ABSTRACT

"Layer moduli during HVS testing: comparing laboratory results with backcalculations from FWD and MDD deflections"

Authors: P. Ullidtz [(1)], J. T. Harvey, M. Riemer [(2)] and J. A. Prozzi [(3)]

During Heavy Vehicle Simulator (HVS) testing of four flexible pavement sections at University of California, Berkeley, Falling Weight Deflectometer (FWD) tests were carried out, both on the test sections and on non-loaded areas. Deflections were also measured under the rolling wheel of the HVS, at several depths, using Multi Depth Deflectometers (MDD). At the same time, extensive laboratory testing was performed on the different layer materials to determine their mechanical properties.

A reasonably good match between calculated and measured MDD deflections could be obtained for all depths and distances, when the subgrade was treated as being highly non-linear elastic. The backcalculations were done in an Excel spreadsheet, using Boussinesq's equations with Odemark's layer transformation, and assuming the subgrade modulus to be decreasing with increasing major normal stress. The moduli of the unbound layers, derived from MDD data, were generally lower than the corresponding moduli from FWD data, whereas the asphalt moduli were similar.

Triaxial testing was performed in the granular materials to determine the variation of the mechanical properties as the confining stress and moisture conditions were changed. In the case of bituminous materials, flexural dynamic stiffnesses were determined at various temperatures and loading frequencies. These tests permitted the comparison with mechanical properties determined from in-situ testing.

Both FWD data and MDD data showed a considerable decrease in asphalt modulus during the HVS loading test. Two test sections with 75 mm asphalt treated permeable bases (APTB) showed a decrease in asphalt modulus to 48% of the original value, whereas the two sections with the conventional aggregate base (AB) had decreased to 33% and 25%.

After the first series of HVS loading tests, two test sections were overlaid by 38 mm of asphalt-rubber hot-mix (ARHM) and two by 75 mm dense graded asphalt concrete (DGAC). On the two sections with ATPB the different overlays caused a similar increase in average asphalt modulus, to about 65% of the original value. On the sections with conventional AB the 75 mm overlay increased the average modulus from 33% to 88%, whereas the 38 mm overlay caused an increase from 25% to 38%.

On one of the sections with ATPB, a second HVS loading test after overlay, caused the average modulus to decrease from 65% to 19% of the original value, or to 29% of the value at the beginning of the second HVS loading test.

FWD data also indicated that the moduli of the granular layers were influenced by the stiffness of the asphalt layer. An increase in the stiffness of the asphalt layer caused an almost equivalent increase in the modulus of the granular layers, up to a certain level.

A temporary recovery of the stiffness of the asphalt concrete layer was observed after the pavement was closed to HVS traffic. However, it was also observed that the stiffness of the asphalt drops rapidly soon after the section was opened to traffic again.

[(1)] Technical University of Denmark, Lyngby, Denmark; [(2)] University of California, Berkeley, USA; [(3)] CSIR, Transportek, Pretoria, South Africa.

Layer moduli during HVS testing: comparing laboratory results with backcalculations from FWD and MDD deflections

by P. Ullidtz [(1)], J. T. Harvey, M. Riemer [(2)] and J. A. Prozzi [(3)]

[(1)] Technical University of Denmark
Institute of Planning
DTU, Building 115
DK 2800 Lyngby
Denmark
e-mail: pullidtz@ivtb.dtu.dk

[(2)] University of California,
Pavement Research Center
1353 South 46th St.
Richmond, CA, 94804-4603
e-mail: jharvey@newton.berkeley.edu

[(3)] CSIR
Transportek
P O Box 1619
El Cerrito, CA, 94530-4619
e-mail: jprozzi@csir.co.za
Phone: +1 (510) 231-9470
Fax: +1 (510) 231-9553

1. INTRODUCTION

The objective of this paper is to present a comparative backcalculation analysis of the accelerated testing of a series of asphalt concrete (AC) pavement sections by the Heavy Vehicle Simulator (HVS). The testing was conducted in an indoor facility at the Richmond Field Station (RFS) of the University of California at Berkeley (UCB) and is part of the CAL/APT Program of the California Department of Transportation (Caltrans). The HVS testing is complemented with extensive laboratory and in situ testing, which enable the characterization of material properties.

Initially, four pavement sections were constructed and tested at UCB, namely: 500RF, 501RF, 502CT and 503RF. The two *drained* sections (500RF and 502CT) have a 76 mm thick asphalt-treated permeable base layer (ATPB) on top of a conventional aggregate base layer (AB), while the remaining two *undrained* sections (501RF and 503RF) contain only a conventional AB. The HVS testing of the four sections was conducted as part of Goal 1 of the CAL/APT Strategic Plan and was undertaken between May 1995 and September 1996. By September 1996, the four sections were trafficked to a terminal condition and were in need of major rehabilitation. In March 1997, sections 500RF and 501RF were overlaid with a 76 mm dense graded asphalt concrete (DGAC) overlay, and sections 502CT and 503RF with a 38 mm thick asphalt rubber hot mix (ARHM) overlay. The HVS testing of the rehabilitated sections commenced immediately afterwards and concluded in May 1999.

Pavement response was measured using Multi-Depth Deflectometers (MDDs), the Falling Weight Deflectometer (FWD), the Road Surface Deflectometer (RSD), and the laser profilometer. Fatigue crack development was monitored using digitized photographs and analyzed using image analysis software. Thermocouples were used to measure air temperature and pavement temperatures at various depths in the asphalt concrete. A temperature control chamber was used to maintain the sections within 20 ±4°C. In-depth moisture content and water table levels were also monitored.

2. LABORATORY TESTING AND DESCRIPTIONS OF PAVEMENT MATERIALS

2.1 Boreholes

Four boreholes were dug within the HVS test sections prior to construction placement of the granular base and asphalt concrete layers in April, 1995. The drained and undrained pavement sections each contained two boreholes each. Subgrade samples were collected and moisture contents were measured at various depths in each hole.

The soil profiles for all the boreholes were fairly similar. The uppermost layer of the subgrade is a stiff, high plasticity black or gray clay. Below, there are repeated layers of stiff brown clay and softer green clay with inclusions of small white stones typically about 4 mm in diameter. The test sections are in an alluvial plane, in which clay deposits have alternated with sand and gravel deposits from floods (Harvey et al., 1996).

The groundwater table was encountered at depths between 3.5 and 4.8 m (11.5 and 15.7 ft.). In 1996, groundwater-monitoring tubes were installed just outside the test sections, and in August 1997 tubes for monitoring subsurface water contents using a nuclear probe were installed about 0.5 m to the side of the HVS test sections. Figures 1 and 2 show changes in the subsurface water contents from August 1997 to December 1998. Figure 1 shows the data recorded at the centerline between Sections 500RF and 502CT. Figure 2 shows the information obtained between Sections 501RF and 503RF. The water content at five different levels was recorded: base (AB), subbase (ASB) and three different levels within the subgrade material (SG1, SG2 and SG3). No significant seasonal variation can be determined from the data; the moisture content seems to be changing following a fairly random pattern.

2.2 Classifications and in-situ conditions

Soils classifications by the Unified Soil Classification System (USCS) and AASHTO systems, relative compaction, and densities for each layer are summarized in Table 1. The classification indicates that the subgrade consists of clayey materials, and the subbase and base layers consist of high quality coarser granular materials. Subgrade samples were found to have plasticity index values ranging from 27 to 41 and liquid limits of 39 to 55. By the USCS, the upper 1.5 to 2.0 meters (5.0 to 6.6 ft.) of subgrade is a high plasticity clay. Below 1.5 to 2.0 m from the top of the subgrade the liquid limit and plasticity index are smaller changing the classification to low plasticity clay. By the AASHTO Soils Classification System, the entire subgrade is classified as an A-7-6 to more than 5.0 m deep (16 ft.). Compaction test results indicated that subgrade samples taken in April 1995 were at or near saturation (Harvey et al., 1996).

TABLE 1: Summary of soil properties at pavement construction, April 1995

Pavement layer	AASHTO Classification	Aggr. Size, 95% passing	UCS Classification	Water contents (April 1995)	Relative density (April 1995)[(a)]
Aggregate base	A-1-a	19 mm (3/4")	GW	5.0-6.0	99-103 percent
Aggregate subbase	A-1-a	25 mm [(b)] (1.0")	GW	4.6-7.9	98-100 percent
Upper subgrade (1.5 to 2.0 m)	A-7-6	0.075 mm (#200)	CH	16.4-23.9	91-98 percent, (average 95)
Lower subgrade (below 2.0 m)	A-7-6	0.075 mm (#200)	CL		

[(a)] Base and subbase compaction relative to Caltrans method 216.
[(b)] Some large aggregates up to 75 mm (3.0") in size were encountered in the subbase.

The subgrade was originally compacted in the mid-1960s, and was not re-compacted for the construction of the test sections because it met Caltrans compaction specifications. It is doubtful that the subgrade was originally compacted at water contents near saturation. It is highly likely that additional water has been taken in through capillary draw since it was compacted 30 years ago.

The subbase also consists of in-place material originally placed in the mid-1960s, and re-compacted in 1995 to meet Caltrans specifications. The aggregate base material was imported. As can be seen from Table 1, there was some variability in water contents and densities across the test sections after construction. This was in part due to the history of the subgrade and subbase, which were originally placed in the mid-1960s, but which met current applicable Caltrans specifications for compaction and gradation for new pavements in April, 1995.

