
Design, Construction, Performance and Future Direction of Dowel Bar Retrofit in Washington State

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INTRODUCTION

In the late 1980's, the Washington State Department of Transportation (WSDOT) was faced with the dilemma of how to rehabilitate the concrete pavements that were constructed as part of the Interstate Highway construction program. The Interstate concrete pavements in Washington state, and other states as well, were originally designed for a 20 year period. As of 1997 over 60 percent (approximately 620 centerline kilometers (385 centerline miles)) of all Interstate PCC pavements in Washington state have been in service for 20 or more years. Since the majority of these pavements are on the Interstate system, they have also experienced anywhere from 2 to 5 times the original traffic design levels.

The management decisions for the distressed concrete pavements had typically been to overlay with thick (100 mm (4 inch)) asphalt overlays. Since the major distress with the concrete pavements in Washington state was faulting, reflective cracking was always a concern and the main factor which drove future rehabilitation. Several different practices were used to try and minimize the effects of reflective cracking, fabrics, SAM, SAMI's; none were found to be effective. Placement of the asphalt overlays was a quick and relatively easy solution to improve the ride characteristics of the concrete roadway for approximately 10 to 14 years, it wasn't believed to be the most cost effective or the solution to the real problem.

Therefore, in the early 1990's WSDOT began to research the various rehabilitation alternatives that were being evaluated nationwide, as well as internationally. The work being conducted in Puerto Rico by retrofitting the concrete panels with dowel bars showed exceptional potential.

This paper will discuss the performance of the existing concrete pavements in Washington state, the development of a dowel bar retrofit test section, modifications that have been made over the years to the specifications, dowel bar retrofit costs, and the future direction WSDOT will be taking with this procedure.

DESCRIPTION OF EXISTING PCCP

The concrete pavements that were constructed from 1959 to 1967, typically consisted of unreinforced 230 mm (9 inch) thick pavements with perpendicular (non-skewed) transverse joints spaced 4.6 m (15 feet) apart. In the 1970's the joints spacing was changed to an unreinforced skewed and random joint spacing (4.3, 2.7, 3.4, and 4.0 m (14, 9, 11, and 13 feet)). The transverse contraction joints are sawed to a depth of D/4, and the longitudinal contraction joints are sawed to a depth of D/3. The width for both joints range from 5 to 8 mm (3/16 to 5/16 inches).

The majority of the concrete pavements have crushed stone base courses, which has a maximum aggregate size of 19 mm (3/4 inch). In some locations the base material consists of asphalt treated base or cement treated base. The majority of the shoulders have been constructed with asphalt concrete.

To date, the main form of distress has been joint faulting and longitudinal cracking in the wheel paths. Typically, in areas where the base material has good drainage, the form of distress has been longitudinal cracking with minimal joint faulting (0 to 6 mm (0 to 1/4 inch)). In areas with poor drainage, base materials consisting of cement treated base or crushed stone, the prevalent form of distress is joint faulting (6 to 22 mm (1/4 to 7/8 inch)). Recently, WSDOT has been experiencing joint faulting on those sections which have asphalt treated bases. The asphalt treated bases are beginning to strip causing a reduction in support which leads to joint faulting. To date minimal cracking has occurred on these sections.

Of the concrete pavements that have fatigued (longitudinal cracking), the cracks developed within 10 to 15 years of construction and have not affected the pavement performance. The faulted pavements, on the other hand, have deteriorated at such a rate that rehabilitation of approximately 80 to 160 kilometers (50 to 100 miles) was required in 1992 and an additional 80 to 160 kilometers (50 to 100 miles) would need rehabilitation between 1997 and 2002. Therefore, the main concern for WSDOT was to better understand the mechanisms of faulting and the most appropriate and cost effective rehabilitation method.

Concrete pavement rehabilitation guides describe the faulting mechanism as a displacement or pumping of fines across the joint from the leave slab (typically) to the approach slab. This action then results in the development of a void and increases in size with continued traffic loading. Void development then results in the leave slab being unsupported and generally leads to pavement faulting.

WSDOT conducted several concrete pavement faulting evaluation studies which involved quantifying slab deflections using the falling weight deflectometer (FWD), determining joint load transfer efficiency, determining void presence and finally, slab removal to investigate the modes of faulting. Load transfer efficiency was determined with the following equation:

$$\text{Load Transfer} = \frac{\text{Deflection of unloaded slab}}{\text{Deflection of loaded slab}} \times 100 \quad (1)$$

The determination of voids or loss of support was determined using the procedure developed by Croveti and Darter (1). This procedure determined that voids (loss of support) existed at the majority of the joint locations evaluated.

In order to verify the results of the FWD testing and the void detection method, WSDOT lifted (removed) nine slabs in various locations across the state. Since the transverse joints were the area of concern, sawcuts were made approximately 1.5 m (5 ft) on either side of the joint and along the longitudinal joint between the concrete lanes and the asphalt shoulder (Figure 1).

The sawcut concrete was then lifted vertically to expose the underlying base material. From this analysis, it was determined that voids developed only in locations where the underlying base material was cement treated base. All other locations, where the base course material was crushed stone, no void could be detected. At these locations it appeared that the fine material mitigated upward as well as horizontally across the joint, resulting in a wedge of fine material directly beneath the slab, presumably causing a reduction in slab support. This hypothesis is supported by the gradations taken on either side of the joint for the wedge material and for the crushed stone base. Without exception, all tests showed that the base material beneath the leave slab was finer than the material beneath the approach slab. In addition, measurements were taken on the topography of the concrete slab and the wedge material, it appears that the faulting measurement and the height of the wedge material are roughly equal. Measurements were taken when the slab temperatures were approximately 16°C (60°F), when slab curling was improbable.

