AN EVALUATION OF RETROFIT LOAD TRANSFER MATERIALS AND DOWEL BAR CONFIGURATIONS

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) constructed experimental test sections of retrofit load transfer across transverse cracks in concrete pavement on U. S. Highway 52 near Zumbrota, Minnesota. The test sections were constructed in September of 1994 to evaluate retrofit load transfer and the effects of different dowel lengths and configurations, and patching materials on performance. The retrofit dowels improved the load transfer efficiency (LTE) of the worst cracks from 23% to more than 80%. After 6 years, the LTE has remained above 80% for all of the test sections, there have been no visible failures of the repairs and very little additional faulting has occurred in any of the configurations, although several repairs exhibited plastic shrinkage cracks and bond failures along the vertical faces of the slots.

INTRODUCTION

There is approximately 1,700 km of jointed reinforced concrete pavement (JRCP) with 8.2m transverse joint spacings in the Minnesota Trunk Highway System. The majority of this pavement was constructed between 1970 and 1993 and had a typical design life of 35 years. This type of concrete pavement was designed with steel mesh reinforcement to keep any mid-panel cracks that would occur from deteriorating and becoming working cracks with no load transfer. However, experience has shown that this steel is rarely effective over the entire life of the pavement and that typical transverse mid-panel cracks almost always eventually result in ruptured or corroded steel and a working crack that usually faults, resulting in a pavement with poor ride quality. This type of distress usually occurs before the pavement is 15 years old, when the greater portion of the pavement design life lies ahead, suggesting that the some type of maintenance work is required to preserve an acceptable ride for the remainder of the pavement's life.

The Minnesota Department of Transportation (Mn/DOT) has been doing concrete pavement rehabilitation (CPR) since the early 1980s in an effort to improve the ride and extend the service life of these pavements. The standard repair for a faulted transverse joint or crack has been a full-depth repair that required the full-depth removal of a section of the concrete pavement that is typically 1m in length by 3.65m in width (full lane width). Both of the vertical faces of this removal area then have twelve 300mm length by 25mm diameter epoxy-coated dowel bars inserted into equally spaced holes drilled at mid-depth to provide load transfer restoration. The removal area is then back-filled with high early strength concrete, curing compound is applied, and the patch is protected from traffic for 12 to 24 hours. This type of removal and repair operation typically takes two days to complete: one day for removal and preparation of a large number faulted areas and a second day for placing and curing of the repair concrete. This process is both time-consuming and costly, with a typical repair costing approximately $500 per 3.65m lane width. If the pavement is suffering from a poor ride (present serviceability rating (PSR) below 3.0) as a result of faulting of these cracks, the pavement is also diamond ground to improve the ride, which can add $3.00- $5.00 /m^2 to the cost of the project. While this repair approach is a somewhat slow and expensive procedure, it should be noted that these repairs typically last 10-15 years or more before failure.

In an effort to reduce the repair time, be less intrusive on traffic and reduce construction costs, Mn/DOT began experimenting with retrofit dowels in September of 1994. After a review of previous work done by Puerto Rico and Washington State [1, 2], Mn/DOT personnel decided to try a similar approach. A review of bid prices from these agencies showed expected bid prices for retrofit load transfer to range from $20 to $35 per dowel, resulting in a per lane price of $60 to $210, depending on the number of dowels installed per lane.

FIELD INVESTIGATION

Background

U. S. Highway 52 (US 52) between reference posts (RP) 79.360 and 82.206 was originally constructed in 1983 and consists of 230mm thick jointed, reinforced concrete pavement (JRCP) with 8.2m transverse joint spacing, 3.65m lane widths, wire mesh reinforcement, and 25mm x 380mm epoxy-coated dowel bars centered at 300mm. The pavement structure was based on a 35-year design lane (right lane) traffic volume of 12,310,000 ESALs, with the 1984 ESALs estimated to be 221,819. The design lane tractor-semi-trailer (TST) volume was 306 per day. The concrete was mixed using recycled coarse aggregate that came from the existing roadway and had a 19mm top size. The pavement is supported by 127mm of dense-graded aggregate base (Mn/DOT Class 5) and sits on a plastic subgrade (AASHTO A-6). The entire project received a 1.1m deep subcut that consisted of removing, blending and recompacting the in-place soils to a uniform density.

By the fall of 1994, the concrete pavement had begun to experience mid-panel cracking. Because of the relatively young age of the pavement, most of the reinforcing steel was still intact and no significant faulting had occurred. However, it was expected that faulting would be considerable problem in the near future due to the use of 19mm top size concrete aggregate. The ride on the northernmost 1.9km of the pavement, between RP 81.000 and 82.206, had degraded to a PSR of 2.6, compared to a PSR of 3.2 on the southern portion of the project. The significant difference in ride between the two segments of roadway is most likely due to the due to the northernmost segment having 95% cracked panels versus only about 10% cracked panels on the rest of the pavement. While the cracks had not yet begun to fault significantly, the shorter panels had probably begun to rock and adversely affect the ride.

Experimental Layout

Dowel bars were retrofit in the transverse mid-panel cracks of northbound US 52 between RP 81.0 and 82.2. The research project had the following objectives:

- evaluate the constructability of retrofit dowel bar installation as a rehabilitation method in Minnesota;
- evaluate the performance of retrofit dowels compared to the "do nothing" option;
- evaluate the relative performance of different retrofit dowel bar configurations;
- evaluate the relative performance of 380mm x 37.5mm dowel bars vs. 457mm x 37.5mm dowel bars; and
- evaluate the effectiveness of Mn/DOT's standard 3U18 patching concrete compared to a proprietary repair material.

The project plan and special provisions were developed using the procedures recommended by the Federal Highway Administration (FHWA) in a draft chapter developed for the National Highway Institute Course on "Techniques for Pavement Rehabilitation" Manual [3].