2.3 Triaxial test results

Two subgrade specimens were compacted from bulk site samples by kneading compaction using a Harvard Miniature Compactor to achieve water contents and densities similar to those encountered at the site in 1995. The first specimen was compacted to a wet density of 2.06 g/cm^3 (129 pcf) at a water content of 22.4 percent, the second to 2.12 g/cm^3 (132 pcf) at a 15.8 percent water content.

The stiffness of clayey soil is typically much higher immediately following compaction, in the "as-compacted" state, than it is after the compacted soil is allowed free access to water during soaking. This is due to the additional binding strength between particles caused by the interfacial tension of water surrounding air bubbles within the voids. To investigate the importance of this effect, the moduli of each of the re-compacted subgrade specimens were investigated both in the as-compacted state and after soaking. Subgrade moduli were measured by triaxial testing and bender element testing (Harvey et al., 1996). Only the results of the triaxial tests are reported in this paper. The triaxial tests were performed following the "Standard Method of Test for Resilient Modulus of Subgrade Soils" (AASHTO Designation T 274-82 (1986)) for fine soils. Some modifications and additions were made to the test procedure (described in Harvey et al., 1996) to address specimen drainage difficulties and pore pressure redistribution between changes in stress state.

Specimen 2 was tested under three conditions: as compacted, soaked, and saturated. Specimen 1 was tested under the as-compacted and soaked conditions. The "B" value was used as a measure of saturation, where a B value close to 1.0 indicated full saturation, a B value of about 0.75 represented the soaked condition, and a B value of 0.96 represented the saturated condition.

The triaxial test results (shown in Figures 3 and 4 for Specimens 1 and 2, respectively) indicated differences between the as-compacted stiffness, soaked, and saturated stiffness for both specimens. Following soaking, the resilient modulus drops to between 45 and 100 percent of the original as-compacted value, with larger changes occurring at smaller confining stresses and larger deviator stresses, which are the stress states causing larger strains. Specimen 1 showed less change in stiffness after soaking but the stiffness remains lower than the stiffness of Specimen 2 regardless of the stress state.

It is also apparent that immediately following compaction, Specimen 2 did not experience a significant decline in modulus with increasing strain levels. The modulus seems to remain fairly constant, or even increase slightly as the loading level increases. This independence of modulus from strain is unusual for fine-grained soils, which are typically strain softening. In this case, it probably results from an increasing mobilization of the capillary forces when the soil structure is deformed, perhaps due to dilation. Once these capillary forces are relaxed by the saturation of the specimen, the resilient modulus does seem to decline with strain, as would be expected for dry or saturated specimens. Intrinsically different stiffness are apparent between the two specimens. The stiffness of Specimen 2 is approximately double that of Specimen 1 for the same stress state because of the increased compaction of Specimen 2.

Several aggregate subbase and aggregate base specimens were tested in a triaxial apparatus in the as-compacted, and saturated states. The drainage system for the test site pavements and lack of capillary potential in these layers suggest that the results for the saturated specimens are not applicable. Technical difficulties made unusable all of the aggregate subbase and all but one of the aggregate base results corresponding to the partially saturated specimens.

Some stiffening of the aggregate base was observed in the ten days after it was placed and compacted, and before the surface layers were placed. A sample of the aggregate base was compacted to a wet density of 2.47 g/cm^3 (154 pcf) at a water content of 5.5 percent using a California kneading compactor. The specimen size was 100 mm (4") in diameter and 200 mm (8") in height. An impervious membrane was placed on the specimen after compaction. The top was allowed to remain exposed to air for 10 days to simulate the effects of exposure to air on the test section. The water content dropped from 5.5 to 2.9 percent during the 10 days. The specimen was then tested following the AASHTO T274 procedure for granular materials. The specimen was then saturated and tested again. The resulting equations for resilient modulus (kPa) are:

$M_R = 229{,}793\ Q^{0.1620}$ after 10 days exposure, and
$M_R = 21{,}181\ Q^{0.4896}$ in the saturated condition, where Q is the bulk stress (kPa).

For a range of bulk stresses between 35 and 600 kPa (5 and 87 psi), the stiffness of the specimen after 10 days exposure is 409 to 648 MPa (59,000 to 94,000 psi). The range for the saturated condition is 121 to 485 MPa (18 to 70 psi). The stiffness in the saturated condition is typical of the other four aggregate base specimens in the same condition. The stiffness of the two aggregate subbase specimens in the saturated condition is about 40 to 60 percent of those of the saturated aggregate base specimens for the same bulk stresses. The in-situ water content measurements (Figures 1 and 2) indicate that neither of the aggregate layers had water contents approaching saturation during the HVS tests.

2.4 Stiffness of asphalt treated permeable base (ATPB)

The asphalt treated permeable base (ATPB) has fairly uniform gradation, with 92 percent passing the 19 mm (3/4") sieve, 27 percent passing the 9.5 mm (3/8") sieve and less than 5 percent passing the No 8 sieve. The asphalt content for the AR-8000 binder was 2.7 percent by mass of mix. Samples were collected during construction, and later reheated and compacted to refusal in the laboratory for testing. The compacted specimens were 100 mm (4") in diameter and 200 mm (8") in height, and had an average air-void content of 35 percent.
The four compacted specimens were tested for compressive triaxial stiffness following the same procedure used for the aggregate base. The ATPB was found to have sufficient asphalt to render its stiffness essentially insensitive to the bulk stress at 20°C. However, there was a great deal of variability in the measured stiffness, in part due to the pneumatic loading system used for the testing. The average stiffness was 1,258 MPa (183,000 psi), and the standard deviation was 406 MPa (59,000 psi) at a bulk stress of 750 kPa (109 psi).

Handling of the ATPB material confirmed that it has considerable tensile strength and stiffness. These properties were, however, not measured. Repetitive unconfined compressive loading resulted in a 27 percent loss of stiffness after 194,000 repetitions, indicating that the material is susceptible to damage from repeated loading. Axial stress of 138 kPa (20 psi) was applied for 0.1 seconds followed by a 2.9 seconds rest period. The axial stress was calculated using the layer elastic theory to represent the deviator stress in the ATPB layer under a 40 kN dual wheel load (9,000 lb.).

2.5 Stiffness of asphalt concrete (AC) at different frequencies and temperatures

The asphalt concrete (AC) used to surface the HVS test sections had a dense gradation with a maximum aggregate size of 19 mm (3/4"). The AC was placed in two lifts, with no tack coat between them. Cores and slabs cut throughout the pavements showed no tensile bond between the two lifts, although some frictional resistance to shear due to aggregate interlock was apparent.

The average asphalt contents for the AR-4000 binder were found to be 4.8 and 5.2 percent by mass of mix for the bottom and top lifts, respectively. Cores and slabs also revealed that the bottom lift had been densified more than the top lift by the additional compaction effort on the top lift while the bottom lift was still hot. Air-void contents in the bottom lift were between 3.0 and 4.5 percent, and between 6.0 and 7.5 percent for the top lift. (Harvey et al., 1996).
Slabs taken from the test section were cut into fatigue beams and subjected to flexural frequency sweeps at temperatures of 15, 20, and 28°C. These temperatures were selected to reflect the range of temperatures encountered in the AC during HVS testing, and are similar to the range of asphalt concrete temperatures encountered during FWD testing. Average flexural stiffnesses calculated from the tested specimens are shown in Figure 5. Considering that the loading time for the Dynatest Falling Weight Deflectometer is about 25 to 30 milliseconds, ranges of average flexural stiffness results

applicable to the FWD testing at the CAL/APT site are shown in Table 2 (from lower temperature, higher frequency to higher temperature, lower frequency).

TABLE 2: Average flexural stiffness for asphalt concrete applicable to FWD test conditions.

Test conditions applicable to FWD testing	Expected flexural stiffness for the top lift in MPa (psi) [a]	Expected flexural stiffness for the bottom lift in MPa (psi) [b]
15°C, 10 Hz	11,410 (1,658,000)	14,160 (2,057,000)
19°C, 5 Hz	8,280 (1,203,000)	10,340 (1,502,000)
28°C, 2 Hz	2,330 (339,000)	2,790 (405,000)

[a] Average specimen air-void content: 6.9 percent
[b] Average specimen air-void content: 3.4 percent.

Additional tests were run at low frequencies and 20°C (Figure 6). The lower frequencies corresponded to the slow wheel speed of the HVS used for deflection testing over the MDDs in the test sections, about 0.5 kph (0.3 mph), which is equivalent to a loading time on the order of 1 second. At these frequencies and 20°C, the flexural stiffness for the top AC lift is expected to be between approximately 2,500 and 3,000 MPa (363,000 and 436,000 psi), and for the bottom lift between 3,000 and 3,500 MPa (436,000 and 509,000 psi).

3. FIELD TESTING AND ANALYSIS

3.1 General

Applied loads and pavement response data obtained with the FWD under an impact pulse load and with the MDD under the slow moving wheel of the HVS were used to backcalculate moduli of the pavement layers at various stages of trafficking. Moduli were estimated for the asphalt concrete layers as well as for the coarser granular materials (base/subbase) and for the finer cohesive layer (subgrade). Backcalculated moduli, in turn, were compared with moduli obtained in the laboratory under various testing conditions as discussed previously.

The responses under the FWD and HVS were obtained under various load magnitudes but under unique environmental conditions, i.e. the conditions prevailing at the date and time the test was carried out. Although the conditions under APT testing were intended to be kept relatively constant, this was not always the case.