Since the major distress in the concrete pavements was in the form of joint faulting and no voids were present at the joint locations, pavement subsealing was viewed to have marginal effectiveness or at best a short lived performance life. Subsealing has been performed on several concrete pavement rehabilitation projects and has performed best when the base material is cement treated base. Since voids are present, the subsealing material is capable of filling the voids and therefore provides some level of support. On the other hand, when crushed stone base is present, no void is present and the subsealing material may be forced into small gaps or channels and can result in the development of a larger void, causing uneven support of the slab, which can lead to premature failure of the slab.

Pavement grinding alone was not believed to be a viable solution since it does not address the initial deficiencies that caused the faulting. In addition, a reduction in thickness would increase the pavement edge stresses, which would then promote the further development of faulting. This rapid fault reoccurrence has been noted on several concrete pavement sections in Washington state, where joint faulting returned within two to three years.

DOWEL BAR RETROFIT TEST SECTION

In July 1992, WSDOT constructed a concrete pavement test section which included four experimental features:

- (1) retrofitted dowel bars only, Section A,
- (2) 1.2 m (4 foot) wide tied and doweled concrete shoulder, Section B,
- (3) retrofitted dowel bars and a 1.2 m (4 foot) wide tied and doweled concrete shoulder, Section C, and
- (4) control section, Section D, which received no treatment.

Following construction of the design features, all sections were ground smooth by diamond grinding.

The concrete pavement in the test section was originally constructed in 1964 and consists of 230 mm (9 inches) of plain jointed concrete on a crushed stone base with a joint spacing of 4.6 m (15 feet). The climate in this area is classified as a wet freeze with approximately 580 mm (23 inches) of annual precipitation. This pavement section, as of 1992, had experienced over 10,000,000 (40 kN (18,000 lb)) equivalent single axle loads. The existing distress consisted of a few slabs with single transverse cracks, and joint faulting from 2 to 16 mm (1/16 to 5/8 inches). Generally, a fault of 5 mm (3/16 inch) is considered "critical" and a fault of 3 mm (1/8 inch) is considered undesirable.

Dowels placed in slots cut in the pavement are effective in restoring load transfer across joints or transverse cracks. Dowels should be 457 mm (18 inches) long and at least 32 mm (1.25 inches) in diameter (2). In addition, the number of dowel bars placed per joint has some significance on the performance of joint load transfer restoration. "In most but not all cases, sections with five dowels per wheel path had slightly higher load transfer efficiencies than sections with three dowels per wheel path. Similarly, sections with 38 mm (1.5 inch) dowels had slightly higher load transfer efficiencies than sections with 25 mm (1 inch) dowels. Dowel length did not appear to affect load transfer efficiency (3)". Therefore, based on the results of the Florida study, the AASHTO Design Guide, and contacts made by WSDOT personnel, it was determined that four dowel bars would be placed in each wheel path and the dowel bar dimensions should be 38 mm (1.5 inch) diameter and have a length of 457 mm (18 inches).

The dowel bar slots were cut to a width of 64 mm (2.5 inches), a depth of approximately 146 mm (5.75 inches) or as required to place the center of the dowel at mid depth, and the required length for bar placement. The dowel bars were spaced 305 mm (12 inches) apart. The first dowel bar in the outer wheel path was placed 305 mm (12 inches) from the lane/shoulder edge. The first dowel bar in the inner wheel path was placed 610 mm (24 inches) from the longitudinal joint with the adjacent lane (see Figure 2 for dowel bar layout).

Light weight jack hammers, with a weight less than 14 kg (30 lbs), were used to break loose the concrete. All exposed surfaces were sandblasted and cleaned prior to installation of the dowel bar. Epoxy coated dowel bars were inserted and held in position by non-metallic chairs. Dowel bars were placed such that horizontal and perpendicular alignment with the existing slabs and joints was maintained. Dowel bar end caps were not used in this project. A mastic filler was placed in the joint to prevent the backfill material from filling the joint and to allow for the expansion and contraction of the filler material. The slot was then backfilled with Burke Fast Patch 928 grout.

A major advantage in using tied PCC shoulders or a widened concrete outside lane is the reduction in slab stresses. Reducing slab stresses has shown to increase the pavement performance life. Therefore, to minimize edge stresses a 1.2 m (4 ft) concrete shoulder was tied to the existing outside lane with 16 mm (5/8 inch) reinforcing bars with a length of 762 mm (30 inches). Three epoxy coated dowel bars were also placed in the transverse contraction joints (see Figure 3).

FWD testing was conducted prior to construction in July 1992, within two weeks following construction in September 1992, and annually every year since construction. On the FWD test days, the deflection measurements were obtained when the air temperature was less than 27°C (80°F) so that the upward curling of the slab was minimized. In addition, faulting measurement were also taken initially, immediately following construction and on a yearly basis. The results of this analysis are shown in Figures 4 through 8.

In summary, the experimental features which contain dowel bars at the transverse joints, Sections A and B, have maintained an average joint load transfer between 80 to 90 percent over the last 4 years. In addition, out of the 48 joints that were measured for joint faulting, only 5 joints have faulted 1.6 mm (1/16 inch) and one joint has faulted 3.2 mm (1/8 inch). Section C (concrete shoulder beam only) has not performed as well as expected. One reason for the lower performance may be that Section C had the lowest initial joint load transfer efficiency and there may be a point at which the load transfer efficiency is too low to expect improvement with only a tied concrete beam. The control section is performing as expected with a reduction in joint load transfer efficiency and essentially all joints having measurable joint faulting.

