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Minne-ALF Project Overview
and
Retro-Fit Dowel Study Results

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A laboratory-based linear loading pavement test stand, the Minnesota Accelerated Loading Facility (Minne-ALF) simulates the passage of heavy traffic loads moving at speeds up to 65 kph (40 mph) over small, full-scale pavement test slabs. Hydraulic actuators control a rocker beam, which simulates loads. Researchers simulated the passage of 40-KN (9-kip) single-wheel loads at a rate of 172,000 per day, although wheel loads up to 100 kN (22 kips) can be simulated at varying speeds. Full-axle simulations are possible with frame modifications.

Concrete slabs were cast and dowels were installed in slots across crack/joints. Test variables included joint fact texture, repair backfill material, and dowel material and length. Test outputs included measurements of load transfer efficiency and differential deflection across the joint/crack.

The effect of joint/crack face texture was great when the joint/crack remained tight. Load carrying performance was improved using Speed Crete 2028 in place of 3U18 concrete backfill with similar joint and dowel bars. Load transfer was unaffected by the use of stainless steel-clad dowel bars in lieu of epoxy-coated dowel bars. Researchers recommend additional testing to examine the effects of dowel length and dowel materials

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"Minne-ALF Project Overview and Retro-Fit Dowel Study Results"

Final Report

Development of an Accelerated Load Test Platform for Pavements

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EXCECUTIVE SUMMARY

The Minnesota Accelerated Loading Facility (Minne-ALF) is a laboratory-based linear loading pavement test stand that simulates the passage of heavy traffic loads moving at speeds up to 65 kph (40 mph) over small full-scale pavement test slabs. Moving wheel loads are simulated using a rocker beam that is controlled by hydraulic actuators. The rocker beam is loaded to 40 kN (9,000 lb) in one direction with the load reduced to 8.9 kN (2,000 lb) in the return direction. This direction-dependent loading is intended to simulate the directional nature of traffic over a given lane of highway pavement. While this directional loading is probably not necessary for structural testing purposes, it will be necessary for the simulation of moisture movement under pavement slabs in future pumping studies. For structural evaluation studies, the test program can be modified to allow loading of the rocker beam in both directions, if desired. The minimum load of 8.9 kN (2,000 lb) is maintained on the vertical actuator at all times to ensure constant contact between the rocker beam and test slab, thereby eliminating the possibility of unintentional impact loads that might otherwise result from loss of beam-slab contact during reversals of rocker direction.

In this study, each rocker cycle required 0.5 seconds, resulting in a loading frequency of 2.0 Hz and the simulation of up to 172,800 40-kN (9-kip) wheel load passages per day. Full-axle simulations are possible with frame modifications.

Demonstration tests were performed to: 1) determine the effects of selected design and construction variables on retrofit dowel load transfer system performance; 2) determine the variability of Minne-ALF test results; 3) demonstrate the general usefulness of the Minne-ALF; and 4) identify the need for any additional test system modifications. Concrete slabs specimens were cast and dowels were installed in slots across cracks and formed joints. Test variables included joint face texture, repair backfill material, and dowel material and length. Test outputs included measurements of load transfer efficiency (LTE) and differential deflection across the joint/crack.

All test slabs were 19-cm (7.5-in) PCC with 38.1 cm epoxy-coated dowels retrofit across smooth-faced joints, except as noted below: 1) 3U18 PCC backfill, tight crack with grain interlock; 2) 3U18 PCC backfill; 3) 3U18 PCC backfill (replicate of slab 2); 4) Speed Crete 2028 PCC backfill; 5) Speed Crete 2028 PCC backfill, 33.0-cm long epoxy-coated dowel bars; 6) Speed Crete 2028 PCC backfill, 46-cm long stainless steel-clad dowel bars.

The following conclusions and recommendations were made based on the laboratory load simulation:

- It appears that the Minne-ALF is a useful tool for evaluating the relative performance of rigid pavement designs and design features. It provided comparable test results for the two replicate specimens that were tested and it indicated significantly improved performance when load transfer was provided by both retrofit dowels and very good grain interlock.
- The LTE and differential deflection histories for the slab containing Speed Crete 2028 concrete backfill is greatly improved over those measured for test slabs containing 3U18 concrete backfill with similar joints and dowel bars. This could be due in part to the higher initial strength of Speed Crete 2028 concrete backfill.
- At this time the effect of dowel bar length could not be properly determined due to poor consolidation of the backfill material used under the 38-cm retro-fit dowel bars. The effect of poor consolidation on LTE and differential deflection appears to be much larger than the effect of relatively small changes in dowel length. Additional tests should be performed to examine the effects of dowel length and dowel materials on the performance potential of retrofit (and original construction) load transfer systems.
- It appears that the load carrying capabilities of the slab were unaffected by the use of stainless steel-clad dowels in lieu of epoxy-coated structural steel dowels.
- Minor modifications and standard maintenance practices are recommended to allow the Minne-ALF to run more effectively (see Chapter 5).
- 3U18 concrete backfill sack mix should no longer be used due to large inconstancies between sacks. This material should be obtained by manually proportioning the 3U18 concrete mixes.

CHAPTER 1
INTRODUCTION

1.1 Problem Statement

Research concerning the performance of new pavement materials and construction techniques has traditionally been accomplished using small-scale laboratory tests and/or field trials. Unfortunately, small-scale lab tests often measure only basic material properties and fail to provide adequate indications of probable field performance. Field trials generally provide good indications of performance potential, but long-term performance potential is often not apparent for many years, resulting in either implementation delays or the construction of pavements that may fail prematurely because their designs are based on short-term performance data.

There is a need to develop accelerated pavement test facilities that rapidly accumulate loads on test pavements to allow the estimation of long-term results in a short time. This would allow the more rapid trial and development of new materials, designs and construction techniques that can eventually be implemented in the field with minimal risk of premature failure.

1.2 Research Objective and Approach

The objective of this project was the development, construction and demonstration of a Minnesota Accelerated Loading Facility (Minne-ALF) for rapidly testing pavements to determine the effects of important design, construction and environmental parameters on pavement performance. The test stand demonstration included tests of a typical Minnesota portland cement concrete pavement design constructed on a composite foundation (natural base and soil over artificial foundation matting). Demonstration test variables included the use of various types and sizes of dowel bars (retrofit across transverse cracks and joints in the test slabs) and the use of different types of backfill material for the dowel slots.

1.3 Benefits

The results of these tests provide Mn/DOT with a good indication of the ability of the Minne-ALF to perform accelerated load testing of concrete pavement systems. The proposed demonstration tests will also provide Mn/DOT with an early indication of the performance potential of dowels retrofit across transverse cracks, and the relative performance potential of different construction materials and designs for this type of concrete pavement repair.

CHAPTER 2
DESCRIPTION OF THE TEST PROGRAM

2.1 Retrofit Load Transfer - Background

Jointed portland cement concrete (PCC) pavement stresses and deflections are decreased when loads are "shared" by adjacent slabs. This sharing or transfer of loads across joints or cracks is generally accomplished through the grain interlock that is present at the fractured joint/crack face and/or by means of a mechanical device, such as a steel dowel bar. Failure to provide good load transfer across joints and cracks can result in the rapid development of many types of pavement distress, including pumping or water bleeding, faulting, spalling and loss of ride quality.

While dowels are typically provided during construction at most PCC pavement transverse joints, they can also be retrofit across transverse cracks that sometimes develop and deteriorate with time. In addition, they can be retrofit at transverse joints where dowels were not originally provided during construction or where the original dowels have deteriorated and are no longer functioning properly. This process is generally accomplished by cutting slots at appropriate locations in the pavement surface, inserting one dowel per slot, and backfilling the slot with a rapid-setting concrete mixture.

Details concerning the process of retrofitting dowels in this test program are provided in later sections of this report.

2.2 Test Frame and Control System

2.2.1 Original Test Stand

The Minne-ALF was initially developed with independently-controlled actuators positioned vertically at opposite ends of a 2.7-m (9-ft) W10 x 68 steel beam. A circular segment of aluminum (radius = 59.3 m [194.4 ft]) was bolted to the bottom of the beam so that it functioned as a rocker capable of simulating the passage of a moving wheel contact area. Complete construction drawings of the original test frame are presented in Appendix A.

Unfortunately, it was not possible to achieve reasonably uniform load profiles (i.e., constant load magnitude regardless of contact area position on slab) at the desired simulated vehicle speed (88 kph [55 mph]) with this test stand configuration. After consultation with MTS staff, the problem was attributed to "cross-coupling" of the actuators, which is to say that the constantly varying load applied by either actuator in rocking the beam produced a constantly varying reaction against the other actuator. The actuator control system was unable to compensate for the rapid variations in reaction loads while attempting to maintain programmed load profiles, resulting in fluctuations in the load applied to the test slab through the contact patch. The magnitude of variation increased with simulated vehicle speed; reasonable load profiles could not be obtained at simulated vehicle speeds exceeding approximately 17 kph (11 mph). It was determined that the test stand and controls required modification to achieve the desired levels of performance.

2.2.2 Test Stand Modifications

As described previously, it was necessary to modify the original Minne-ALF so that it could more accurately simulate the movement of heavy vehicle loads on test pavements at highway speeds. The following sections briefly describe the modifications that were performed under Task 1A of this project.

Rocker Beam/Hydraulic Actuator Configuration

The rocker beam/actuator arrangement was modified to eliminate the "cross-coupling" difficulties that were apparently present in the original system. A short section of W10 x 60 steel beam was attached to the center of the original rocker beam to form a "tee." One actuator was attached vertically to this short section and connected to an overhead reaction beam. This actuator was operated in load control and applied the programmed load profile to the rocker beam.

The other actuator was oriented horizontally in the same plane as the rocker beam; it also was attached to the top of the short beam section and to a reaction beam that was added at one end of the frame. This actuator was operated in stroke control to push the rocker beam back and forth. Figure 2.1 is a photograph of the modified rocker beam/actuator design.

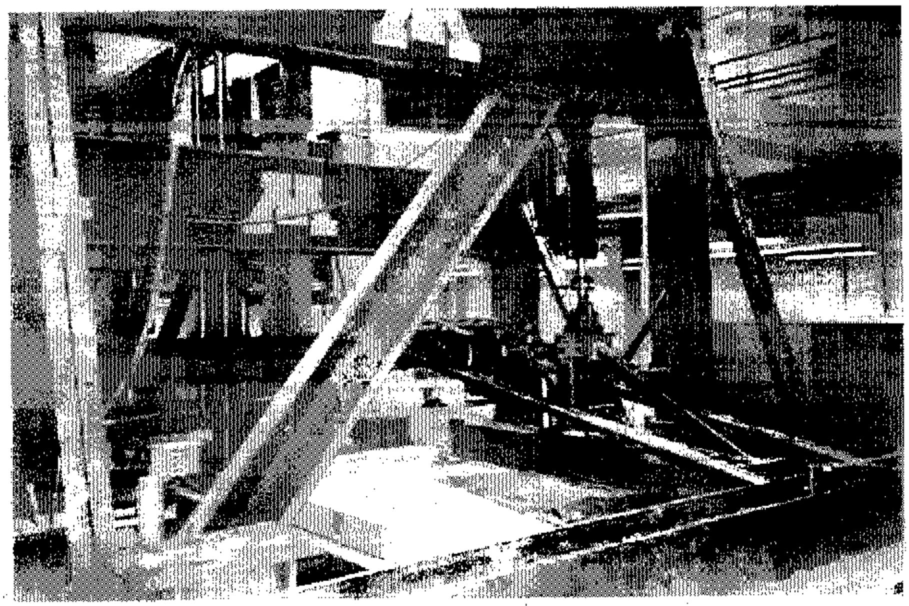

Figure 2.1. Photo of modified ALF rocker beam/actuator design.

The new rocker beam/actuator configuration necessitated the addition of several stiffeners and other test frame modifications. These are described in the following subsection.

Minne-ALF Frame Modifications

The "shakedown" operation of the Minne-ALF suggested that the implementation of several frame modifications might improve the operating efficiency of the test frame. The following is a summary of the test frame modifications that were implemented prior to the testing of additional PCC slabs.

- Modifications due to rocker beam/actuator modifications described previously:
  1. relocation of four columns and one overhead reaction beam
  2. addition of two 0.629-m (2.06-ft) W12 x 72 steel columns
  3. addition of two 3.1-m (9.9-ft) W12 x 72 horizontal reaction beams
  4. addition of two sets of double angle braces to reduce deflections in horizontal reaction beams

- Modifications to reduce vibration and increase the resonant frequency of test frame:
  5. addition of lateral bracing to columns
  6. addition of a diagonal "knee brace" to reduce deflections in the overhead reaction beam

- Modifications to control movement of rocker beam:
  7. addition of a two-bearing roller to the end of the rocker beam and addition of a steel plate "hinge" or guide path to the horizontal reaction beam to ensure that the horizontal actuator produces a rocking motion and not a sliding motion
  8. addition of a steel-wheel guide roller connected to an arm mounted on the column near the center of the rocker beam to reduce the lateral movement of the rocker beam produced by surface irregularities present on the PCC slab

A photograph of the modified test frame is presented in figure 2.2 (2). Complete construction drawings of the modified test frame are presented in Appendix B.

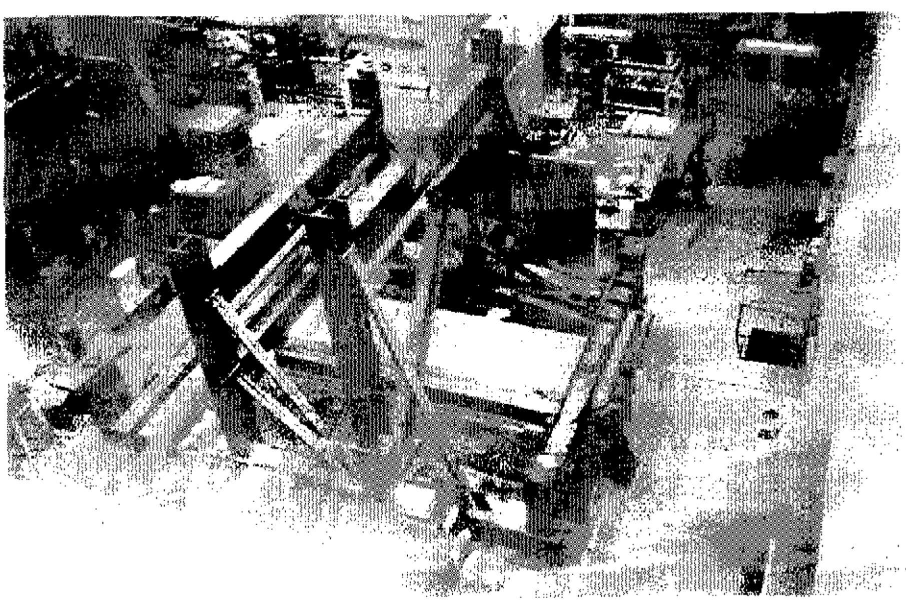

Figure 2.2. Photo of modified ALF test stand.

Modifications to Hydraulic Power Supply System

A close-coupled servovalve and accumulator assembly was added to each actuator in an effort to improve fluid flow rates, reduce pressure spikes and increase the rates at which the actuator pistons can change directions.

A hydraulic service manifold (HSM) was installed to improve the flow control and shut off capabilities of the hydraulic system. The HSM allows the hydraulics for the Minne-ALF to be shut down (in case of an emergency or hydraulic interlock condition) without shutting down the main hydraulic pump, which would disable other testing programs in the laboratory. Other test hydraulic testing operations in the lab have been fitted with HSMs as well, which prevents the Minne-ALF from being shut down inadvertently by hydraulic interlock and emergency shut down conditions at other lab test sites.

Computer Software and Hardware Upgrades

After the new actuator/rocker beam configuration was adopted, it was discovered that the original MTS TestStar software (which was several years old) was not capable of running the new actuator configuration. Attempts were made to upgrade the software through the addition of an advanced function generator module, which would allow the actuators to follow a repeated predetermined wave pattern in either stroke or force control mode.

After much testing, it became apparent that a more current version of the MTS TestStar software would be required to control the system. This software upgrade required an upgraded Pentium-class computer operating under Windows NT and a new processor board for the MTS TestStar system. The software upgrade also necessitated rewriting the TestWare program and TestStar configuration files. All of this was accomplished under Task 1A.

2.2.3 Load Simulation Capabilities

The Minne-ALF is currently configured to simulate the movement of a 40-kN (9,000-lb) wheel load along a path length of up to 2.75 m (9 ft) over a test pavement measuring up to 3.7 m by 4.6 m (12 ft by 15 ft). It is currently being operated with an average simulated wheel speed of 44 kph (27 mph), although the peak speed as the load crosses the center of the test area approaches

65 kph (40 mph). These speeds were selected because they allow rapid testing without significant loss of load control. Higher speeds can be simulated with some sacrifice in the accuracy of the load magnitude. Load magnitudes of up to 100-kN (22 kips) can be applied; actuator capacity is the primary limitation on load magnitude. A 6.4-mm (0.25-in) neoprene pad was placed below the rocker beam during testing to minimize the effects of pavement surface irregularities on the load profiles (i.e., to smooth the actual load profiles) and movement of the rocker beam.

The tests described in this report were performed with the rocker beam being loaded to 40 kN (9,000 lb) in one direction and with the load reduced to 8.9 kN (2,000 lb) in the return direction. This direction-dependent loading is intended to simulate the directional nature of traffic over a given lane of highway pavement. While this directional loading is probably not necessary for structural testing purposes, it will be necessary for the simulation of moisture movement under pavement slabs in later pumping studies. For structural evaluation studies, the test program can be modified to allow loading of the rocker beam in both directions, if desired. The minimum load of 8.9 kN (2,000 lb) is maintained on the vertical actuator at all times to ensure constant contact between the rocker beam and test slab, thereby eliminating the possibility of unintentional impact loads that might otherwise result from loss of beam-slab contact during reversals of rocker direction.

Figure 2.3 illustrates the load and stroke profiles that were used to control the magnitude and speed of the simulated moving wheel load. Each rocker cycle required 0.5 seconds, resulting in a loading frequency of 2.0 Hz and the simulation of up to 172,800 40-kN (9-kip) wheel load passages per day.

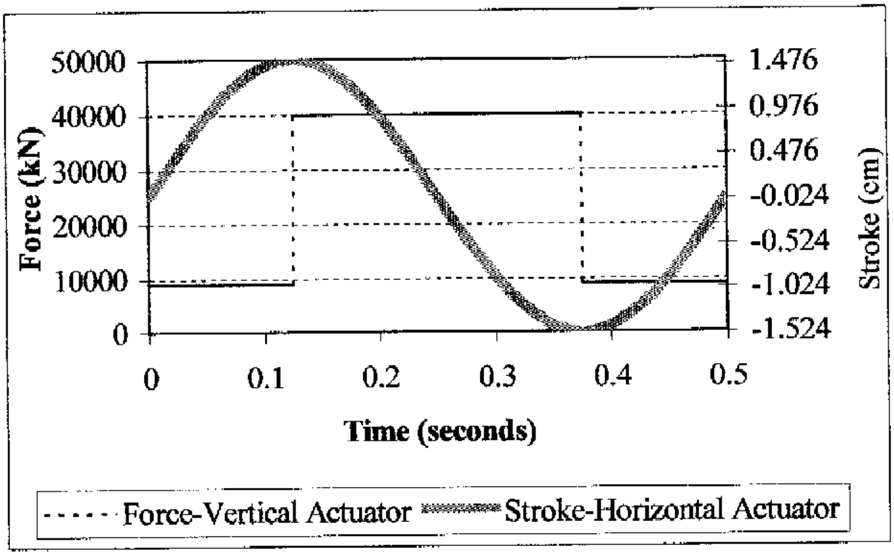

Figure 2.3. Actuator load and stroke profiles for current test program.

2.3 Foundation

A composite foundation was prepared to model the foundation present under test section 6 at the Minnesota Road Research Project (Mn/ROAD). The foundation was comprised of 13 cm (5 in) of Mn/DOT class 5 crushed stone base over a 23-cm (9-in) layer of clay-loam obtained from the Mn/ROAD test site. These materials were placed over a 6.4-mm (0.25-in) layer of neoprene (which simulates the support provided by deeper foundation layers) which, in turn, rests on a rigid steel plate and beam bedding. The natural materials were compacted at moisture contents that were 0.5 percent above optimum.

2.4 Data Collection

MTS TestWare software allows the collection of data from both internal sources (e.g., load, stroke, etc.) and external sources (e.g., linear variable differential transducers (LVDTs), thermocouples, etc.). The program written for the current Minne-ALF test program collects load and stroke data from the hydraulic actuators, as well as slab deflection data produced by the LVDTs that are mounted over the slab joint or crack (as described below). These data sources are sampled at a rate of 400 Hz at predetermined cycle counts and are saved in an ASCII data file.

Estimates of joint/crack load transfer are currently measured using two LVDTs placed in the vicinity of the joint/crack and 2.5 cm (1 in) from the edge of the slab. The LVDTs were originally mounted on a short cantilever arm that was attached to the test frame near the center of the rocker beam. However, significant "noise" observed in the data collected from slab 1 was attributed to vibration of the mounting frame. Prior to the testing of slab 2, a different LVDT stand was designed (as a part of Task 1A) to minimize vibration effects and to more accurately measure differential deflections between the approach and leave side of the slabs. This stand was bolted to the approach side of the slab with an extended bracket to allow an LVDT to rest on the leave side of the joint, thereby allowing direct measurement of differential deflections with a single LVDT. The original LVDT stand was used for the second LVDT, but was stiffened to reduce vibration. This stand was attached to the central reaction column; the attached LVDT measures the overall deflection of the leave side of the joint/crack. These two LVDT stands are illustrated in figures 2.4 and 2.5.