The project was divided into 5 test sections (see figure 1) where different load transfer configurations were installed. There was an additional control section between RP 81.0 and 81.2 where no dowels were installed. Each test section was divided in half, with the first 20 slots using 380mm by 37.5mm dowel bars (subsection A) and the second 20 slots using 457mm by 37.5mm dowel bars (subsection B). The slots cut at the first four cracks in each subsection were filled with the proprietary repair material and the next 16 were filled with the standard 3U18 patching mix. Table 1 shows the mix design for the 3U18 material. The proprietary mortar material was mixed per the manufacturer's instructions and was extended with aggregate passing the 9.5mm sieve at the rate 1.4kg aggregate per 1kg of mortar. Table 2 shows the typical strength gain characteristics of both materials.

The typical dimensions and details of the slot and retrofit dowel installation are shown in figures 2, 3, and 4. The slots were made using parallel pavement saw cuts approximately 65mm apart, approximately 150mm in depth and 900mm in length. The contractor used a piece of equipment that could cut three slots simultaneously. This system allowed for much faster removal of the concrete and better alignment of the slots with the centerline of the pavement. The concrete was then removed by placing a light jackhammer on one end of the sawed slot and applying a small amount of force, causing the concrete in the slot to "pop" out of the pavement. A minor amount of additional jackhammer work was necessary to clean the bottom of the slot to the required dimensions. The slot was then sandblasted to remove the sawing residue and air-blasted to remove any remaining loose material.

Prior to placing the dowel bar assemblies, silicone joint sealant was applied to the crack face in a strip approximately 10mm wide by 1mm deep to keep any grout from entering the cracks and causing compression problems later on. After the silicone had cured, the slots were then coated with a thick cement/water slurry and the dowels were installed. All of the dowel bars had plastic end caps with built-in chairs that allowed the dowel bars to remain 12-15mm above the bottom of the removal area, thereby allowing the backfill material to fully encapsulate the bars. The end caps/chairs also were designed to allow 6mm of space between the end of the bar and the cap that would allow the pavement to expand without placing the concrete at the end of the dowels into compression and cause a failure of the patch. The dowels were also fitted with a minimum 6mm thick tab of compressible material that filled the slot full depth and width at the crack and allowed room for pavement expansion. The slots were then quickly filled with either the proprietary patching material or 3U18 patching concrete, vibrated, finished and cured with liquid membrane curing compound. The next morning (16 – 24 hours after placement) the repair was then sawed to reestablish the crack, the crack was sealed and the pavement was opened to traffic.

EVALUATION

Traffic Volume

A traffic analysis was done for the roadway in the spring of 2001 by Mn/DOT's Office of Transportation Data and Analysis. The forecast showed that the estimated traffic used to design the pavement was significantly lower than what the roadway had actually experienced. The traffic volume the design lane (right lane) had received by 2001 was approximately 8,000,000 ESALs, approximately 50% more than the original 1982 forecast used for design had estimated for the same period. The revised traffic forecast showed a 35-year estimated design lane traffic volume of 20,582,000 ESALs and that the TST volume had grown to 716 per day. The estimated design lane ESALs for 2002 is 519,000. The left lane traffic volume was assumed to be 11.1% of the right lane or approximately 57,000 ESALs and 80 TSTs in 2002.

Evaluation of Patches and Backfill Material

In November 1995 it was noticed that the surface of the majority of the repairs that were filled with 3U18 exhibited hairline cracks along their edges and also had cracks perpendicular to the edges, forming squares measuring approximately 65mm by 65mm. It was also noticed that the surface of several of the repairs made using the proprietary material had begun to scale to a depth of 2-3mm below the surface of the pavement.

Subsequent coring of several of the slots and analyses of the cores showed that many of the slots suffered from a lack of bond between the repair material and either one or both vertical faces of the slots (see figure 5). The cores also showed that the repair materials were mainly bonding to the rough surface at the bottom of the slot that had been fractured off by the jackhammer. Both repair materials also exhibited a substantial amount of plastic shrinkage cracking, although it was more pronounced in the 3U18. Both materials also exhibited significant voids beneath the dowel bars. Several of the repairs also exhibited hairline cracks radiating from the caps on the ends of the dowel bars that appeared to be caused by shear forces. [4]

It was hypothesized that the lack of bond between the repair material and the slots was caused by excess water in the patching material, causing excessive shrinkage of the patches. The voids beneath the dowel bars and chairs also appeared to be caused by the presence of excess water that coalesced under the bars instead of bleeding to the surface of the patch. This theory is supported by a lack of aggregate in the repair material adjacent to these voids, indicating a high water content (see figure 6).

It was also thought there might have been insufficient sandblasting of the slots to completely remove the slurry left behind by the wet saws during the removal process. It was also possible that the smooth vertical face that results from using saws didn't provide an adequate surface for the repair material to bond to, suggesting that much more aggressive sandblasting would be useful in any future work of this type.

Field Review of Pavement Condition

A thorough field review of the condition of the repair slots was performed in November 2000. It appeared that the surface deterioration of the proprietary repair material had not increased significantly beyond the level first noticed in 1995. The surfaces of most of the repairs patches were recessed 1 to 4 mm below the surrounding pavement surface. There appeared to be no relationship between lane location and the amount of scaling, indicating traffic was not an issue in the deterioration.

The plastic shrinkage cracking and debonding of the vertical faces of the slots was still evident in the surface of the 3U18 patching material and, to a lesser degree, in the proprietary material. There was no visible deterioration in the surface of the patches or in the cracks, indicating that the condition of the repair material had not changed significantly since 1995.

None of the repairs constructed using either material had deteriorated or completely lost bond with the adjoining concrete. There was no noticeable faulting evident in any repair, indicating that the bond with the original pavement concrete still existed to some degree, even with the partial debonding of the vertical faces of the patches at an early age.

There was significant deterioration in the condition of the pavement itself. Several of the panels had broken into smaller pieces 1-2m x 1-2m in size. There was also cracking in several panels that looked similar to the crack pattern typically observed in continuously reinforced concrete pavement. Virtually all of these cracks were still tight and did not appear to be detrimental to the performance of the pavement, but their presence will probably have a negative effect on the long-term performance of the pavement.