MODIFICATIONS TO DOWEL BAR RETROFIT PROCEDURE

Based on the results of the test section and the experience gained from the dowel bar retrofit projects over the last 4-1/2 years, the following modifications have been made to this procedure:

1. Require three dowel bars per wheel path. The Florida study indicated a slight improvement with an increase in the number of dowel bars per slot. Therefore, it was decided that using only three dowels per wheel path would provide for a cost savings without sacrificing performance
2. Require the use of dowel bar end caps. This will allow the dowel bar to move in relation to the expansion and contraction of the pour back material.
3. Require the use of prepackaged patching material. In order to keep innovation of the process open to improvements, it was believed that using a product that allowed for the use of a mobile mixer (3.8 cubic meters (5 CY) capacity) would provide a cost savings. Upon trying this process, it was realized that an inconsistent mix resulted causing problems with shrinkage and bond. Therefore, the use of this mobile mixer is not recommended.
4. Extension material for the slot backfill shall be in accordance with WSDOT Standard Specification Section 9-03.1(4) A through C using AASHTO Grading No. 8 with the following exceptions: The 9.5 mm (3/8 inch) square sieve shall have a minimum of 95% passing, the 4.75 mm (US No. 4) sieve shall have a maximum of 25% passing, and the extension material shall be non fractured to ensure workability.

WSDOT Standard Specification 9-03.1(4)

A Deleterious Substances

The amount of deleterious substances shall not exceed the following values:

Amount finer than 0.075 mm (US No. 200)	1.00 percent by mass
Pieces of specific gravity less than 1.95	2.00 percent by mass
Clay lumps	0.50 percent by mass
Shale	2.00 percent by mass
Wood waste	0.05 percent by mass

For coarse aggregate with a minimum single face fracture content of 25 percent by mass, the material finer than 0.075 mm (US No. 200) sieve may increase to a maximum of 1.5 percent by mass.

B Wear in Los Angeles Machine

Coarse aggregate shall not have a percentage of wear in Los Angeles machine in excess of 35 after 500 revolutions.

C Grading

Coarse aggregate for Portland cement concrete when separated by means of laboratory sieves shall conform to one or more of the following gradings:

AASHTO Grading										
Passing Sieve	No 467		No 57		No 67		No 7		No 8	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
50 mm	100	---	---	---	---	---	---	---	---	---
37.5 mm	95	100	100	---	---	---	---	---	---	---
25.0 mm	---	---	95	100	100	---	---	---	---	---
19.0 mm	35	70	---	---	90	100	100	---	---	---
12.5 mm	---	---	25	60	---	---	90	100	100	---
9.5 mm	10	30	---	---	20	55	40	70	85	100
4.75 mm	0	5	0	10	0	10	0	15	10	30
2.36 mm	---	---	0	5	0	5	0	5	0	10
1.18 mm	---	---	---	---	---	---	---	---	0	5

After approximately two winters, it was noted that wear had begun to occur in the concrete grout material. Measurements were made and it is believed that the wear is attributed to the use of studded tires. The modifications to the extension material were made to try and get larger aggregate into the pour back material without changing the dimensions of the retrofit. The larger the aggregate the higher the resistance to studded tire wear.

DOWEL BAR RETROFIT PROCESS

According to the Special Provision for this process the following installation procedures are required:

1. Saw cut slots in the pavement as required to place the center of the dowel at mid-depth in the concrete slab. Multiple saw cuts parallel to the centerline may be required to properly remove material from the slot.
2. To prevent damage to the existing pavement any jack hammers used to break loose the concrete shall have a weight less than 14 kg (30 lbs).
3. All exposed surfaces and cracks in the slot shall be sand blasted and cleaned of saw slurry and parting compound prior to installation of the dowel.
4. Prior to placement, the dowel bars shall be lightly coated with a parting compound and placed on a chair that will provide a minimum of 13 mm (1/2 inch) clearance between the bottom of the dowel and the bottom of the slot. The dowel bars shall be placed as shown in the Plans, parallel to the centerline and parallel to the pavement surface of the lowest panel at the transverse joint. Tolerances for alignment will be within 6 mm (1/4 inch).
The chair design shall hold the dowel bar tight in place during placement of the concrete patch. Any chair design that may allow movement of the bar during the placement of the concrete patch will be rejected by the Contracting Agency.
5. Immediately prior to placement of the dowel bar and concrete patch, the Contractor shall caulk the existing transverse joint crack at the bottom and sides of the slot as shown in the Plans. The caulking shall be placed and screeded off to provide a smooth level surface. The transverse joint crack shall be caulked sufficiently to satisfy the above requirements and to prevent any of the patching material from entering the joint crack at the bottom or sides of the slot.
6. A 9.5 mm (3/8 inch) thick foam core board filler material shall be placed at the middle of the dowel to maintain the transverse joint as shown in the Plans. The filler shall fit tight around the dowel and to the bottom and edges of the slot. In addition sealant shall be placed where the core board contacts the bottom of the slot to provide a tight fit. The filler shall be capable of remaining in a vertical position and tight to all edges during placement of the patch. If for any reason the filler shifts during placement of the patch the work shall be rejected and redone at the Contractor's expense.
7. Patching material shall be consolidated by using a vibrator as approved by the Engineer.
8. The joint shall be maintained by saw cutting the surface with a hand pushed single blade saw, then sealed with a joint sealant filler conforming to the requirements of Section 9-04.2. The cut width shall be 6 mm (1/4 inch) and the depth shall be 38 mm (1-1/2 inches) deep, and the saw cut shall be 0.76 m (2-1/2 feet) long centered over the three retrofit epoxy-coated dowel bars. The joint shall be sawed within 24 hours after placement of the concrete patch.