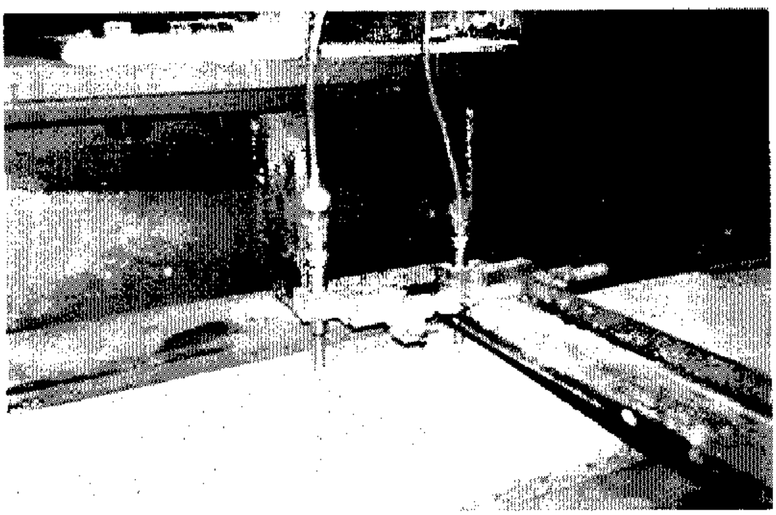

Figure 2.4. Original LVDT mounting bracket.

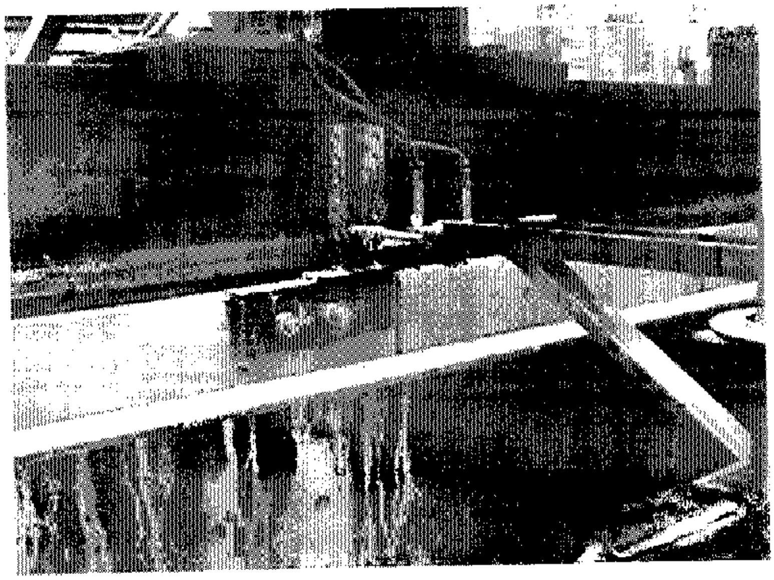

Figure 2.5. Modified LVDT mounting brackets.

The LVDTs used in the current test program are capable of measuring deflections over a range of ±6.4 mm (0.25 in) with an accuracy of 0.001 mm (0.00004 in). LVDT voltage outputs are processed by a signal conditioner box before being sent to the TestStar, which converts the voltages to deflection readings.

2.5 Test Specimens

2.5.1 Fabrication of Portland Cement Concrete Test Slabs

Six PCC slabs measuring approximately 4.6 m (15 ft) in length, 1.8 m (6 ft) in width and 19 cm (7.5 in) thick were tested in the Minne-ALF under Task 2. A standard paving mix design (Mn/DOT 3A41) was obtained from the Minnesota Department of Transportation and used for each slab. Mix proportions for the test slabs are presented in table 2.1.

The PCC test slabs were cast and moist-cured for a period of seven days, in accordance with ASTM C 192 "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory." The air content and slump of the plastic mixes was measured using the Press-R-

Meter and standard slump cone, respectively. The results of these tests are also presented in table 2.1.

Table 2.1. Mix proportions and air content and slump measurements for test slabs.

Slab No.	Coarse Aggregate, kg/m^3	Fine Aggregate, kg/m^3	Cement, kg/m^3	Hydration Water, kg/m^3	A/E, mL/m^3	WR, mL/m^3	W/C Ratio	Air Content, %	Slump, mm
1	1119	760	352	132	136		0.38	4.25	70
2	1068	779	349	160	136		0.46	6.2	127
3	1116	783	354	148	136	384	0.42	6.8	121
4	1101	745	350	165	151		0.47	8.5	191
5	1116	760	352	164	128	407	0.47	7.8	165
6	1068	787	357	167	136		0.47	7.2	165

Companion strength specimens were cast (in accordance with ASTM C 192 procedures) at the same time as the test slabs. Three modulus of rupture test beam specimens (150 mm x 150 mm x 610 mm [6 in x 6 in x 24 in]) and three cylindrical (150 mm x 300 mm [6 in x 12 in]) 28-day compressive strength specimens were cast along with test slab 1. Twelve cylindrical (150 mm x 300 mm [6 in x 12 in]) compressive strength specimens (for testing after 1, 3, 7 and 28 days of curing) were cast along with all other test slabs. The modulus of rupture and compressive strength tests were performed in accordance with ASTM C 78 "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" and ASTM C 39 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," respectively. In addition, the static modulus of elasticity was determined in accordance with ASTM C 469 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression" prior to performing the compression tests on the 12 cylinders cast along with test slabs 2 through 6. Table 2.2 summarizes the strength data for the six task 2 test slabs.

Table 2.2 Strength data for test slab mixes.

Slab Number	Specimen Number	Compression Strength (MPa)				Static Modulus of Elasticity (GPa)			
		Days of Curing							
		1	3	7	28	1	3	7	28
1	1				29.3				
	2				30.5				
	3				31.0				
	Avg.				*30.3*				
2	1	8.3	16.9	24.7	32.3	14.4	19.8	23.0	26.8
	2	9.1	17.2	25.6	33.8	14.9	19.5	24.3	26.8
	3	7.7	16.4	25.3	32.5				
	Avg.	*8.4*	*16.8*	*25.2*	*32.9*	*14.7*	*19.7*	*23.7*	*26.8*
3	1	23.3	25.6	32.1	39.7			25.7	27.4
	2	22.6	29.2	33.6	39.4			24.4	27.2
	3	22.5	25.5	31.9	39.9			26.0	27.8
	Avg.	*22.8*	*26.8*	*32.5*	*39.7*			*25.4*	*27.5*
4	1	9.4	13.3	23.6[a]	27.7	12.6	16.9	22.0[a]	23.8
	2	9.5	13.7	21.9[a]	26.3	12.7	17.4	21.3[a]	23.4
	3	9.7	13.4	21.6[a]	26.7	13.0	16.8	22.2[a]	23.6
	Avg.	*9.6*	*13.5*	*22.4[a]*	*26.9*	*12.8*	*17.4*	*21.9[a]*	*23.6*
5	1	8.3	16.1	22.6	32.6	10.3	18.4	22.6	25.0
	2	8.1	16.4	23.2	32.9	11.4	19.0	21.7	25.8
	3	8.2	17.4	23.2	32.7	11.4	16.9	21.9	25.0
	Avg.	*8.2*	*16.6*	*23.0*	*32.7*	*11.0*	*18.1*	*22.1*	*25.2*
6	1	8.3	17.4	23.8	35.2	11.1	18.8	20.2	26.7
	2	10.7	16.6	23.0	34.0	12.2	16.9	21.1	25.5
	3	7.9	16.8	25.8	35.9	10.1		22.8	26.0
	Avg.	*9.0*	*16.9*	*24.2*	*35.0*	*11.1*	*17.9*	*21.4*	*26.1*

[a] Testing performed at 14-days of curing.

Slab 1

Test slab 1 was cast at a contractor's yard in St. Michael, Minnesota. This test specimen was transported to the University of Minnesota laboratory (while still in the steel channel casting frame), where it was placed on the Minne-ALF composite foundation. A thin layer of sand was spread on the foundation surface to help remove foundation surface irregularities and to provide more uniform support to the test slab. The test slab was positioned so that the center of the rocker beam would sit 15.2 cm (6 in) from the edge of the slab and would be centered at the midpoint of the test slab (2.3 m [7.5 ft] from either end).

After the test slab had been placed on the test stand foundation, it was apparent that there were still numerous areas of nonuniform support. This was due, at least in part, to irregularities in the casting foundation surface at the contractor's yard. To correct this situation, an attempt was made to fill the voids between the slab and foundation by injecting a cement-based grout under pressure. A homemade pressure injection system was fabricated using a lawn chemical sprayer. A highly fluid cement grout was prepared using three parts flyash, one part cement, and two parts water (by weight). A funnel with a capacity of 1000 mL (33.8 fl oz) and an outlet orifice diameter of 35 mm [1.4 in] was used to test the fluidity of the cement grout mixture. The cement grout mixture passed through the funnel in nine seconds, while the same volume of water passed through the funnel in 7.5 seconds.

A total of six holes were drilled in the slab surface to provide ports for grout injection. One hole was drilled on each side of the crack at approximately the center of the slab; the other four were drilled at various locations along the edge of the slab at a distance of approximately 30 cm (1 ft) from the slab edge. Figure 2.6 illustrates the location of the grout injection holes.

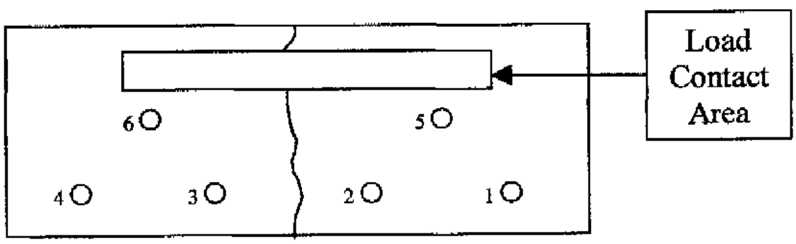

Figure 2.6. Slab 1 grout injection hole locations.

Holes one, two and three accepted grout readily. Hole one was injected twice until grout ran from the end of the slab. Grout immediately ran out the side of the slab when hole 2 was injected. Holes four, five and six did not accept significant quantities of grout. A partially successful attempt was also made to inject grout from the side of the slab at the slab-foundation interface.

The grout injection of the slab was not considered completely successful. Upon completion, there still appeared to be large voids along and under the slab, and the slab did not appear to be well seated during testing.

Slabs 2 through 6

After considering the apparent problems with uniformity of foundation support in slab 1, slabs 2 through 6 were cast in place. Debris from the demolition of the previous test slab was removed, and a sheet of 3-mil plastic was laid on the foundation. The steel casting frame was placed on the base, assembled and positioned such that the center of the rocker beam would sit 15 cm (6 in) from the edge of the slab and 2.3 m [7.5 ft] from either end. The form sides were tied together using 6.4-mm (0.25-in) diameter threaded rod to prevent distortion of the forms during concrete placement and consolidation.

2.5.2 Formation of Joint/Crack

One goal of this study was to determine the effects of different crack/joint faces on the load transfer efficiency of retrofit dowel installations. For slab 1, the dowels were retrofit across a transverse crack, thereby allowing aggregate interlock to supplement the load transfer provided by the retrofit dowel bars. For slabs 2 through 6, the dowels were retrofit across full-depth smooth transverse joints, where any load transfer observed would be due solely to the retrofit dowel bars. The following subsections detail the formation of the crack and joints.

Slab 1

The transverse crack was formed using the Minne-ALF. The arc was removed from the bottom of the rocker beam and a metal bearing plate measuring 15 cm (6 in) wide and 30 cm (12 in) long was placed at the longitudinal center of the slab, near the slab edge. The load beam was then used to repeatedly load the slab through the bearing plate to produce a fatigue crack. A total of 41,569 load cycles were applied; the first 21,358 cycles ranged between 4.4 kN (1,000 lb) and 75.5 kN (17,000 lb), and the last 20,211 cycles ranged between 4.4 kN and 93.4 kN (21,000 lb). A crack was visible along the side of the slab after the application of approximately 4,000 load cycles; the remaining load cycles were necessary to propagate the crack across the slab.

After the crack had propagated transversely across the slab, small metal plates were epoxied on the slab surfaces on each side of the crack to serve as bearing seats for the tips of the deflection-measuring equipment (LVDTs).

Slabs 2 through 6

A smooth, full-depth transverse joint without aggregate interlock was formed using a piece of 6.4-mm (0.25-in) thick plywood placed transversely at the center of the casting frame. A plywood form was used rather than a saw cut because the required depth of cut precluded the use of a hand-held concrete saw and the use of a walk-behind saw was considered impractical for use in the lab. After 24 hours of curing, a hand-held saw was used to remove the top edge of the plywood.

Immediately after finishing the surface of the PCC slab, small metal plates were placed on the slab surfaces on each side of the crack to serve as bearing seats for the tips of the deflection-measuring equipment (LVDTs).

2.5.3 Dowel Bar Retrofit Process

The dowel bars were retrofit into the test slabs using the practices described below, which were adapted from those required by the Minnesota Department of Transportation (Mn/DOT) for use by contractors. Appendix C contains a brief summary of Mn/DOT's dowel retrofit field practices.

Cutting and Preparation of Dowel Bar Slots

The test slabs were allowed to moist cure for a period of 7 days, after which time slots were cut in test slabs 2 through 6. The cutting of slots in slab 1 was delayed for approximately one year while the test frame was redesigned and modified to address deficiencies in the original design, as described previously.

Three slots were cut across the joint or crack in each slab, with slot centers at distances of 15, 45 and 76 cm (6, 18 and 30 in) from the edge of the slab. All dowel slots were cut using a hand-held concrete saw with a 35-cm (14-in) blade. The nominal slot dimensions were 13 cm (5 in) deep, 7.0 cm (2.75 in) wide and approximately 76 cm (30 in) long. These dimensions allowed the retrofit dowels to be centered at the mid-depth of the slab (9.5 cm [3.75 in] from the top and bottom surfaces).

A light-weight (6.8-kg [15-lb]) electro-pneumatic chipping hammer was used to remove the concrete in the slots. After the larger pieces of concrete were removed, the slots were further cleaned with a wire brush and thoroughly blown out with compressed air. The sides of the slots were then wiped down with a damp cloth to remove any residual dust.

For test slab 1, the crack was sealed where it intersected the bottoms and sides of the dowel slots. This was done to prevent any of the backfill material from entering the crack. Crack sealing was not necessary for slabs 2 through 6 because the joint forming material remained in place through the entire slab thickness.

Installation of Dowel Bars

Epoxy-coated dowel bars measuring 3.8 cm (1.5 in) in diameter and 38 cm (15 in) in length were selected for use in test slabs 1, 2, 3 and 4. To test the effects of dowel bar length on load transfer and potential field performance, epoxy-coated dowel bars measuring 3.8 cm (1.5 in) in diameter and 33 cm (13 in) in length were selected for use in test slab 5. Stainless steel-clad dowels measuring 3.7 cm (1.44 in) in diameter and 46 cm (18 in) in length were used in slab 6. The finished diameter of the stainless steel-clad dowels was slightly less than 3.8 cm (1.5 in) due to buffing procedures used to smooth irregularities produced by the stainless steel cladding process. Each dowel was lightly coated with a form release agent prior to being placed on chairs that provided a clearance of 13 mm (0.5 in) between the bottom of the dowel and the bottom of the slot. Plastic expansion caps were placed over the ends of the dowels to allow up to 6.4 mm (0.25 in) of longitudinal movement at each end of each bar. Layers of tape were wrapped around the stainless steel-clad dowels where the plastic expansion caps and chairs contacted the bars, since the chairs and caps were sized for 3.8 cm (1.5 in) bars.

The dowel/chair assemblies were placed in the slots and centered over the transverse crack or joint. The bars were oriented so that they were parallel to the edge of the slab and slab surface. A 6.4-mm [0.25 in] thick foam board filler was placed in close fit with the dowel and slot at the location of the crack or joint to ensure the function and location of the crack/joint through the retrofit backfill material for slabs 1, 2, 3 and 4. It was noted that the foam board in the innermost slot had bent slightly after consolidation of the test slab 4 backfill material. Therefore, a piece of 6.35-mm (0.25-in) plywood was used in place of the foam board filler for test slabs 5 and 6 to prevent any deformation of the formed joint that might aid in providing load transfer across the joint.

A portland cement mortar bonding agent was applied to the slot walls and bottoms immediately prior to placing the backfill material around the bars. The bonding agent consisting of 50% cement and 50% sand (by weight) with enough water to achieve a creamy consistency. The following batch quantities were used to produce this non-air-entrained mortar:

- 900g (2 lb) cement
- 900g (2 lb) fine aggregate
- 360g (0.8 lb) water

The mortar was applied to the dowels and was worked into the slot surfaces using a brush to ensure that all surface irregularities were coated.

Mn/DOT Concrete Patching Mix Grade 3U18 (sack mix) was used to backfill the slots and anchor the dowels in place for test slabs 1, 2 and 3; Tamms Speed Crete 2028 was used to backfill the slots for slabs 4 through 6. Table 2.3 presents the mix proportions for the concrete backfill materials used in the slots of each test slab. Mix proportions varied somewhat between batches due to apparent inconsistencies in the material provided in each premixed sack. The air content of the backfill material was not measured because this study was not concerned with durability testing of the materials.

The backfill material was placed in the slots immediately after the bonding agent application was complete to prevent drying of the bonding agent. The concrete mix was placed in two lifts and each lift was consolidated using a pencil vibrator. The slots were finished with wood and magnesium floats, and were covered with wet burlap and plastic during curing.

Table 2.3. Concrete backfill sack mixture proportions.

Batch No.	Concrete Backfill Type	Number of Sacks	Water, g	HRWR, mL	A/E, mL	Slump, mm	Slot Filled[a]
Slab 1 – 38.1-cm long dowel bars, tight crack with grain interlock							
1	3U18	1	3069	84.22	9.0	32	#1 & ¾ of #2
2	3U18	1	3069	84.22	10.0	19	#3 & ¼ of #2
Slab 2 – 38.1-cm long dowel bars, smooth faced joint							
1	3U18	1	3069	44	10	25	#2
2	3U18	1	3069	44	10	19	#1
3	3U18	1	3069	44	10	25	#3
Slab 3 – 38.1-cm long dowel bars, smooth faced joint							
1	3U18	1	2886	44	10	30	#3
2	3U18	1	3219	44	10	19	#1
3	3U18	1	3851	44	10	0	#2
Slab 4 – 38.1-cm long dowel bars, smooth faced joint							
1	Speed Crete 2028[b]	1	2515			0	#3
2	Speed Crete 2028[b]	1	2515			0	#2
3	Speed Crete 2028[b]	1	2515			0	#1
Slab 5 – 33.0-cm long dowel bars, smooth faced joint							
1	Speed Crete 2028[b]	1	2515			0	#3
2	Speed Crete 2028[b]	1	2515			0	#2
3	Speed Crete 2028[b]	1	2515			0	#1
Slab 6 – 46-cm long dowel bars, smooth faced joint							
1	Speed Crete 2028[c]	2	5039			0	#3
2	Speed Crete 2028[c]	2	5039			0	#2
3	Speed Crete 2028[c]	2	5039			0	#1

[a] Slot #1 – critical dowel bar location (center of slot is 15.2 cm from edge of slab).
Slot #2 – middle dowel bar location (center of slot is 45.7 cm from edge of slab).
Slot #3 – inside dowel bar location (center of slot is 76.2 cm from edge of slab).

[b] 22.7 kg (50 lb) of washed-pea gravel (CA8) was incorporated into mixture.

[c] 45.4 kg (100 lb) of washed-pea gravel (CA8) was incorporated into mixture.

Cylindrical compression test specimens measuring 10 cm x 20 cm (4 in x 8 in) were cast along with each batch of concrete backfill and were tested after 24 hours of curing. Mn/DOT specifications require that these specimens achieve a compressive strength of at least 21 MPa (3,000 psi) within 24 hours. For the purposes of this study, load testing was begun 24 hours following the dowel installation and backfill only if the companion specimens exhibited the required 24-hour strength. Otherwise testing was delayed and additional curing was performed until a compressive strength of at least 21 MPa was achieved. Compression testing was also performed on selected specimens prepared from the Speed Crete 2028 concrete backfill after 3 to 5 hours of moist curing. Speed Crete 2028 typically has a compressive strength of 34.5 MPa (5,000 psi) after 3 hours of curing (3). It should be noted that the compressive strengths measured for the backfill used in slab 5 were significantly less than 34.5 MPa after 3 hours of curing. It took 17 hours of moist curing for the compressive strength of this concrete backfill to reach 30.4 MPa (4,410 psi).

Table 2.4 presents the compressive strengths of the backfill material used for each test slab. It can be seen that the backfill material used in the outermost dowel slot of test slab 3 exhibited a compressive strength less than 21 MPa (3000 psi). In this case, the slots and remaining compression test specimens were moist-cured for an additional 12.5 hours. The remaining compression test specimens were then tested to ensure that the compressive strength of the backfill material in each slot exceeded 21 MPa (3000 psi).

Table 2.4. Compressive strengths of concrete backfill material.

Specimen Number	Compression Strength (MPa), 24-hour		
	Batch Number		
	1	2	3
Slab 1 – 3U18 concrete backfill, 38.1-cm long dowel bars, tight crack with grain interlock			
1	22.3	24.5	
2		23.3	
Avg.	22.3	23.9	
Slab 2 – 3U18 concrete backfill, 38.1-cm long dowel bars, smooth faced joint			
1	47.7[a]	49.9[a]	45.6[a]
2	47.8[a]	50.7[a]	47.1[a]
Avg.	47.8[a]	50.3[a]	46.4[a]
Slab 3 – 3U18 concrete backfill, 38.1-cm long dowel bars, smooth faced joint			
1	22.4	16.3	21.2
2	32.5[b]	23.7[b]	30.2[b]
Avg.	22.4 / 32.5[b]	16.3 / 23.7[b]	21.2 / 30.2[b]
Slab 4 – Tamms Speed Crete 2028 concrete backfill, 38.1-cm long dowel bars, smooth faced joint			
1	31.2[c]	31.3[d]	27.7[e]
2	31.5[c]	31.8[d]	28.2[e]
3	41.2	40.4	38.6
4	39.5	40.1	40.0
Avg.	31.3[c] / 40.3	31.5[d] / 40.2	28.0[e] / 39.3

Table 2.4. Compressive strengths of concrete backfill material (continued).

