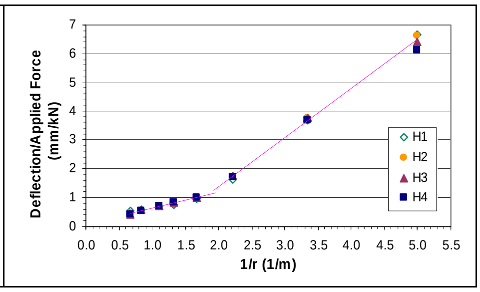


Figure 5
Mean normalized deflections test site nr. 1A

Examination of the trend actually observed would appear to suggest the presence of a subsurface half-space (starting from a depth close to the radial distance at which the diagram of the composite moduli exhibits a practically horizontal trend) with decidedly superior mechanical characteristics as compared to those of the surface layer of material. However, this deduction is clearly in contrast with the fact that the structure tested is composed of material with homogeneous characteristics. It also contrasts with the assumption that the laying and compaction procedures should, if anything, have resulted in the opposite situation, with a more rigid surface part. This peculiar trend of surface moduli with respect to variations in distance from the centerplate, which was common to numerous stations at other sites investigated, cannot be attributed to a genuine "double layer" mechanical behavior, emerging from analysis of the theoretical model. Rather, it is more likely to be attributed to a particular diffusion of the stress within the unbound material examined: around the loading plate (at radial distances and depths ranging from 300 to 600 mm) particle slippage and particle breakage do not allow stress to be transmitted according to the elastic theory. Therefore the deflection measured at elevated distances from the loading plate is produced only by a portion of the total stress applied. This implies that the elevated stiffness revealed by the trend of the surface moduli for these radial distances cannot be considered reliable. It more probably indicates that the deflection values measured are lower than those obtainable by modeling the structure tested as elastic, linear and isotropic.

Furthermore, analysis of the diagram representing the deflections normalized with the inverse of distance from the loading center plate, shown in Figure 5, could be regarded as justifying the adoption of a double-layer model for analyzing the behavior of the structure at investigation site 1. The stiffness contributions of the different layers would then be computed by backcalculation. However, as shown below, this model may lead to results that are not representative of the actual behavior of the mass tested.

Figure 6 shows the modulus values calculated by backanalysis through a non linear constitutive model of the material versus the test pressure level applied. For each stress level applied, all upper layer moduli are concentrated around the same value. By contrast, the underlying half-space modulus values, which are higher, are noticeably scattered. Furthermore, the half-space shows an anomalous trend: with varying test pressures, the moduli presented first a decreasing and then an increasing trend, which could not be attributed to any of the mathematical models normally adopted for expressing the response behavior of granular materials. This phenomenon, which characterizes the values of half-space moduli computed by backcalculation, may instead be attributed to the deflections utilized for estimating such moduli (those at greater radial distances), which should not be considered representative of an elastic and isotropic behavior of the material.

