Performance of Dowel Bar Retrofitted Concrete Pavement Under HVS Loading

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ABSTRACT
The California Department of Transportation uses Dowel Bar Retrofit (DBR) as a rehabilitation strategy for concrete pavements. Two test sections were dowel bar retrofitted and a third section was designated for control on US 101 near Ukiah, California. All three sections were subjected to accelerated pavement testing using the Heavy Vehicle Simulator (HVS). The HVS results demonstrated a large improvement in load transfer efficiency (LTE), and decreases in maximum vertical deflections and vertical deflection differences from DBR. LTE was not damaged by trafficking on the DBR sections and was less sensitive to temperature changes than the control section. Falling Weight Deflectometer testing showed damage to interlock at the joint on the control section and no damage on the DBR sections. Joint/crack deflections and deflection differences increased with trafficking. A total equivalent loading of approximately 11,000,000 equivalent single axle loads was applied to each of the DBR sections without failure occurring.

INTRODUCTION

The California Department of Transportation (Caltrans) operates a state highway pavement network of over 78,000 lane-kilometers of which 32 percent are rigid pavements.(*1*) It has been estimated that approximately 90 percent of California rigid pavements were constructed between 1959 and 1974.(*2*)

In 1999, nearly 32 percent of the network required corrective maintenance or rehabilitation; rigid pavements comprised 24 percent of the lane-kilometers requiring immediate attention (*1*). Caltrans places the highest priority for maintenance and rehabilitation spending on pavements with poor ride quality and high average daily traffic. Faulting is the primary cause of roughness on Caltrans' concrete pavements.(*3*)

Extensive evaluation of pavements across the country indicates that Load Transfer Efficiency (LTE) is one of the primary variables controlling the rate of faulting development.(*4*)

Load transfer and energy distribution in rigid pavements can come from three sources:

1. Aggregate interlock between the slabs across the joint;
2. Base material, although this is the weakest means of obtaining load transfer; and
3. Load transfer devices, typically dowels.

Caltrans rigid pavements are plain jointed concrete, except for several experimental sections. They have no dowels at the transverse joints except for a few pavement sections built since 1999. Caltrans did not use dowels in the 1950s and 1960s when most California rigid pavements were built because of the inability of construction equipment to obtain correct dowel alignment at that time. Further, typical practice in the early 1950s was to use 19- to 25-mm diameter dowels, which Caltrans researchers at that time found to be in effective in reducing faulting even when placed correctly (5).

Historically, Caltrans has relied on improving the non-erodability of base materials and aggregate interlock to control faulting. Nearly all of the rigid pavements currently needing rehabilitation were built with cement treated bases (CTB). Designs from the mid-1960s consist of 200 mm PCC (225 mm for higher traffic levels); 100 mm CTB (150 mm for higher traffic levels); 150 mm aggregate subbase; joint spacing of 3.7, 4.0, 5.5, 5.8 m; skewed joints; and an asphalt concrete shoulder.

Caltrans experience with CTB bases is that cemented bases alone have not prevented rapid development of faulting. A study performed by McLeod and Monismith in 1979 indicated that faulting generally occurred within 1 to 4 million equivalent single axle loadings (ESAL) after construction. (6)

Mechanistic-empirical models have been developed to predict faulting development based on the presence, size, and distribution of dowels, traffic, climate, and base type. Models for faulting development (*4*, *7*) applied to 1960s Caltrans undowelled designs indicate that faults as small as 3 mm can cause roughness that triggers maintenance or rehabilitation, and that the time to develop this level of faulting and roughness can take as little as one year to develop for traffic levels on the most heavily trafficked routes in the state.(*8*)

Various states and researchers have looked at DBR as a means of restoring load transfer across joints, and therefore extending the life of concrete pavements, and reducing the time between maintenance and rehabilitation treatments to reduce roughness. These studies have included the original testing in Puerto Rico, field evaluation of widespread deployment (*9*) and scaled down accelerated loading tests (*10*). The results of the WSDOT study indicated good performance when good construction is obtained, but no sections had failed so that estimation of design life for life cycle cost analysis is difficult. The Minnesota study showed the benefits of DBR based on the scaled down accelerated testing.

Caltrans has used dowel bar retrofit (DBR) as an experimental rehabilitation strategy for several years and has recently made it a standard strategy. Primary concerns with DBR include:

- Relative performance of design options, such as the number of dowels per wheelpath, dowel type, and dowel durability;
- Relative performance in different climate regions;
- Relative performance for different levels of existing distress and age of candidate pavements;
- Overall performance and cost, to permit life cycle cost analysis and comparison with other rehabilitation strategies; and
- Construction quality control and assurance and its effect on performance.

Objectives

This paper presents work that is part of a larger research goal to establish whether, compared to other strategies, dowel bar retrofit (DBR) provides adequate performance relative to its cost for the rehabilitation of rigid pavements. The test plan for this research includes an element for Field Accelerated Pavement Testing with the Heavy Vehicle Simulator (HVS), the subject of this paper.

The accelerated pavement testing element includes a factorial of test site and climate condition variables: climate, pavement age, and existing pavement conditions. The test sections at Ukiah fit the factorial cell for older pavements in a wet environment.

TEST SECTION LAYOUT AND PAVEMENT CONDITION

The three tested sections had the following features:

- 555FD is a control section that was not retrofitted with dowel bars. It was tested across two sawn joints and uncracked slabs.
- 554FD has two sawn joints and a longitudinal crack and was dowel bar retrofitted.
- 553FD has two transverse cracks that were dowel bar retrofitted.

The test sections are in the northbound truck lane of a four-lane freeway in a cut section with steep side slopes and no edge drains. These sections are located on US 101 near Ukiah, California; a four-lane freeway built in 1967. Average annual rainfall near the site is 1,011 mm. The total cumulative loading since 1967 is between 3.4 and 5.1 million ESALs, estimated from recent data from a nearby Weigh-In-Motion station installed in the 1990s. This estimate of previous traffic may be low because truck traffic may have diminished with reductions in logging activity in the decade prior to WIM installation.

Two joints or cracks are included in each 8-m long HVS test section, resulting in loading across all of the center slab and the ends of the two adjacent slabs. All slabs are 3.7 m wide with joints skewed at a 1:6 ratio, and asphalt concrete shoulders.

Slab dimensions for the HVS test sections are shown in Figure 1. Section 553FD was originally one 11.4-m long slab caused by the transverse joint not being sawed. The center slab of this section is formed by two transverse cracks in the original slab.

Live traffic test sections (553LT, 554LT, 555LT) were selected on nearby slabs based on replication of dimensions and conditions in the HVS test sections, including DBR, and are currently being monitored.

Fault heights measured on some of the slabs ranged from 6 to 12 mm. The transverse cracks also had faulting, ranging between 6 and 9 mm. There was some minor spalling at some of the joints and transverse cracks, and none on the longitudinal cracks.

100-mm diameter cores, including PCC and CTB, were taken on the center slabs of each test section and tested for thickness and unconfined compressive strength (Table 1). Stiffness was measured by laboratory compressive loading on one core. The CTB appeared to be in good condition.