Specimen Number	Compression Strength (MPa), 24-hour		
	Batch Number		
	1	2	3
Slab 5 – Tamms Speed Crete 2028 concrete backfill, 33.0-cm long dowel bars, smooth faced joint			
1	3.0[e]	5.1[e]	30.3[f]
2	30.5[f]	34.1	30.4[f]
3	33.8	37.0	33.2
4	32.2		33.3
Avg.	3.0[e] / 30.5[f] / 33.0	5.1[e] / 35.6	30.3[f] / 33.3
Slab 6 – Tamms Speed Crete 2028 concrete backfill, 46-cm long dowel bars, smooth faced joint			
1	39.9	41.0	42.3
2		44.3	
Avg.	39.9	42.7	42.3

(a) Test specimens for test slab 2 were not broken until 23 days after retrofitting the dowel bars due to difficulties with the TestStar upgrade. The slots and cylinders were moist-cured for a period of 48 hours and air-cured until testing of the specimen commenced (after 23 days).
(b) Test specimens broken after approximately 40.5 hours of moist-curing.
(c) Test specimens broken after 5 hours of moist-curing.
(d) Test specimens broken after 4 hours of moist-curing.
(e) Test specimens broken after 3 hours of moist-curing.
(f) Test specimens broken after 17 hours of moist-curing.

2.5.4 Minimization of Pavement Surface Irregularities

A 6.4-mm (0.25-in) neoprene pad was placed below the rocker beam during the testing of slabs 1 through 3 to minimize the effects of pavement surface irregularities on the load profiles (i.e., to smooth the actual load profiles) and movement of the rocker beam. This matting broke down quickly (daily), which resulted in deterioration of the applied load profiles due to the presence of small irregularities in the finished PCC surface. To reduce this type of problem, a 6.4-mm (0.25-in) poron-urethane pad with special abrasion-resistant backing was used for test slabs 4 and 5. This type of matting was also used for the first 4.3 million cycles of slab 6, after which a 6.4-mm neoprene pad was used because funding was not available for the purchase of a new poron-urethane pad.

CHAPTER 3
LABORATORY TESTING

3.1 Calibration of LVDTs

The LVDTs were re-calibrated prior to testing each slab in order to ensure that all deflection measurements would be within the linear range of each LVDT. In addition, the TestStar software was re-calibrated for each LVDT. This entailed determining the TestStar range required to produce an output of ± 10 volts for each LVDT. It was also necessary to determine the TestStar offset by reading the voltage indicated by the TestStar system when the LVDT plunger is set at its mechanical zero. The TestStar range and offset for each LVDT were then entered into the appropriate input lines of the *Edit Input Signals* window of the TestStar program and the *Assign Temporary* button was used to set up a temporary calibration file.

After the temporary calibration file was set up in TestStar, the LVDTs were mounted on the test stands so that they were at rest near their mechanical zeroes and the plungers rested on the steel plates previously attached to the test slab surface. Table 3.1 presents the final "time zero" positions of the LVDTs for each test slab.

Table 3.1. Initial LVDT positions prior to testing.

	LVDT A		LVDT B	
Test Slab Number	Distance from Edge of Slab (mm)	Distance from Joint/Crack (mm)	Distance from Edge of Slab (mm)	Distance from Joint/Crack (mm)
1	25.4	82.6	25.4	76.2
2	19.1	54.0	28.6	165.1
3	14.3	54.0	19.1	155.6
4	19.1	54.0	19.1	158.8
5	18.5	54.3	29.2	165.1
6	20.7	50.2	18.2	148.8

3.2 Establishing the Drive File

A drive file is created prior to the testing of each Minne-ALF test slab. The drive file is essentially the program that drives the test actuators to achieve the desired load and/or stroke wave forms during testing. The drive file is a version of the user-input load/stroke profiles that has been modified to account for compliance and response of the test system and test specimen. It is created by the TestWare software following the development of an ITF (inverse transfer function) file. The ITF file is created by the following process, which is initiated in the *Iteration Method and Compensation Parameters* window in the TestWare software program (4):

1. The program sends a preprogrammed command signal to the system.
2. The sensor response to this command indicates how the system as a whole reacts to the command signal.
3. The program analyzes the difference between the response and the command signals and attempts to close the difference by modifying the command signal.
4. The process continues iteratively, with the command signal being modified repeatedly until the response approaches the desired shape.

When the user stops the system from further iterations, the last 1024 data points (which define the ITF file) are saved by the system and used to create the drive file. This drive file is then used in the testing regime where it can be played back as many times as desired.

Appendix D presents a log of the generation of the ITF and drive files for each test slab. This information was considered important because it was not always possible to generate acceptable drive files on the first try. Since the creation of the ITF involves some vibration and loading of the slab, it was thought that consideration of differences in the ITF/drive file generation process might help to explain any differences in the performance of supposedly comparable test specimens.

3.3 Load Testing Procedures

3.3.1 Pre-Test Procedures

Testing for any given slab begins immediately after the drive file has been created. Initial LVDT readings must be taken to establish the zero position of each LVDT just before starting the test. These values must be taken after the hydraulics are turned on and the rocker beam is leveled (zero stroke) and unloaded (force is approximately zero). The zero LVDT values produced under these conditions are recorded in the data log sheet and used to adjust the LVDT readings obtained during testing. The testing should be stopped and new LVDT "zero" readings should be obtained at least twice weekly to ensure that any changes in the established zeros (due to slab settlement and other possible sources of systematic or random change) are recorded.

3.3.2 Data Collection

The TestWare program was written to automatically collect data from the vertical actuator (load), the horizontal actuator (stroke) and from the two LVDTs (deflection) at the following pre-determined load repetition or "cycle" counts: 1, 1000, 2000, 5000, 10000, 20000, 50000, 100000, 300000, 600000, and every 600000 cycles thereafter. In addition, provisions were also made to allow manual triggering of data collection at any desired time.

Whether the data collection is automatically or manually triggered, the program is set up to obtain data at a sampling rate of 400 Hz for a duration of two seconds at the predetermined or desired cycle count. In this way, about 100 data points are obtained for each of four load cycles (assuming a test load rate of 2 Hz) for each of the four sampled channels. In addition, TestWare automatically records the last two seconds of data prior to its shutdown if a control interlock condition develops (i.e., the system automatically shuts down because load, stroke or deflection measurements exceed user-defined levels).

3.3.3 Test Termination

Testing of each slab was to be terminated when the load transfer efficiency fell below 70 percent and the differential deflection across the joint/crack exceeded 127 μm (0.005 in). The definitions and computation of these performance measures are described in section 4.1 of this report.

CHAPTER 4
RESULTS AND ANALYSIS

The following subsections present and discuss the results of the tests of the six retrofit load transfer installations.

4.1 Performance Measures

4.1.1 Load Transfer Efficiency

Load transfer efficiency (LTE) is a measure of the ability of a joint/crack to transfer load from one side of a joint/crack to the other (5). The data required to compute LTE are generally acquired by measuring the deflections of both sides of a joint or crack under an impulse load that simulates the passage of a heavy vehicle. This is typically accomplished using a Falling Weight Deflectometer (FWD), which drops a mass package onto a load plate positioned on one side of the joint or crack. Geophones or deflection sensors are placed at the center of the load plate and on the unloaded side of the joint (as well as at other locations). Sensor signals are used to measure or compute deflection measurements.

Although there are at least three different equations that use FWD data to produce a measure of load transfer efficiency, the most commonly used and widely accepted equation is the following:

$$\text{LTE}\ (\%) = (d_{UL} / d_L) \times 100$$

where:

LTE = percent load transfer efficiency
d_{UL} = deflection of unloaded side of crack/joint
d_L = deflection of loaded side of crack/joint

With this expression, perfect load transfer (where both sides of the crack/joint deflect equally under an applied load) exists when the ratio is 100 percent. Conversely, no load transfer exists when the ratio is 0 percent and both sides of the joint/crack move independently. Many agencies consider LTE values less than 70 percent to be unsatisfactory and may consider retrofitting load transfer devices in such cases.

Table 4.1 present summaries of the LTE values obtained for each test slab when the load is on the approach or "upstream" side of the joint or crack. Table 4.2 presents the same information computed when the load is on the leave or "downstream" side of the joint or crack.

Table 4.1. Test slab performance summaries – load on approach side of joint.

Cycles (Million)	Minne-ALF Test Slab											
	1		2		3		4		5		6	
	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)
2E-06			202	78	27	96						
5E-05									10	96		
1E-04									12	95		
1E-03			49	95	9	99			9	97		
2E-03			64	91	23	97			12	97		
5E-03			70	91	33	96			14	97		
0.01			61	94	63	93			8	98		
0.02			51	93	87	92			15	97		
0.05			74	93	146	89	56	94	26	95		
0.1			82	91	100	89	54	95	41	94		
0.3											26	95
0.3			74	93	118	90	63	94	46	93	35	93
0.6			215	80	260	78	98	92	38	95	28	94
1.2			261	79	339	79	104	91			58	91
1.8			244	77	231	79	139	88			55	91
2.0											47	93
2.4			251	76	258	78	143	89	21	98		
3.0			292	70	253	79	150	89	47	95	47	94
3.6	34	98	325	69	364	75	168	88	45	96	48	93
4.2	34	98			322	78	170	88	68	94	53	93
4.8	53	96			348	70	150	86	57	94	45	93
5.1					331	65						
5.4	69	96					158	79	75	92	47	93
6.0	67	96					143	87	80	92	63	92
6.4												
6.6							176	85	70	93	69	91
6.8											65	91
7.6											74	90
7.8											71	92
8.3											70	91
9.0											68	92
9.4											81	90
9.8											77	90

Table 4.2. Test slab performance summaries – load on leave side of joint.

Cycles (Million)	Minne-ALF Test Slab											
	1		2		3		4		5		6	
	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)	Diff. Defl. (μm)	LTE (%)
2E-06			58	93	39	95						
5E-05									18	93		
1E-04									20	92		
1E-03			186	80	57	92			28	91		
2E-03			208	80	45	94			19	95		
5E-03			176	83	47	94			24	94		
0.01			236	78	25	97			28	94		
0.02			226	80	21	98			14	97		
0.05			201	81	65	94	40	96	7	99		
0.1			204	82	117	87	37	97	11	98		
0.3											20	97
0.3			205	82	153	87	52	96	33	95	22	96
0.6			89	92	198	78	75	94	49	93	16	97
1.2			68	94	245	75	101	92			15	97
1.8			327	64	268	71	87	93			33	95
2.0											23	97
2.4			341	62	300	68	115	91	24	98		
3.0			169	81	233	78	112	91	17	98	27	97
3.6	130	92	154	83	192	84	106	92	14	99	30	96
4.2	99	94			174	87	107	92	12	99	35	96
4.8	122	92			302	72	87	89	19	98	48	93
5.1					198	84						
5.4	145	92					96	90			53	92
6.0	173	91					89	90			106	85
6.4												
6.6	132	94					94	89			109	83
6.8											104	85
7.6											105	83
7.8											106	86
8.3											106	85
9.0											115	85
9.4											92	86
9.8											89	87

4.1.2 Differential Deflection

Differential deflection is the difference in deflections of the two sides of a joint or crack during loading (e.g., $d_L - d_{UL}$). Differential deflection measurements are often used along with the LTE to provide added insight into the performance of a joint or crack. One could consider the deflection test results for two different joints, for example. The first might have $d_L = 0.5$ mm (0.020 in) and $d_{UL} = 0.25$ mm (0.010 in), while the second might have $d_L = 0.2$ mm (0.008 in) and $d_{UL} = 0.1$ mm (0.004 in). Both joints would have the same LTE of 50%, but the first joint would clearly be of greater concern because the higher differential deflections would be more likely to produce pumping, faulting and spalling.

Table 4.1 present summaries of the differential deflection obtained for each test slab when the load is on the approach or "upstream" side of the joint or crack. Table 4.2 presents the same information computed when the load is on the leave or "downstream" side of the joint or crack.

4.2 Effect of Joint Face Texture

The effect of joint face texture on load transfer and potential field performance was evaluated by comparing the LTE and differential deflection history of test slab 1 (which featured a tight crack with grain interlock) with those of slabs 2 and 3 (which contained smooth-faced joints approximately 6.4 mm (0.25 in) in width. Tables 4.1 and 4.2 summarize the LTE and differential deflection data obtained for the three test slabs. Figures 4.1 and 4.2 are graphical presentations of the same data.

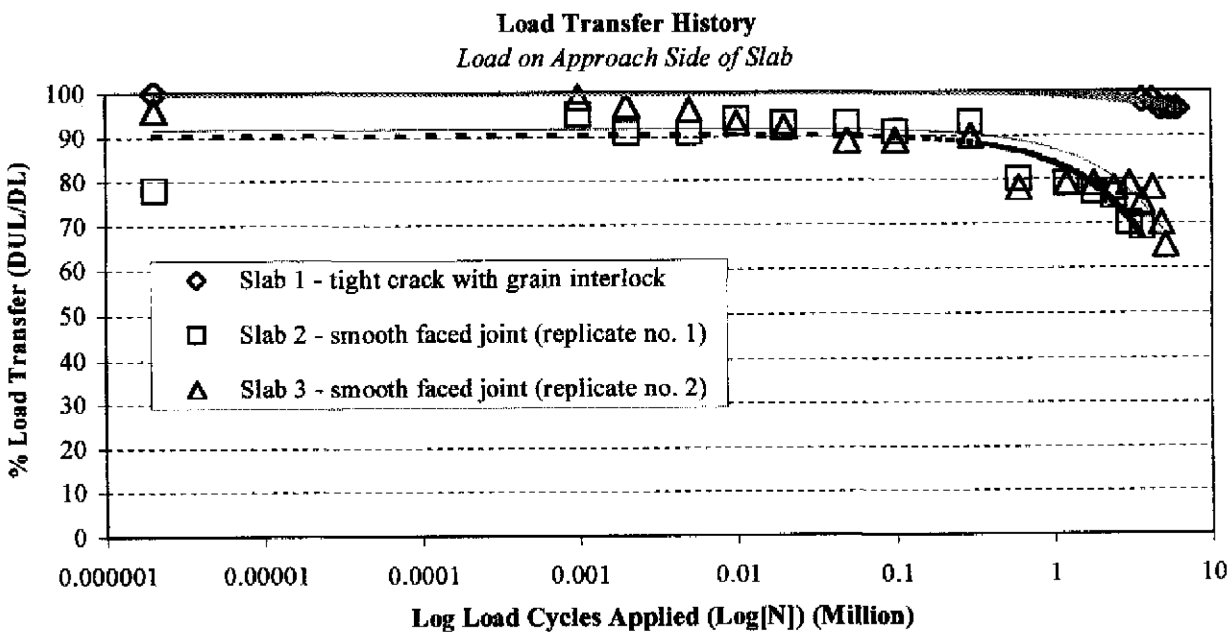

Figure 4.1. Effect of Joint Face Texture on Load Transfer Efficiency.

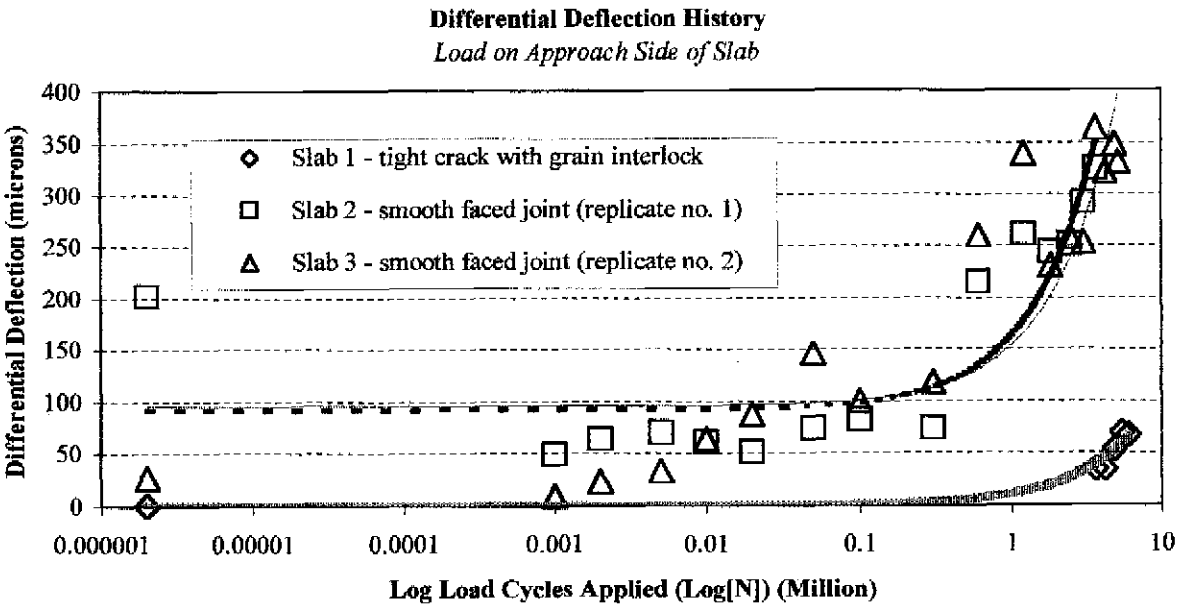

Figure 4.2. Effect of Joint Face Texture on Differential Deflection.

Performance measures could not be computed for slab 1 using any of the data collected during the first 3 million load cycles because the LVDT data collected appeared to be extremely offset in time with respect to where the load placement should have been when considering horizontal actuator stroke measurements. Figures 4.3 and 4.4 illustrate the problems with the LVDT readings during the first 3 million cycles. Attempts were made to adjust these LVDT readings using known zero values (e.g., the differential deflection should approach zero when the stroke is at zero [i.e., when the rocker beam is crossing the transverse joint]); these attempts did not yield reasonable results and were discarded. The LVDT problem was eventually attributed to inadequate stiffness in the LVDT mounting brackets. This problem was corrected, as described previously, and subsequent LVDT data were reasonable and useful.

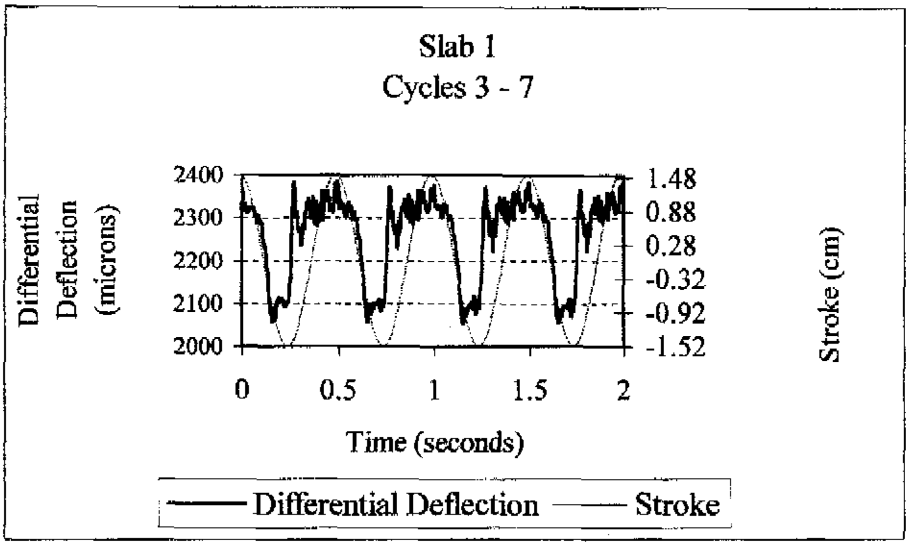

Figure 4.3. Illustration of slab 1 deflection data when LVDTs mounted on original bracket.

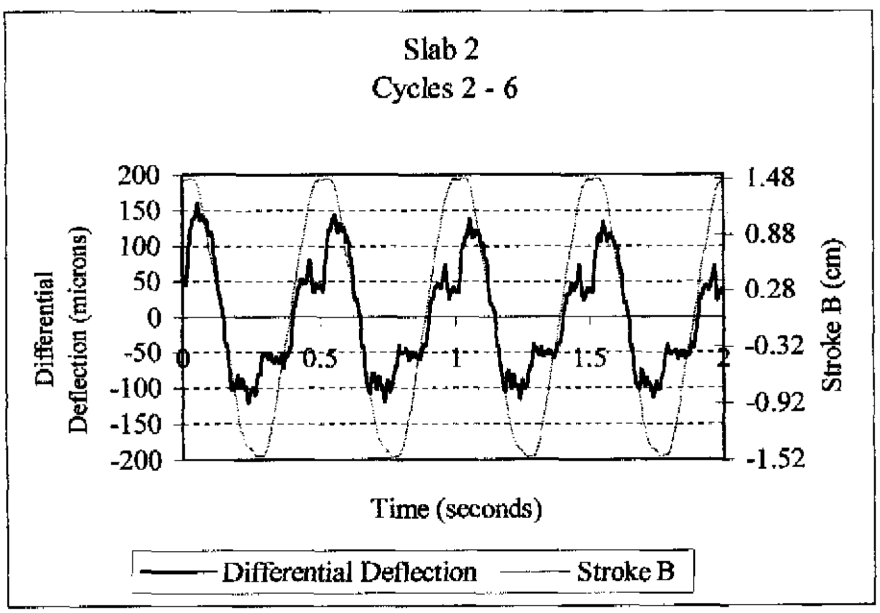

Figure 4.4. Illustration of slab 2 deflection data when LVDTs mounted on modified brackets.

Slabs 1, 2 and 3 were subjected to simulated single-wheel load applications totaling 6.7, 3.6 and 5.1 million cycles, respectively. The following LTE and differential values were calculated at test termination for each specimen when loaded on the approach side of the joint:

	LTE	Differential Deflection
• Slab 1:	96 %	67 μm (0.002 in)
• Slab 2:	69 %	325 μm (0.013 in)
• Slab 3:	65 %	331 μm (0.013 in)

As stated previously, the original test program plan was to terminate testing when LTE dropped below 70 percent and differential deflection exceeded 127 μm (0.005 in). The testing of slabs 2 and 3 was terminated using these criteria. Slab 1, however, showed no signs of significant further degradation after 6.7 million cycles; testing was terminated early to allow more timely testing of other specimens.