On the other hand, non-destructive laboratory testing enabled material properties to be evaluated under different conditions, i.e., a range of temperatures, different types of loading, loading magnitudes and times, moisture conditions, etc. However, it is usually difficult (often impossible) to simulate actual field conditions and therefore manipulation of the data is required to compare the results. Differences between laboratory and in-situ conditions include:

(i) specimen preparation which fails to reproduce the material matrix encountered in the field (density, particle orientation, etc.),
(ii) differences in testing conditions such as support conditions, temperature, moisture, etc., and
(iii) differences in loading conditions such as magnitude, time and especially the way in which the load is applied to the specimen.

Furthermore, the model used in the backcalculation analysis also often differs: finite element methods, multi-layer linear elastic theory, method of equivalent thickness, etc. All these methods would only yield the same answer if the materials were linear-elastic in nature. Pavement materials are, by no means, linear elastic and often their behavior differs substantially from such an assumption. Nevertheless, the assumption generally made by highway engineers is that under the “small” strains induced by traffic loading, linear-elastic behavior holds approximately true. Often, this assumption is made solely because of the ease in applying linear elastic (LE) analysis as opposed to models that better simulate the actual material behavior and response.

Another general simplification, when using LE theory, is that Poisson's ratios are assumed within a narrow range (0.40 to 0.30). Even though the backcalculation analysis is not significantly affected by ratios within this range, it has been observed that the influence becomes significant with Poisson's ratios larger than 0.45. Ratios larger than 0.5 have been measured in the laboratory under loading conditions intended to simulate in-situ field conditions. However, Poisson's ratios larger than 0.5 are not feasible within the context of linear elasticity, and the concept lacks meaning outside the context of the theory. The phenomenon observed is actually the dilation of the material.

Because of the above mentioned differences it cannot be expected that the backcalculated moduli from these different methods and different theories to be the same. In order to obtain meaningful comparisons, manipulation of the results is required. Each of the approaches has advantages that can be capitalized upon if the various methodologies are properly combined. For routine purposes at a network, or even at a project level, the use of all the approaches simultaneously would be unfeasible and impractical. Such an extensive analysis should be limited to only a few specially designed projects that have the technical and financial resources required to analyze all the information that the data provide.

FWD testing is preferred for network and often project level analysis due to its high productivity and proven reliability. However, for a meaningful analysis of the FWD data, additional information of the pavement structure and materials is always required. Such information can be provided by the MDD system. FWD backcalculation is based on deflection data from the surface deflection bowl, typically 7 to 9 points. This deflection bowl usually extends up to 1200-1500 mm away from the loading point. In this region, the stress-strain conditions of the materials are very different from those in the area immediately under the loading pad. Thus, the strain sensitivity of the materials is directly incorporated into the response data. This strain sensitivity (especially when it affects various layers) cannot be assessed correctly with the response from such a reduced number of sensors.

The MDD system, on the other hand, records up to six quasi-continuous deflection bowls at various depths (256 measurements per bowl at 6 different depths = 1536 data points). This comprehensive set of data facilitates a better estimation of the stress-sensitivity of the various materials. This advantage of the MDD is also its main disadvantage: an increase in the quantity of response data increases the complexity of the analysis.

Another advantage of the MDD system is that it records not only the elastic behavior, but also the in-depth plastic response and performance of the various materials. This information is necessary for establishing failure criteria associated with surface rutting and for determining the percentage each layer contributes to the total permanent deformation measured at the surface. Plastic analysis, however, is out of the scope of this paper.

3.2 Analysis based on FWD data

From April 1995 to January 1998, 11 series of FWD tests were carried out on the four flexible test sections at Richmond Field Station. The dates of the tests are given in Table 3. Most of the testing was done in four lines: line A was outside test areas 503RF and 502CT, line B over test areas 503RF and 502CT, line D over test areas 501RF and 500RF, and line E outside of these areas (Figure 7).

All pavement sections were treated as a 3-layer system with an asphalt concrete surface, a base course (combining base and subbase) on a semi-infinite subgrade (an alternative model with a non-infinite subgrade was also used). For the *undrained* sections (501RF and 503RF), the asphalt thickness (before overlay) was assumed to be 137 mm (5.4") and on the *drained* sections (500RF and 502CT) 213 mm (8.4"). Between FWD test series 6 and 7, a 38 mm (1.5") thick asphalt rubber hot mix (ARHM) overlay was placed at lines A and B (test sections 502CT and 503RF). The ARHM thickness was increased to 62 mm from chainage 24 to 38 m (80 to 130 ft.) (Figure 7). Also a 76 mm (3") thick dense graded asphalt concrete (DGAC) overlay was placed at lines D and E (test sections 500RF and 501RF).

TABLE 3: FWD and HVS testing dates.

AC test 1 AC test 2 AC test 3 AC test 4 AC test 5 AC test 6	Before overlay	20-Apr-95 22-Sep-95 28-Feb-98 09-Oct-96 06-Nov-96 29-Jan-97
	Section 500RF Section 501RF Section 502CT Section 503RF	3-May-95 / 9-Nov-95 20-Nov-95 / 26-Feb-96 5-Dec-95 / 20-Sep-96 5-Mar-96 / 9-Sep-96
AC test 7 AC test 8 AC test 9 AC test 10 AC test 11	After overlay	28-Mar-97 21-Apr-97 06-Oct-97 17-Nov-97 06-Jan-98

The assumed thickness of the granular layers is given in Table 4. Test sections 501RF and 503RF were situated from chainage 6.0 to 13.7 m (20 to 45 ft.), measured from the starting point of the FWD testing (Figure 7). Sections 500RF and 502CT were situated from chainage 54.9 to 62.5 m (180 to 205 ft.) approximately.

The Method of Equivalent Thickness (MET) was used in the inverse analysis (backcalculation). This method is based on Boussinesq꞉s solution for a semi-infinite half-space, combined with Odemark꞉s transformation of a layered system (Ullidtz, 1987, Ullidtz, 1998). Boussinesq꞉s solution has been modified to accommodate a non-linear subgrade layer, where the modulus at a point varies with the major principal stress, S_1, on the form:

$$E = C \times \left(\frac{S_1}{p} \right)^n$$

Where p is atmospheric pressure and C and n are constants (n is negative). The corresponding surface modulus, for calculating deflections, is found from:

$$E = (1 - 2 \times n) \times C \times \left(\frac{S_1}{p} \right)^n$$

On instrumented test pavements, the layer moduli are first derived from FWD deflections, and then used to calculate stresses and strains at the position of the gauges. This method has repeatedly been found to provide results that are as good as results obtained with the Finite Element Method (FEM). Linear elastic theory normally results in a rather poor agreement between measured and calculated subgrade strains (Baltzer et al., 1998).

It should be noted that the "non-linearity" may be only partly due to a true non-linearity of the subgrade. Part of it may be caused by dynamic effects or may be due to the granular nature of some of the layers. Treating it as if it were a true non-linearity, and using the simple Odemark-Boussinesq method has, however, been found to result in a reasonably correct calculation of the pavement response, in terms of deflections, stresses and strains (Ullidtz, 1998).

Some examples of the calculated moduli from line B are given in Figures 8 and 9 for the backcalculations corresponding to <u>before overlay</u> and <u>after overlay</u>, respectively. It can be observed that before any HVS tests were carried out (FWD test series 1 and 2), the two HVS test areas had approximately the same moduli as the rest of the pavement length, although there is a considerable variation in the moduli along the length of the area. Part of this variation is due to the stochastic nature of the layer materials, but part can be attributed to a systematic variation in thickness of the asphalt layer. The area between chainage 24 m and 38 m has a thicker ARHM overlay. This results in an overestimation of the asphalt moduli for this area.

TABLE 4: Assumed thickness of the base/subbase layer.

Section	Line	Base/subbase thickness in mm (")
500RF	D	356 (14)
500RF	E	330 (13)
501RF	D	432 (17)
501RF	E	406 (16)
502CT	A	432 (17)
502CT	B	406 (16)
503RF	A	508 (20)
503RF	B	483 (19)

The density of the test points within the HVS test areas was increased after test series number three. After the HVS tests, the asphalt modulus within the HVS test areas is considerably lower than within the non-loaded areas.

Even the granular materials show a certain decrease in backcalculated modulus as a result of the HVS testing (Figure 10). This is more obvious from the summary of the test results discussed in the next section. For the subgrade, no effect of the HVS loading can be seen (Figure 11) so the variability is only attributed to the stochastic nature of the material.

3.3 Summary of FWD results

A summary of the backcalculated asphalt moduli is given in Figure 12. In this figure, the mean modulus of the asphalt is shown for each of the test areas, as well as the mean value for all the FWD test points outside of the HVS test areas - that is the unloaded or non-trafficked areas (legend "Rest" in the figure). All values are shown as a function of the date of FWD testing. Figures 13 and 14 gives the backcalculated moduli for the base and subgrade layers, respectively.

Until February 1997, the modulus is for the original asphalt only. After February 1997 the modulus includes the overlay. No significant difference in modulus was found between the drained and the undrained sections.

In the following three figures (Figures 15 to 17) the *"relative modulus"* is shown as a function of FWD testing date. The relative modulus is the modulus at the HVS test section divided by the modulus of the non-tested area ("Rest"). Thus, the relative moduli should exclude any effects of temperature, time, or any other seasonal variation. The periods of HVS tests are also indicated on the graphs.