DBR DESIGN, MATERIALS AND CONSTRUCTION

The same dowel bar retrofit design and specifications were used on all of the retrofitted sections. The design required four dowels per wheelpath, following current Caltrans and previous WSDOT design practice. The dowels were epoxy coated steel, 32 mm diameter and 457 mm long. The back-fill material was prepared from aggregate, sacks of patch cement, and water brought to the site by the contractor.

Beam and cylinder specimens of the mixed back-fill material were made from material sampled at the site (ASTM C 31). The specimens were transported to the laboratory and cured at 97 percent humidity and 20°C. The beams and cylinders were tested (Table 1) in replicate pairs for 3rd point loading modulus of rupture (ASTM C 78) and compressive strength (ASTM C 39).

A diamond grinder removed the faults on all of the joints to be tested.

HVS AND FWD TEST PLANS

The HVS testing was performed in the following order:

- Section 555FD (no retrofit), from 9 through 23 February, 2001
- Section 554FD (retrofit), from 26 February through 29 March, 2001
- Section 553FD (retrofit), from 5 April through 4 May, 2001

All HVS testing was performed in the bi-directional mode, with the dual wheel load passing over the center of the dowel bar assembly in the outside wheelpath, centered 0.76 m from the longitudinal joint between the concrete slab and the asphalt concrete shoulder. The dual wheel was mounted with Goodyear G159A radial tires inflated to 730 kPa. The dual wheel centerlines are 0.36 m apart on the axle. The HVS wheel speed is approximately 7 km/hr.

HVS Instrumentation and FWD Test Program

Joint deflection measurement devices (JDMD) are Linear Variable Displacement Transducers (LVDT) fastened vertically to the sides of the concrete slabs. The JDMDs are numbered as follows:

- JDMDs 1 and 2 on each side of the first joint or crack, measuring vertical deformation,
- JDMDs 4 and 5 on each side of the second joint or crack, measuring vertical deformation,
- JDMD 3 mid-slab along the longitudinal joint between slab and shoulder between the joints or joint and transverse crack, measuring vertical deformation, and
- JDMD 6 across the second joint or transverse crack laterally halfway across the slab, measuring horizontal joint opening and closing.

Falling Weight Deflectometer (FWD) data included in this paper come from four measurement dates:

- March, 2000, one year before dowel bar retrofitting (DBR) of the test sections,
- January, 2001, just before DBR,
- February, 2001, just after DBR and just before HVS testing began, and
- May, 2001 after HVS testing was completed.

Deflections were measured twice in a 24-hour period at each of the times listed above:

- in the early hours of the morning when the average slab temperature is lowest, and the temperature gradient results in the corners and joints of the slab pushing down on the base (not performed in March, 2000), and
- in the late afternoon when the average slab temperature is highest, and the temperature gradient results in the corners and joints of the slab lifting up from the base (actual lift off is difficult determine).

FWD deflections were measured on the HVS test sections, on the live traffic test sections (553LT, 554LT, 555LT) and on the additional 30 slabs within the lane closure that had no DBR or other treatment applied to them (referred to as “Outside Slabs”).

Measurement of Load Transfer Efficiency, Maximum Deflection and Maximum Deflection Difference

Important differences between the HVS loading and highway loading are:

- Bi-directional loading (HVS) does not cause faulting; while uni-directional loading (highways) causes faulting by movement of material from under the downstream (departure) slab to under the upstream (approach) slab. Since faulting is highly correlated with load transfer efficiency (*4*), damage to LTE is used as a surrogate in this study. Deflections and deflection difference provide an indication of energy applied to the underlying layers, which is required to move material and cause faulting although models have not yet been specified for faulting as a function of energy applied.
- The HVS maintains a fairly constant load at low speed, while highway traffic applies loading at high speeds which can cause increasing dynamic loads once faulting begins to develop, and potentially increase faulting and the rate of damage to LTE.

Load Transfer Efficiency (LTE) was measured using the JDMDs during HVS testing. A full deflection response is recorded for each JDMD as the wheel moves from one slab to another across the joint (Figure 2). Under the rolling wheel, there is an "approach LTE" which is the load transfer as the load rolls up to the joint on the upstream slab, and a "departure LTE" which is the load transfer when the load passes onto the downstream slab.

A 'pair' definition of LTE is used in the UC/Caltrans HVS database and in this paper, because it provides more stable results, and therefore makes it easier to track changes in LTE with damage. The pair definition uses the deflection of both instruments when the wheel is at the same location, and so compares the deflections of the two slabs under the same loading condition. The deflections are measured by the JDMDs attached to the outside edge of the slab, while the load is over the dowel bars in the wheelpath.

$$LTE_{pair}^{approach} = \frac{y_{rel}^{2}}{y_{peak}^{1}} \quad LTE_{pair}^{depart} = \frac{y_{rel}^{1}}{y_{peak}^{2}}$$

Negative LTE is also possible under rainy conditions when there is no aggregate interlock and the unloaded slab is pushed upwards by hydraulic pressure caused by the downward deflection of the loaded slab.

Load transfer efficiency is measured somewhat differently with the Falling Weight Deflectometer, with the load placed on the upstream side of the joint, and deflections measured simultaneously on the upstream and downstream slabs. FWD measurements were taken at the corner of the slabs and at the center of the transverse joint (laterally halfway across the slab). The joint was placed between the deflection sensors that were 200 and 300 mm from the center of the load plate for FWD measurements. FWD deflections are measured at the same distance from the longitudinal edge of the slab edge as the load. LTE is calculated from FWD measurements using Westergaard's definition (*11*).

Differences in loading time between the HVS and the FWD should not significantly change LTE and deflection measurements, assuming that the response of the concrete slab, steel dowels and underlying layers is primarily elastic. The definitions and values of LTE for the HVS and FWD are somewhat different, as they must be since one comes from a rolling wheel and the other from a point load, however they should be similar and show similar trends with climate and load damage.

Maximum deflection is the maximum vertical downward movement of the loaded slab under both the. *Maximum deflection difference* is the maximum difference in deflection between the loaded and unloaded slabs when the wheel is on the loaded slab, provides an indication of the shear energy applied to the base. Under HVS loading this would be the difference $y^{1}_{peak} - y^{2}_{rel}$ and $y^{2}_{peak} - y^{1}_{rel}$ (Figure 2). Maximum deflection difference was measured when the wheel is moving in one direction, from JDMD 1 towards JDMD 2, and from JDMD 4 towards JDMD 5.

SUMMARY OF HVS AND FWD RESULTS

Observations from HVS Test Sections

Rainfall and air temperature conditions and average mid-depth (100 mm) slab temperature and top to bottom temperature difference during the HVS testing and data collection are shown in Figure 3.

555FD – Control Section

HVS loading on Section 555FD consisted of 80,282 repetitions with a 40-kN dual wheel load, followed by 89,397 repetitions with a 90-kN dual wheel load, for a total of 169,679 repetitions. Using the Caltrans load equivalence exponent of 4.2, 2,775,000 equivalent single axle loads (ESAL) were applied by the HVS. Some problems arose with the data acquisition system during testing on Section 555FD, and there are gaps in the recorded data between 0 and 24,752 load repetitions, and between 72,809 and 136,510 repetitions.

Water was observed flowing laterally in the joints at the beginning of loading and for some time afterward. Water and pumped material from the underlying layers was observed flowing out of the joints under loading during and just after rain events, at times squirting out of the joint up to 100 mm above the surface of the slab. No new cracking was observed on the slabs.