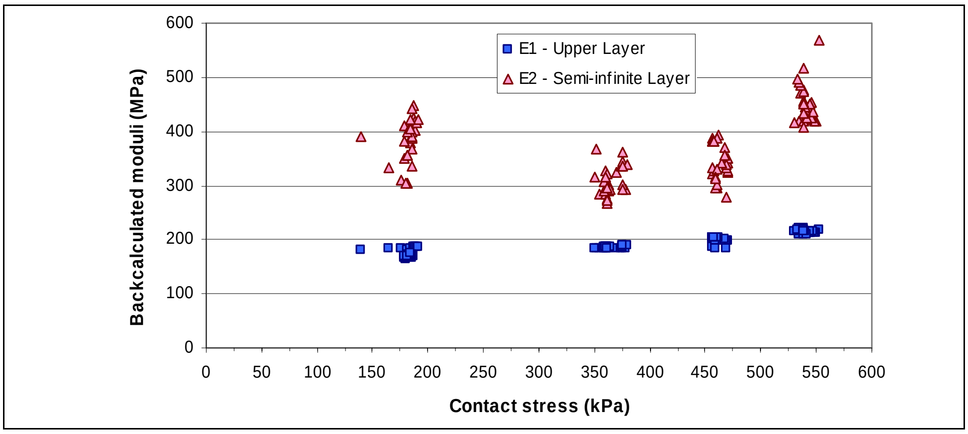


Figure 6 Backcalculated elastic moduli, investigation area 1A, station 2

The results of measures undertaken at investigation site 1 show that when the deflection basin presents this particular shape, i.e. a concavity inversion between the radial distances of 300 and 600 mm, then the only significant measures of the entire deflection basin are those close to the loading plate, in particular the value measured at the center of the loading plate. These latter values, as shown in Figure 4, also clearly indicate that the material examined effectively presented non linear stress-hardening behavior. This deflection indicates the deformation of the entire structure tested. Therefore, it allows determination, through Boussinesq's equation, of the elastic modulus of the equivalent half-space, which represents the only bearing capacity parameter that is always significant. In such cases, the reliable data acquirable from FWD tests are the same as those that can be acquired using less sophisticated instruments (e.g. Light FWD), which generally allow evaluation only of the equivalent surface modulus below the loading plate or in its immediate proximity. Naturally, caution should be exerted in elaborating experimental measurements, even when the construction techniques adopted and the materials utilized are known, because similar deflection basin shapes, with significant variation in concavity, are in any case always possible [COST 336 (1999)], although limited to certain clearly identifiable conditions.

At investigation sites 2 and 3 (which cannot be represented on the basis of their stratigraphic composition as homogeneous half-spaces), deflectometric measurements revealed a different pattern of the structures analyzed as compared to the behavior described above. This was observed above all in areas presenting an elevated degree of compaction, which was often partly due to construction site traffic. Thus, the deflection basins detected can be analyzed by the common theoretical models, which hypothesize a homogeneous elastic and isotropic pattern of the layers investigated.

The graph showing surface modulus values versus distance from the loading plate center is characterized by good regularity and absence of significant concavity inversions at a radial distance between 300 and 600 mm. Overall, decreasing trends were detected (Figures 7, 8), similar to those characterizing investigations on finished pavements.

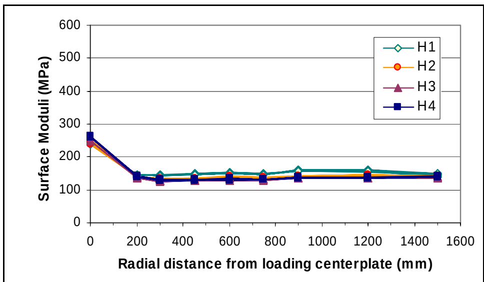


Figure 7 Elastic surface moduli, test site nr. 2, representative station

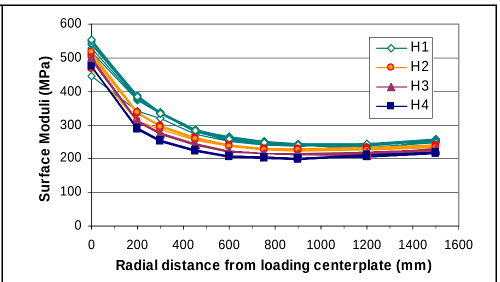


Figure 8 Elastic surface moduli, test site nr. 3, representative station

The structural model adopted for backanalysis was chosen as a function of the stratigraphic composition of the structure examined. Figure 9 shows the structure concerning stations of site nr. 3. Results of computational analysis carried out for stations tested at site nr. 3 are of particular importance as they demonstrate that it is possible not only to estimate bearing capacity by merely calculating the elastic surface modulus of the equivalent half-space, but also to evaluate the stiffness contribution of individual layers composing the structure tested.

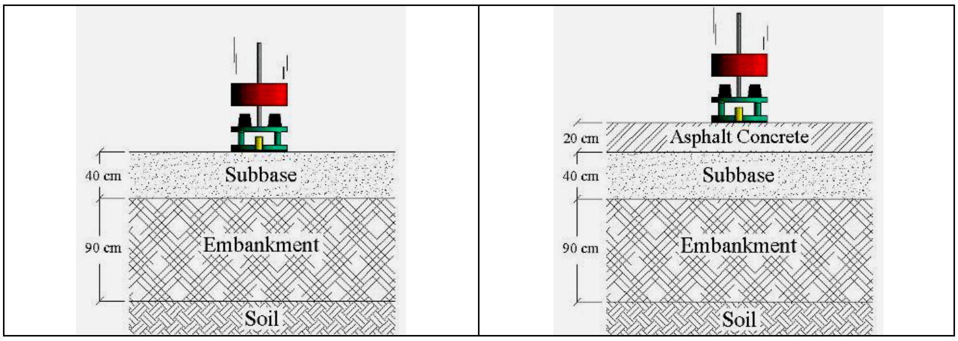


Figure 9 Test site nr. 3, structures tested (without and with asphalt concrete)

Deflectometric measurements performed on the same stations both before and after the laying of asphalt concrete layers allowed comparison of moduli obtained in the two test conditions. In addition, it allowed assessment of the reliability of values estimated by the investigations conducted directly on the unbound layer.