If the maximum relative asphalt modulus before HVS testing is taken as 100%, the two drained sections drop to 48%, whereas the undrained sections drop to 33% (501RF) and 25% (503RF). After overlay, the two drained sections have moduli of about 65% of the maximum values before HVS testing. The modulus of the undrained section overlaid by 76 mm (3") DGAC increases to 88% (from 33%, 501RF) and that of the section overlaid by 38 mm (1.5") ARHM increases to 38% (from 25%, 503RF). Some of the increase after overlay could be attributed to temporary recovery of stiffness ("*pseudo-healing*") of the original asphalt layer. The temporary recovery of stiffness is commonly observed after HVS trafficking is discontinued, however, this condition reverts rapidly after trafficking is restarted. Therefore, this condition could be referred to as pseudo-healing rather than healing because the change in the properties is only temporary.

After overlay, section 500RF was tested again with the HVS. This caused the asphalt modulus to decrease from 65% of the maximum to 19%.

With the current analysis, it was not viable to estimate to what extent the decrease in stiffness of the asphalt layer is due to a lack of bonding between the asphalt layers or to the formation of (micro) cracking in the asphalt. It is important to highlight that the relative decrease in the moduli of the granular layers is very similar to that of the asphalt layer. For the subgrade, there is no effect (or a very limited effect) of the HVS tests.

The variation of the modulus of the granular material with the stiffness of the asphalt appears to be a general phenomenon throughout the experiment. This is evidenced in Figure 18, where the modulus of the granular layer (base/subbase) has been plotted against the modulus of the asphalt. The modulus of the granular layer is roughly one tenth of the asphalt modulus, up to an upper limit of about 500 MPa. The same phenomenon, of increasing modulus of granular layers with increasing asphalt stiffness, has been observed on other test roads, and deserves further exploration (Ullidtz & Ekdahl, 1998).

Inaccuracies in the inverse analysis (backcalculation) may lead to overestimation of the modulus of one layer if another layer is underestimated. This would produce the opposite of the above phenomenon, i.e., the modulus of the granular layer would be decreasing with increasing stiffness of the asphalt.

3.4 Analysis based on MDD data

At test section 501RF, MDD data (De Beer et al., 1988) was used to backcalculate the layer moduli after 203,498; 595,552 and 1,426,467 load repetitions. The measured deflection basins are shown in Figures 19 to 21 for the surface, base, and subgrade, respectively. The differences in deflections from one measurement to another are consistent and the data shows good repeatability.

The backcalculation was done in an Excel spreadsheet using Odemark-Boussinesq's method with a non-linear subgrade. Deflections were calculated at the asphalt surface, the top of the base course, the top of the subbase, and the subgrade. The deflections were calculated at the same positions where MDD data were collected and the Root Mean Square (RMS) difference between the measured and calculated deflections was minimized by changing the elastic moduli of the layers. This was done using the built-in function "Solver". An example of the agreement between measured and calculated deflections is shown in Figures 22 to 24 for the surface, base, subbase, and subgrade, respectively.

The backcalculated moduli obtained from the MDD data are given in the Table 5. It should be kept in mind that the MDD deflections were measured under a slow moving dual-wheel with 100 kN load (22.5 ksi). Moduli were backcalculated at three stages of trafficking, i.e. 203,498; 595,552 and 1,426,467 load repetitions respectively.

TABLE 5: Backcalculated moduli from MDD data for Section 501RF.

Repetitions	E1 (MPa)	E2 (MPa)	E3 (MPa)	C (MPa)	n	E_{om} (MPa)
203,498	8,637	129	60	9.6	-1.00	62.7
595,552	3,973	87	36	12.6	-1.00	58.9
1,426,467	1,268	123	74	16.2	-0.94	59.1

Moduli derived from FWD testing at test section 501RF, before and after the load testing, are given in Table 6. Also shown are the moduli on the unloaded area, before and after the load testing.

The asphalt modulus from the MDDs is larger than the FWD modulus at the beginning of the loading, but smaller at the end of the test. The value at the beginning of the loading is in good agreement with the FWD modulus derived for the non-loaded area, although a smaller value would have been expected due to the longer loading time under the slow-moving wheel of the HVS.

TABLE 6: Backcalculated moduli from FWD data for Section 501RF.

FWD	E1 (MPa)	E2/E3 (MPa)	C (MPa)	n	Eom (MPa)
501, before	3,917	434	134.0	-0.07	165
501, after	2,127	109	52.0	-0.31	122
Unload before	5,862	450	91.7	-0.17	149
Unload after	9,750	462	79.9	-0.19	137

The moduli of the unbound materials are much lower under the slow moving rolling wheel than under the impulse load of the FWD, and the apparent non-linearity of the subgrade is much more pronounced under the MDDs (a lower limit of -1.0 was imposed in the use of "Solver"). An exception is for the granular layers at the end of the HVS loading test, where the FWD modulus for the combined layers is similar to the moduli derived from MDD data.

MDD data collected at the beginning of the experiments were not well suited for backcalculation due to poor repeatability and reproducibility. Examples from test section 501RF, measured with one MDD for three repetitions of the load, are shown in Figure 25. The figure shows deflections at the surface of the pavement. Had the repeatability been good, the three curves would have been identical.

The MDDs are anchored at a depth of approximately 3 m, so that the deflection below this level does not contribute to the overall deflection. The calculated (theoretical) deflection at this depth should, therefore, be subtracted from the overall deflection. The "effective" depth to the anchor point appears to be less than the actual depth. There exist in the literature various ways of estimating this depth; in this study, the depth was determined from a plot of surface deflection as a function of the distance from the center of the load versus the inverse of this distance (Ullidtz, 1987). The depth estimated in this way was on average 1864 mm. The non-linearity constant was fixed to n = -0.20, and the backcalculated moduli are given in Table 7.

In this case, there seem to be a better agreement between MDD- and FWD-based asphalt moduli except for the asphalt concrete modulus at 1,426,467 repetitions, which is rather low. For the granular materials and subgrade there is not a significant change when using this approach.

TABLE 7: Backcalculated moduli from MDD data considering the anchor depth.

Repetitions	E1 (MPa)	E2 (MPa)	E3 (MPa)	n	E_{om} (MPa)
203,498	3,386	164	51.4	-0.20	52.2
595,552	1,671	109	35.1	-0.20	51.5
1,426,467	333	140	52.9	-0.20	54.2

The last figure (Figure 26) compares the deflection at the top of the base course when measured with two different MDDs under the same load and for the same pavement structure. This difference can be attributed to the variability of the quality of the material across a supposedly homogeneous section. Part of that difference can be also attributed to measurement error.

3.5 Comparison with laboratory data

From triaxial tests on the subgrade material, relationships between the modulus and the major principal, dynamic stress, may be derived in the same format as used with the FWD analysis. The weight of the pavement layers was about 14 kPa (2 psi) so, selection of the triaxial test at the confining stress of 7 kPa (1 psi) and in the soaked condition seemed to be appropriate. For the two specimens the following relationships were obtained:

$$Specimen\,1: \quad E = 36.4 \times \left(\frac{S_1}{p}\right)^{-0.34}$$

$$Specimen\,2: \quad E = 52.4 \times \left(\frac{S_1}{p}\right)^{-0.20}$$

From the FWD test at the beginning of the experiment the relationships were:

$$Section\,501: \quad E = 46.6 \times \left(\frac{S_1}{p}\right)^{-0.32}$$

$$Section\,502: \quad E = 36.9 \times \left(\frac{S_1}{p}\right)^{-0.29}$$

$$Section\,503: \quad E = 49.0 \times \left(\frac{S_1}{p}\right)^{-0.30}$$

At a typical FWD subgrade stress of 30 kPa (4.4 psi), the two triaxial specimens have moduli of 54.8 MPa (8,000 psi) and 66.7 MPa (9,700 psi), respectively. From the FWD test the moduli are 501: 68.5 MPa, 502: 52.3 MPa and 503: 70.3 MPa (10,000; 7,600 and 10,200 psi, respectively). The scatter is larger for the MDDs. The values are 501: 46.7 MPa, 502: 33.4 MPa and 503: 80.8 MPa (6,800; 4,900 and 11,700 psi respectively). Thus, the averages are 60.8 MPa (8,800 psi), 63.7 MPa (9,300 psi) and 53.6 MPa (7,800 psi) for the triaxial, FWD and MDD, respectively. This is considered a very good agreement, and it is far better than the factor of three between triaxial and FWD tests normally reported in North America (Darter et al., 1991).

For the granular layers the triaxial tests result in moduli from 409 to 648 MPa (59,400 to 94,100 psi) for the unsaturated condition, which is most relevant for the test pavements. For the pavement sections that had no HVS tests, the modulus of the combined base and subbase layers from backcalculated FWD data was mostly in the range of 400 to 500 MPa (full range 250 MPa to 600 MPa (36,300 to 87,200 psi)).

Unfortunately, during FWD testing, the asphalt temperature was not recorded. Therefore, the unexplained scatter on the backcalculated moduli can be partly attributed to unobserved temperature effects. In an attempt to evaluate the effect of temperature, backcalculated moduli were adjusted using the air temperature obtained from a weather station. The test number, the month of testing, the average modulus on the non-loaded test sections, and the average maximum air temperature for the corresponding month are given in Table 8.

There is a tendency for the moduli to be higher during the winter months, but there is probably also an increase in moduli with time due to aging. The range of moduli is within the range given in Table 2, i.e., 2,330 MPa to 11,410 MPa. In the last column of Table 8, the calculated moduli are adjusted to an (air) temperature of 15°C by dividing the measured modulus by the factor F_C = {1-2.5*log10 (temperature/15°C)}. There is some indication that the modulus (after some months of hardening) at a temperature around 15°C would be about 10,000 MPa, which is within the range of expected values reported in Table 2.