Load Transfer Efficiency

Prior to the start of the project, a falling weight deflectometer (FWD) was used to measure the load transfer efficiency (LTE) of each of the cracks in the right wheelpath of the right lane for both the control section and the test sections (see figure 7). The majority of the cracks still had LTE values above 90% (calculated using the equation $((D_l/D_{ul})x100\%)$, but there were several cracks where the LTE had dropped below 50% (between RP 81.3 and 81.5 and between RP 82.0 and 82.2). The LTE of the right lane test sections were measured again in November of 1995, one year after the retrofit dowels had been placed, and a substantial improvement was seen in these areas (see figure 8). The LTE of the entire section appears to be lower than in 1994, with an overall average of 85% and several cracks below 80%. This is most likely due to the relatively high pavement surface temperature during the testing in 1994 that may have contributed to the load transfer between slabs due to expansion of the pavement. The pavement temperatures at the time of the FWD testing were between 22^oC and 41^oC in 1994 and between 5^oC and 8^oC in 1995.

The LTE of the right lane test sections was measured again in October of 2000 using an FWD (see figure 9); the average had remained 85% with few crack LTE values below 80%. The majority of the cracks with LTE values below 80% were formed after the time the retrofit installation was performed and therefore had no dowels inserted. The pavement surface temperature at the time of testing was between 3^{o}C and 16^{o}C.

A comparison of the different dowel bar configurations shows no significant difference exists in the LTE of any of the test sections after 6 years of traffic (Figure 11). All of the test sections had average LTE's above 80% and 6 of the 10 were at 85% or above and are summarized in Table 2. The average LTE of the right lane, right wheel path dropped from 85.6% to 85.1% during this time period. All of the test sections had approximately the same loss in LTE with time, losing only an average of 0.5% in 5 years. Test section 2A had the greatest loss of 4%. This test section also had one of poorest LTE's prior to the dowel bar retrofits being installed. Test section 4B showed a gain in LTE of 3% between 1995 and 2000. This increase may be due to the higher pavement temperature during testing in 2000 relative to 1995.

A comparison of difference in performance between the 380mm and 457mm long dowel bars shows that the sections with the shorter bars lost an average of 1.4% LTE between 1995 and 2000, while the sections with the 457mm length bars gained 0.4% during that same period.

With only 4 sets of dowels per test section filled with the proprietary backfill material, it was not possible to do a meaningful comparison between the performance of that material in each dowel configuration versus the 3U18 backfill material. A comparison of the LTE for all the retrofit dowel bars backfilled with the proprietary material showed that it had an average LTE of 83.0% in 1995 and an average LTE of 82.9% in 2000, or a loss of 0.1%. This was the same percentage loss as all the retrofit dowels for the entire project, meaning the performance is comparable to the 3U18.

The most noteworthy difference is the change with time in the LTE of the control section. The control section had an LTE of 88% in September 1994 with a few crack LTE values measured below 50%. By October 2000 the average LTE of the control section had dropped to 48% with only one crack remaining above 80% and half the cracks being below 50%.

Load transfer efficiencies were measured on the cracks in the left lane on May 23, 2001 (see figure 10). The measurements were taken during a mild rainstorm that kept the pavement temperature between 10^{o}C and 15^{o}C. A comparison of the retrofit dowel test sections 4A, 4B, 5A and 5B shows that there is no significant difference in the LTE of any of the sections (see figure 11). All of the dowel configurations show LTEs between 92 and 95%, or slightly higher LTEs than the comparable test sections in the right lane, as could be expected due to the significantly lower traffic the left lane has seen.

Also noteworthy is the relatively high LTE of the undoweled sections, control and 1A-3B. The LTEs of these sections range between 73-91%, which is significantly higher than the undoweled right lane control section value of 48%. The LTEs in the left lane are comparable to or higher than the LTEs measured in the right lane in 1994, prior to the dowel retrofitting procedure. This supports the work of Vandenbossche [6], which showed that the load-induced fatigue was a primary contributor to the failure of the temperature steel in JRCP.

Faulting Evaluation

Faulting measurements were obtained from the outer wheel path of the right lane during September 1994, November 1995 and October 2000 using a Georgia Faultmeter (see figure 12). Unfortunately, it was not possible to accurately determine which measurement went with which crack for the 1994 data, so it was not used for this analysis.

Figure 13 shows a summary of the average faulting measurements for each test section and the difference between the averages measured in 1995 and 2000. It can be seen that several of the test sections had negative faulting in 1995 and that none of the sections had an absolute average fault measurement greater than 0.53mm (test section 2B, -0.53mm). By October 2000 all of the test sections had seen an increase in faulting, with an average increase for each test section of 0.52mm. Test section 2A showing the greatest average faulting measurement of 0.75mm, and test section 5B had the lowest with 0.30mm. The control section had an average faulting measurement of 1.92 mm in 2000. The control section faulting was not measured in 1995 so a comparison cannot be made. However, the faulting in the control section is significantly greater than that of all of the test sections, which indicates that the retrofit dowels do reduce faulting.

Ride Evaluation

When this section of US 52 was originally constructed in 1994, it had a PSR of 3.7 which was typical for a new concrete pavement built in Minnesota at that time. Figure 14 shows the PSR for the pavement from 1985 through 2000, broken down into three segments. By 1994 the ride for the first two segments, RP 79.360 and 80.000 and RP 80.000 and 81.000, was substantially better than that of the segment between RP 81.000 and 82.206. This was due in large part to the relatively low number of uncracked panels in the first two segments, less than 10%, compared to the third segment which had over 95% cracked panels. The ride was measured again in 1995, after the retrofit load transfer was installed, and every year thereafter. The figure shows that the ride for all sections increased in 1996, most likely due to changes in the measuring equipment, and has remained relatively constant in each segment since then. It is unlikely that the 0.2km between RP 81.030 and 81.110, which was undoweled and had a larger amount of faulting, contributed negatively to the overall ride measurement of the third segment because of its short length relative to the entire segment.

CONCLUSIONS AND RECOMMENDATIONS

1) The water content of any repair material used to backfill retrofit dowel bar slots should be carefully controlled and the amount used should be the minimum necessary to allow for the proper placement of the material in order to reduce the probability of shrinkage cracks and debonding.
2) Extraordinary effort should be made in to insure that all faces of the removal area are thoroughly cleaned and abraded (by sandblasting or other effective means) to assure the best possible bond between the repair material and the pavement.
3) There appears to be no significant performance difference between 380mm length bars and the 457mm length bars. This indicates that the shorter bars are of sufficient length to provide adequate load transfer if properly centered over the crack or joint. There was a 6% lower bid price for the shorter bars ($33.00 each vs. $35.00 each) on this project, which would amount to substantial savings on a large project.