Table 3 - Site test nr. 3: mean values of elastic moduli (MPa)

Site test nr. 3	**Station 56**			**Station 58**		
Asphalt concrete layer	NO	YES	Δ %	NO	YES	Δ %
Asphalt concrete	5335 (@ 8.1°C)			5650 (@ 2.9°C)		
Subbase	680	640	-6	620	580	-6
Embankment	176	204	16	200	240	20

Table 3 shows the mean values of elastic moduli obtained by backcalculating the deflection basins resulting from application of the different pressure levels. Analysis of the data acquired highlights an optimal correspondence with the estimated bearing capacity of the subbase layer. Thus the difference between mean values of the elastic moduli obtained through tests performed directly on the unbound layer and on the finished pavement was roughly 6%. Furthermore, the same non linear behavior of this layer is demonstrated by the values shown in Figure 10, which indicates modulus values obtained in the two different tests conditions as a function of the vertical stress induced at the mid-depth of the layer thickness.

In contrast, modulus values for the underlying layers show more marked differences. After the laying of asphalt concrete, appreciable increase was observed at both the stations examined for the embankment. This increase was probably caused by the non linear behavior (stress softening type) of these materials and to the fact that the state of stress acting on the materials was reduced due the presence of the asphalt concrete layer.

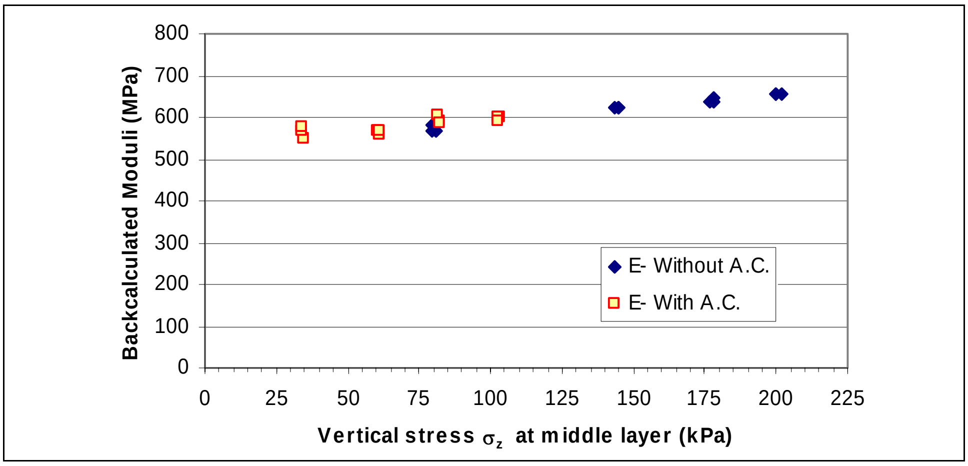


Figure 10 Site test nr.3 - subbase elastic moduli obtained in the two test conditions

4. CONCLUSIONS

Deflectometric investigations conducted with FWD directly on unbound layers allow estimation of the bearing capacity of the structure tested. Such estimation can be related to the individual layers or it can be global, according to whether the response behavior of the structure analyzed can or cannot be assimilated to that of an elastic and isotropic multilayer. This distinction can be successfully evaluated through analysis of variation in the values of surface moduli (obtained with Boussinesq's equation) as a function of radial distance from the center of the load application area. If this diagram is characterized by a marked inversion of concavity at distances ranging roughly from 300 to 600°mm from the center of the loading place, then the only significant bearing capacity value may be the elastic surface modulus of the equivalent half-space calculated using the central deflection value. As is known, if the stiffness contribution of the different layers is evaluated by means of specific backcalculation software, with such basins being considered as genuinely representative of elastic and isotropic behavior, then the results obtained may be devoid of significance.

If the trend of equivalent elastic moduli presents no anomalies between geophones D_3 (300 mm) and D_5 (600 mm), then the mechanical behavior of the structure tested can, with acceptable approximation, be considered elastic and isotropic. Therefore, once the stratigraphy is known, it is possible to estimate the stiffness of the different layers by means of deflection basin analysis, using the backcalculation procedures commonly adopted for analysis of tests conducted on finished pavements.

The comparison, described in this study, between modulus values obtained through deflectometric tests performed both during construction, directly on subbase layers built using unbound aggregates, and values obtained after laying of the asphalt concrete layers, contributes to demonstrating the plausibility and reliability of the test procedure adopted.

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