TABLE 8: Effects of temperature in asphalt concrete moduli.

Test No	Month	Moduli in MPa (psi)	Avg. Max Temp (°C)	Adj. Moduli in MPa (psi)
Before overlay				
1	April	3,300 (480,000)	17	3,800 (552,000)
2	September	5,900 (857,000)	23	11,000 (1,598,000)
3	February	9,800 (1,426,000)	16	10,500 (1,527,000)
4	October	6,300 (915,000)	21	9,900 (1,438,000)
5	November	8,700 (1,264,000)	17	10,000 (1,453,000)
6	January	10,000 (1,453,000)	14	9,300 (1,351,000)
After overlay				
7	March	4,400 (639,000)	17	5,100 (741,000)
8	April	5,700 (828,000)	17	6,600 (959,000)
9	October	6,100 (886,000)	21	9,600 (1,395,000)
10	November	8,600 (1,250,000)	17	10,000 (1,453,000)
11	January	10,400 (1,511,000)	14	9,700 (1,409,000)

4. CONCLUSIONS AND RECOMMENDATIONS

This paper describes some of the results of the analysis which were carried out to characterize the layer material properties of the various pavement sections that were tested as part of the CAL/APT Program. The sections were subjected to accelerated pavement testing (APT) under the Heavy Vehicle Simulator (HVS) at the Richmond Field Station (RFS) of the University of California at Berkeley from 1995 to 1999. Some of the main findings during this study are the following:

(i) Under the particular conditions of this study, the main and most important differences between FWD and MDD analysis relate to the type of loading. The loading time under the Dynatest FWD is 25 to 30 milliseconds, while under the HVS wheel it is in the order of 1 second. As indicated earlier, the type of loading also has a important influence in the response of the pavement structure. Notwithstanding these differences, both methodologies (FWD and MDD) were able to assess adequately the damage to the pavement structure as the trafficking increased. This damage was evaluated as the decrease in the stiffness of the various layers. All asphaltic and unbound granular layers showed a decrease in stiffness with an increase in the traffic load applications. In this study, this was expressed as the reduction in the relative modulus that was insensitive to temperature and moisture conditions and thus, only reflected the effect of traffic. Besides, backcalculated moduli for the asphalt and subgrade layers matched well in most cases. The bigger differences were encountered with the moduli of the base/subbase layer.

(ii) To date analysis of in-depth deflection bowls has been limited to ad-hoc investigations where manual and time-consuming backcalculation procedures have been applied. The Method of Equivalent Thickness (MET), described and used in this paper, is a first attempt to develop a standard methodology for analyzing the information contained in the in-depth deflection bowls. The preliminary results obtained indicate the potential of the method to be used as a routine analysis procedure to evaluate MDD data (especially under heavier loads). This, in turn, can be used to analyze the extensive data available in the HVS database that presently contains information of approximately 400 pavement test sections under many different conditions

The moduli backcalculated based on MDD data using the MET approach are well within expected and reasonable ranges and compare well with moduli backcalculated using FWD data and more traditional approaches.

(iii) Laboratory testing is the only procedure that can evaluate the pavement materials before the pavement sections are built. Thus, it is extremely valuable during the design stage as well as for the initial sensitivity analysis of materials to design variables. In this regard, laboratory testing is irreplaceable.

Furthermore, by evaluating material properties under a range of conditions (temperature, frequency, moisture, and density), laboratory determined properties enable a more objective comparison of results obtained with different methods. In particular for the analysis reported in this paper, the effect of loading time on the moduli of the asphaltic materials can be estimated so that meaningful comparisons between FWD and MDD backcalculated moduli can be established. Theoretically, the information derived from the use of any technology can be converted into a standard testing conditions to permit meaningful comparisons.

Laboratory testing also enables the comparison of the material properties according to the design specifications with the material properties of the final product as delivered to the site and also with the properties of the materials as placed in situ. Another advantage of laboratory testing is that the mathematical calculation of material moduli does not require the assumption of a value for the Poisson=s ratio. This eliminates one source of uncertainty as compared to FWD and MDD backcalculated values.

The laboratory testing that was carried out as part of this study was of non-destructive nature. Thus, a limited number of samples enabled a wide range of information to be obtained.

(iv) The concept of "pseudo-healing" was introduced in this paper to describe the temporary recovery of the stiffness of the asphalt concrete layer after the pavement section is closed to traffic. During the HVS testing, it has been observed that this is only a temporary phenomenon, and that the stiffness of the asphalt concrete drops rapidly soon after the section is open to traffic again.

(v) The final comment deals with the stochastic nature of the properties being backcalculated. In the case of pavement materials, the calculated moduli are an integrated measure of the response of various components of the pavement structure. Because such a deterministic value does not exist, moduli should be treated as a random variable with an inherent distribution associated with it. Tighter specifications and quality control will only ensure a reduction in the variability but never a deterministic property. HVS testing showed that even within a small, well-controlled experimental section, natural variability could be significant. As engineers, an attempt should be made to estimate the distribution function of the properties and the sensitivity of the distribution to the above-mentioned variables.

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6. FIGURES

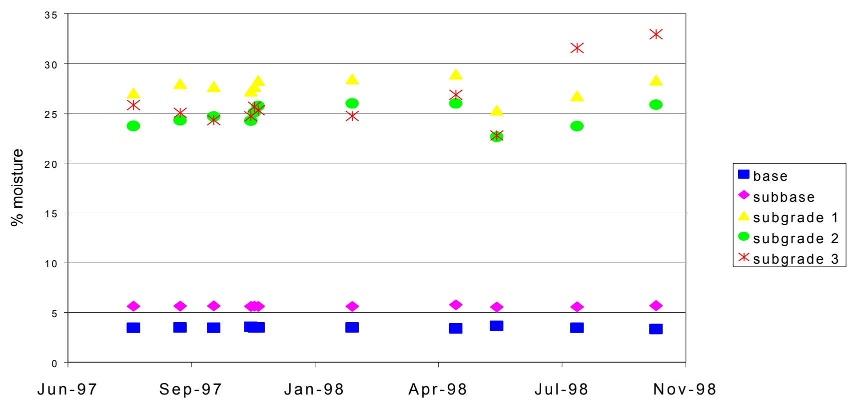

Figure 1: Subsurface water content measured between Sections 500RF and 502CT.

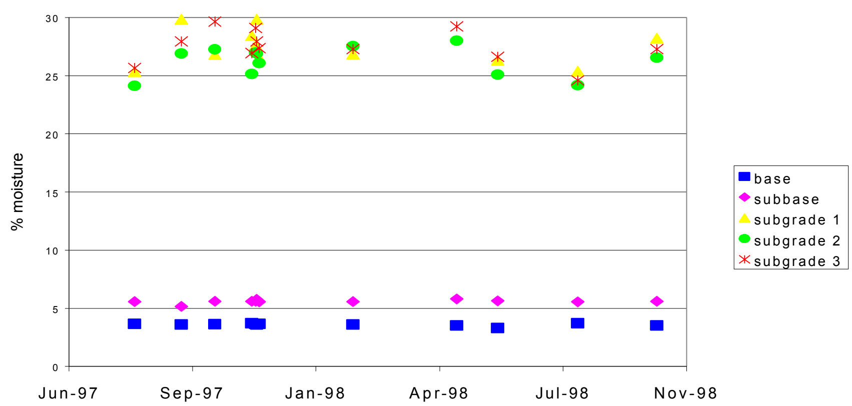

Figure 2: Subsurface water content measured between Sections 501RF and 503RF.

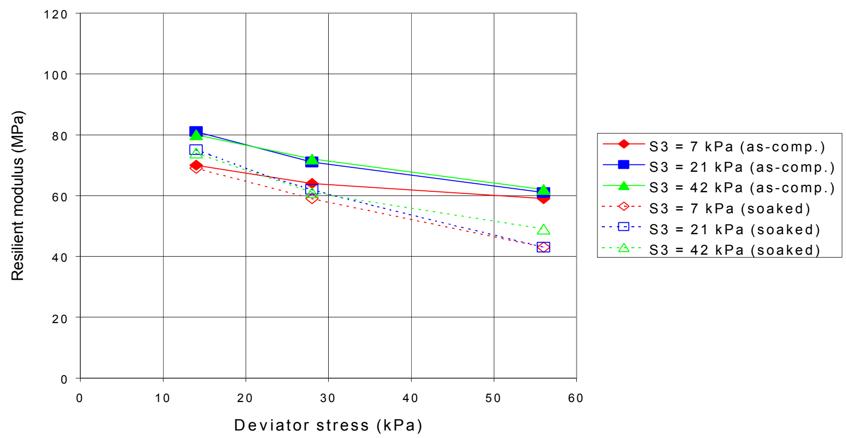

Figure 3: Resilient modulus of Specimen 1 (S3 = confining stress).

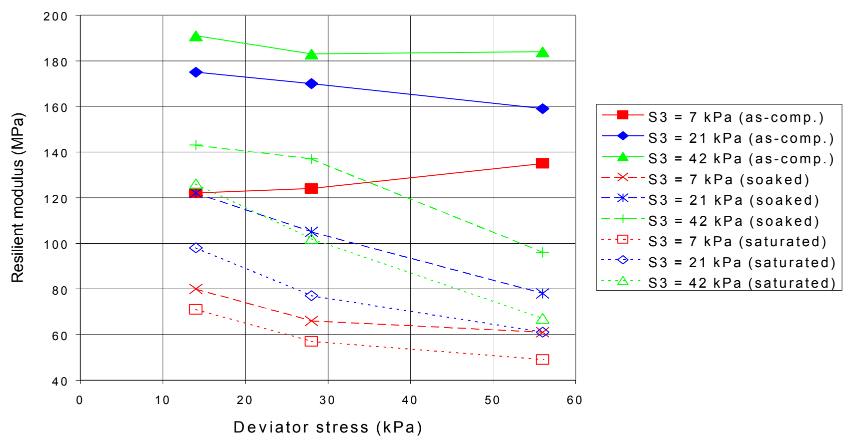

Figure 4: Resilient modulus of Specimen 2 (S3 = confining stress).