4) There appears to be no significant performance difference between repairs constructed using the Mn/DOT 3U18 patching material and the proprietary material. Both suffered bond and void problems but both were still performing satisfactorily after 6 years. The material selected should, therefore, be based on other parameters such as cost, ease of use, and time required for opening the road to traffic.
5) There appears to be no difference in the load transfer efficiency and faulting measured in the right wheel path of the right lane relative to any of the dowel patterns used in this experiment. Additional load transfer and faulting measurements should be taken in the left wheel path and in outside wheel path of the left lane to determine the value of using dowels in those locations.
6) Retrofit load transfer did not significantly improve the LTE of the left lane over that of the "do nothing" option. This suggests that, depending upon traffic volumes, it may be sufficient to provide retrofit load transfer in only heavily trafficked lanes. However, since the installation is only seven years old, it is too early to determine whether this will remain the case in the long term. Also, since there is no ride data available for the left lane test sections, it isn't known what effect the presence of dowels would have had on long-term ride performance.
7) Retrofit load transfer reduced the increase in faulting by a substantial amount over doing nothing. However, the poor ride that existed prior to the retrofit was maintained afterwards, which strongly suggests that poor riding pavements should be diamond ground after the load transfer is established.

U. S. 52 Retrofit Dowel Bar Test Section Patterns

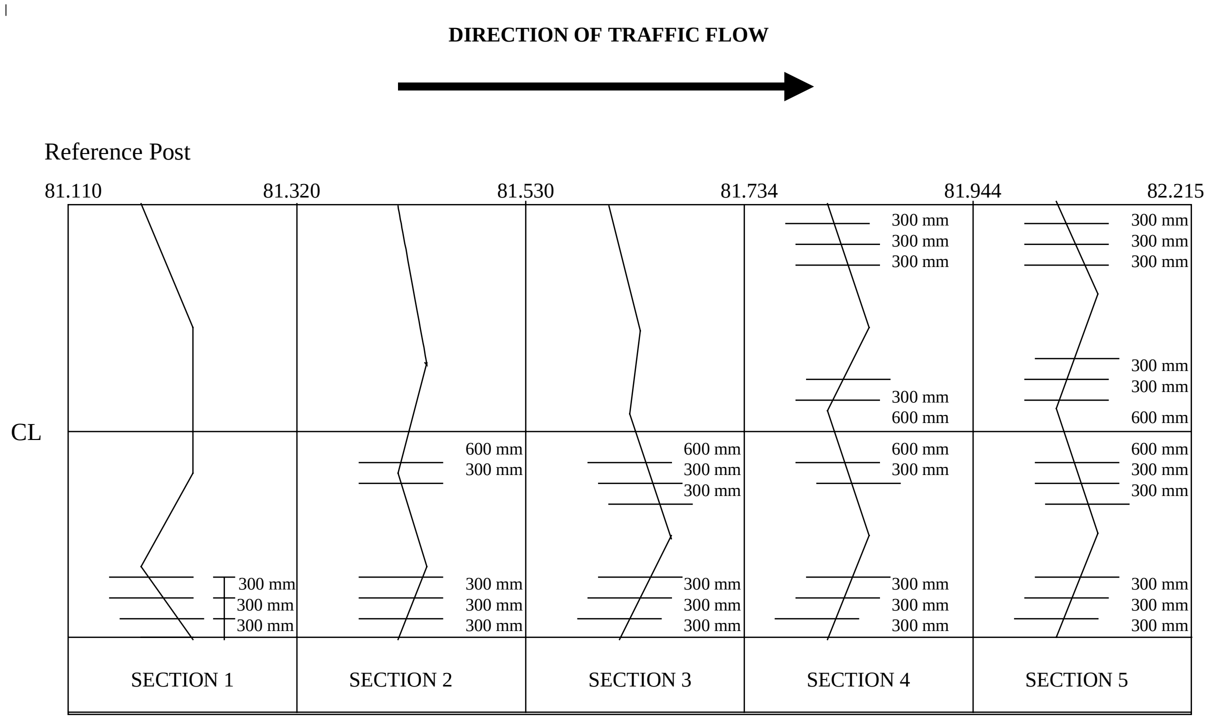

Figure 1. Test Section Layout.

Table 1. Mn/DOT 3U18 Patching Concrete Mix Design

Material	Mass/m^3
Type I cement	501kg
9.5mm max coarse aggregate	795kg
Fine aggregate	782kg
Water	176kg
Air	6.5%
Type E Admixture	25 ml/kg cement

Table 2. Strength Gain Characteristics for US 52 Patching Materials (at 21^{0}C)

Time	3U18	Proprietary
1 hr		14 MPa
2 hrs		28 MPa
4 hrs	5 MPa	
8 hrs	11 MPa	
12 hrs	17 MPa	
24 hrs	29 MPa	48 MPa

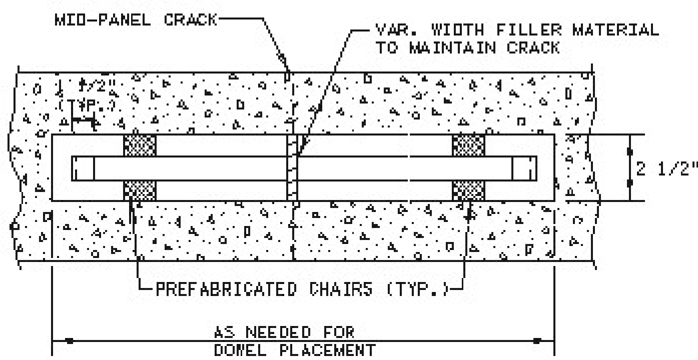

Figure 2. Typical Plan View of Retrofit Dowel Installation [3]