Typical deflection histories for various HVS load repetitions are shown in Figure 4a for JDMDs 1 and 2. The gradually increasing joint deflections caused by damage under the 40- and 90-kN loading are apparent (last two deflection measurements), as is the increase in deflections from the 90-kN. JDMD 2 in the figure indicates that the slab rocked under 90-kN loading, meaning that when the wheel was on one end of the slab the other end of the slab tilted up. The rocking resulted in an upward tilt of about 0.25 mm across the slab length of approximately 4.75 m.

554FD – DBR on Transverse Joints

HVS loading on Section 554FD consisted of 102,578 repetitions with a 40-kN dual wheel load, followed by 370,000 repetitions with a 90-kN dual wheel load, for a total of 472,578 repetitions and 11,255,000 ESALs.

No water was observed flowing across the test section or in the joints, except during the major rain event near the beginning of the test. No pumping, or new cracking or other distress was observed during the test.

Typical deflection histories for HVS loading are shown in Figure 4b for JDMDs 1 and 2. There is a trend of gradually increasing joint deflections caused by damage under the 40- and 90-kN loading apparent in Figure 4b (last two deflection measurements shown). The increase in deflection under the 90-kN load can also be seen. However, the deflections are much lower than those of the undoweled Section 555FD, less than 0.5 mm under the 90-kN loading even after nearly 500,000 repetitions. The figure also shows that there was no slab rocking on the dowel bar retrofitted slab.

553FD – DBR on Transverse Cracks

HVS loading on Section 553FD consisted of 73,984 repetitions with a 40-kN dual wheel load, followed by 359,990 repetitions with a 90-kN dual wheel load, for a total of 433,986 repetitions and 10,925,000 million ESALs.

No water was observed on the test section or in the joints during the test except during the rainfall events. No pumping or new cracking or other distress was observed during the test.

The JDMD deflection histories for various repetitions of HVS loading for Section 553FD are nearly identical in shape and magnitude to those shown in Figure 4b for Section 554FD. The deflections on both sections are less than 0.5 mm under the 90-kN loading even after nearly 500,000 repetitions. The JDMD deflection histories for Section 553FD indicate that there was very minor slab rocking.

HVS Test Results

Static Horizontal Joint Opening

Unloaded horizontal joint position was measured using JDMD 6 mounted on the slab surface on all three HVS sections. A trend was observed between joint position and mid-depth slab temperature, although there was scatter due to temperature curling and unrecorded interaction with other slabs. The trend was less clear for Section 555FD because of the limited range of temperatures. In addition, the surface location of JDMD 6 makes the measurements also a function of curl, and 555FD has greater curling deformations due to lack of dowel restraint, which resulted in some correlation of measured joint opening and temperature gradient. The trend lines were extrapolated to 30° C, the temperature above which FWD testing indicated 100 percent LTE on nearly all of the undowelled joints. With the JDMD 6 position at 30° C considered as the joint "closed" position, static horizontal joint openings were estimated for comparison with LTE and deflections.

Load Transfer Efficiency

Load transfer efficiency (LTE) for the three HVS test sections is shown versus load repetitions in Figure 5. The LTE results can be summarized as follows:

- The undoweled 555FD has very low LTE, less than 40 percent, even after only 25,000 40-kN load repetitions;

- The doweled section have high LTE, typically greater than 75 percent throughout the HVS testing;
- Both the doweled and undoweled sections show greater LTE under the 90-kN loading compared to the 40-kN loading. This is likely due to the larger deflections under greater load driving the slab into the base, resulting in greater load transfer through the base than occurred under the lighter load.

LTE would also be expected to increase with increasing temperatures, which cause the joints to close as the slabs expand. LTE was evaluated against measured horizontal joint opening, with the scale for joint opening based on the assumption that the joints have maximum closure at 30 C. The results under HVS loading showed no clear trend of increasing damage with load repetitions. However, these results were confounded by the two loads used and the increasing temperatures with time on each section. It was clear that LTE should have been measured at 40-kN for the entire test to provide a consistent variable across temperature and repetitions.

Joint Maximum Deflections

Joint maximum deflections (JDMDs 1, 2, 4 and 5) are compared for the three HVS test sections in Figure 6. The results show the much larger joint maximum deflections on the undoweled sections (555FD) compared to those on the dowel bar retrofitted sections (553FD, 554FD). The deflections are in the range of 2 to 10 times greater on the undoweled section than on the doweled sections under identical loads and number of load repetitions. The smaller and less variable deflections on the doweled sections can be attributed to greater load transfer to the next slab. Another likely contributor is the restraint of vertical temperature curling deformations, which results in more support to the slab from the base, and less incidence of slab lift-off under positive temperature gradients.

Deflections on both the doweled and undoweled sections increased as expected under the 90-kN loading compared to the 40-kN loading, especially on the undoweled section, as can be seen in Figure 7 plotted against estimated horizontal joint opening.

Mid-Slab Edge Maximum Deflections

All of the sections had about the same length slab at the middle of the test (Figure 1). The two dowel retrofitted sections (554FD, 553FD) had smaller mid-slab edge deflections (JDMD 3) than the undoweled section under 40-kN loading. This indicates that the dowels are providing some support to the middle of the slab at the edge, probably by resisting bending about the longitudinal axis of the short slabs through load transfer at the joint. The HVS exaggerates this effect compared to truck loading because it only applies a half axle.

Under the 90-kN loading, the mid-slab edge deflections increased rapidly on the section with a longitudinal crack (554FD), but not on the other two sections. The mid-slab edge deflections are nearly identical for the two sections without a longitudinal crack (555FD and 553FD), although only 553FD is dowel bar retrofitted. The mid-slab edge deflections on Section 554FD are about 2 to 3 times greater than those on Sections 553FD and 555FD after about 100,000 repetitions of the 90-kN load. This indicates that the longitudinal crack on DBR sections may deteriorate and result in greater deflections and therefore greater erosion along the longitudinal joints, even though the DBR remedies joint deflection and LTE problems. The eventual impact of the larger mid-slab edge deflections on overall performance is not known. It may be that this effect is secondary to long-term deterioration of the pavement, since the deflections appeared to eventually stabilize on Section 554FD.

Joint Maximum Deflection Differences

Joint maximum deflection differences for JDMDs 1 and 4 showed that the dowel bar retrofit significantly reduced the deflection differences compared to the undoweled section. This means that shearing deformations at the joint is greatly reduced in the retrofitted joints and crack, which is expected to reduce the development of faulting.

Deflection difference was seen to continue to increase with increasing load repetitions for the doweled sections, showing increasing damage, while load transfer efficiency did not. Measurement of LTE with the 40-kN load throughout the HVS testing would have probably indicated damage more effectively than measurement with the 90-kN load under which the damage to LTE was confounded by more load transfer through the base.

FWD Test Results

The FWD results included in this paper are for 44.5-kN loading. Results for heavier FWD loads are included in Reference 8. Overall, the FWD results generally confirmed those measured under the HVS wheel, although there are some differences caused by measurement under the 90-kN load during the later HVS repetitions and continuous measurement under the 44.5-kN FWD load.