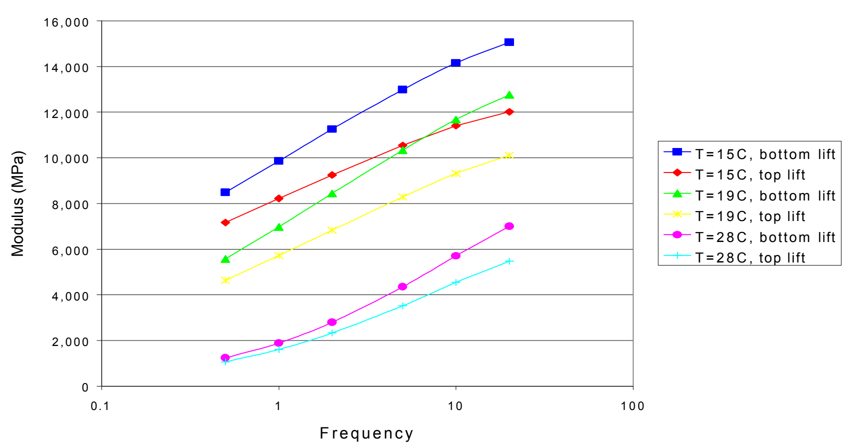

Figure 5: Average asphalt moduli determined in the laboratory (0.5 to 20 Hz)

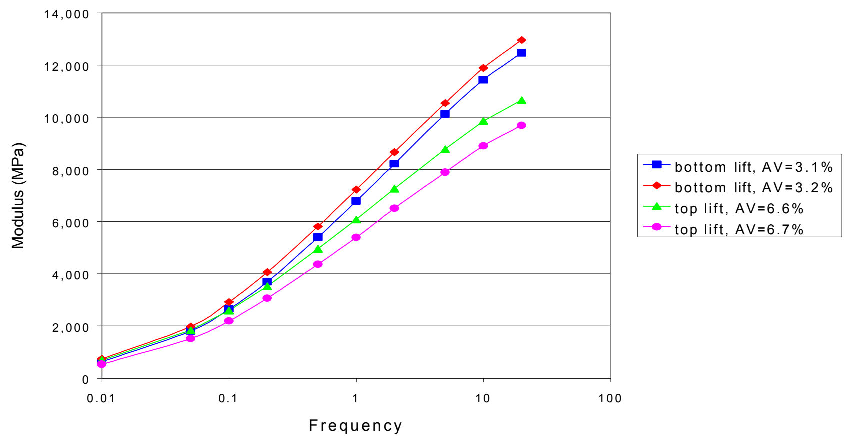

Figure 6: Frequency sweeps at lower frequencies and lower temperatures.

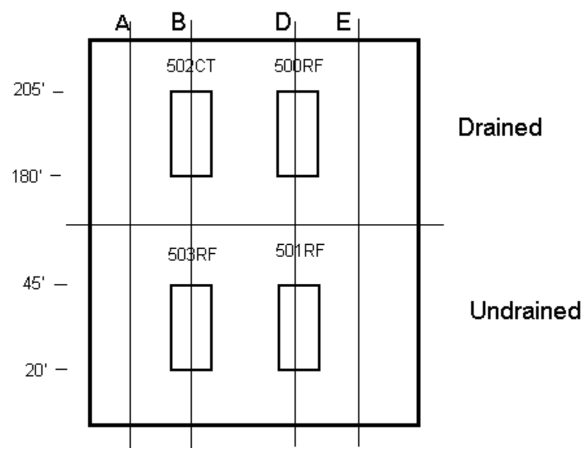

Figure 7: Sketch of the layout of the test sections and FWD test lines.

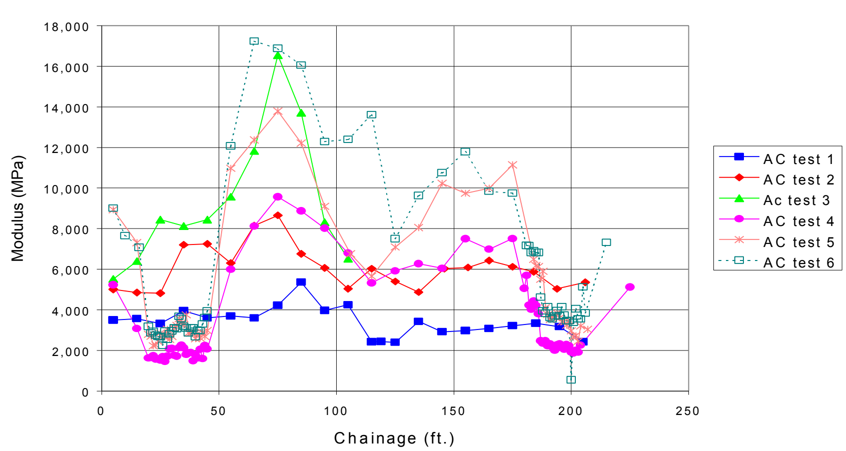

Figure 8: Asphalt concrete modulus at line B <u>before overlay</u>.

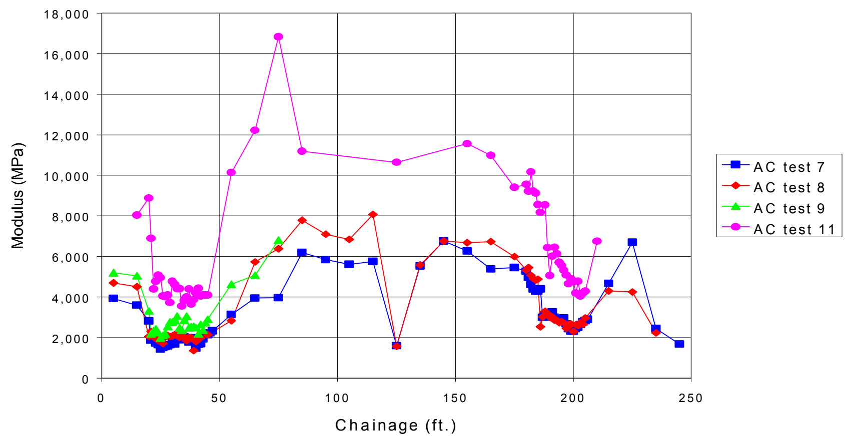

Figure 9: Asphalt concrete modulus al line B <u>after overlay</u>.

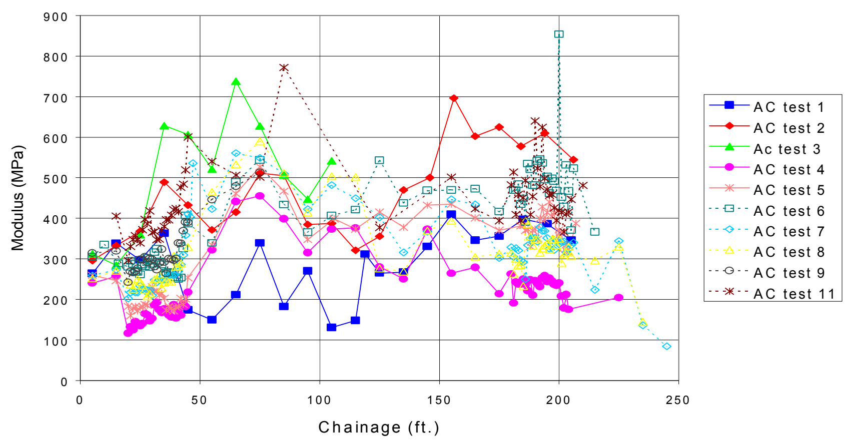

Figure 10: Modulus of granular materials (base/subbase) at line B.

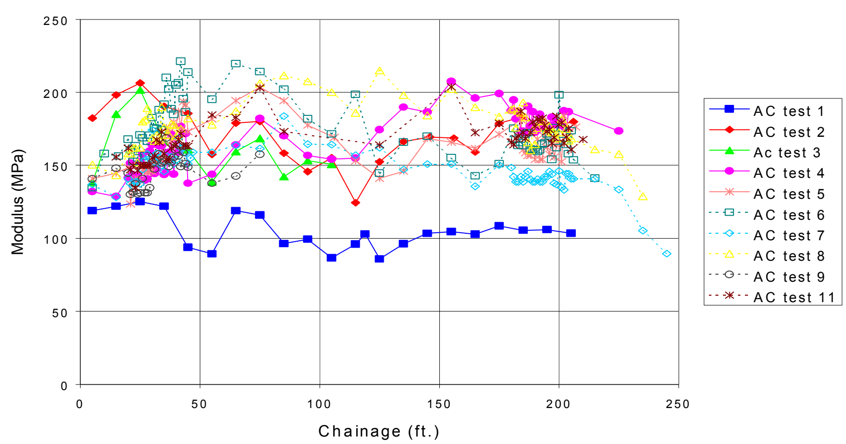

Figure 11: Modulus of subgrade layers at line B.