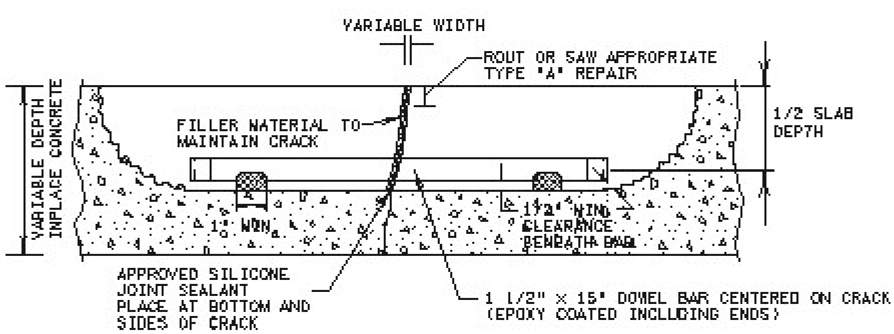

Figure 3. Typical Elevation View of Retrofit Dowel Installation [3]

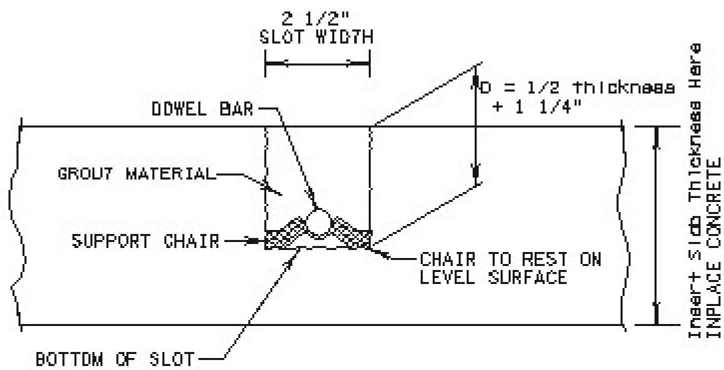

Figure 4. Typical End Elevation View of Retrofit Dowel Installation [3]