Load Transfer Efficiency

The poor LTE (centerline transverse joint) on this section of highway prior to dowel bar retrofit can be seen in Figure 8. The effects of temperature can also be seen; LTE for the 30 "outside" slabs averaged 41 percent at 10 C in rainy January, 2001 and 96 percent at 33 C in May, 2001. These results indicate that LTE can be very low on Caltrans cement treated bases under cool temperatures.

The damage to LTE caused by HVS loading on the undoweled 555FD can be seen at high daytime temperatures (Figure 8a), and is particularly apparent at cool nighttime temperatures (Figure 8b). The damage is confirmed by comparison of LTE before and after HVS loading on 555FD, and comparison of trafficked 555FD with untrafficked 555LT and the average of the 30 untrafficked "outside" sections.

The LTE measurements from the FWD on the doweled sections show little damage from the HVS testing, similar to the LTE measurements from the HVS wheel (Figure 5). Figure 8 shows that at hotter temperatures DBR doesn't make much difference, since LTE is already high due to joint closing and slab corners getting greater support from the base due to temperature gradient induced curling. The measurements for February, 2001 in Figure 9 show that under cooler temperatures, the DBR provides about half of the .

Maximum Deflections

Maximum vertical deflection measurements on the loaded slab were greater at cooler temperatures. They were similar for all of the sections before DBR. After DBR, the deflections were similar for the doweled and undoweled sections at hot temperatures, but larger for the undoweled sections compared to the doweled sections. At cool temperatures, deflections were largest on the trafficked undoweled Section 555FD, indicating the damage caused by the HVS loading.

Maximum Deflection Differences

Maximum deflection differences followed similar trends to those of maximum deflections. The effect of the damage caused by the HVS on undoweled Section 555FD can be seen in Figure 9. The results do not show any damage to the doweled sections from the HVS trafficking.

CHECK ON SLAB FATIGUE

It might be expected that the 90-kN HVS trafficking would have caused fatigue cracking in the concrete slabs. The worst case for this would have been Section 555FD, which had the largest temperature gradients, and longest slab length. As a simple check, mid-slab tensile stresses were calculated for HVS test section 555FD using the 3-dimensional finite element program EverFE 2.11 for the 90-kN HVS wheel load with the following information:

- Actual slab length;
- Wheel located mid-slab at the offset from the longitudinal joint used during trafficking;
- Maximum and minimum measured temperature gradients measured during HVS testing (actually occurred on 553FD, although others were similar): 0.049 and –0.032 C/mm;
- Two-layer system with a composite k-value of 0.135 MPa/mm (491 pci) from FWD back-calculation; and
- Loads located at mid-slab and next to the transverse joint.

Calculated stresses and estimated flexural strengths were used as input for the fatigue equation developed by Darter and Barenberg (*12*) for field slabs. The maximum tensile stress of 2.66 MPa occurred with the maximum daytime gradient and the load at mid-slab. The strength was assumed to be 5.0 MPa (Table 1), which resulted in a stress strength ratio of 0.53, and no damage for the actual 90-kN repetitions on 555FD. The low fatigue damage is primarily due to low temperature gradients, short slab dimensions, relatively high concrete strength, and the location of the wheelpath away from the shoulder.

CONCLUSIONS

1. Dowel Bar Retrofit (DBR) provided very good Load Transfer Efficiency (LTE) and reduced deflections and deflection differences between slabs. On the control sections, LTE increased and deflections and deflection differences decreased as temperatures increased.

2. Approximately 11,000,000 ESALs were applied to the DBR sections with no cracking or other failure observed in the dowel bar retrofits, and no new cracking of the slab was observed on any of the HVS test sections.

3. Load Transfer Efficiency (LTE) was heavily damaged on 555FD, the control section. This damage was identified by FWD testing and primarily at cool temperatures when the joint was opened and there was typically a negative temperature gradient. No damage to LTE was observed on the DBR sections.

4. Maximum deflections and deflection differences between slabs showed the same trends as those of LTE. Some damage was seen on the DBR sections in terms of maximum deflections and deflection differences as measured with the 90-kN HVS wheel load.

5. The reason for insubstantial decrease in Load Transfer Efficiency on DBR may be insufficient load repetitions applied. It is not clear why deflections increased under loading, indicating damage to the aggregate interlock and dowel/concrete contact surface, but LTE did not decrease. These results require comparison with the live traffic sections at Ukiah (553LT, 554LT and 555LT), the Palmdale DBR HVS test sections (currently underway), and long-term Washington State DOT field data from DBR sections, for explanation.

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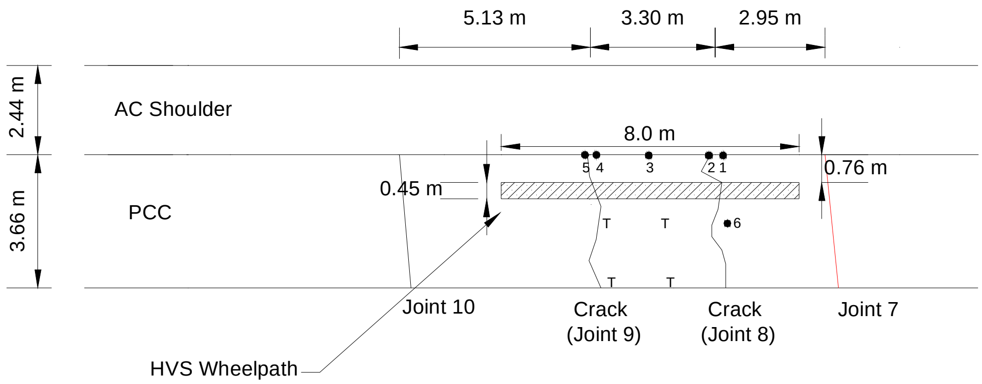
TABLE 1 PCC and CTB Thickness and Strength from Cores, and DBR Grout Strength

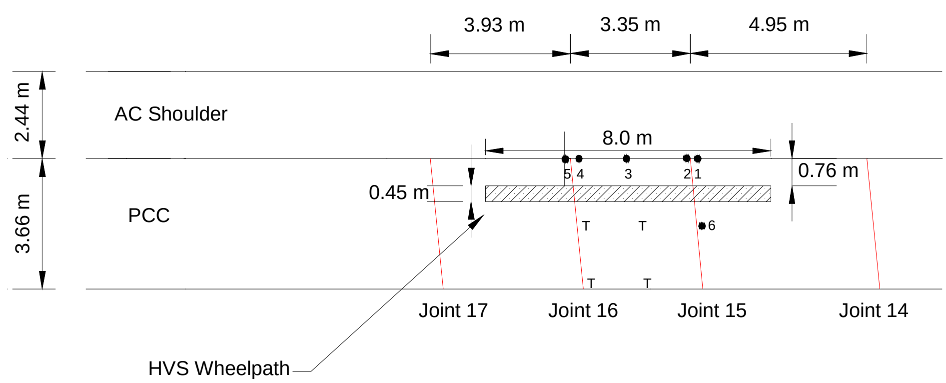
Laboratory Tests on PCC and CTB Cores					
HVS Section	**PCC thickness (mm)**	**PCC Compressive Strength, MPa (psi)**	**PCC Stiffness, MPa (psi)**	**Estimated PCC Flexural Strength, MPa (psi)***	**CTB Thickness (mm)**
553FD	205	56.7 (8,226)		5.0-6.25 (726-907)	89
554FD	198	61.7 (8,961)		5.22-6.52 (757-947)	82
555FD	212	57.6 (8,354)	13,291 (1,929,000)	5.04-6.30 (731-914)	85