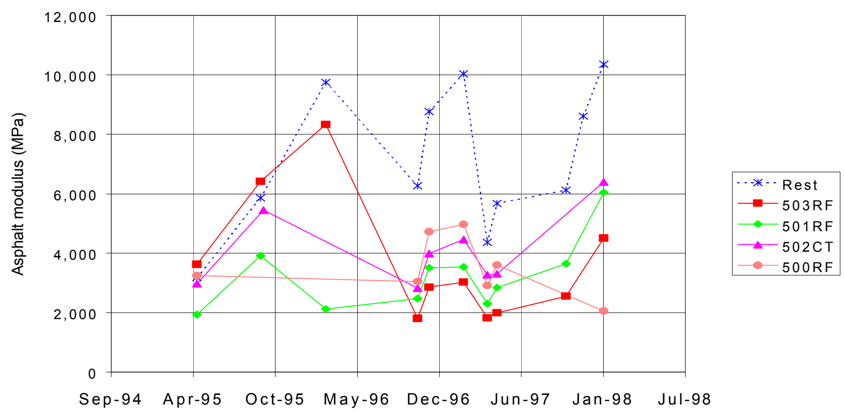

Figure 12: Summary of backcalculated asphalt moduli.

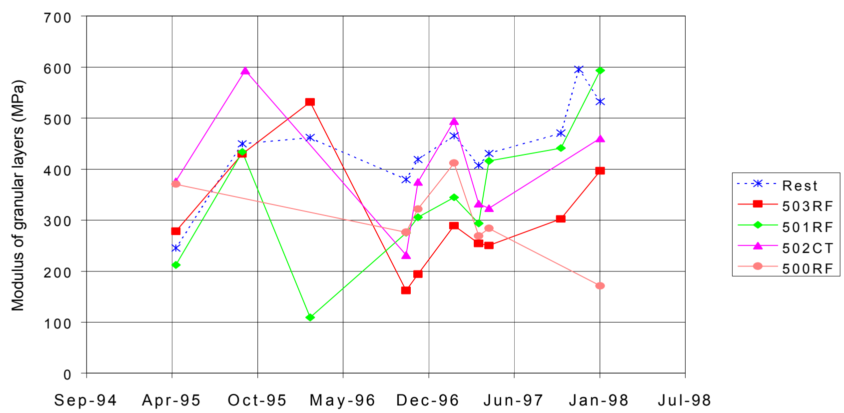

Figure 13: Summary of backcalculated base/subbase moduli.

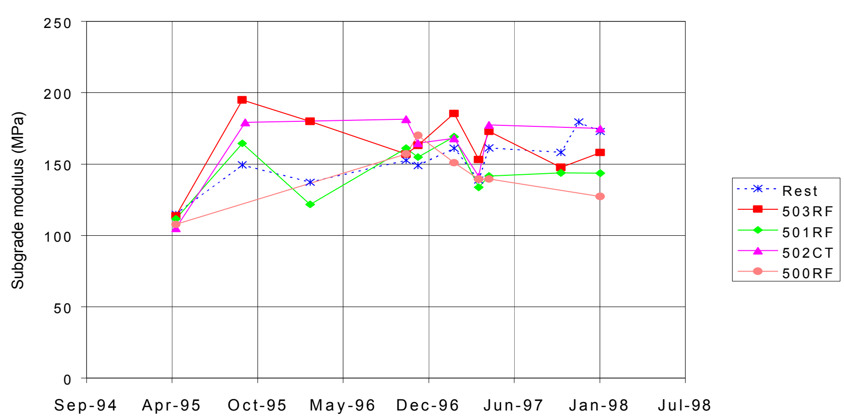

Figure 14: Summary of backcalculated subgrade moduli.

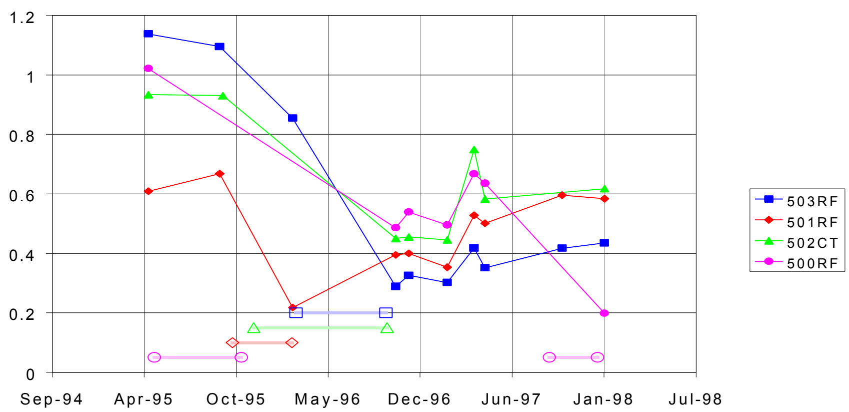

Figure 15: Relative modulus of the asphalt concrete layer.

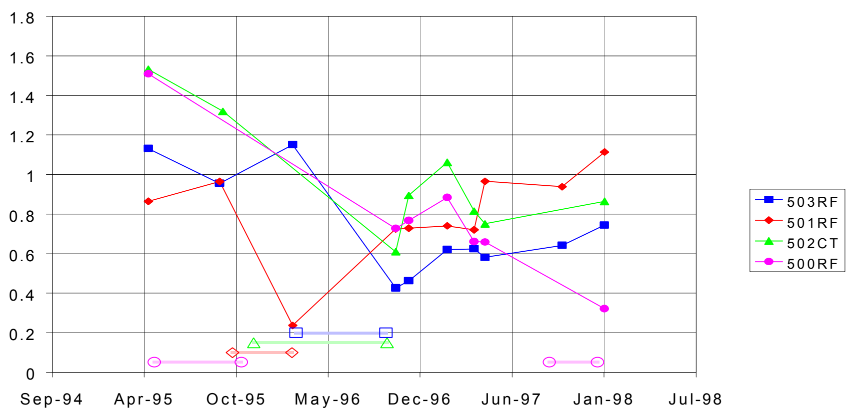

Figure 16: Relative modulus of the base/subbase course.

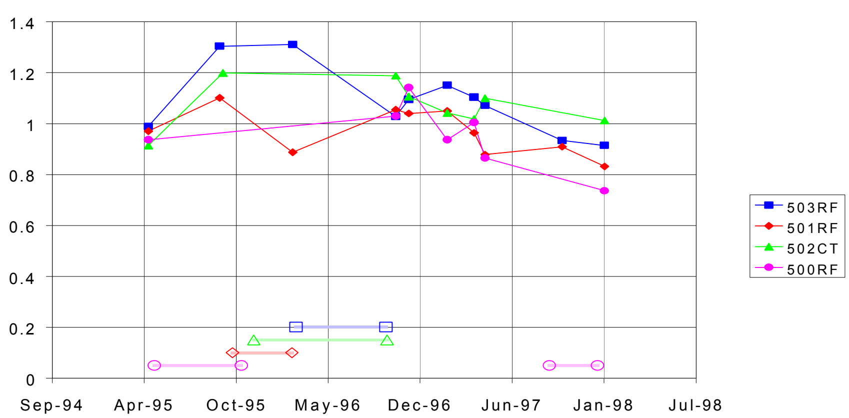

Figure 17: Relative modulus of the subgrade layer.

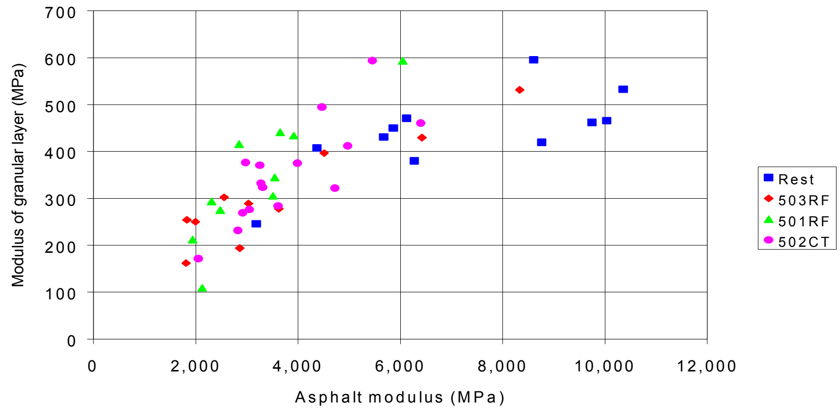

Figure 18: Correlation between AC modulus and the modulus of the base/subbase layer.

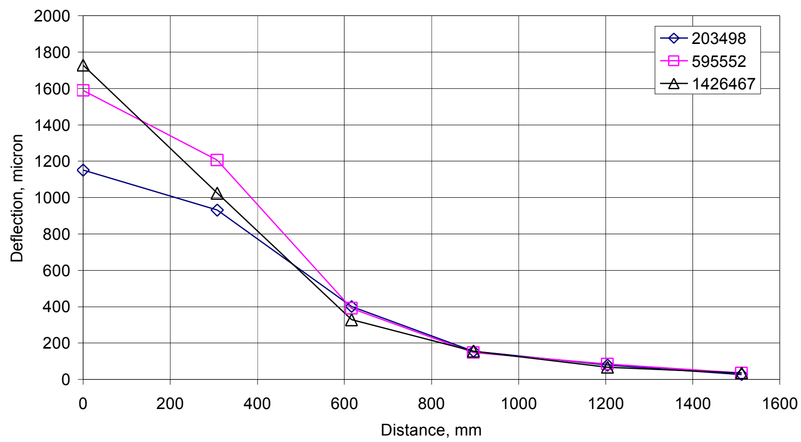

Figure 19: Surface deflection measured with the MDD system.