WSDOT Standard Specification 9-04.2

Joint Sealants

Hot Poured Joint Sealants

Hot poured joint sealants shall meet the requirements of AASHTO M 173 Concrete Joint Sealer, Hot Poured Elastic Type and be sampled in accordance with ASTM D 5167. In addition, the sealant shall have a C.O.C Flash Point (AASHTO T 48) of 205°C minimum. In lieu of the specified bond test in M 173, the bond test shall be in accordance with WSDOT Test Method 412.

Two Component Poured Rubber Joint Sealer

The physical properties of the joint sealer, when mixed in accordance with the manufacturer's recommendations, shall be as follows:

1. Color: Gray or Black
2. Viscosity: Must be pourable and self-leveling at 10°C
3. Application Life: Not less than 3 hours at 22°C and 50 percent relative humidity
4. Set to Touch: Not more than 24 hours at 22°C and 50 percent relative humidity
5. Curing Time: Not more than 96 hours at 22°C and 50 percent relative humidity
6. Non Volatile Cement: Not less than 92 percent
7. Hardness Rating (Durometer "Shore A"): 5-35
8. Resiliency: Not less than 80 percent
9. Bond test methods shall be in accordance with WSDOT Test Method No. 412

Suitable primer, if required by the manufacturer, shall be furnished with each joint sealer. The primer shall be suitable for brush or spray application at 10°C to pour the joint within 24 hours. It shall be considered as an integral part of the sealer system. Any failure of the sealer

in the test described herein, attributable to the primer, shall be grounds for rejection or re-testing of the sealer.

Acceptance of joint sealing compound for use on a project shall be on the basis of laboratory tests of samples representative of each batch of material to be used on the job. A period of at least two weeks shall be allowed for completion of tests. Each container of the compound shall be clearly identified as to batch number.

9. Refer to Figure 9 for detail plan view of dowel bar placement.

To date, WSDOT has dowel bar retrofitted approximately 225 lane kilometers (140 lane miles) of existing concrete pavement (refer to Table 1 for summary of bid items). An additional 210 lane kilometers (130 lane miles) will be retrofitted by the end of the 1996-1997 construction season. The future plans, beyond the 1997 construction season, will be to rehabilitate approximately 80 lane kilometers (50 lane miles) per biennium of concrete pavement.

CONCLUSION

In previous years, WSDOT has either delayed the rehabilitation of its concrete pavements or has placed thick asphalt overlay to improve the driving surface. Based on the experience of Puerto Rico and WSDOT, dowel bar retrofit can be considered a viable option for the rehabilitation of faulted concrete pavements. At this point in time, retrofitted dowel bars appear to be the most cost effective option for restoring load transfer to the faulted PCC pavements in Washington state. It is estimated that the total cost for dowel bar retrofit, including grinding, would be approximately \$155,000 per lane kilometer (\$250,000 per lane mile) and the cost of a 100 mm (4 inch) asphalt overlay would be approximately \$236,000 per lane kilometer (\$380,000 per lane mile). Keeping in mind that the dowel bar retrofit is applied to only the faulted right lane, while the asphalt overlay must be placed over the entire roadway section (including shoulders).

The first PCC rehabilitation project in Washington state has been in service for approximately 5 years and the results support the use of retrofit dowel bars for load transfer restoration. Based on the initial analysis and the performance obtained in Puerto Rico, the expected performance life of the retrofitted dowel bars is estimated to be 10 to 15 years.

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Table 1. Summary of Dowel Bar Retrofit and Diamond Grinding Costs

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Contract	Dowel Bar Retrofit Cost	Dowel Bar Quantity (Each)	Diamond Grinding Cost	Grinding (sq m)	Quantity (SY)	Length Retrofitted (lane km)	(lane mi)
4235	\$34.40	61,698	\$3.10	248,740	297,500	53.1	33.0
4340	\$44.00	12,225	\$3.40	20,581	24,615	5.8	3.6
4616	\$24.50	24,220	\$3.80	134,997	161,460	18.4	11.5
4706	\$36.00	10,850	\$4.50*	43,787	52,370	8.5	5.3
4902	\$33.00	84,265	\$3.82	490,598	586,770	69.7	43.3
5009	\$35.50	107,990	\$3.75	246,369	294,664	86.4	53.7
5122	\$42.00	15,372	\$6.00	50,680	60,615	13.8	8.6
5193	\$35.00	24,536	\$6.15	145,500	174,022	18.5	11.5

*price reflects removal of grinding slurry and depositing off site

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 Figure 7. Fault Measurements - Section C Tied Shoulder Only
 Figure 8. Fault Measurements - Section D Control
 Figure 9. Detail Plan View of Dowel Bar Placement