Laboratory Tests on Field Prepared Specimens of DBR Grout				
Curing Time, hours	Compressive Strength, MPa (psi)	Caltrans Compressive Strength Specification, MPa (psi)	Flexural Strength (3^{rd} point), MPa (psi)	Caltrans Flexural Strength Specification, MPa (psi)
3	21.1	21	3.82	3.5
24			2.48	
37	54		6.66	

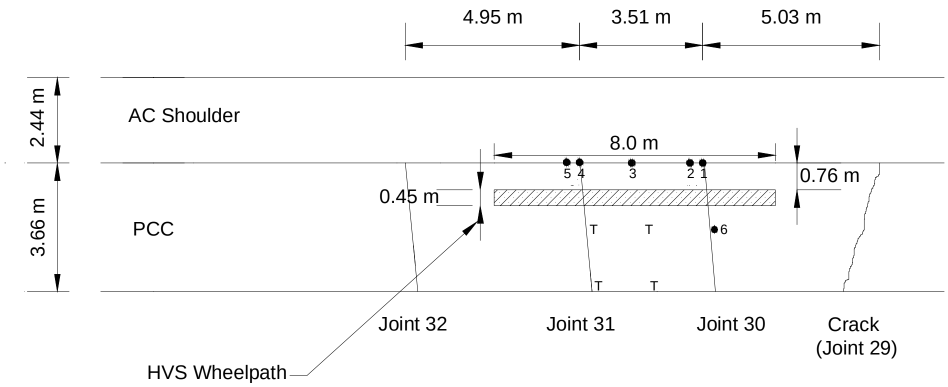
*estimated from compressive strength using ACI equation



(a) Section 553FD (dowel bar retrofit)



(b) Section 554FD (dowel bar retrofit)



(c) Section 555FD (control)

FIGURE 1 HVS test sections, with existing cracks and slab dimensions.

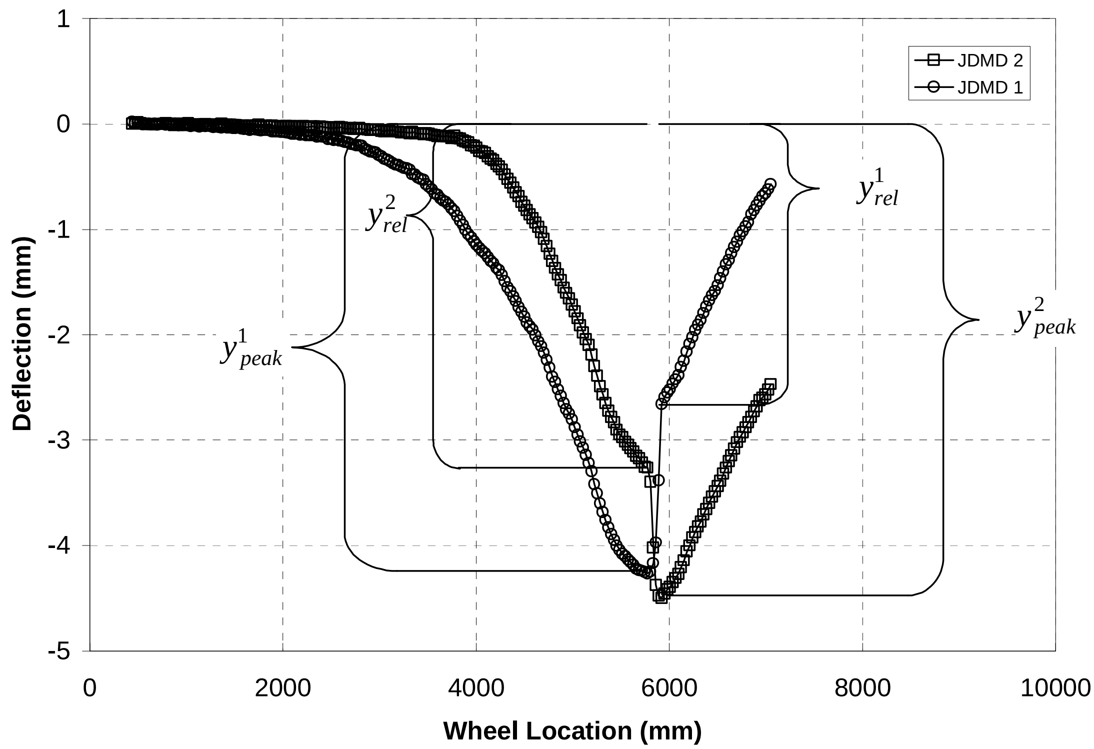


FIGURE 2 Example of LTE calculation from JDMD measurements.