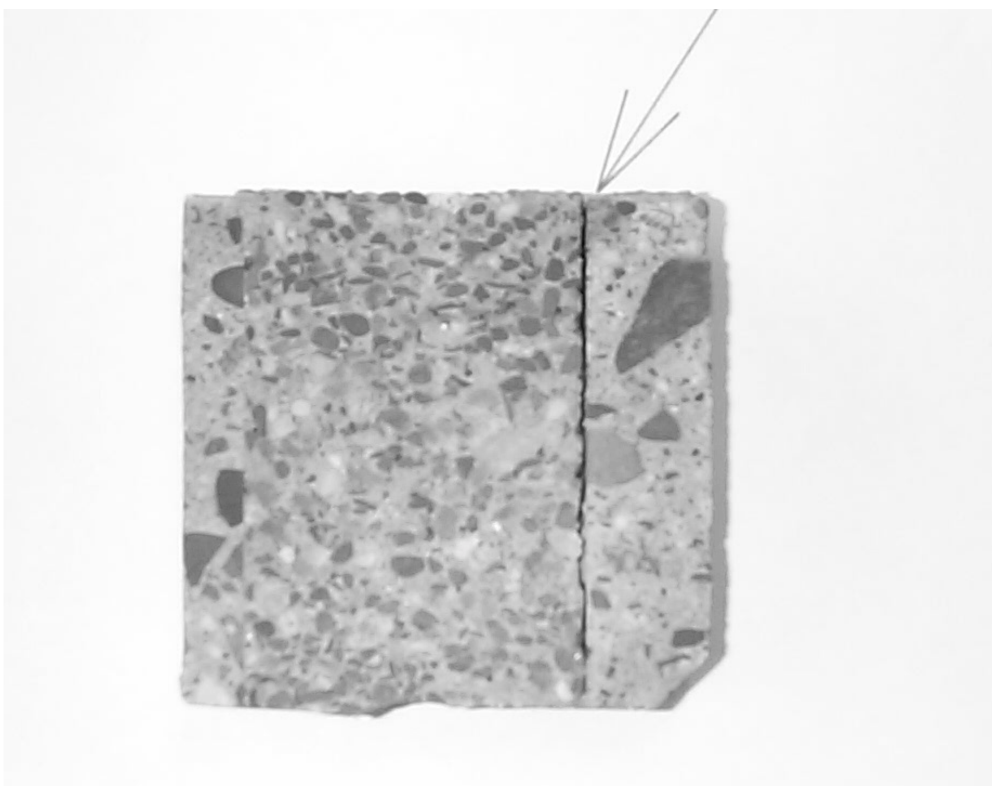

Figure 5. Typical Debonded Vertical Face

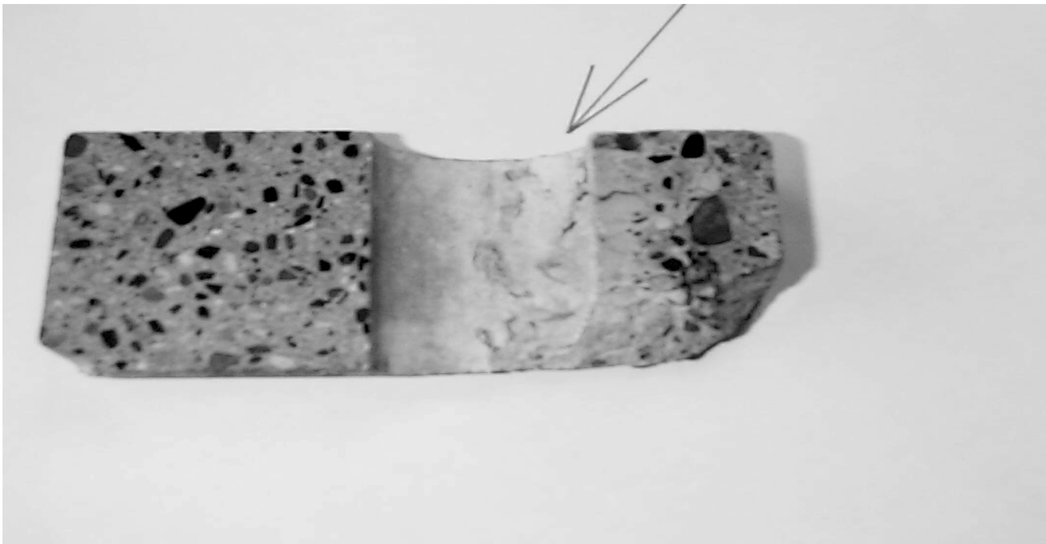

Figure 6. Example of Voids in Mortar Around Dowel Bar

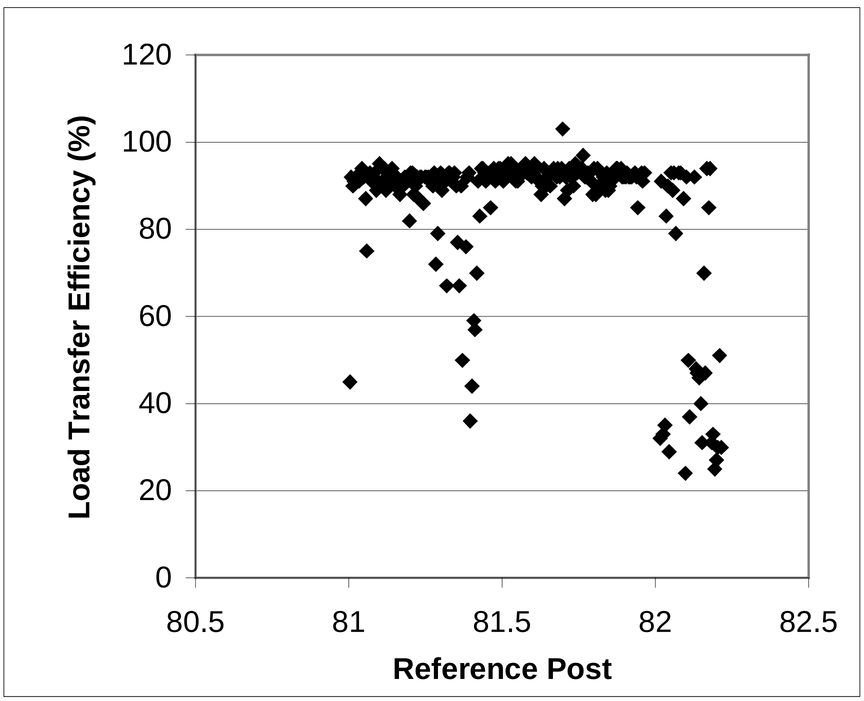

Figure 7. Retrofit Dowel Load Transfer Efficiency, 9/14/1994 (Right Lane, Right Wheel Path)

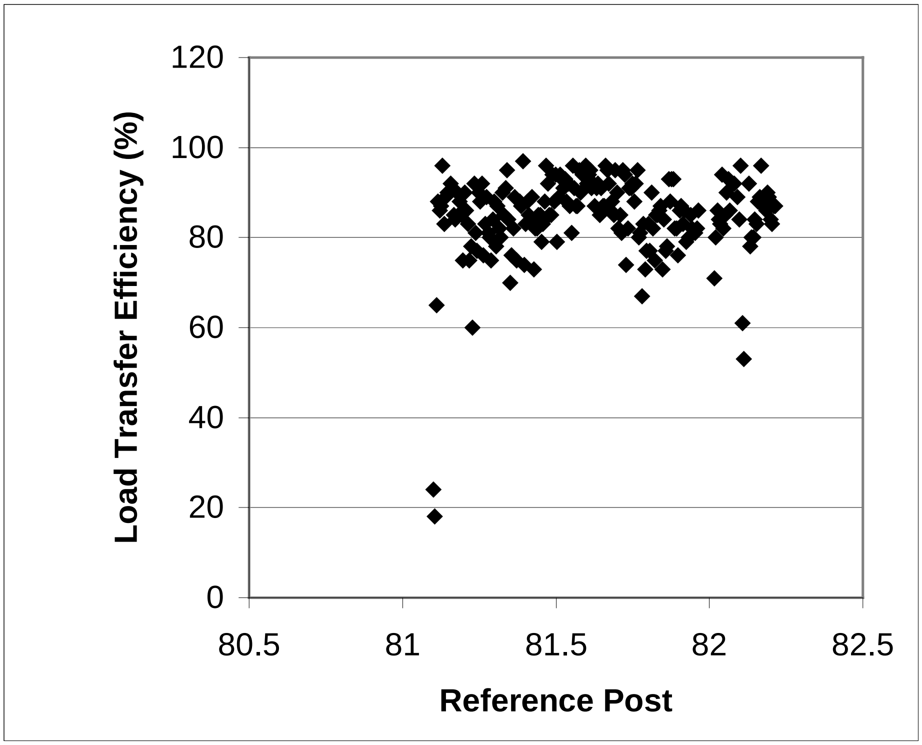

Figure 8. Retrofit Dowel Load Transfer Efficiency, 11/07/1995 (Right Lane, Right Wheel Path)