Even though slab 1 was subjected to more load repetitions than either slabs 2 or 3, it never exhibited significant losses of load transfer or increases in differential deflection. This can be attributed to the contribution of aggregate interlock across the very tight crack that was present in slab 1. This aggregate interlock probably carried a very high proportion of the load across the crack, thereby reducing the dowel-concrete stresses that produced increasing looseness in the joints of slabs 2 and 3.

While some aggregate interlock contribution is likely in the field, one should not expect this type of performance from field installations because temperature fluctuations and drying shrinkage will generally open any cracks or joints, thereby significantly decreasing the effective contributions of any grain interlock. It is generally accepted that grain interlock becomes ineffective when the joint or crack width exceeds 0.75 mm (0.03 in) (6); this condition is not difficult to achieve in Minnesota, even with very short joint or crack spacings.

Further studies should be performed to assess the performance of the retrofit dowel system associated with varying joint or crack widths (i.e., widths that produce grain interlock contributions somewhere between the tight crack contribution associated with slab 1 and the zero contribution associated with the smooth joints used in slabs 2 and 3).

4.3 Repeatability of Test Results

Slabs 2 and 3 were essentially replicate specimens with identical designs and subjected to identical tests. Figures 4.1 and 4.2 illustrate that tests of these replicate specimens using the Minne-ALF produced fairly repeatable test results. Plots of LTE and differential deflection vs. log of the number of load cycles (semi-log plots are typically used for fatigue-type analyses such as this) show very nearly identical profiles. The results obtained at test termination are also nearly identical: slab 2 produced LTE and differential deflection values of 69 percent and 325 μm (0.013 in), respectively, while slab 3 produced LTE and differential deflection values of 65 percent and 331 μm (0.013 in), respectively.

It is worth noting that there was an apparently large discrepancy in test results between slabs 2 and 3 after 2 loading cycles. At this point, slab 2 was apparently exhibited LTE and differential deflection values of 78 percent and 202 μm (0.008 in), respectively, while slab 3 exhibited values of 96 percent and 27 μm (0.001 in). These apparent differences are due to the fact that the TestStar system generally requires several load cycles before it has properly adjusted the command signal to minimize differences with the measured feedback response signal. Therefore, any data collected during this initial period of adjustment can be affected by spikes in the loading profile. In future testing, the first data point should be collected after 50 cycles in order to avoid the load spikes associated with initial signal compensation algorithms.

4.4 Effect of PCC Backfill Material

The effect of PCC backfill material on load transfer and potential field performance was evaluated by comparing the LTE and differential deflection history of test slab 4 (containing Speed Crete 2028) with those of slabs 2 and 3 (which contained 3U18 concrete backfill). Figures 4.5 and 4.6 graphically present the LTE and differential deflection data obtained for these three test slabs.

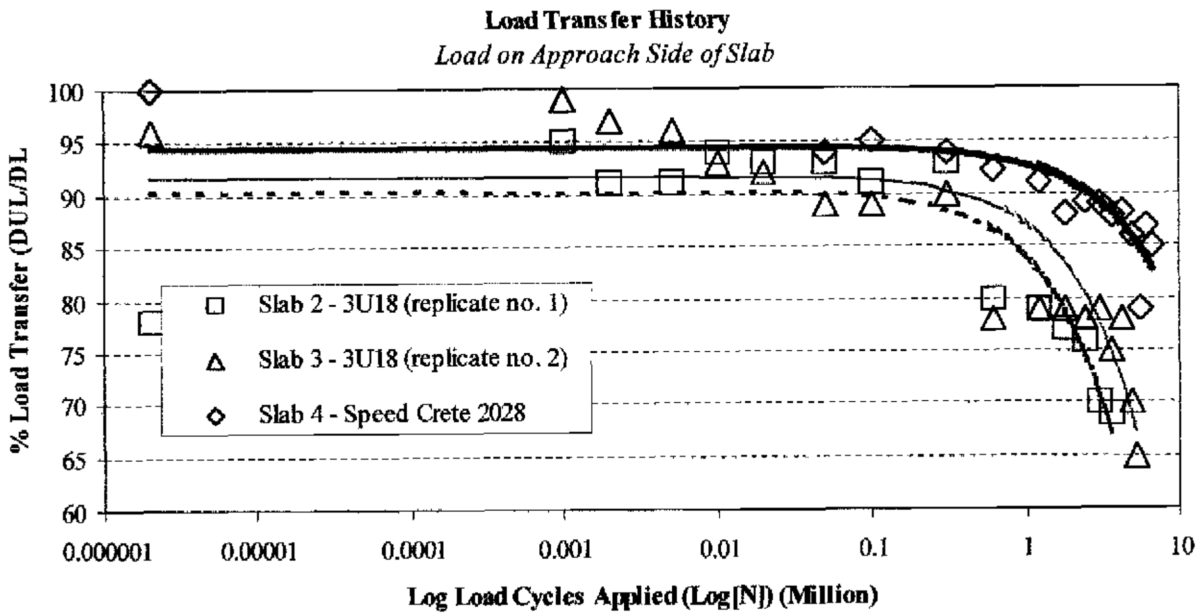

Figure 4.5. Effect of Concrete Backfill Material on Load Transfer Efficiency.

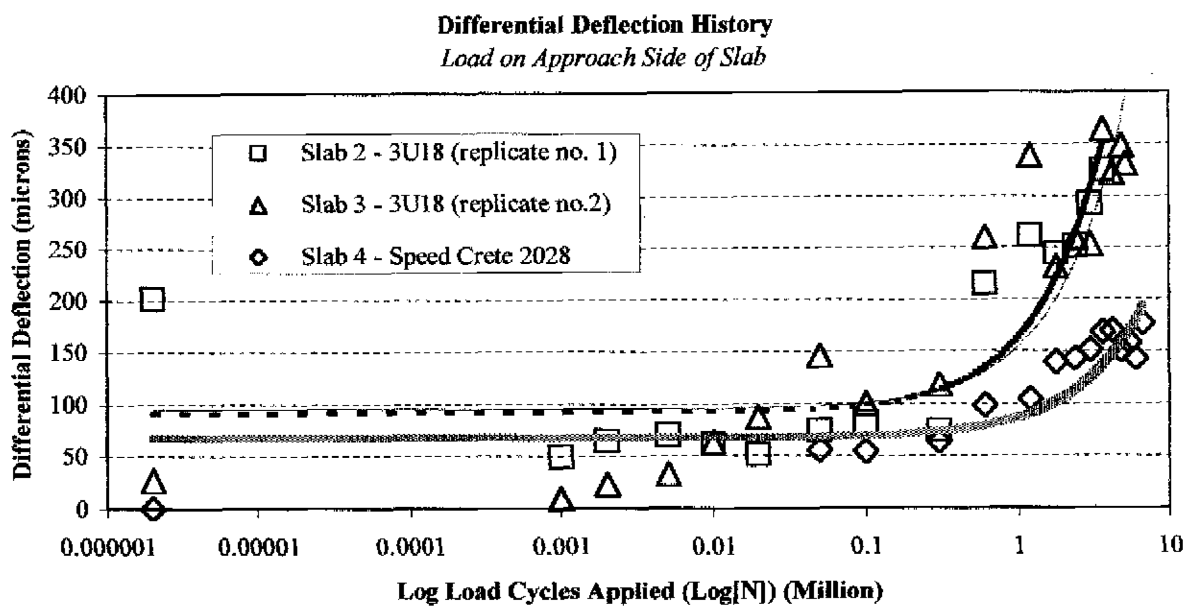

Figure 4.6. Effect of Concrete Backfill Material on Differential Deflection.

Slabs 2, 3 and 4 were subjected to simulated single-wheel load applications totaling 3.6, 5.1 and 6.6 million cycles, respectively. The following LTE and differential values were calculated at test termination for each specimen when loaded on the approach side of the joint:

	LTE	Differential Deflection
• Slab 2:	69 %	325 μm (0.013 in)
• Slab 3:	65 %	331 μm (0.013 in)
• Slab 4:	85 %	176 μm (0.007 in)

As stated previously, the original test program plan was to terminate testing when LTE dropped below 70 percent and differential deflection exceeded 127 μm (0.005 in). The testing of slabs 2 and 3 was terminated using these criteria. The testing of slab 4 was terminated after 6.6 million cycles to allow testing of other specimens within the contract time frame; this was justified because the behavior trend of slab 4 was well established after 6.6 million load cycles.

As illustrated in figures 4.5 and 4.6, the LTE and differential deflection histories for slab 4 (which contained Speed Crete 2028 concrete backfill) is greatly improved over those measured for test slabs 2 and 3 (which contained 3U18 concrete backfill). This could be due in part to the higher initial strength of the Speed Crete 2028 concrete backfill. The differences in initial strength are shown in table 2.4, which suggests that the 2028 material used in slab 4 was significantly stronger than the 3U18 material used in slab 3 (Note that the compression test results for slab 2 were obtained after 23 days of curing and are not directly comparable to those obtained after 24 hours for slabs 3 and 4.) The higher strength of the 2028 material would make it more resistant to microcracking and the development of "socketing" around the dowel bar.

The increased LTE and reduced differential deflection of slab 4 (compared to slabs 2 and 3) could also be due to the use of different matting under the rocker beam during the testing of slab 4. The matting type is not believed to significantly affect LTE and differential deflection, however.

4.5 Effect of Dowel Bar Length

The effect of dowel bar length on load transfer and potential field performance was evaluated by comparing the LTE and differential deflection history of test slab 4 (containing Speed Crete 2028 and 38-cm [15-in] long dowel bars) with that of slab 5 (which contained Speed Crete 2028 and 33-cm [13-in] long dowel bars). Figures 4.7 and 4.8 present the LTE and differential deflection data obtained for these two test slabs.

Slabs 4 and 5 were each subjected to 6.6 million load cycles, after which the following LTE and differential values were measured when loaded on the leave side of joint:

	LTE	Differential Deflection
• Slab 4:	85 %	176 μm (0.007 in)
• Slab 5:	93 %	70 μm (0.003 in)

As illustrated in figures 4.7 and 4.8, the LTE and differential deflection histories of slab 5 (containing 33-cm [13-in] long dowels) were better than those obtained for test slab 4 (containing 38-cm [15-in] long dowels). This result was unexpected and was investigated further.

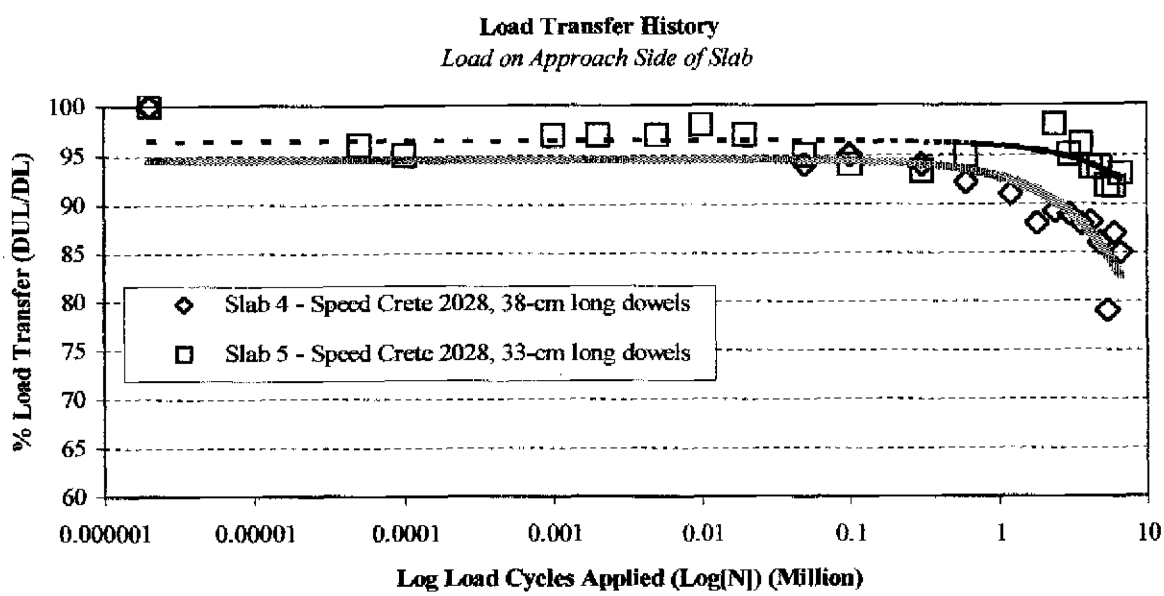

Figure 4.7. Effect of Dowel Bar Length on Load Transfer Efficiency.

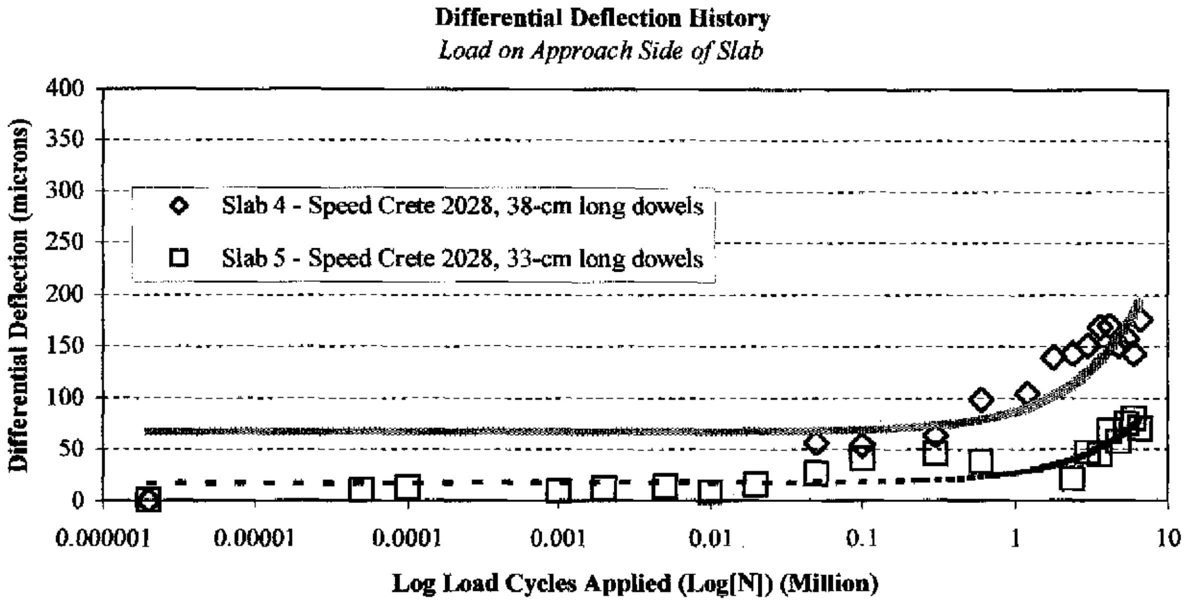

Figure 4.8. Effect of Dowel Bar Length on Differential Deflection.

Cores was retrieved from the joint and through the dowel closest to the pavement edge for each of these two slabs. These cores were examined for evidence of poor consolidation of the backfill material. Figure 4.9 is a photograph of the backfill material beneath the dowel in slab 4, which shows evidence of poor consolidation. The honeycombing evident in this photo was attributed to the poor workability (0 cm [0 in] slump) of the material used to backfill all three slots of this test slab. This lack of consolidation probably resulted in reduced load transfer for these dowels. There was no apparent socketing present around the dowel bar even though the backfill material was severely honeycombed.

Figure 4.10 is a photo of the backfill material beneath the critical dowel bar from slab 5. This material shows little evidence of honeycombing, which may help to explain why this slab performed better than slab 4, in spite of the reduced dowel length. It is likely that there would have been little difference in the performance of these two slabs if they had both had comparable dowel support. Replication of these test slabs is recommended to confirm this hypothesis.

Figure 4.9. Photo of Backfill Material Beneath Critical Dowel Bar of Slab 4.

Figure 4.10. Photo of Backfill Material Beneath Critical Dowel Bar of Slab 5.

4.6 Effect of Dowel Bar Type

The effect of using stainless steel-clad dowels on load transfer and potential field performance was evaluated by comparing the LTE and differential deflection histories of test slab 4 (containing Speed Crete 2028 and 38-cm [15-in] long epoxy-coated dowels) with those of slab 6 (containing Speed Crete 2028 and 46-cm [18-in] long stainless steel-clad dowels). Figures 4.11 and 4.12 present the LTE and differential deflection data obtained for these two test slabs.

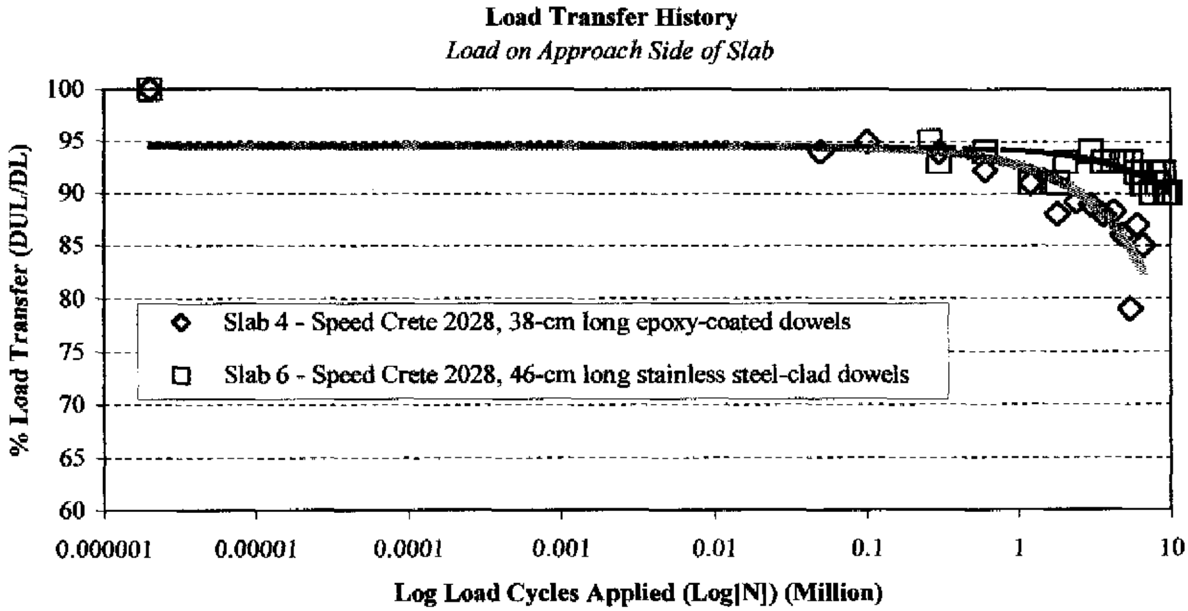

Figure 4.11. Effect of Dowel Bar Type on Load Transfer Efficiency.

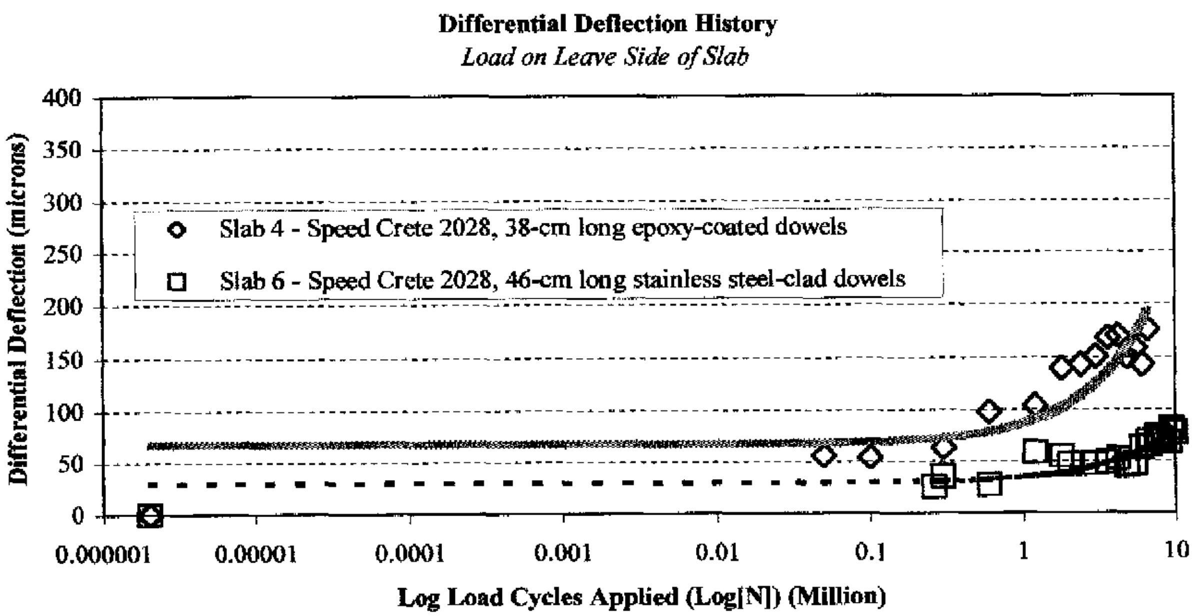

Figure 4.12. Effect of Dowel Bar Type on Differential Deflection.