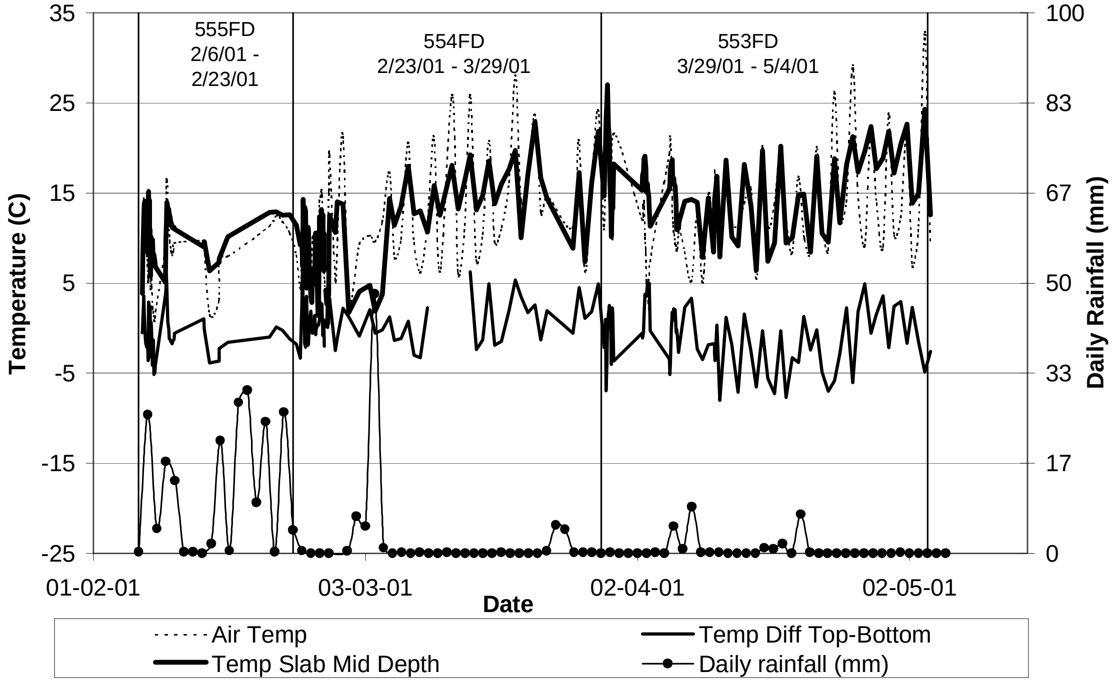
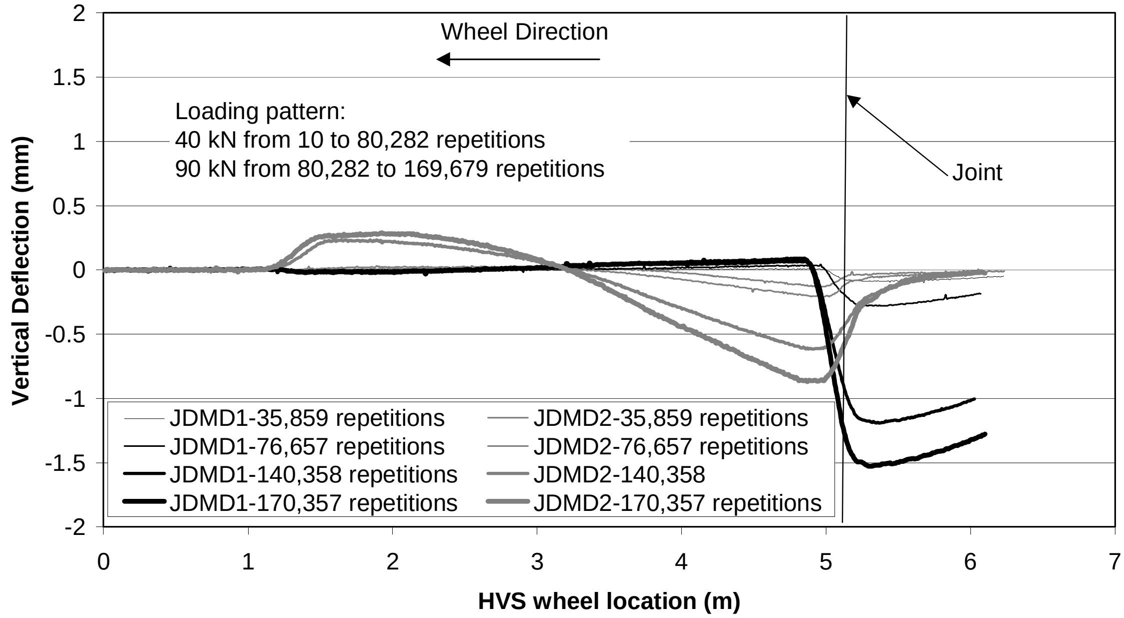


FIGURE 3 Rainfall, air temperatures and slab temperature and temperature gradient during HVS testing.



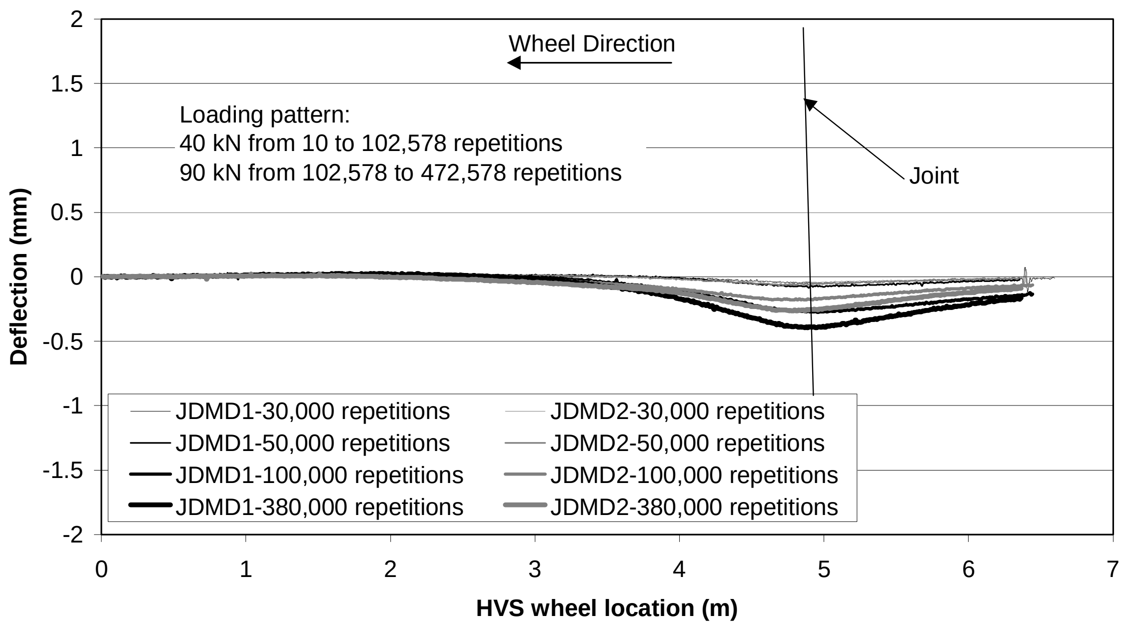


FIGURE 4 Influence lines for JDMDs 1 and 2 versus load repetitions under HVS loading on Sections 555FD (4a - top) and 554FD (4b - bottom).

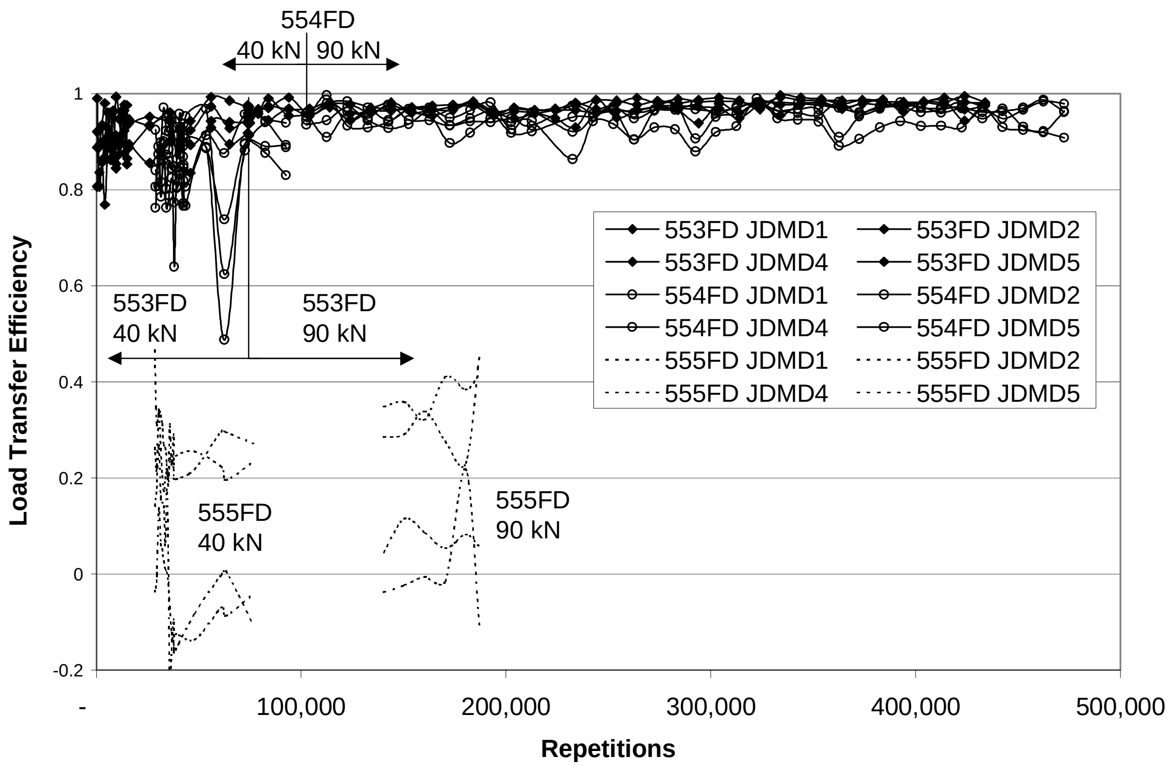


FIGURE 5 Comparison of load transfer efficiency on all HVS test sections during HVS trafficking.