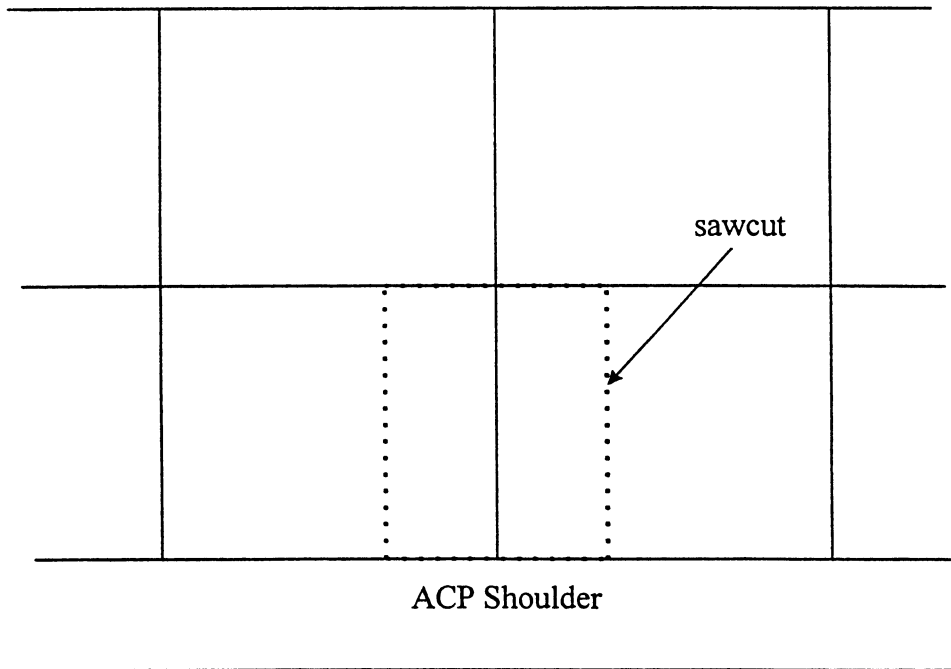


Figure 1. Sawcut Detail for Panel Removal.

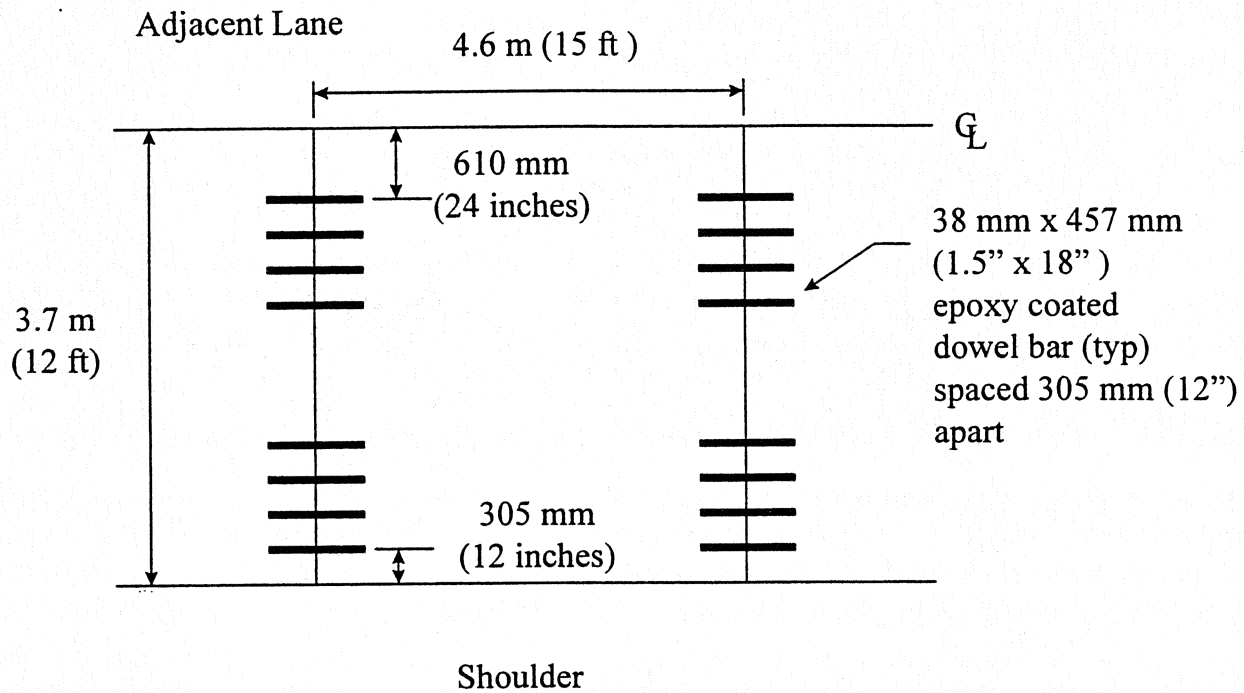


Figure 2. Dowel Bar Retrofit Layout.

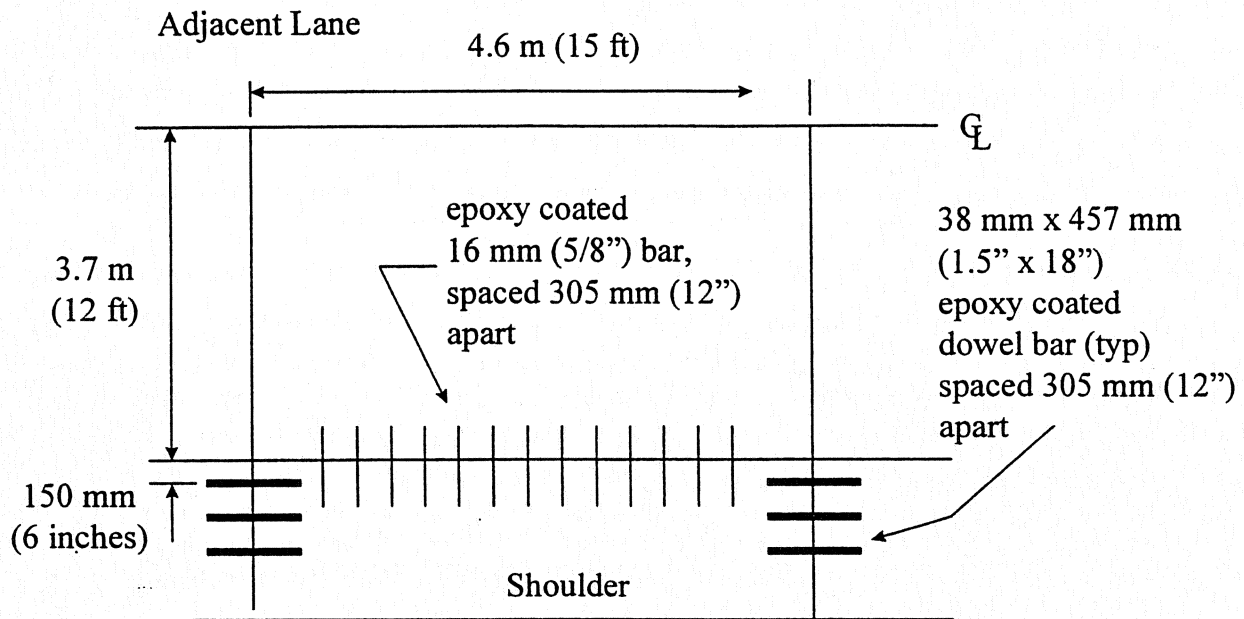


Figure 3. Tied Shoulder Layout.