Slabs 4 and 6 were each subjected to 6.6 and 9.8 million load cycles, respectively, after which the following LTE and differential values were measured when loaded on the approach side of the joint:

	LTE	Differential Deflection
• Slab 4:	85 %	176 μm (0.007 in)
• Slab 6:	90 %	77 μm (0.003 in)

As illustrated in figures 4.11 and 4.12, the LTE and differential deflection histories of slab 6 (containing 46-cm [18-in] long stainless steel-clad dowels) are improved over those obtained for test slab 4 (containing 38-cm [15-in] long dowels). Statistical analyses performed at a 95 percent confidence level indicated the compression strength of the concrete backfill used in slab 6 was not significantly different from that of the same material used slab 4. However, there were apparent consolidation problems with slab 4, as noted previously. It is believed that this is probably the primary source of the performance difference between these two specimens. It is also possible that the increased dowel length used in slab 6 provided some improvement in performance, but it is believed that this contribution would have been relatively small compared to that of the consolidation effect.

It appears that the load carrying capabilities of the slab 6 were unaffected by the use of stainless steel-clad dowels in lieu of epoxy-coated structural steel bars. The long-term performance potential of this type of dowel bar is still a concern because of the lower strength of stainless steel and the potential for pitting of the cladding. However, after 9.8 million simulated heavy traffic loadings, the LTE and differential deflection histories were excellent (LTE = 90 percent, differential deflection = 77 μm [0.003 in]), indicating good resistance to pitting in the laboratory load test program.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

- At this time, it is very difficult to relate the number of load applications in the Minne-ALF to a number of load applications in the field. On one hand, every load applied by the Minne-ALF can be considered a critical load in both placement and magnitude; this would suggest that even higher numbers of loads might be expected in actual field applications where loads are not so highly channelized and not all loads are the maximum allowed by law. However, there are many factors present in the field (e.g., pavement curling and warping, the opening and closing of joints, changes in concrete strength and condition with time, seasonal variations in foundation stiffness, etc.) that have not yet been (or cannot be) adequately considered in laboratory-based accelerated testing programs. These factors often significantly reduce performance in the field. For these reasons, it is best to consider the Minne-ALF (and most other accelerated load testing facilities) to be capable of providing a good indication of only the <u>relative performance potential</u> of different designs and design features.
- It appears that the Minne-ALF is a useful tool for evaluating the relative performance of rigid pavement designs and design features. It provided comparable test results for the two replicate specimens that were tested and it indicated significantly improved performance when load transfer was provided by both retrofit dowels and very good grain interlock (through a tight crack).
- The LTE and differential deflection histories for the slab containing Speed Crete 2028 concrete backfill is greatly improved over those measured for test slabs containing 3U18 concrete backfill with similar joints and dowel bars. This could be due in part to the higher initial strength of Speed Crete 2028 concrete backfill.
- The effect of dowel bar length could not be properly determined due to poor consolidation of the backfill material used under the 38-cm retro-fit dowel bars. The effect of poor consolidation on LTE and differential deflection appears to be much larger than the effect of relatively small changes in dowel length.

- It appears that the load carrying capabilities of the slab were unaffected by the use of stainless steel-clad dowels in lieu of epoxy-coated structural steel dowels.

5.2 Recommendations

The following recommendations are suggestions for future modifications to the Minne-ALF test stand, modifications to the test procedures, recommended test stand maintenance activities, and recommendations for retrofit dowel installations in the lab or field.

5.2.1 Test Stand Modifications

- The current 2-bearing hinge system for the rocker beam should be replaced with a 4-bearing system. The current system incurs high maintenance costs and frequent downtime as the bearings fail and require replacement after 2.5 million load cycles. Furthermore, the bearings begin to bind as they deteriorate, which prevents the free movement of the rocker beam and results in unacceptable variances in the load and stroke profiles. These, in turn, affect the measured deflections. A four-bearing system design similar to that developed by Mauritz (2) should be implemented as soon as possible to provide more accurate, efficient and reliable testing.
- The rocker beam guide at the end of the beam opposite the 2-bearing hinge system should be redesigned and replaced. The current system consists of a single guide plate that is attached to the foundation frame and another that is bolted to the test slab. With this arrangement, the test slab twists slightly during loading when the rocker beam is being restrained by the slab-mounted guide plate. This twisting can affect measured deflection test results. The problem can be remedied by developing a guide bracket that is completely supported by and attached to the foundation or test frame, rather than the test slab. This should also be performed before further testing is completed.
- The poron-urethane pad with abrasion-resistant backing that was used for slabs 4 through 6 should be used for all future testing. This pad was much more durable, requiring less frequent replacement than the neoprene pad originally used and providing more effective protection against load spikes caused by small irregularities in the finished PCC surface.

5.2.2 Test Procedure Modifications

- Initial performance data should first be collected after about 50 load cycles (instead of after 1 or 2 cycles) to avoid the effects of load spikes produced while the system is compensating for differences between the command and response signals.

5.2.3 Test Stand Maintenance

Safety interlocks have been established within the TestStar program to shut the system down automatically in the event of a catastrophic failure, (i.e., the system goes unstable, or the stroke/load exceed normal operating ranges). However, routine maintenance must be performed to ensure the safety of people in the immediate area and to ensure that the data collected is usable. Recommended maintenance activities include the following:

- The actuators should be flushed prior to starting each new test slab.
- The hydraulic hoses must be checked daily to ensure that the hoses are not wearing significantly or showing signs of blistering or internal leakage.
- The rocker beam hinge bearings must be greased at least twice weekly.
- The steel-wheeled roller support at the center of the rocker beam should be checked periodically to ensure that enough grease is present between the roller and the rocker beam column.
- When neoprene matting is used beneath the rocker beam, it must be changed when the load profile shows evidence of increasing load spikes. This may be daily or even more frequently.

5.2.4 Retrofit Dowel Load Transfer Installation Procedures

- Due to apparently large inconsistencies in content of different sacks of 3U18 concrete sack mix, it is recommended that the preproportioned sack mixes no longer be used. Improved consistency in backfill material workability and strength will be obtained by manually (and carefully) proportioning the 3U18 concrete mixes.
- Extreme care must be taken to ensure good consolidation of whatever backfill materials are used. If current materials and procedures lead to difficulties in consolidation in the laboratory where conditions are highly controlled, then it is highly likely that field installations will also be subject to problems.

- Additional tests should be performed to examine the effects of dowel length and dowel materials on the performance potential of retrofit (and original construction) load transfer systems. It may be possible to significantly reduce dowel length without sacrificing performance in retrofit installations. Smaller bars and shorter slots would significantly reduce the cost of this type of activity. Newer materials (such as fiberglass composites) may also offer performance advantages through improved corrosion resistance.

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1. Beer, Michael G. "Development of an accelerated loading test platform for pavements." M.S. Thesis. University of Minnesota, Department of Civil Engineering. Minneapolis, MN. 1997.

2. Mauritz, Josh A. "Design and Development of Modifications for an Accelerated Pavement Testing Facility." M.C.E. Project. University of Minnesota, Department of Civil Engineering. Minneapolis, MN. 1997.

3. Tamms Industries. *Technical Data Sheet – Speed Crete 2028.* Tamms Industries. May 1994.

4. MTS Systems Corporation. "790.1x Enhancements for TestWare-SX." TestStar™ II Control Systems Manual – 150330-06A. MTS Systems Corporation. March 1997.

5. Darter, M. I., S. H. Carpenter, M. B. Snyder, K. T. Hall, et al. "Module 3H: Load Transfer Restoration." Techniques for Pavement Rehabilitation, Participants Notebook, Third Revision. National Highway Institute, U.S. Department of Transportation. October 1987.

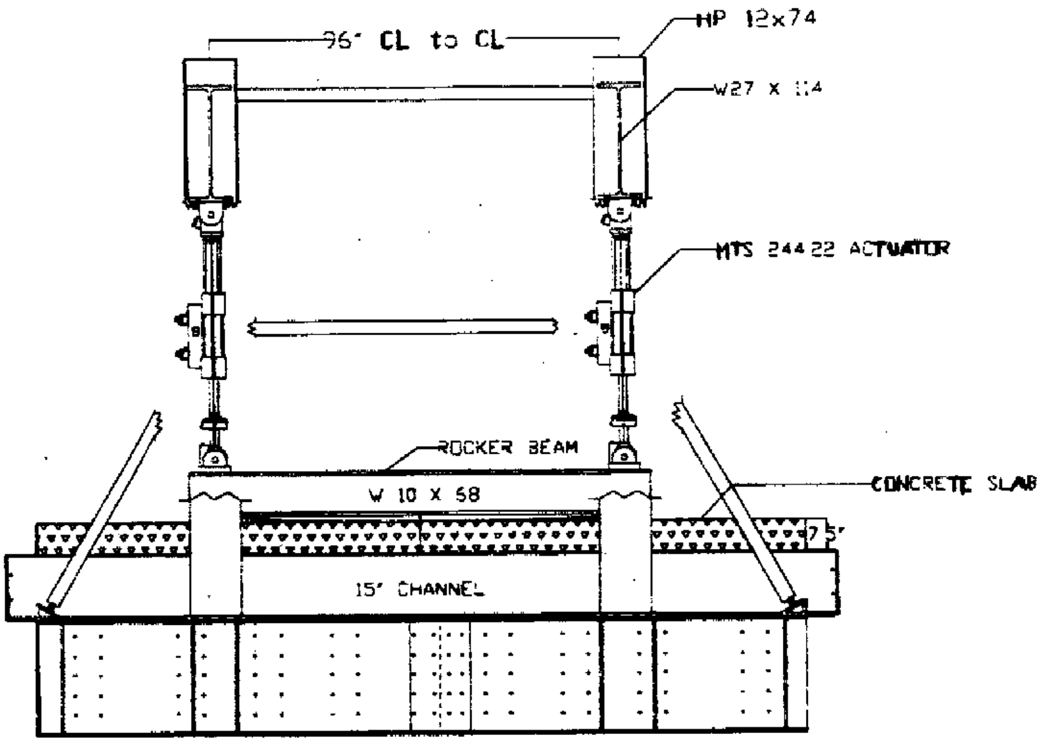
6. Kelleher, K. and R. M. Larson. "The Design of Plain Doweled Jointed Concrete Pavement." Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation. Purdue University, West LaFayette, IN. April 18-20, 1989.

7. Mn/DOT Concrete Pavement Rehabilitation Standards. July 1999.

8. Mn/DOT Special Provisions – SP5-38. "(0301) Epoxy Coated Dowel Bar Retrofit." Maintenance Project Number 2506. 1994.

APPENDIX A

CONSTRUCTION DRAWINGS FOR ORIGINAL MINNE-ALF



TRANSVERSE ELEVATION
CUT-AWAY ACTUATOR VIEW

Figure A-1. Transverse elevation cut-away view of complete test stand.

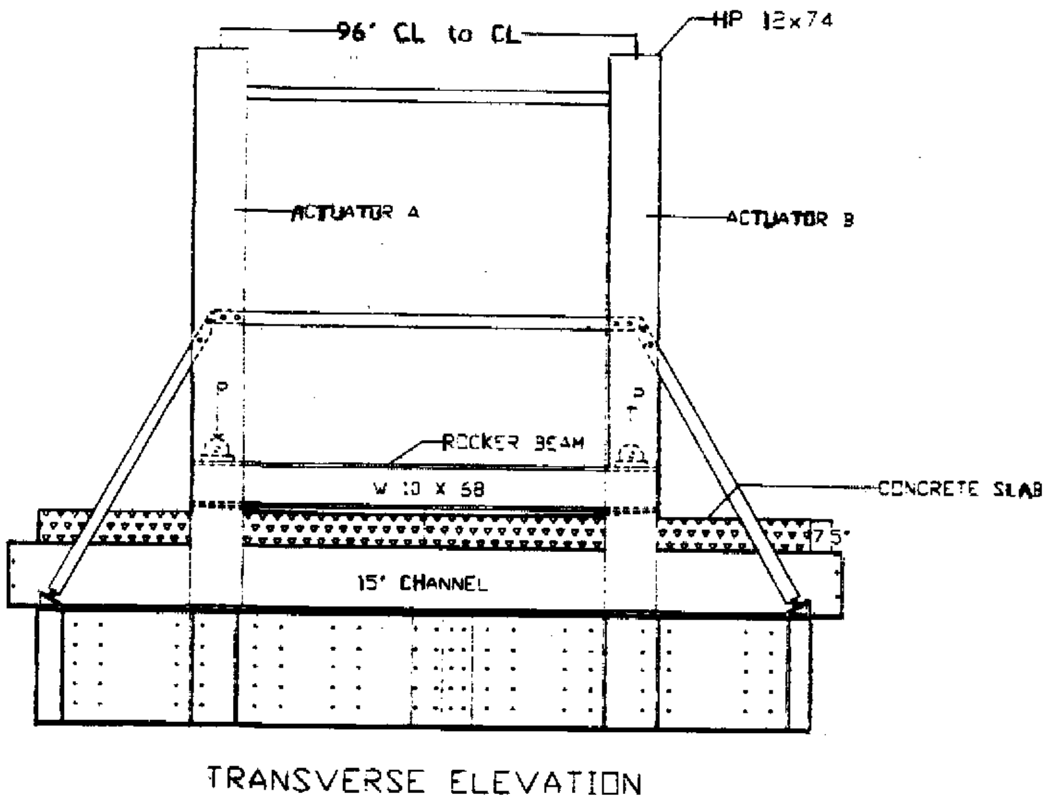


Figure A-2. Transverse elevation of test frame.

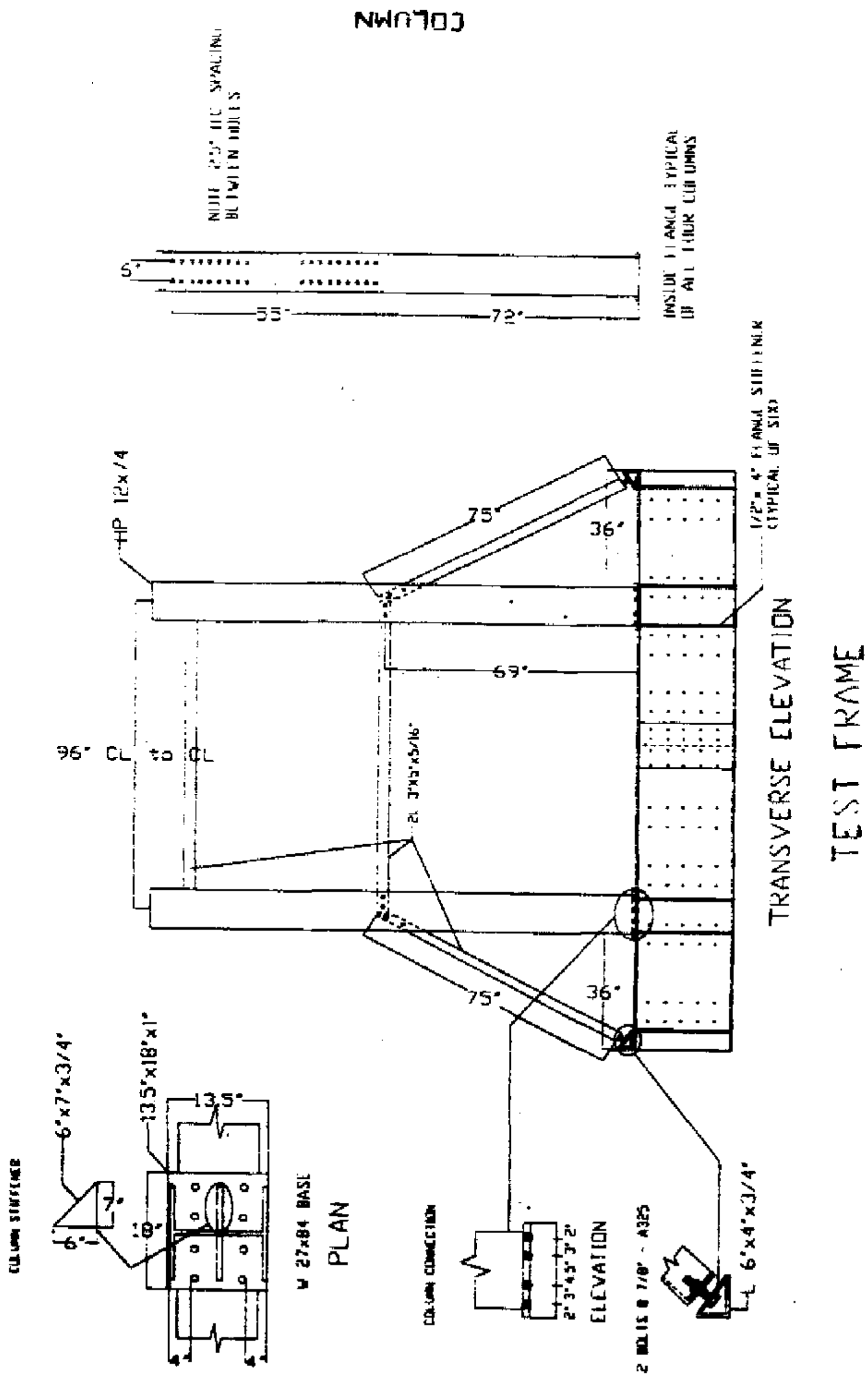


Figure A-3. Transverse elevation view of frame with details.

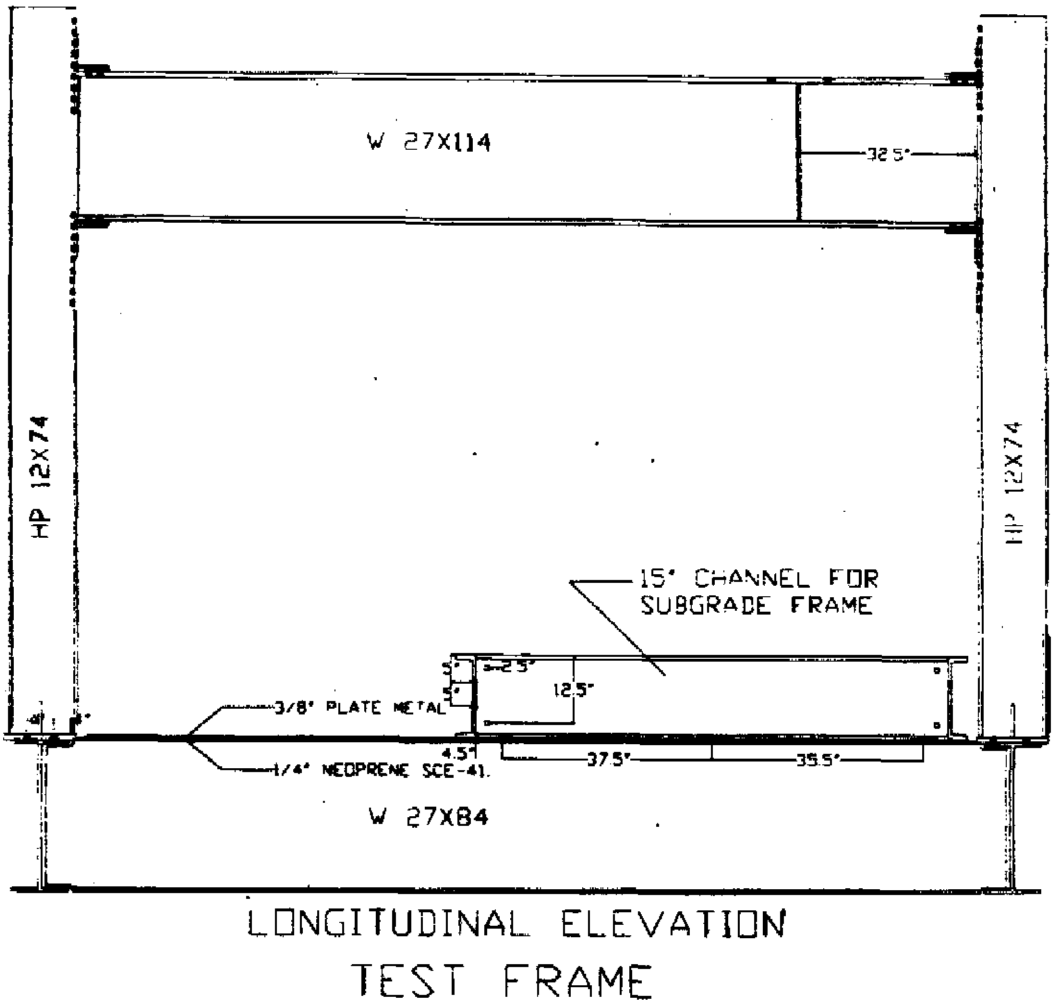


Figure A-4. Longitudinal elevation view of test frame.

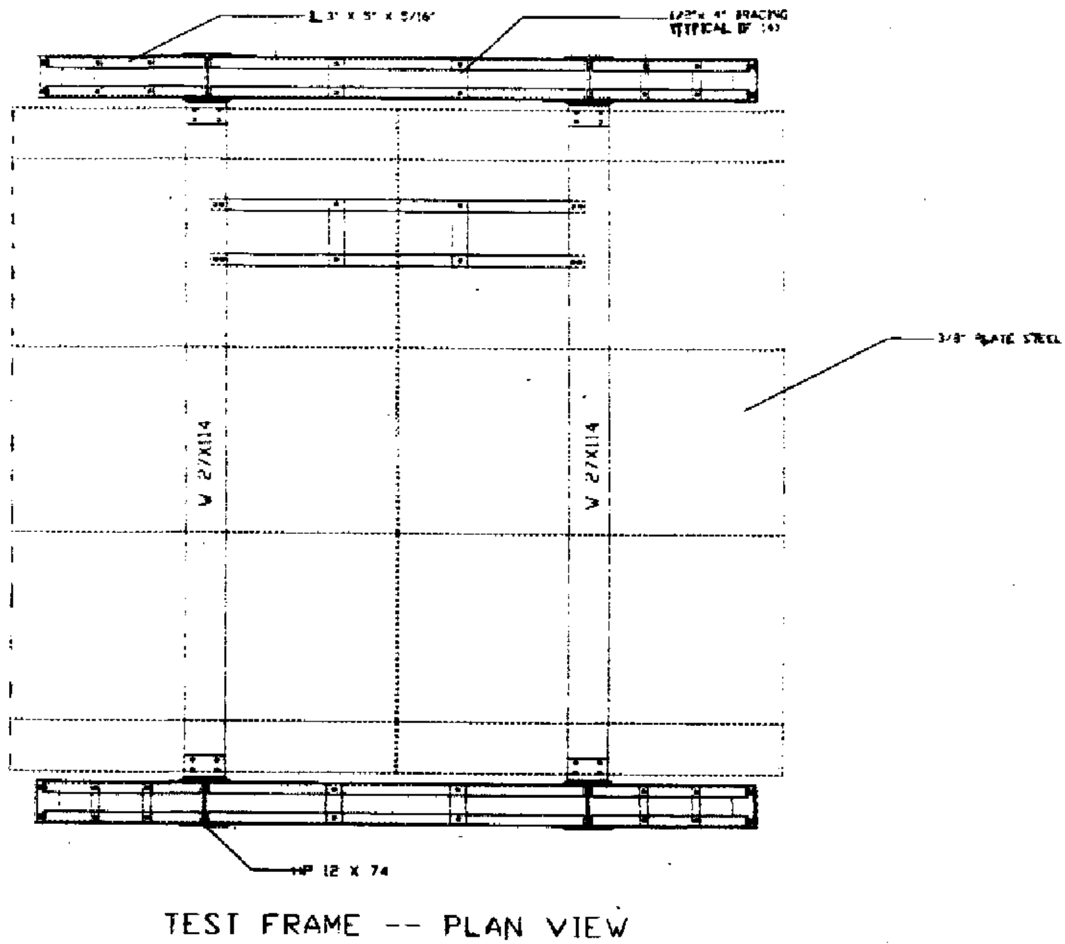


Figure A-5. Plan view of test frame.