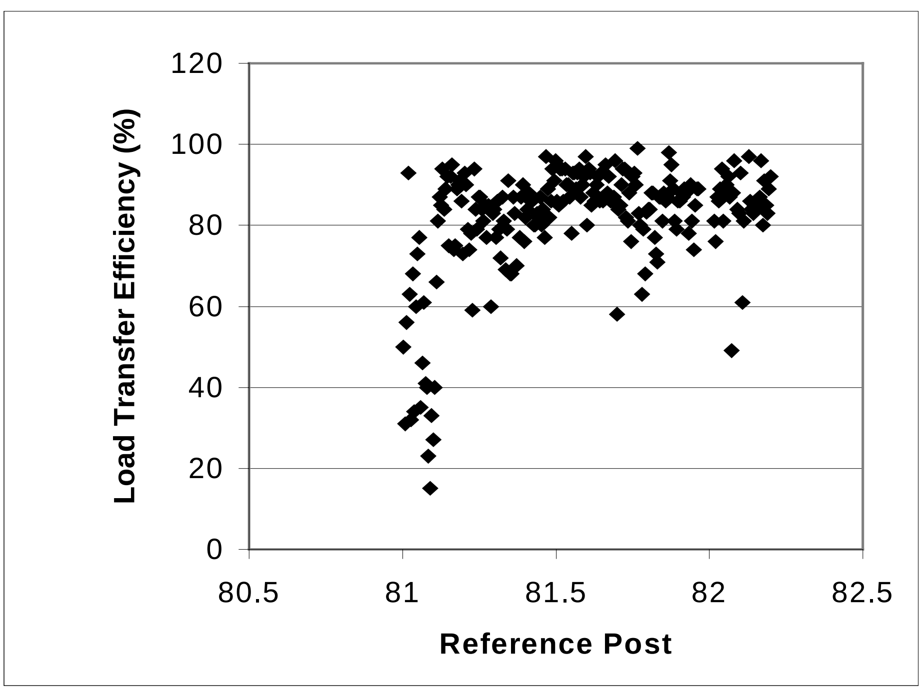

Figure 9. Retrofit Dowel Load Transfer Efficiency, 10/30/2000 (Right Lane, Right Wheel Path)

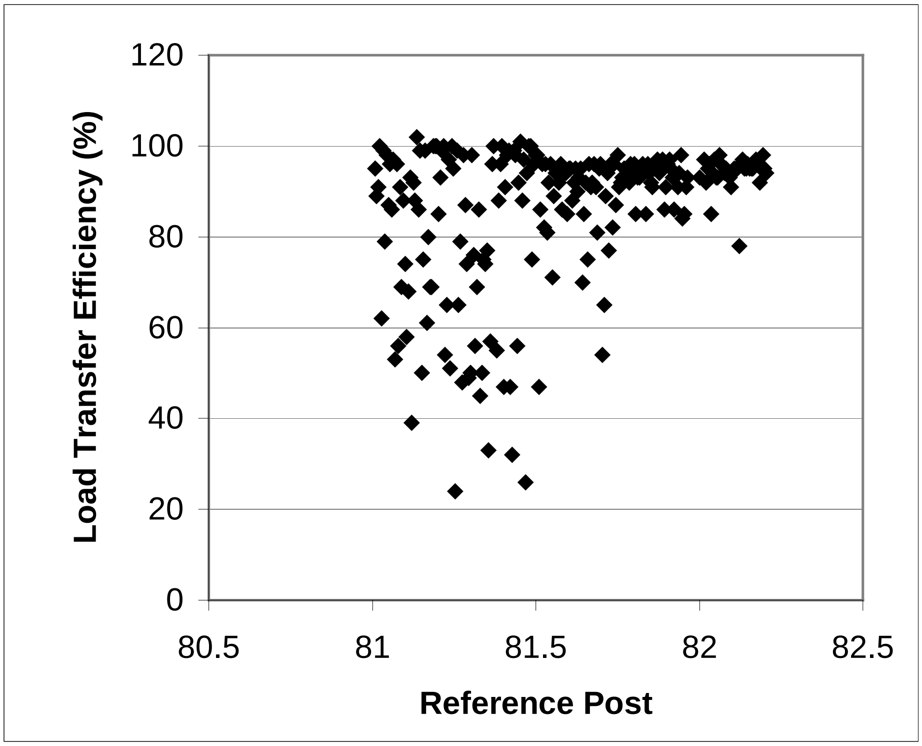

Figure 10. Retrofit Dowel Load Transfer Efficiency, 5/23/2001 (Right Lane, Right Wheel Path)