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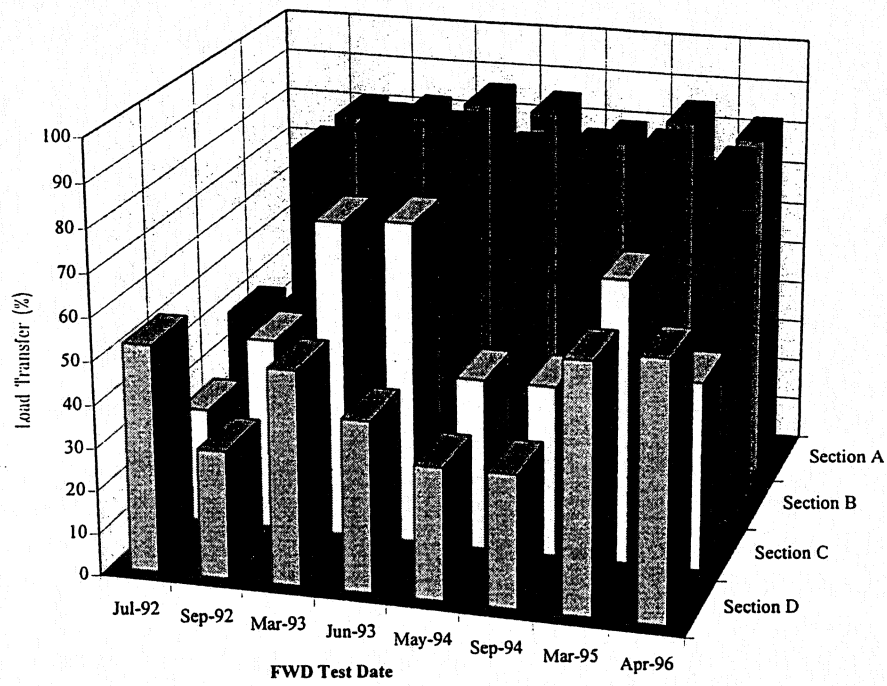


Figure 4. Joint Load Transfer Efficiencies.

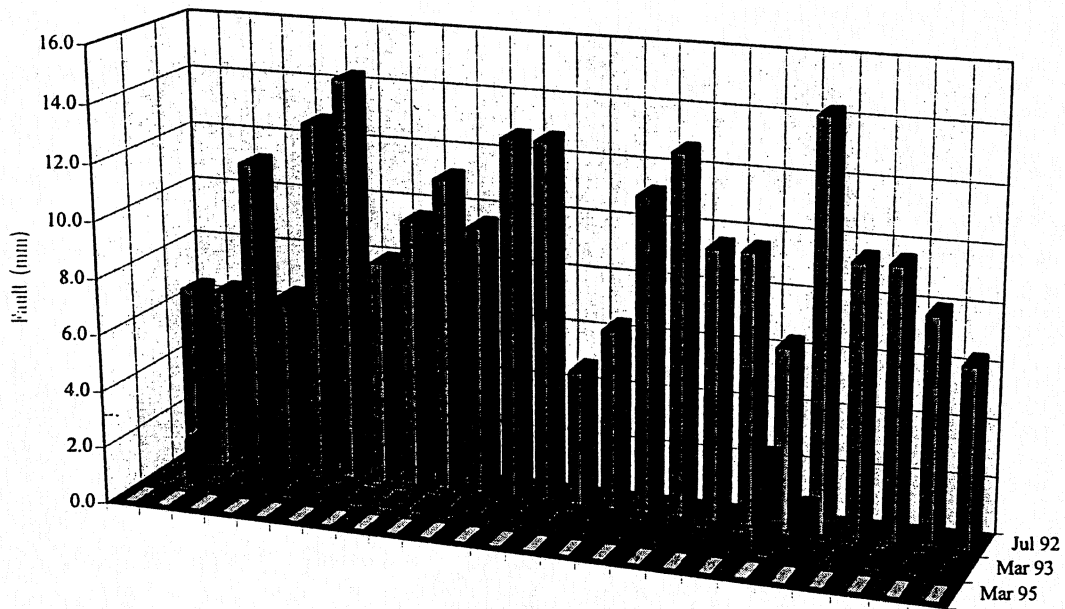


Figure 5. Fault Measurements - Section A Dowels Bars Only.