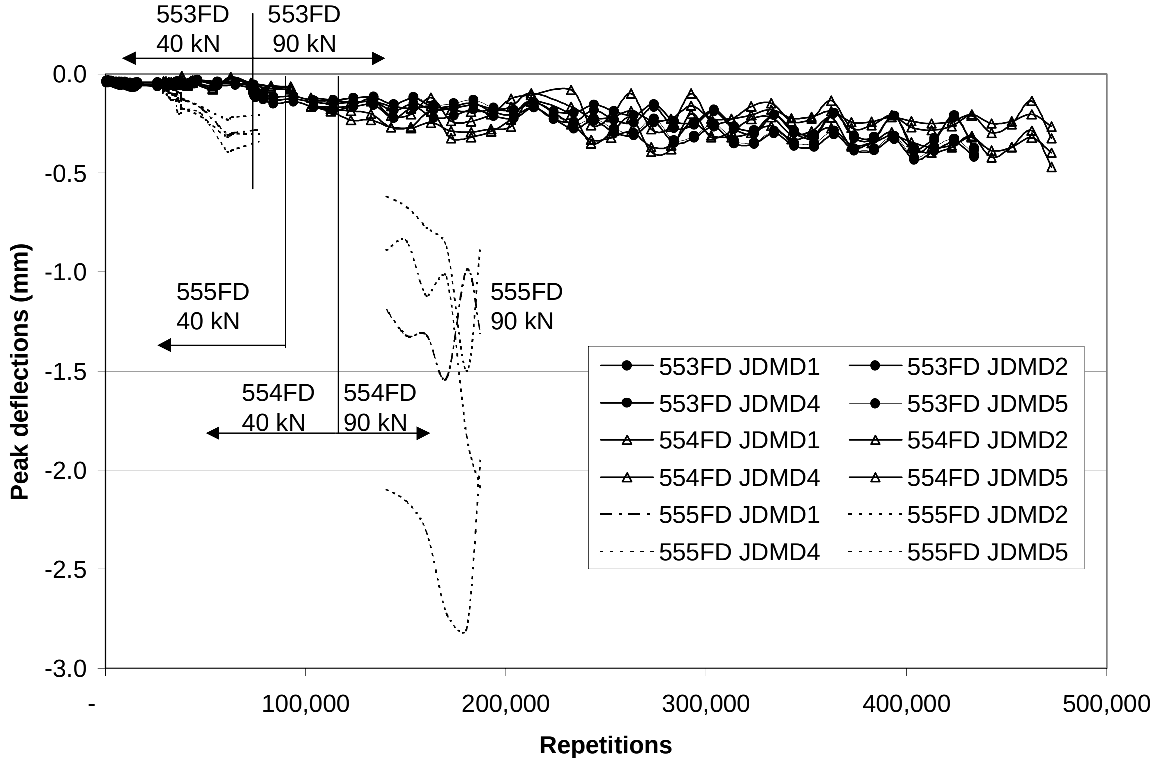


FIGURE 6 Comparison of mid-slab edge and horizontal joint maximum deflections on all HVS test sections during HVS trafficking.

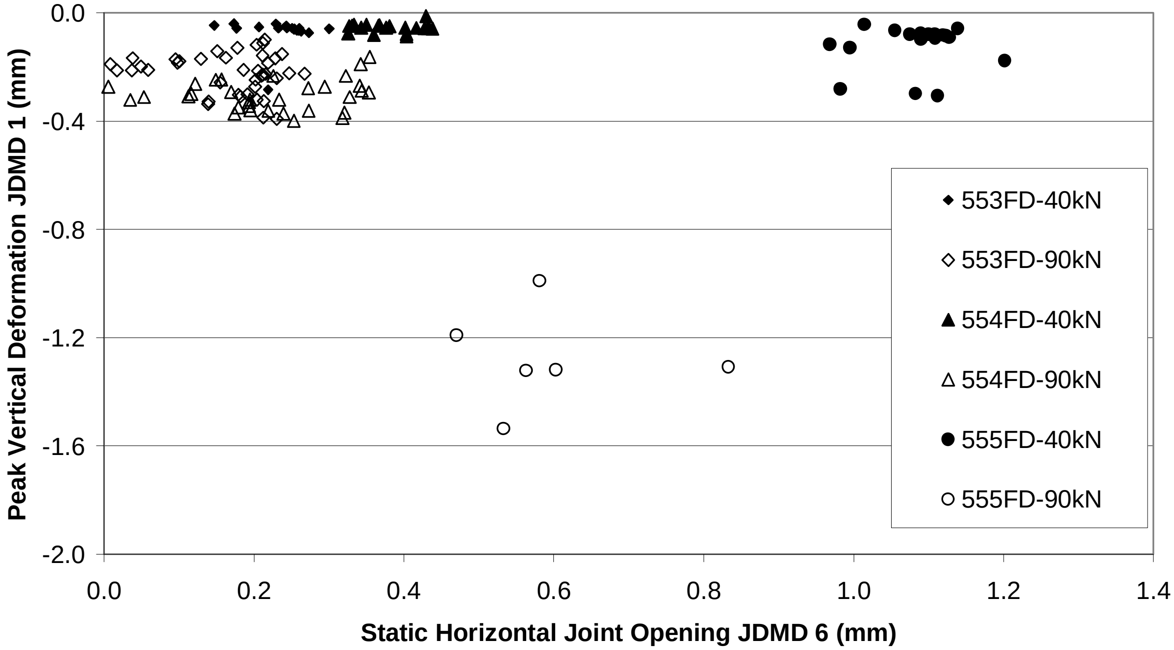
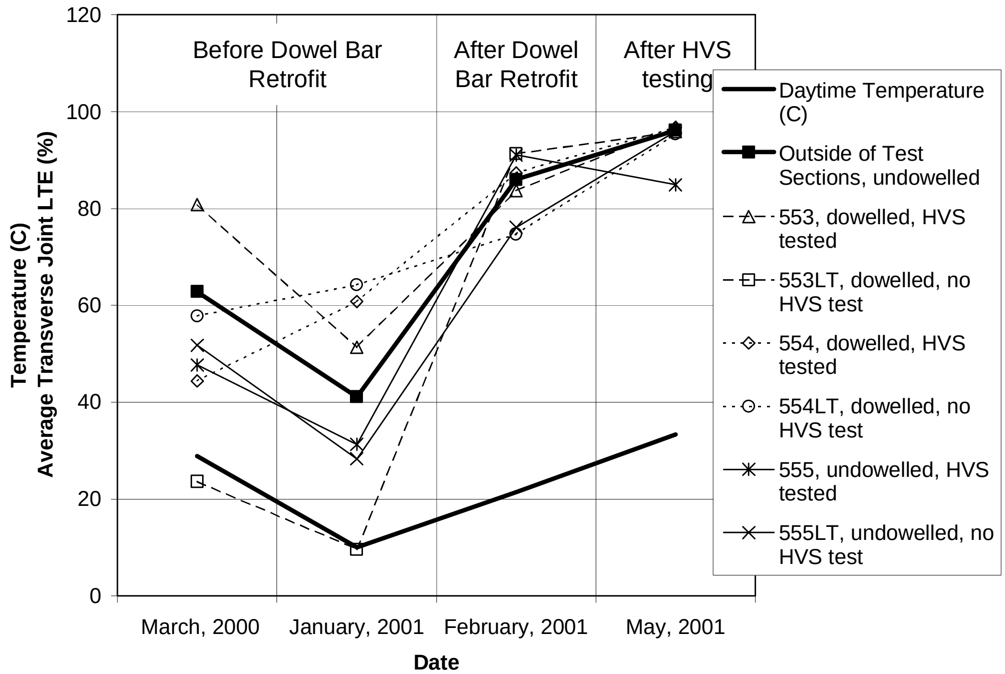