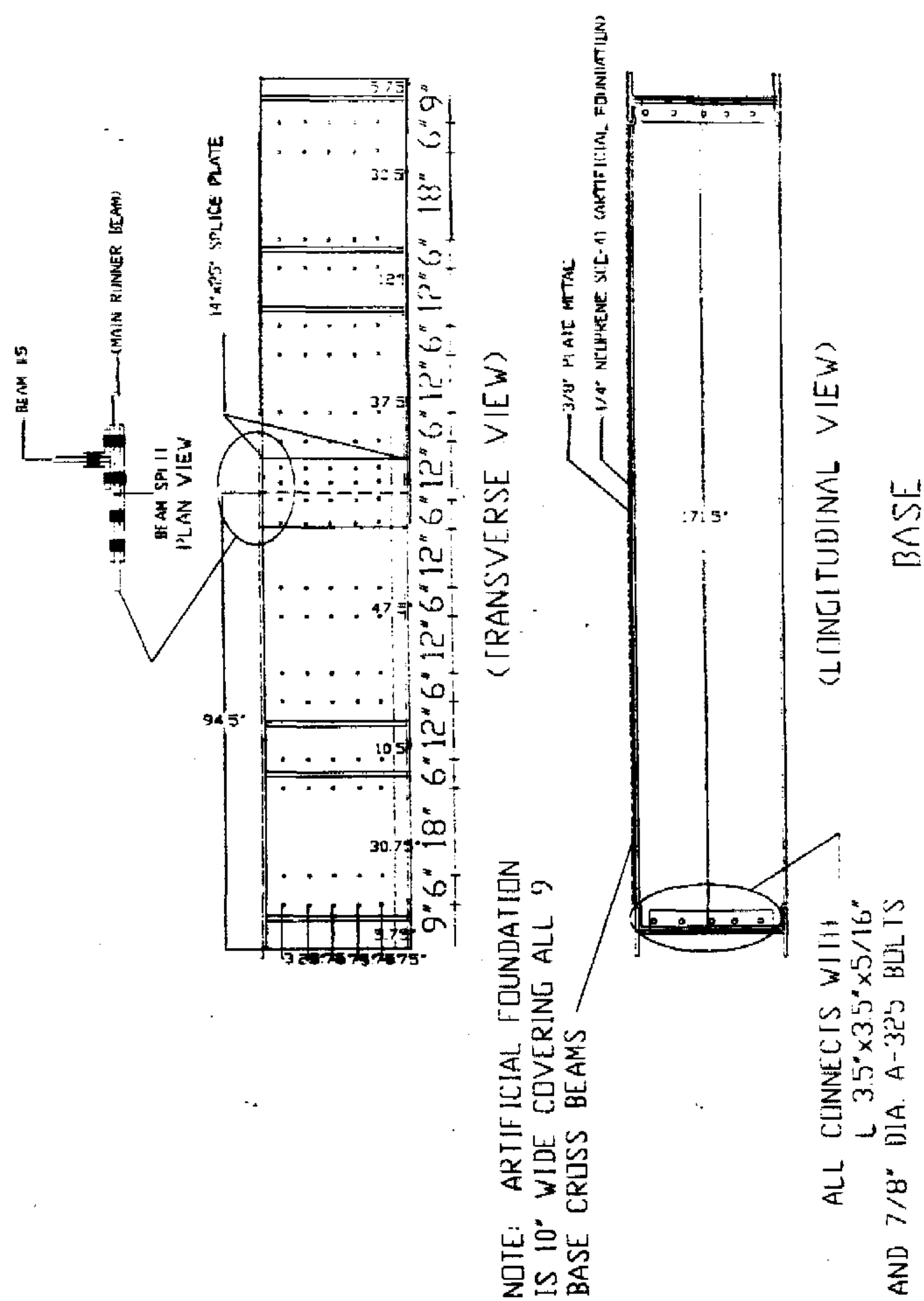


Figure A-6. Base longitudinal view with details.

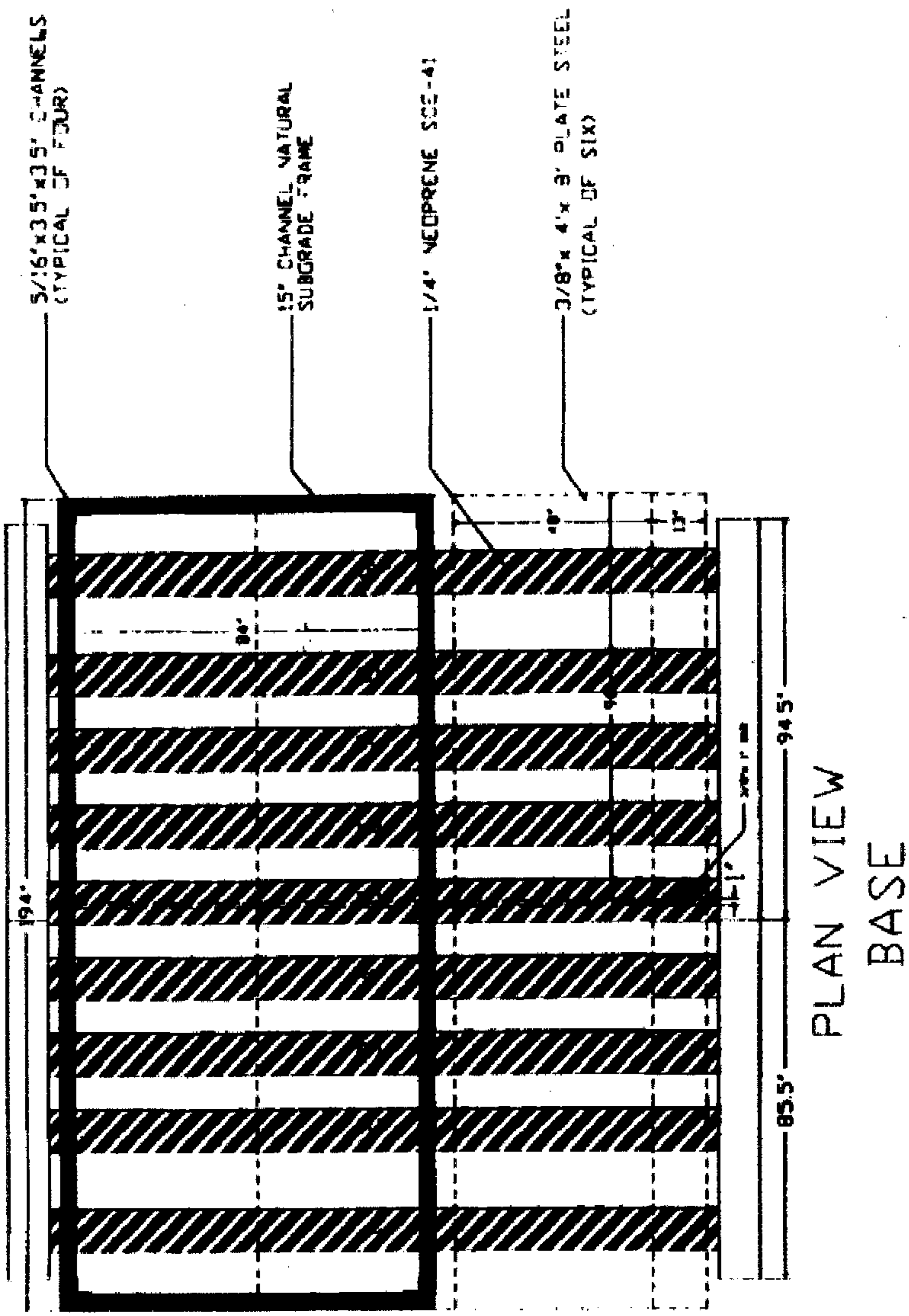


Figure A-7. Base plan view.

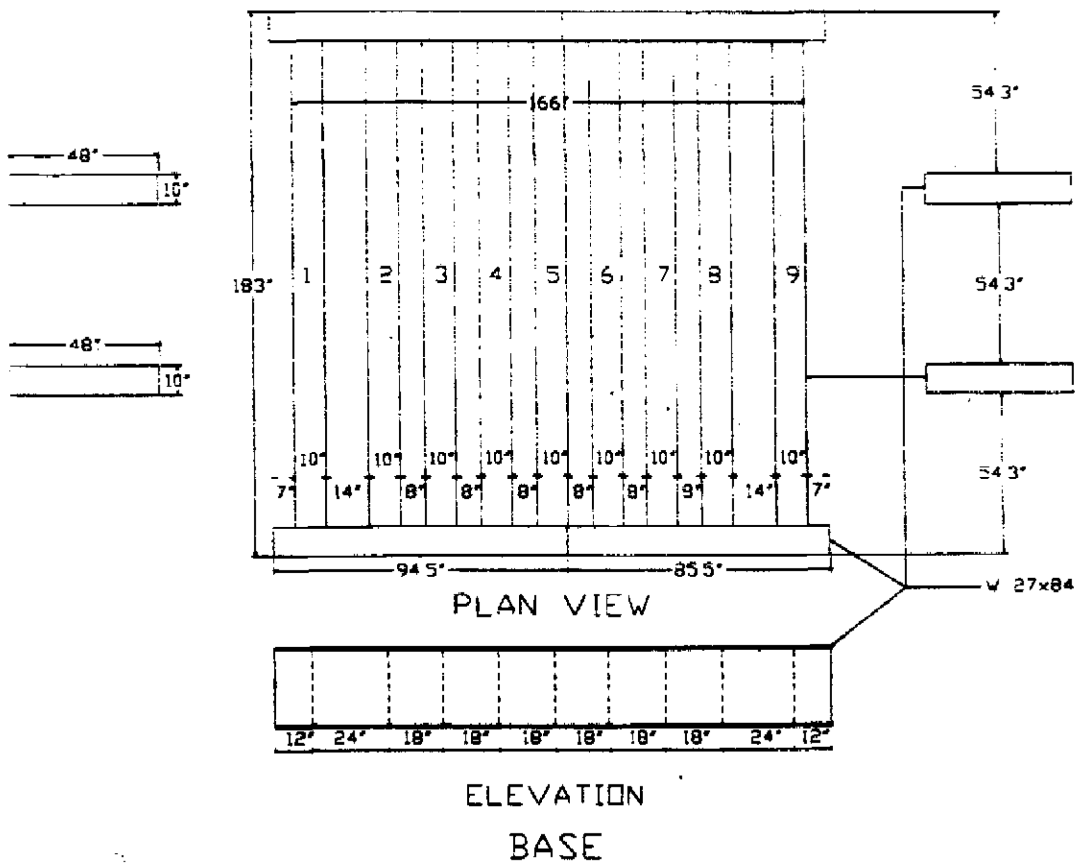


Figure A-8. Base plan and elevation view of girders.

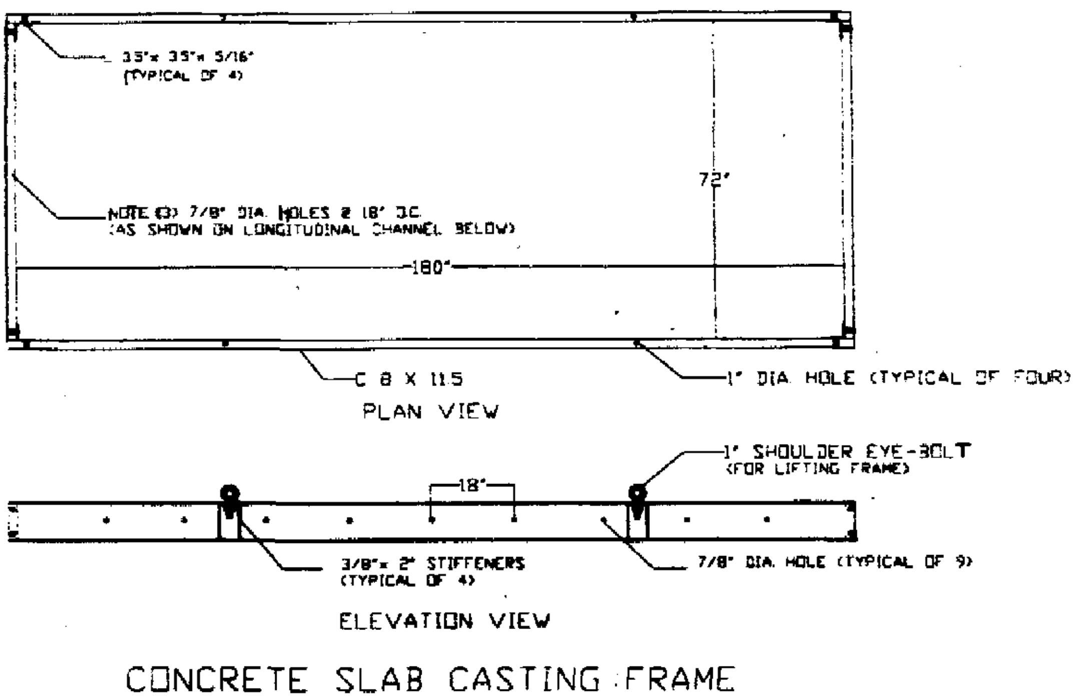


Figure A-9. Plan and elevation view of concrete slab casting frame.

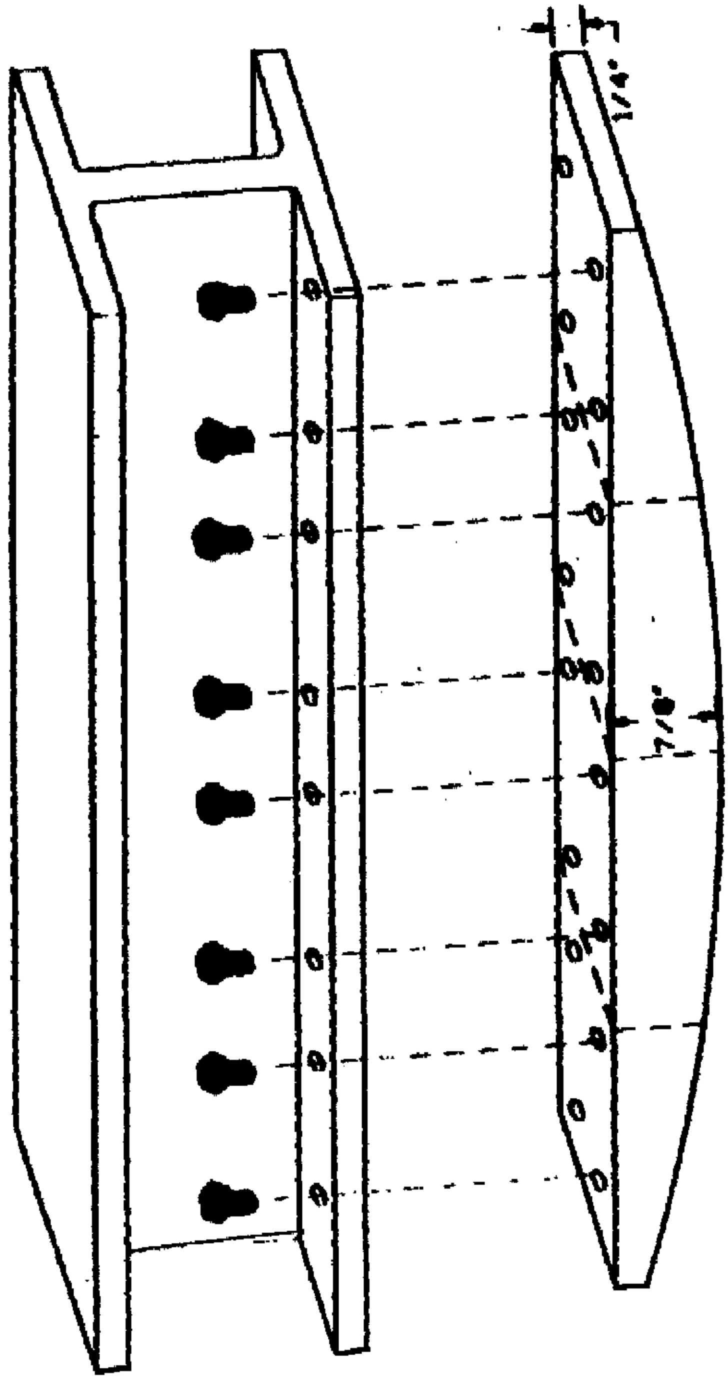


Figure A-10. Rocker-beam connection.

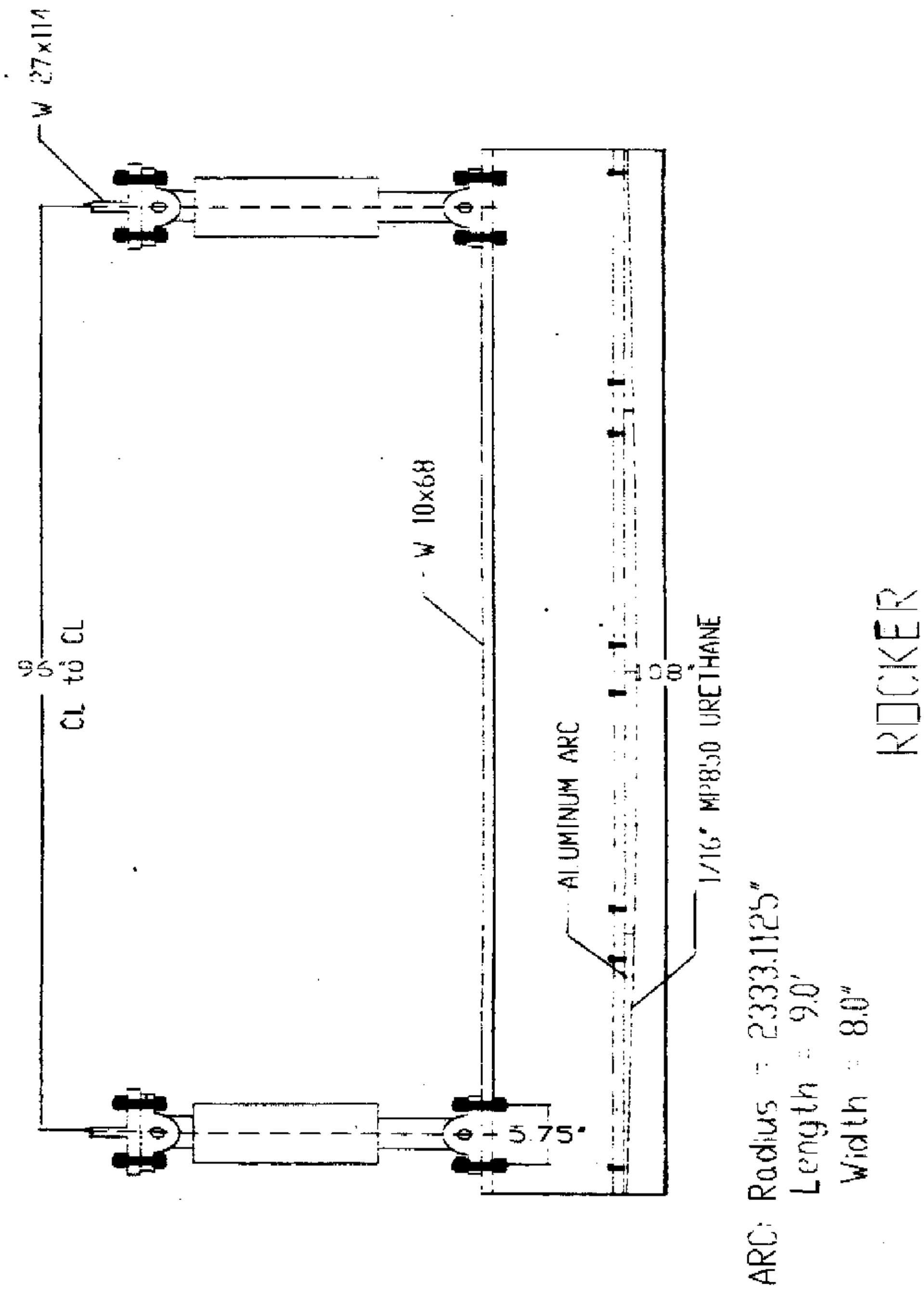


Figure A-11. Rocker and actuator assembly.