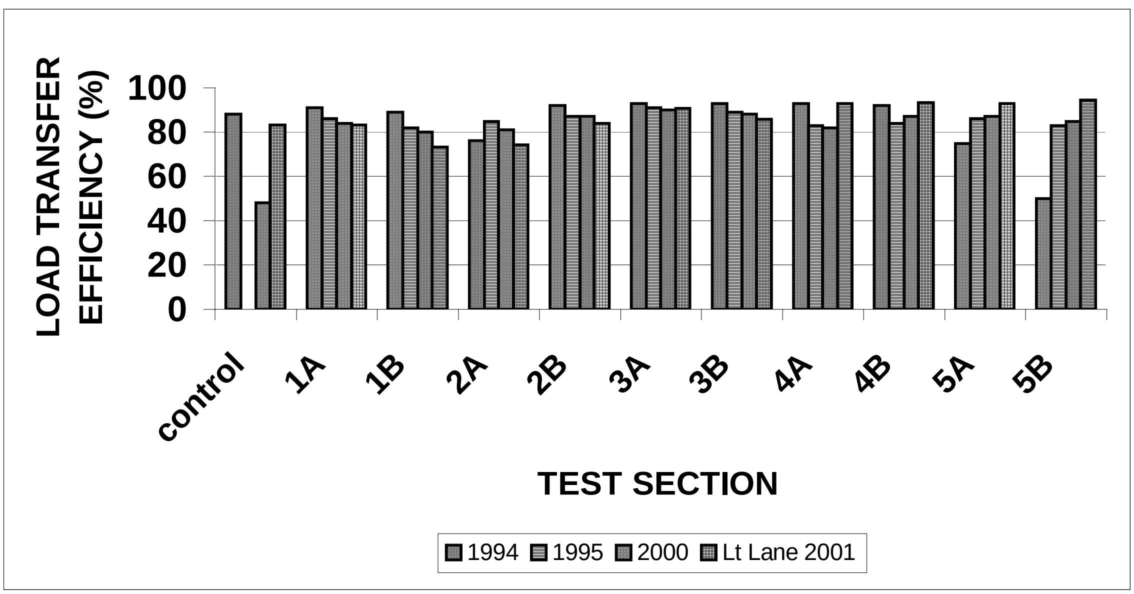

Figure 11. Retrofit Dowel Load Transfer Efficiency By Test Section

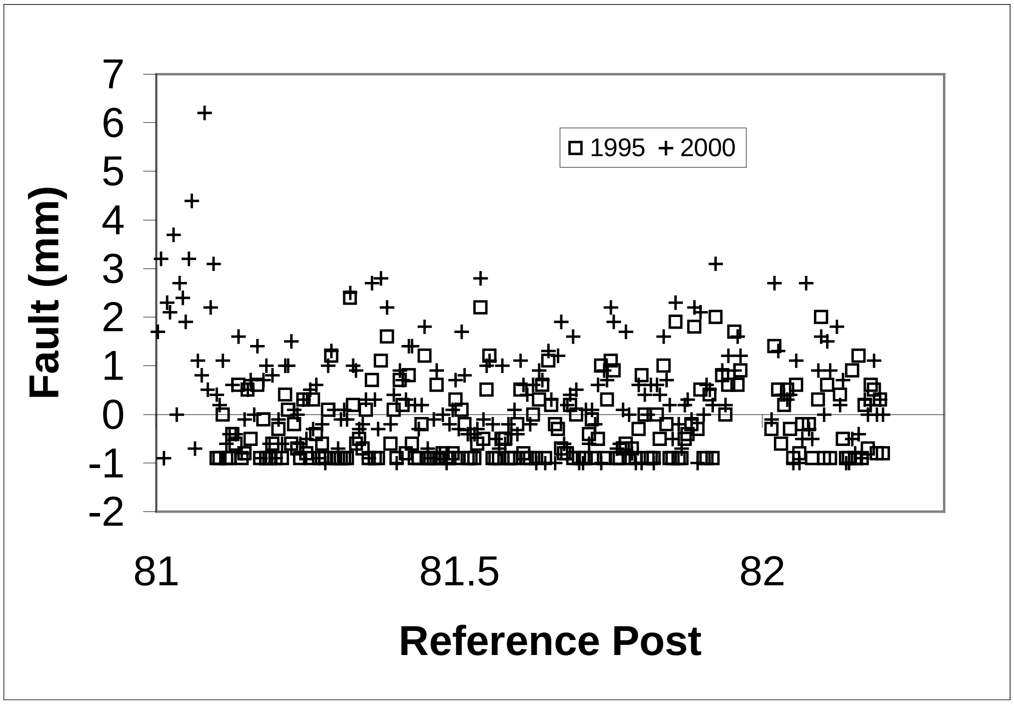

Figure 12. Summary of Test Section Faulting Measurements

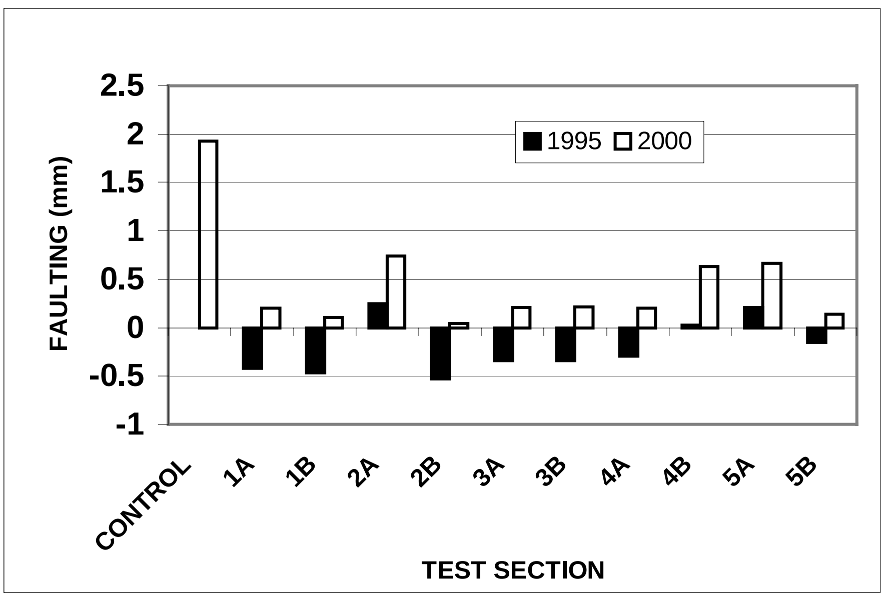

Figure 13. Change in Test Section Faulting Averages over Time

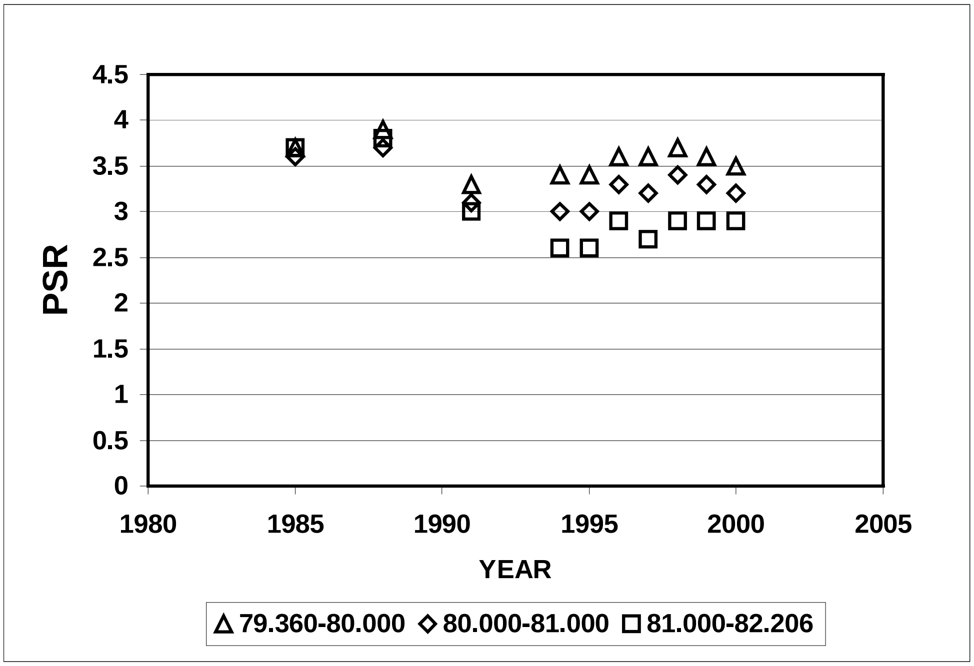

Figure 14. US52 Retrofit Dowel Test Sections Control Section Ride by Mile

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