FIGURE 7 Comparison of maximum vertical deflection versus horizontal joint opening on all HVS test sections during HVS trafficking.



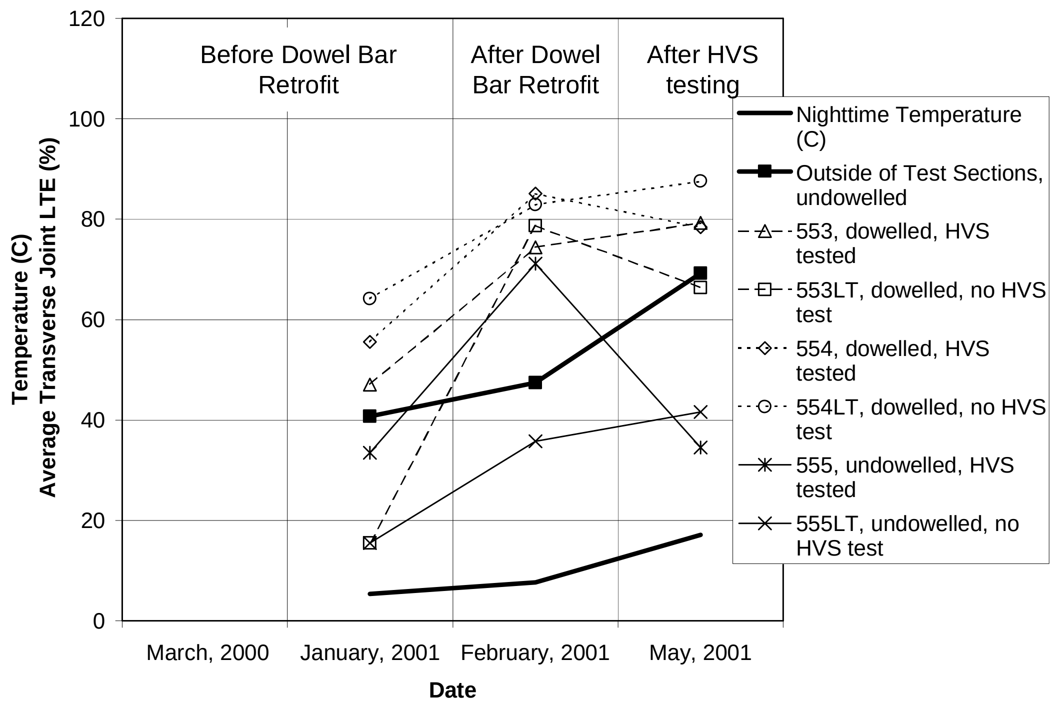


FIGURE 8 Comparison of load transfer efficiency for daytime (8a – top) and nighttime (8b – bottom) measured with the Falling Weight Deflectometer on all HVS test sections and “outside” control section before and after HVS trafficking.

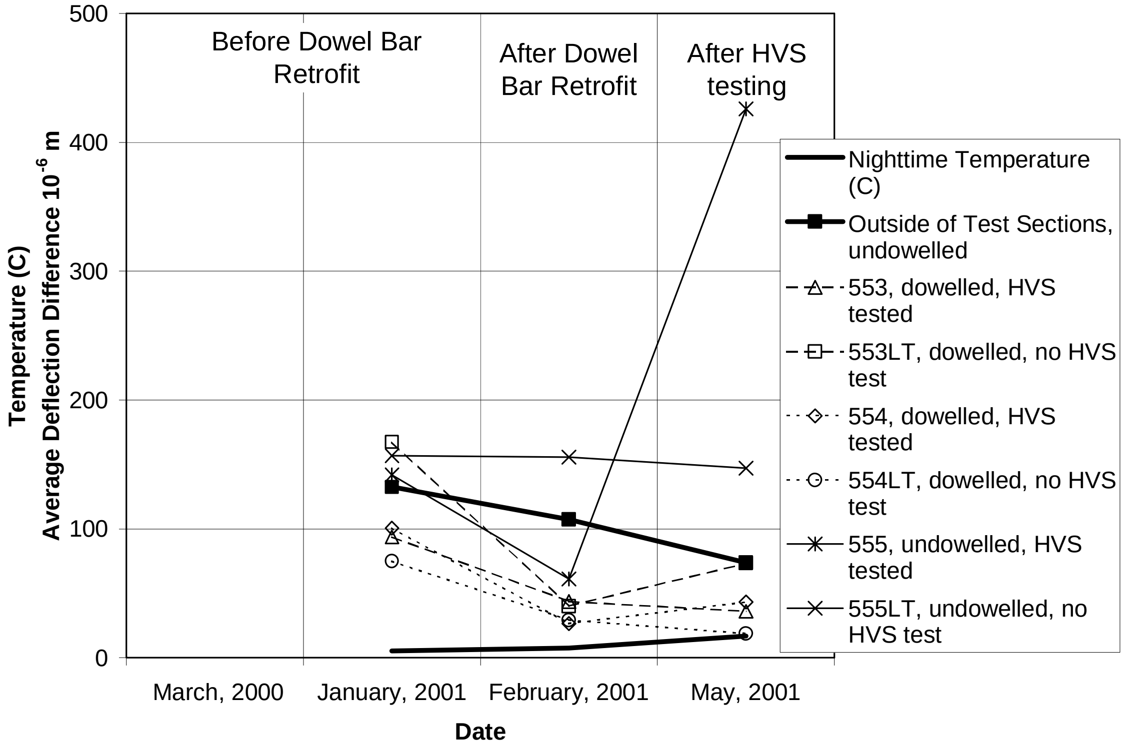


FIGURE 9 Comparison of deflection difference between slabs for nighttime measured with the Falling Weight Deflectometer on all HVS test sections and "outside" control section before and after HVS trafficking.