APPENDIX B

CONSTRUCTION DRAWINGS FOR MODIFIED MINNE-ALF

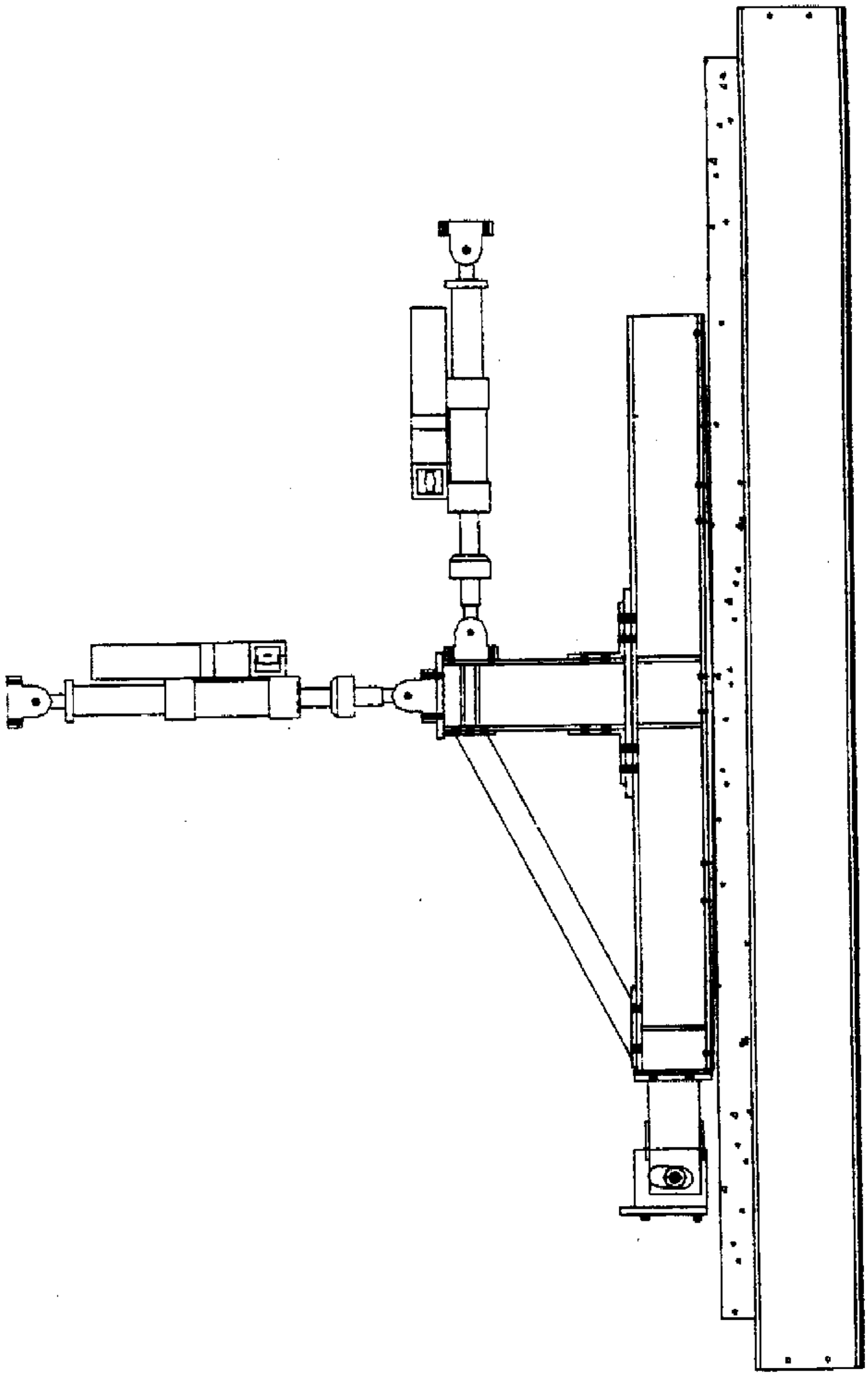


Figure B-1. Rocker beam and slab configuration.

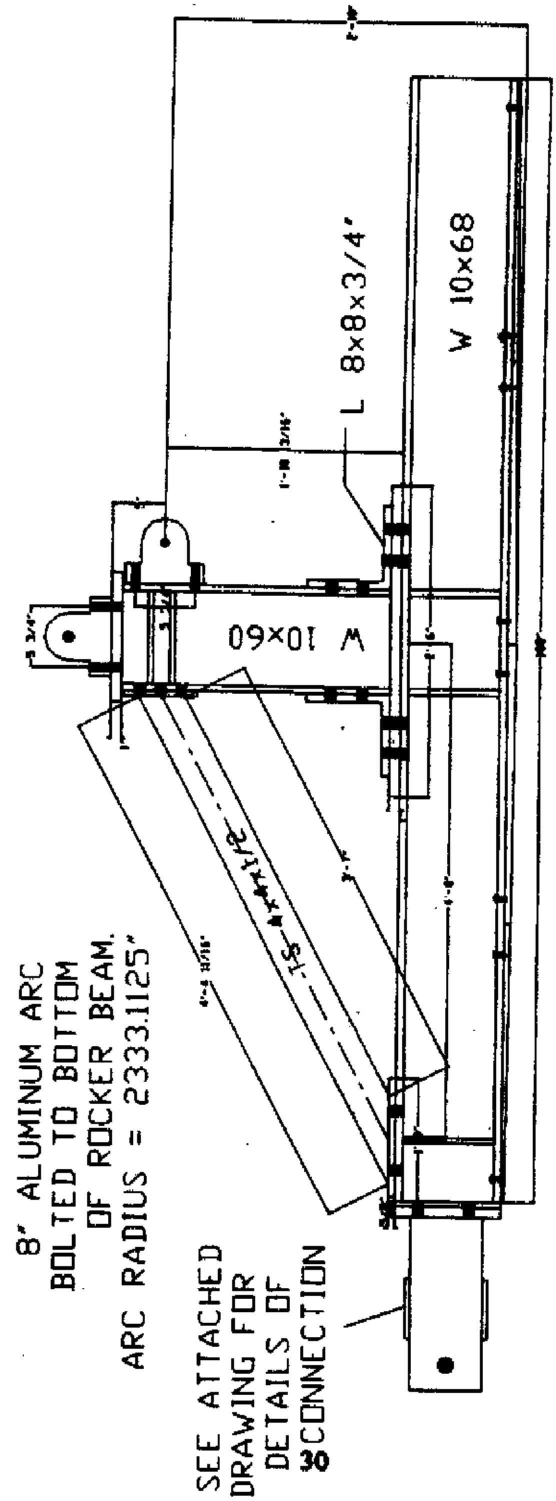


Figure B-2. Redesigned rocker beam.

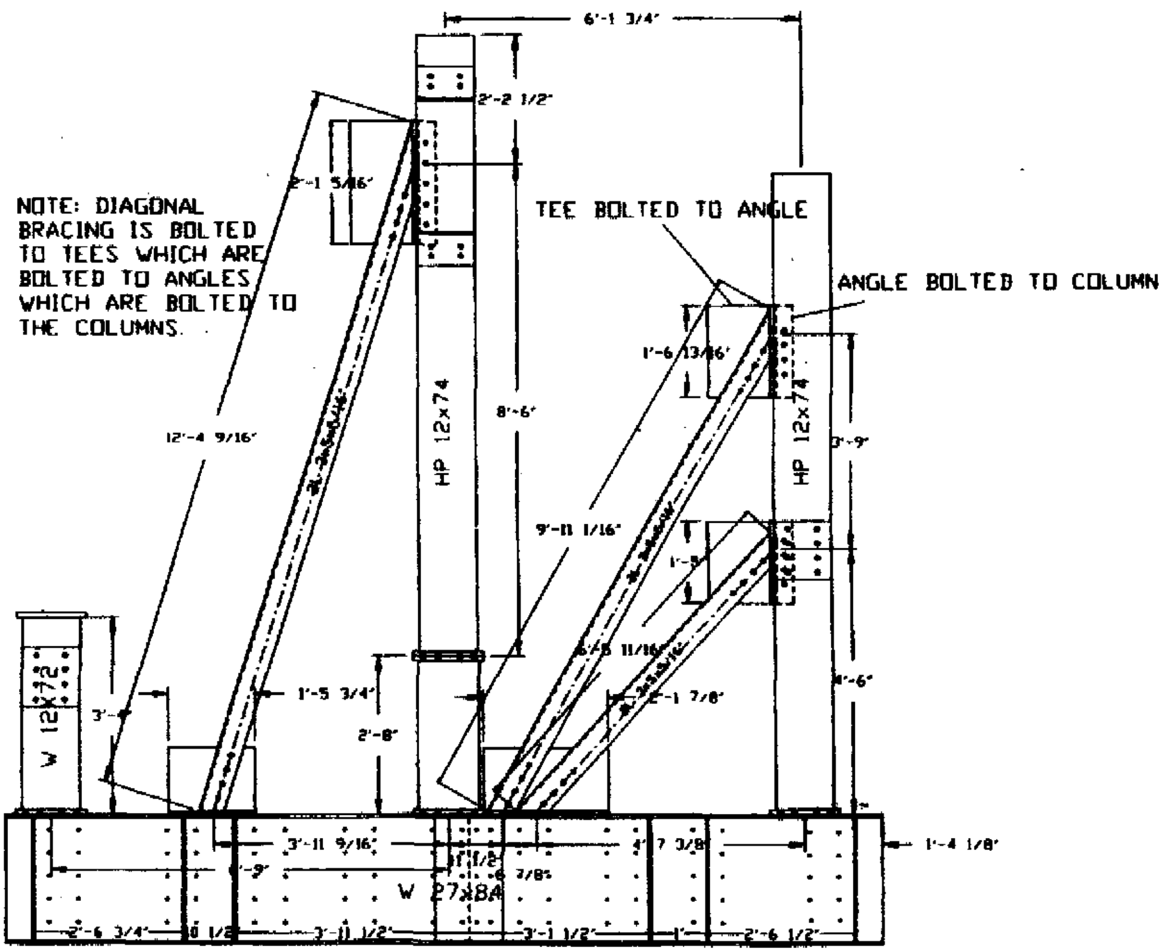


Figure B-3. Transverse elevation of modified Minne-ALF.

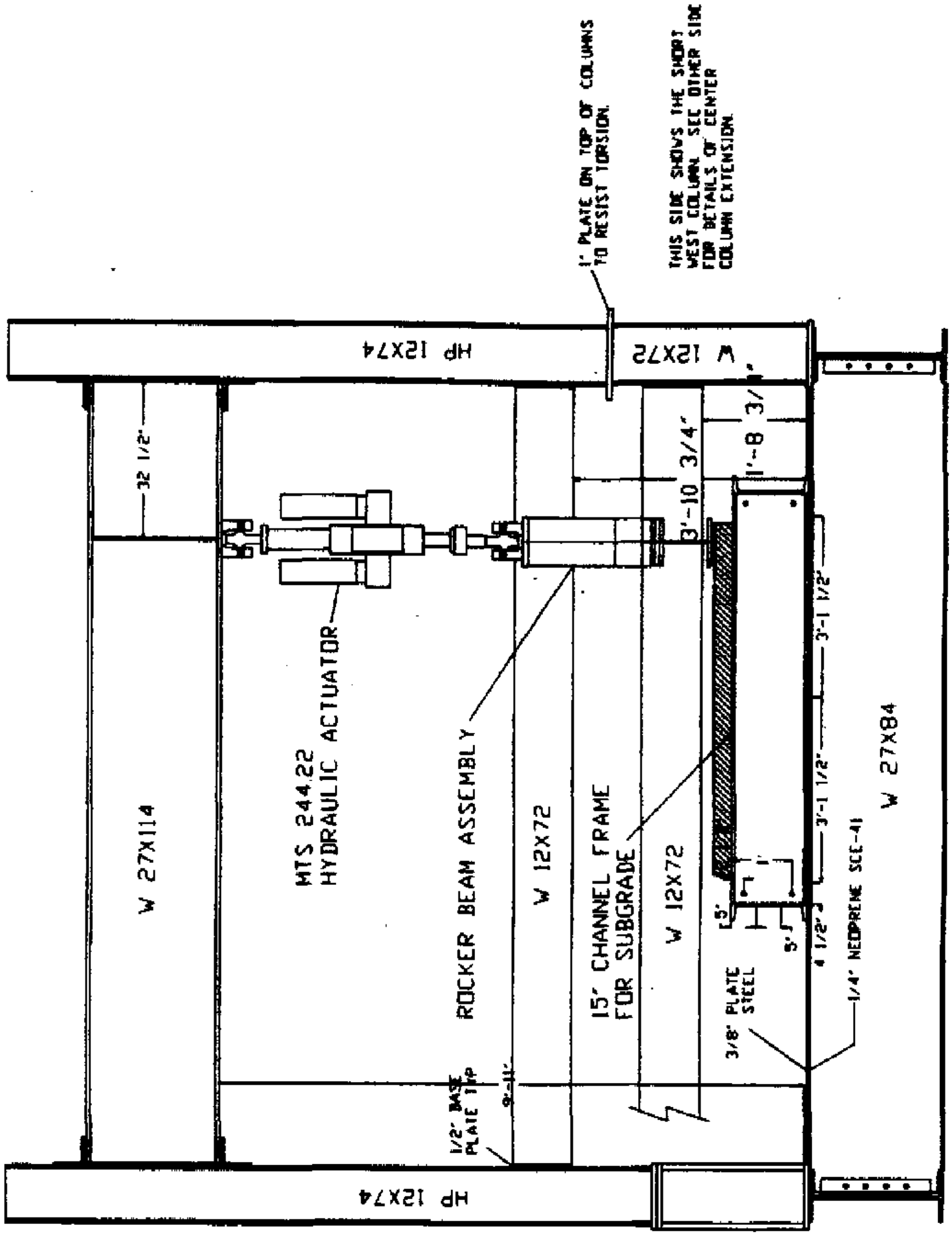


Figure B-4. Longitudinal elevation of modified Minne-ALF.

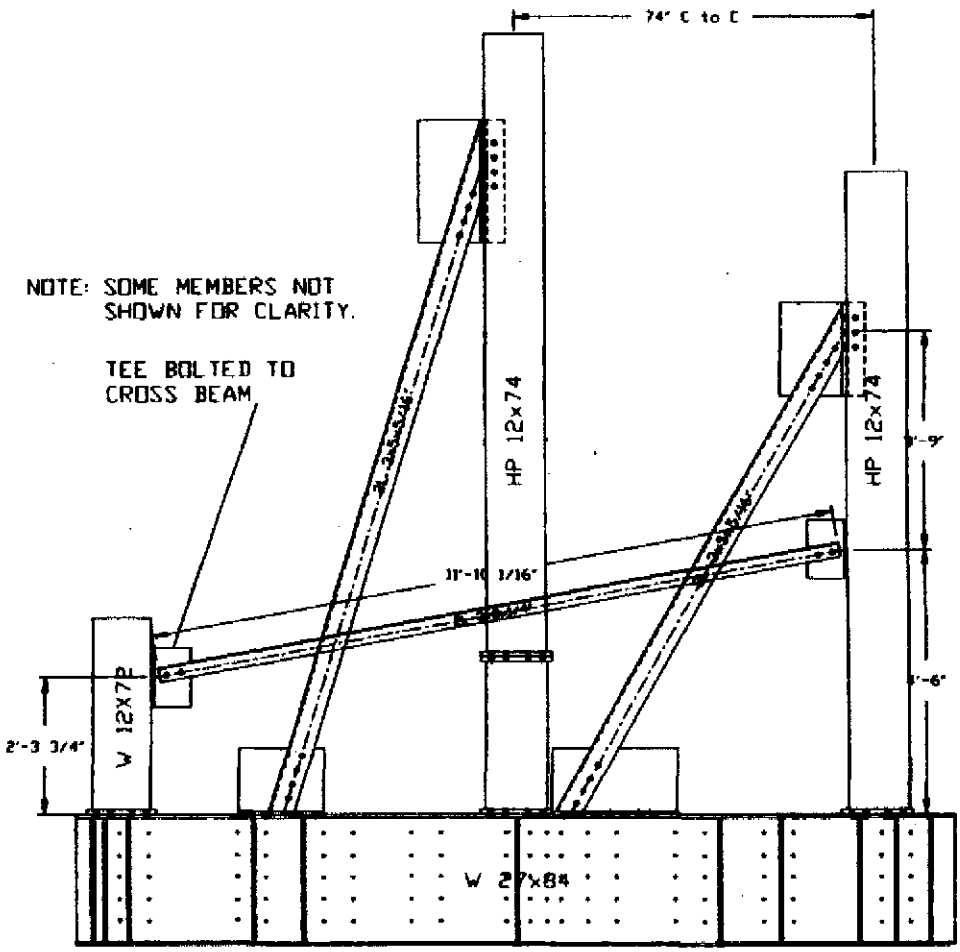


Figure B-5. Side view of diagonal stiffeners.

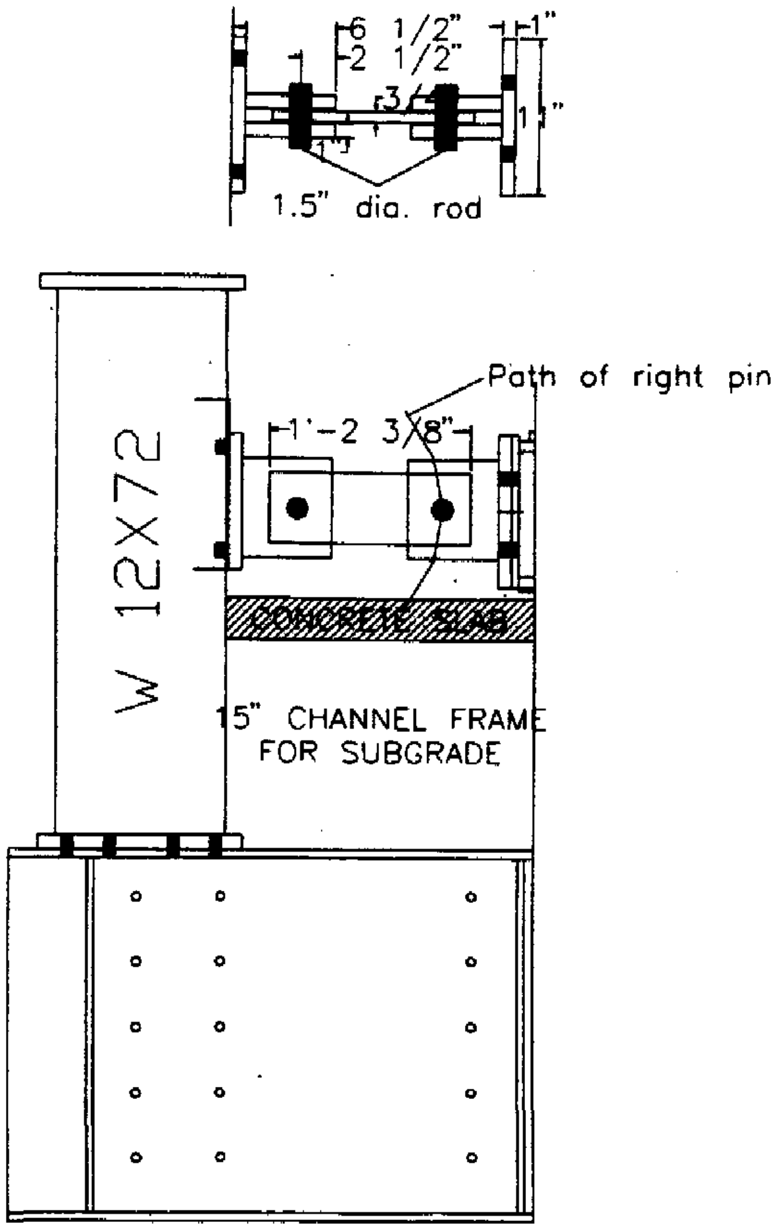


Figure B-6. Two-pin hinge connection.

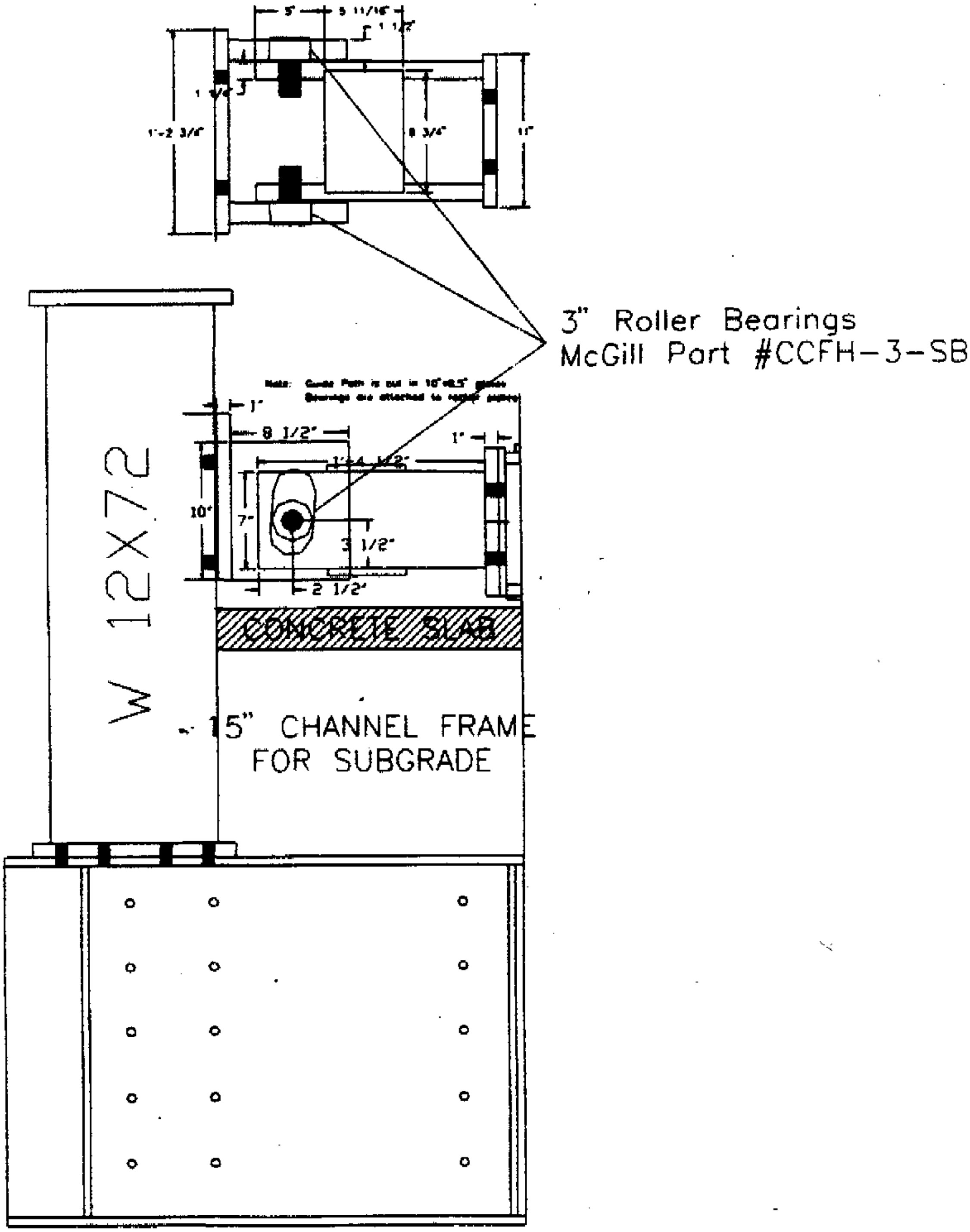


Figure B-7. Hinge connection between rocker beam and frame.