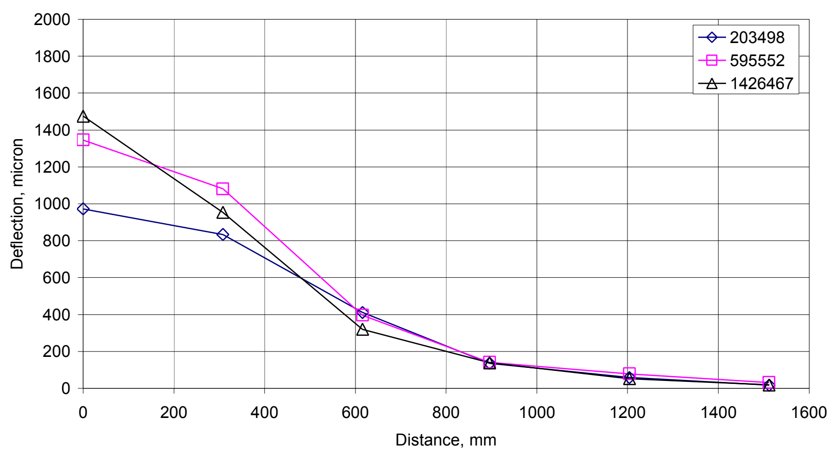

Figure 20: Base course deflection measured with the MDD system.

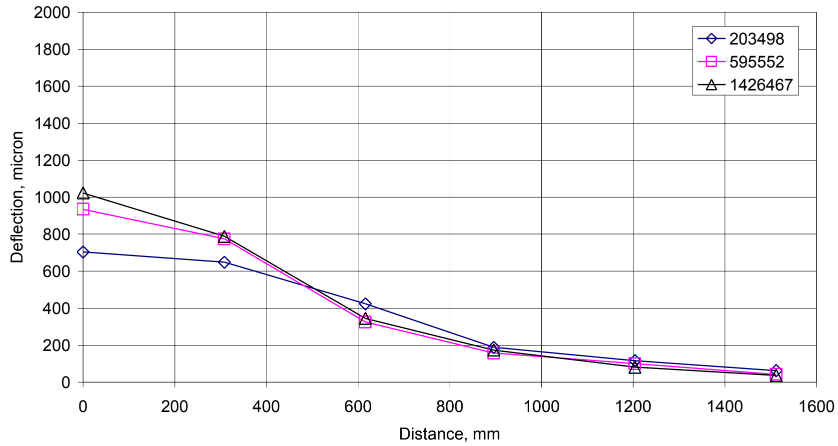

Figure 21: Subgrade deflection measured with the MDD system.

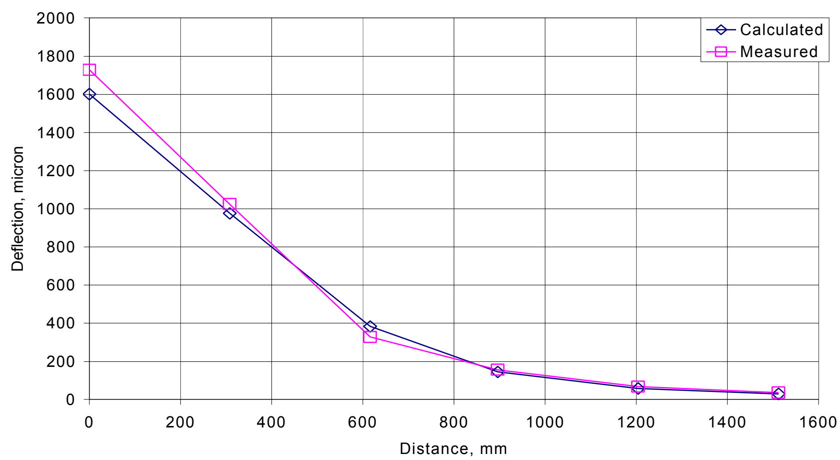

Figure 22: Measured and calculated deflections at surface level.

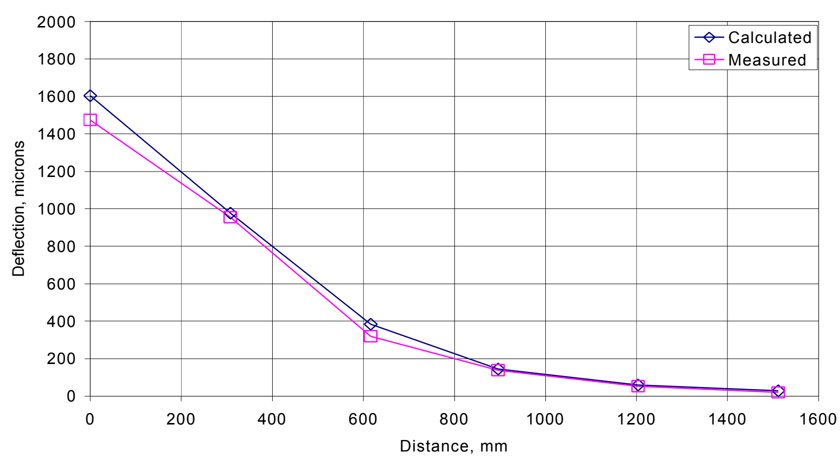

Figure 23: Measured and calculated deflections at base level.

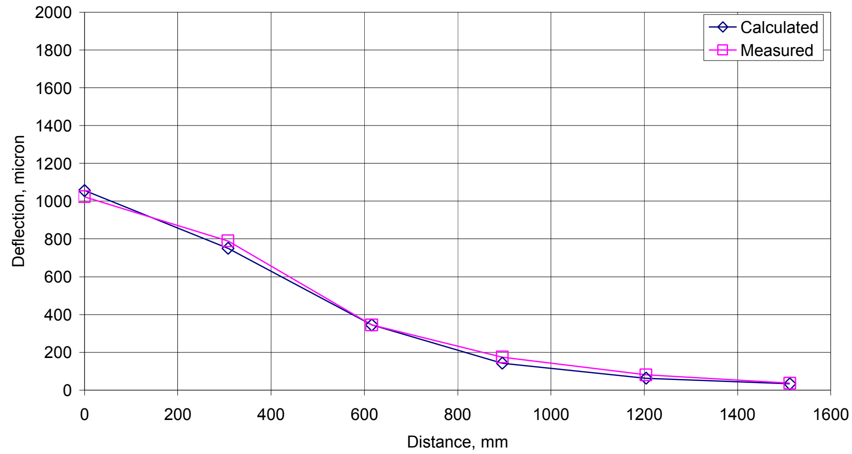

Figure 24: Measured and calculated deflections at subgrade level.

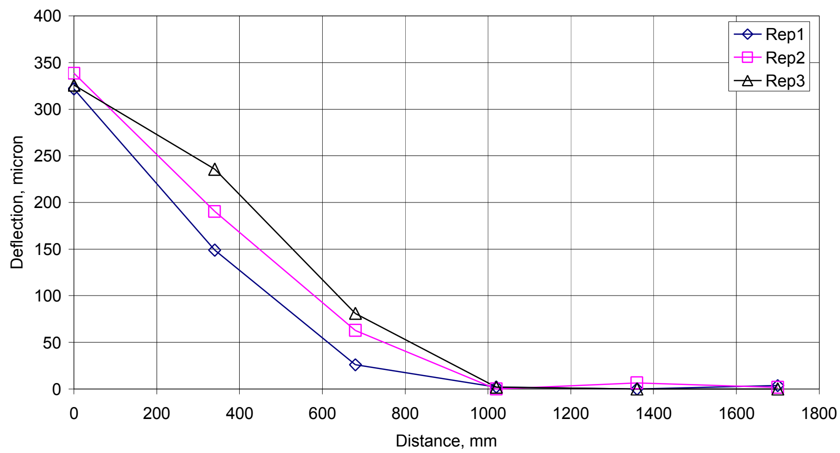

Figure 25: Repeatability of MDD deflection measurements at surface levels.

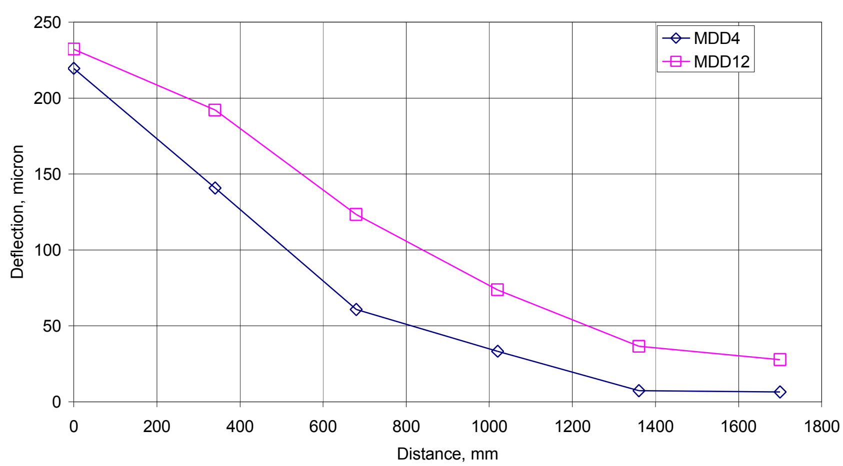

Figure 26: Variability of MDD deflections within the test section.