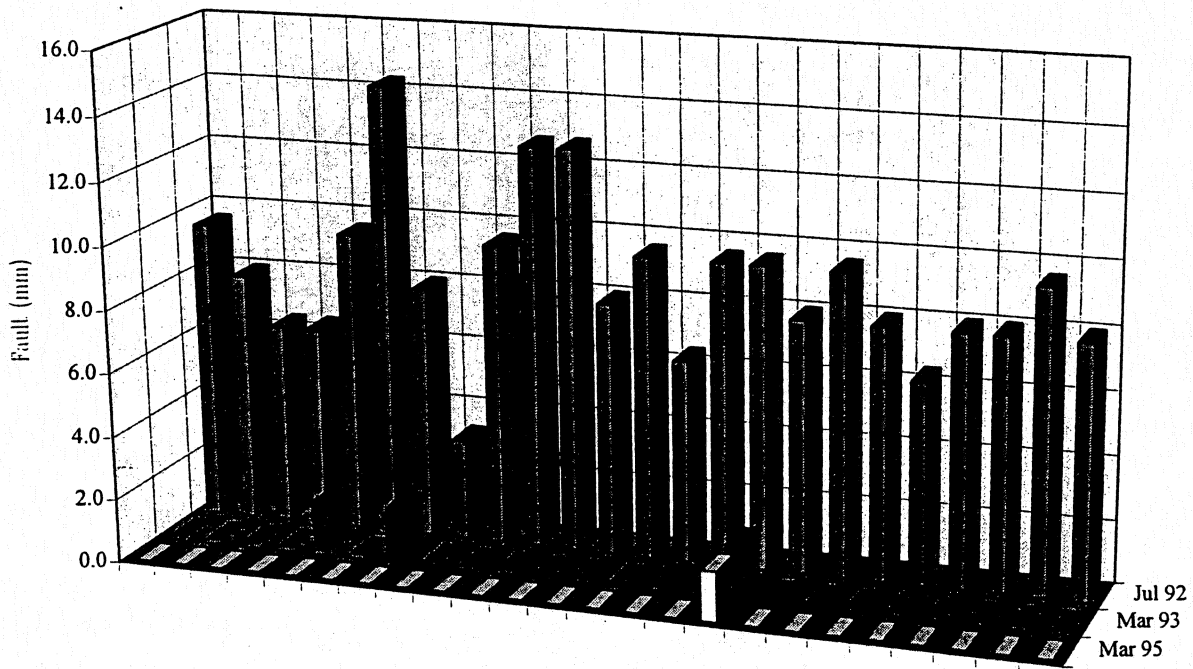


Figure 6. Fault Measurements - Section B Dowel Bars and Tied Shoulder.

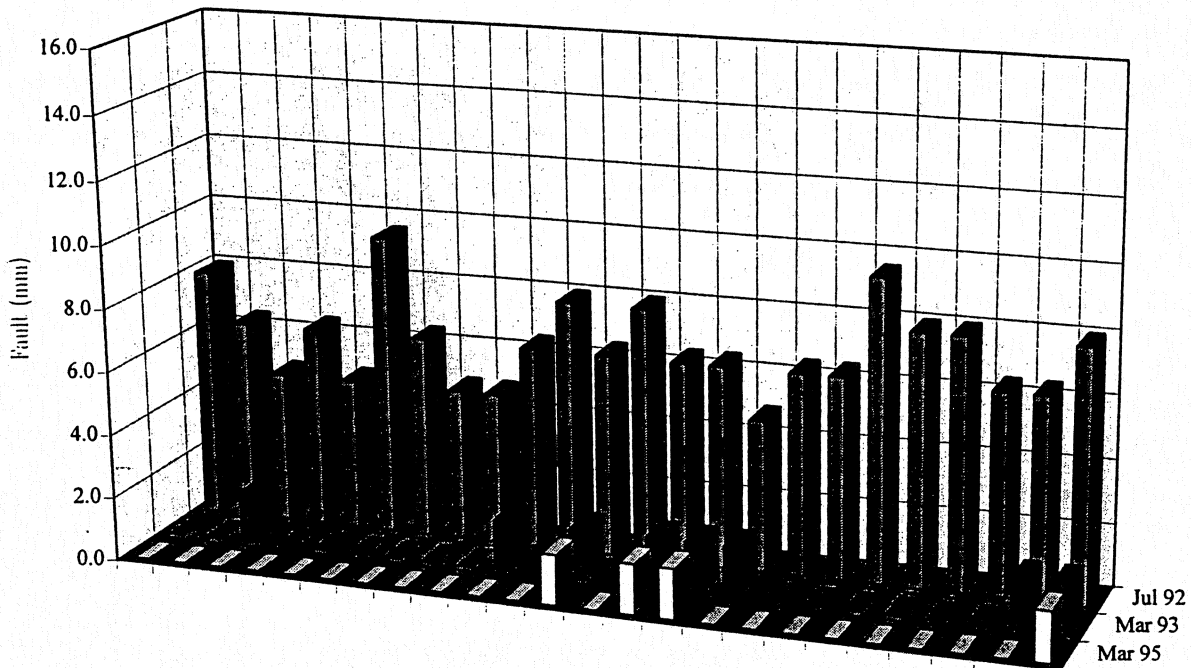


Figure 7. Fault Measurements - Section C Tied Shoulder Only.

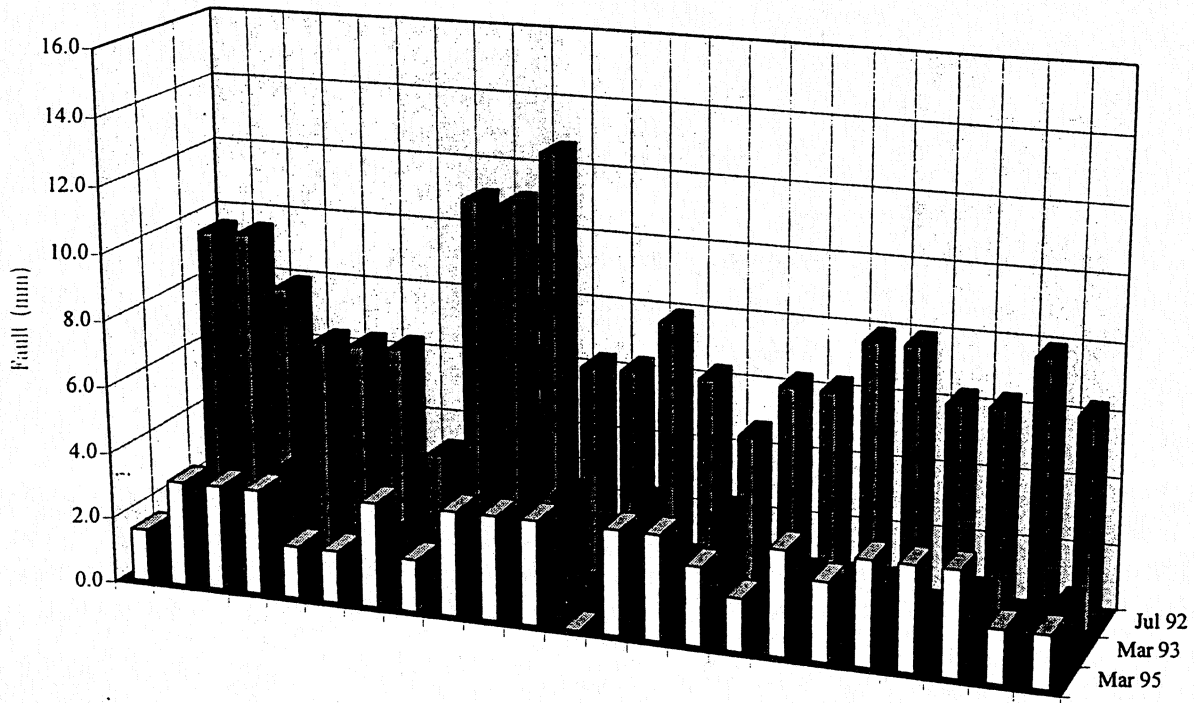


Figure 8. Fault Measurements - Section D Control.

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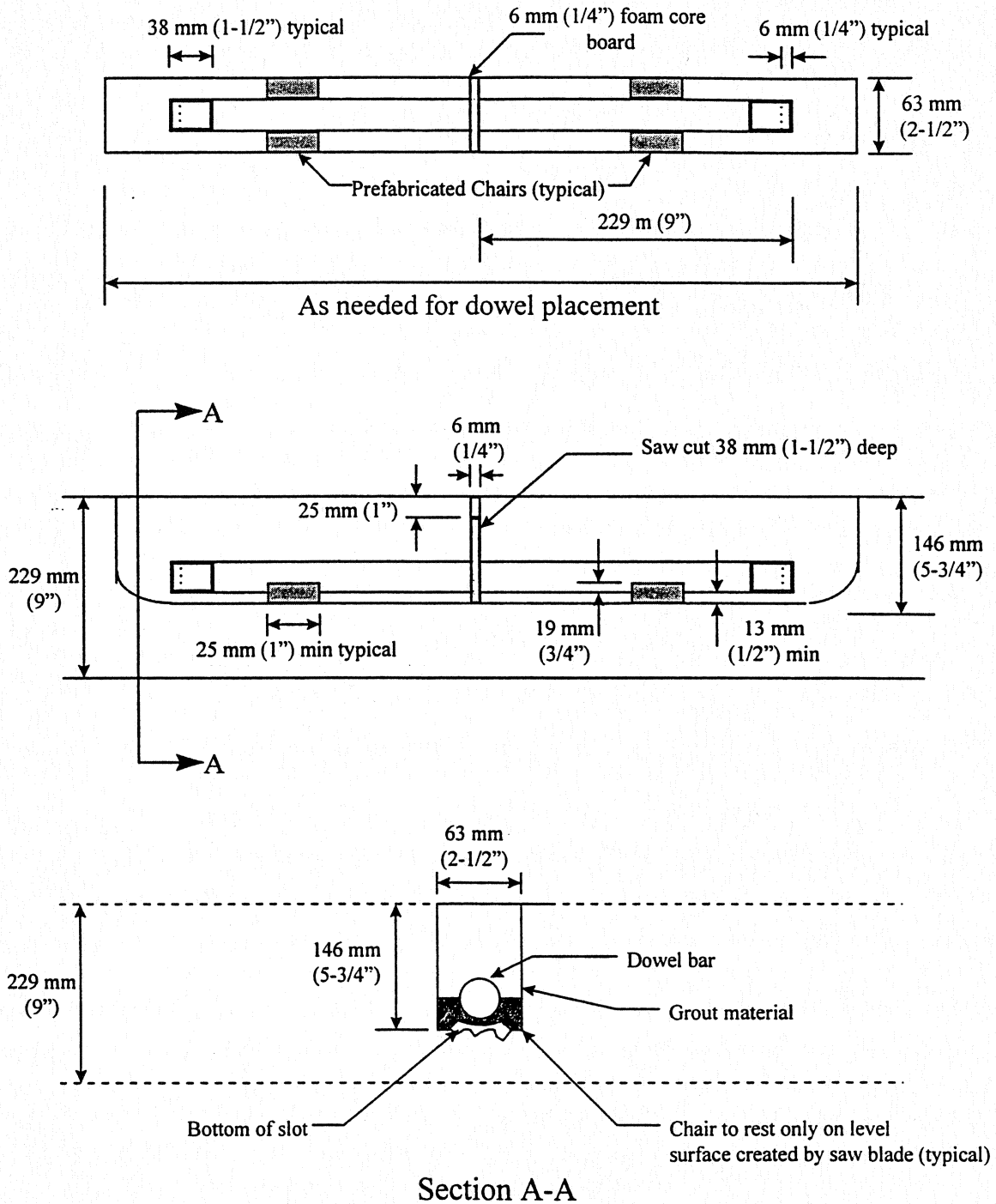


Figure 9. Detail Plan View of Dowel Bar Placement

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