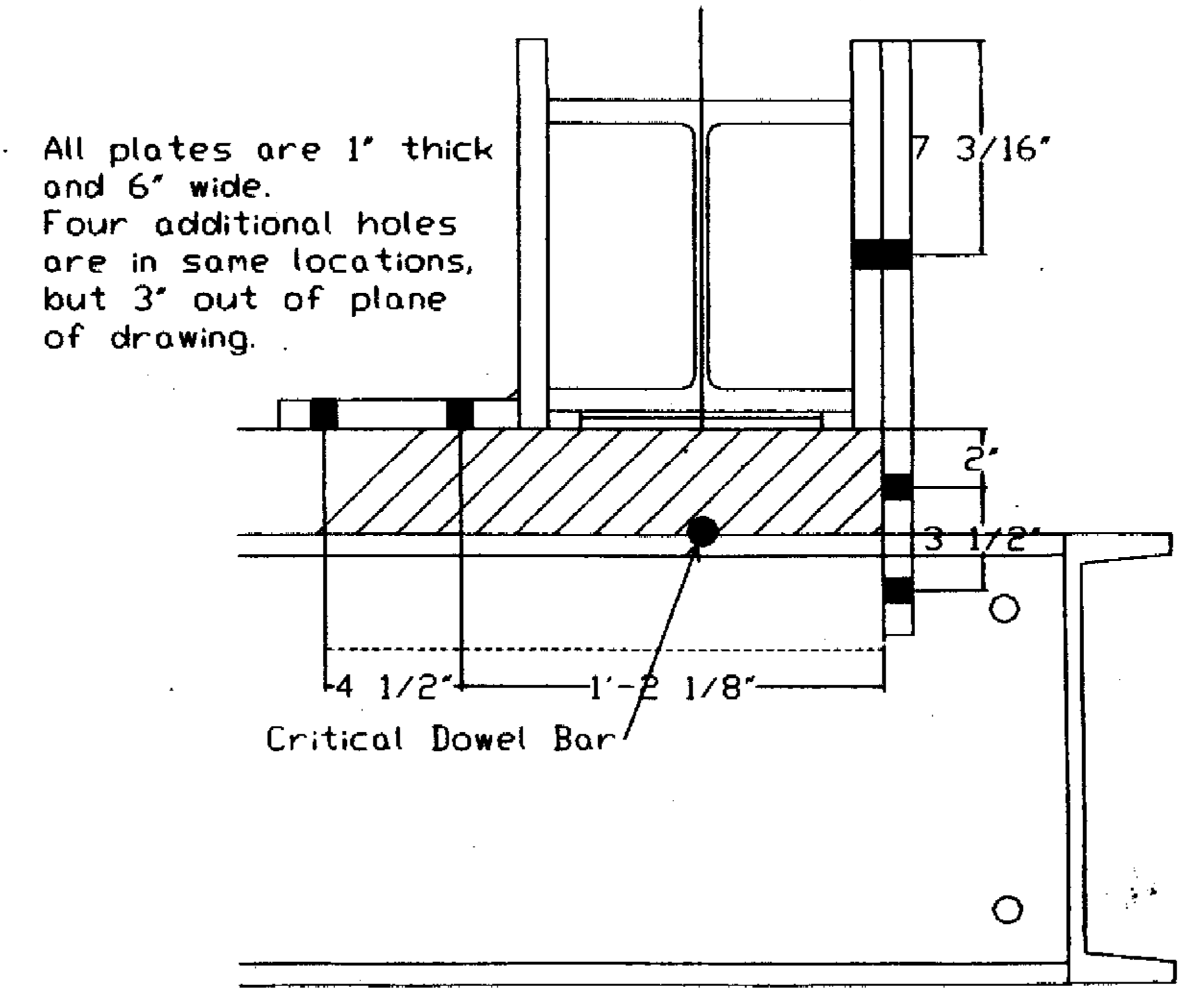


Figure B-8. Rocker beam guide plates.

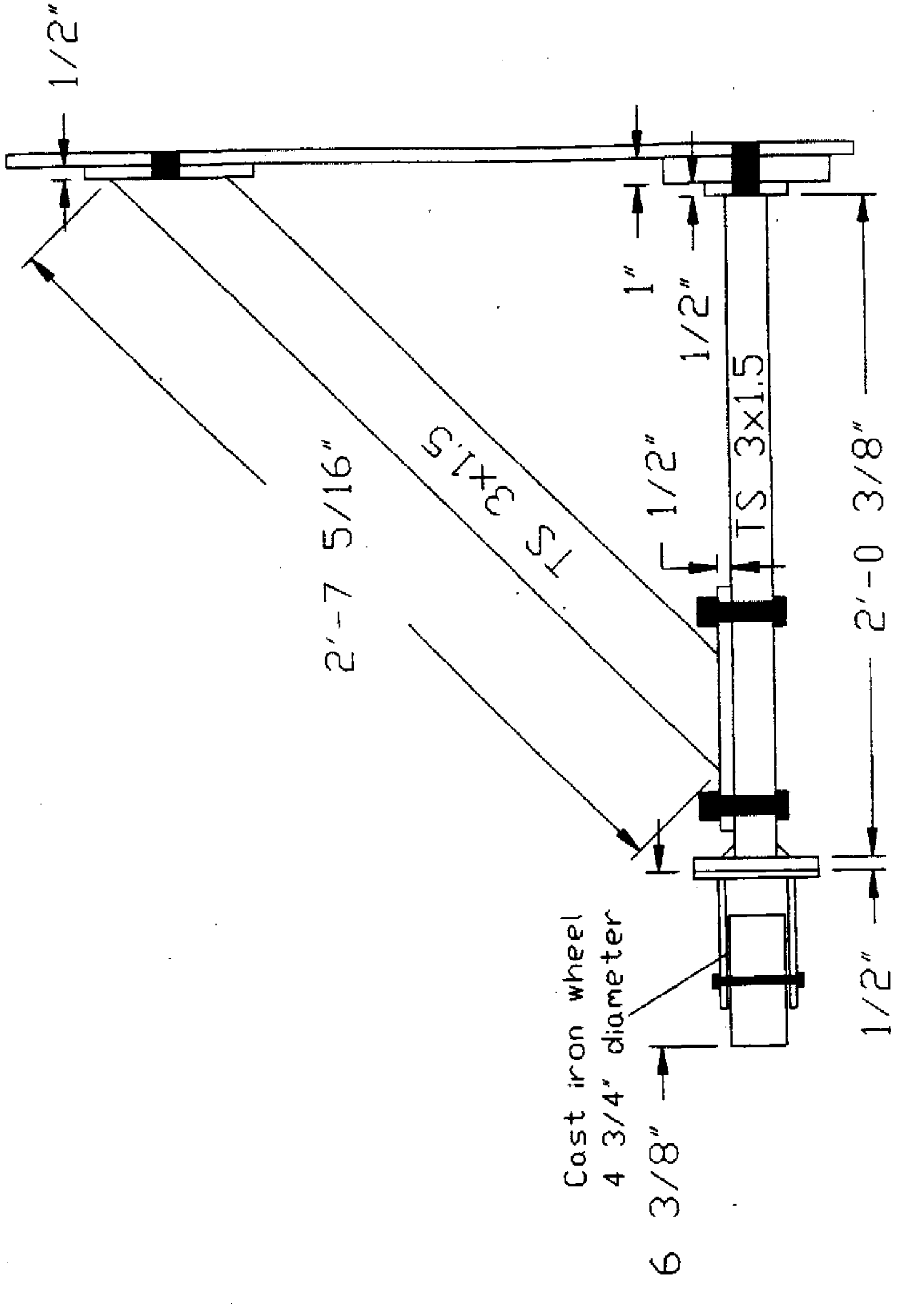


Figure B-9. Wheel brace for side of rocker beam.

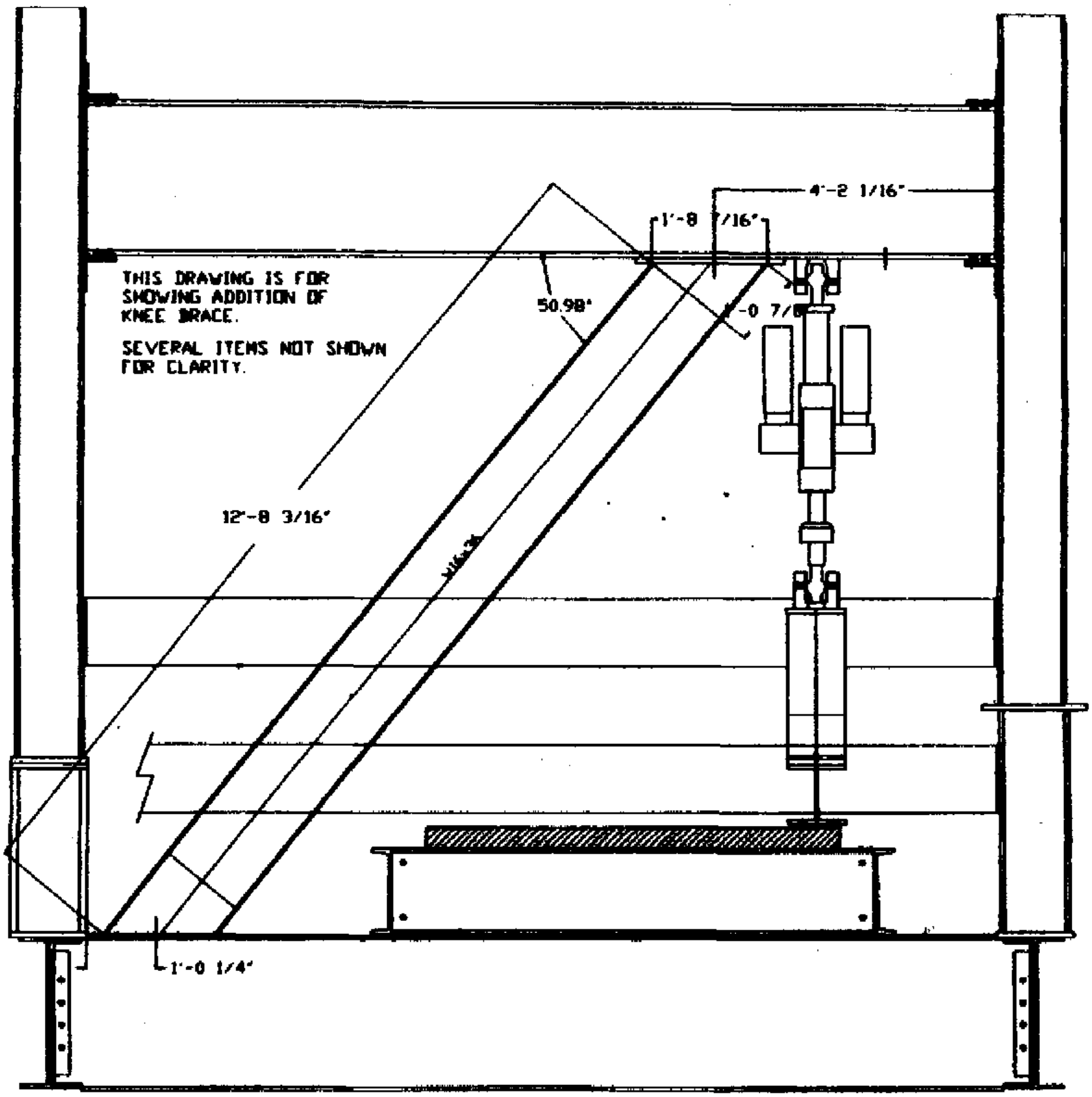


Figure B-10. Knee brace.

APPENDIX C

MN/DOT STANDARD SPECIFICATIONS FOR DOWEL BAR RETROFIT WORK (FIELD INSTALLATIONS)

APPENDIX C

Mn/DOT Concrete Pavement Rehabilitation Standards and Special Provisions for Epoxy Coated Dowel Bar Retrofits

Mn/DOT Concrete Pavement Rehabilitation Standards (7)

Construction Notes

This work shall meet the provisions of Specification 2301, except as modified herein:

Concrete Mixes

Repair Type[7]	Concrete Mix Grade	Minimum Time To Opening (hours)[1]	Admixture Dosage[2] & Type/ Curing Requirement
B	3U18	24	Maximum Type A
B	3U18	12	40% of Maximum Type E
C	3A32HE	24	Maximum Type A
C	3U22	12	40% of Maximum Type E[3]
C	3U27	12	40% of Maximum Type E[3]
C	3U28	12	40% of Maximum Type E[3]
C	3A32HE	12	40% of Maximum Type E[3] / Use curing blankets and insulation[4]
D (Less than 50' long)	3U22	24	25% of Maximum Type E[3]
D (Less than 50' long)	3U27	24	25% of Maximum Type E[3]
D (Less than 50' long)	3U28	24	25% of Maximum Type E[3]
D (Less than 50' long)	3A32HE	72	Maximum Type A[5]
D (From 50 to 200' long)	3A32HE	72	Maximum Type A
D (From 50 to 200' long)	3A32HE	[6]	[6]
D (More than 200' long)	3A32	[6]	[6]
D (More than 200' long)	3A41	[6]	[6]
D (More than 200' long)	3A32HE	[6]	[6]

[1] Providing that ambient and concrete temperatures exceed 60° F.
[2] Recommended dosage is from the manufacturer of the admixture.
[3] Shall be added as a slump increaser.
[4] Materials must meet Mn/DOT Specifications 3756 and 3760.
[5] Contact the Concrete Engineering Unit if an earlier opening time is required.
[6] Per Mn/DOT Specification 2301.

[7] Type B – Type B repairs generally consist of partial depth milling or chipping to remove deteriorated or delaminated concrete and preparation and placement of the repair. Type B-2D and B-2E repairs include removal to the bottom of the pavement if necessary.
Type C – Type C repairs consist of full depth removal of the concrete at joints or cracks and preparation and placement of the repair. Type CX repair is used in conjunction with Type C repairs if removal is required beyond the required 1 m (3ft – 6in) width, but less than 4 m (13 ft) total along centerline.
Type D – Type D repair is generally used for removal and replacement of one ore more concrete pavement panels. It is also used if the length of full depth repair within a panel exceeds 4 m (13 ft) along centerline.

Concrete Mix Grades 3U27, 3U28, and 3U22 shall have water reducing accelerator (Type E) used as a slump increaser, and the water reducing accelerator solution shall be included as part of the total recommended mixing water.

Accelerators should generally not be used when the ambient air temperature exceeds 27 °C (80 °F). If accelerators are used, they must be used with caution (Contact the Concrete Engineering Unit).

The air content for all grades of concrete shall be 6.5 % plus or minus 1.5 %. Specification 2461.4A4b shall be adjusted accordingly based on the 6.5 % target value.

The CA-80 gradation in Table 3137-2 of the 1995 Specifications shall be 100 % passing the 9.50 mm (0.375 in) sieve and 55-95 % passing the #4 sieve. For CA-80, not more than 5 percent shall pass the 300 μm (#50) sieve.

The contractor shall establish traffic control one day in advance of the beginning of the rehab operation for rehab surveys and locations.

Pavement Removal

1. Tapering of the edges of Type B partial depth repairs will generally not be required where the removal is accomplished by milling, either transversely or longitudinally. Secondary spalling resulting from milling, shall be jackhammered out and repaired at the contractor's expense.
2. For partial depth removals, no "jackhammers" shall be allowed. Removal hammers shall be limited to a maximum rated weight of 13.6 kg (30 lb).

3. Milling machines used for concrete removal shall be equipped with a device for stopping at preset depths to prevent damage to dowel bars.
4. The sawing of the adjoining slab full-depth 101.6 mm (4 in) wide and the removal of the concrete prior to installing a full-depth, full-width repair may be required as approved by the Engineer. Payment shall be made under the bid item 2301.603 "Relief Cut".
5. Overlays in saw cuts from removal operations shall be filled with silicone or hot pour (3725) joint sealant.

Repair Preparation

6. The bonding grout shall consist of an equal mass of portland cement and sand, mixed with sufficient water to form a slurry with the consistency of thick cream. The grout will require brushing or scrubbing (with a stiff bristle broom) into the in place concrete. The grout shall be mixed by mechanical means. Concrete shall be placed immediately after grouting. If the grout whitens, sand blast and regrout. The life of the grout shall not exceed 1.5 hours.
7. Dowel bars shall be placed parallel within a tolerance of 3.2 mm (0.125 in) in relation to the top of the planned pavement profile and the pavement centerline.
8. Either non-shrink grout or an epoxy anchorage shall be used for bonding reinforcing tie bars and dowel bars to in place concrete (contact the Concrete Engineering Unit for an approved products list). The drilled holes shall be clean and dry prior to any bonding agent placement. The bonding agent shall be fully set prior to placing concrete for all repairs. Epoxy coating is required on tie bars, dowel bars, and reinforcing steel.

Joint Reestablishment

9. To ensure that cracks are reestablished in their original locations for Type B repairs, parallel scribe their locations on the adjoining pavement outside the removal area prior to removal operations. The cracks shall be re-established at their original locations using a compressive material.
10. Sawing and sealing of joints and cracks for Type B and Type C repairs shall be in accordance with the appropriate Type A repair.
11. Immediately after sawing, all joints shall be thoroughly cleaned by water flushing.

12. Concrete repairs shall not protrude beyond the original cross-section of the pavement by 9.5 mm (0.375 in). The edges shall be formed or sawn full-depth.
13. Longitudinal and transverse joints shall be reestablished throughout all repairs.
14. Edging is required adjacent to all inserts in fresh concrete.
15. Misaligned dowel bars and those with cross-section loss shall be burnt off or otherwise severed. This work is incidental to repair. If this involves mores than three adjacent dowels, remove and replace the dowels using the appropriate repair detail. The placement of compression relief material is required.
16. All Type B-2 repair procedures must conform to the procedures for Type B-1 or Type B-3 when appropriate.
17. The removal of any inserts used in the reestablishment of joints in Type C repairs will not be allowed before 24 hours, except by sawing or as approved by the Engineer.
18. Restoration of contraction and longitudinal joints by green sawing on all Type C and D repairs shall be to a depth of one-third of the pavement thickness.

<u>Concrete Placement, Finish, and Cure</u>

19. Concrete for Type B partial depth repairs shall not be placed at air temperatures below 10 °C (50 °F).
20. The concrete surface texture shall consist of brooming in the direction of the long dimension of the repair.
21. Immediately after final finishing, all concrete shall be cured in accordance with Spec. 2531.3G2 "Membrane and Extreme Service Membrane Curing Method". The material for the Membrane Curing Method shall meet Specification 3754, and may not be water based. The material for Extreme Service Membrane Cure shall meet the requirements of 3755. Hudson sprayers may be used if the coverage rate is doubled and the curing material is from an agitated source (extreme service membrane cure is not required for Type B repairs).
22. Insulation of patches will be required in cool weather, (below 16 °C [60 °F] or when in place pavement temperatures are below 10 °C [50 °F]). When texture planing is required and the temperatures are below 16 °C (60 °F) (night or day), a blanket cure shall be applied for a minimum of 48 hours after placement. Beams or cylinders should be cast, if earlier opening times are required.

Special Provisions - Epoxy Coated Dowel Bar Retrofit (8)

Materials:

- The release agent shall be a liquid membrane-forming compound that conforms to the requirements of Section 3902 of the Mn/DOT Standard Specifications for Construction.
- Epoxy-coated dowel bars shall be in accordance with Section 3302 of the Mn/DOT Standard Specifications for Construction.
- The dowel bars shall have tight fitting end caps made of non-metallic materials that will allow for a 6.4 mm (0.25 in) movement of the bar at each end.
- The 6.4-mm (0.25-in) thick foam core board filler material shall be a closed cell foam faced with poster board material on each side commonly referred to as foam core board.
- The CA-8 gradation in table 3137-1 of the specifications shall be modified to read 55 to 95 percent passing the 4.75 mm (#4) sieve.
- Chairs for supporting and holding the dowel bar inplace shall be completely epoxy coated in accordance with AASHTO M 294, or made out of non-metallic materials.

Construction Requirements:

- 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.
- To prevent damage to the existing pavement designated to remain, any jackhammers used to break loose the concrete shall have a weight less than 13.6 kg (30 lb).
- All exposed surfaces and cracks in the slot shall be sandblasted and cleaned of saw slurry and any release agent prior to installation of the dowel.
- Dowel bars shall be lightly coated with a release agent prior to placement and placed in a chair that will provide a minimum of 12.7 mm (0.5 in) clearance between the bottom of the dowel and bottom of the slot. The dowel bars shall be placed to the depth as shown in the Plans, parallel to the centerline, and parallel to the pavement surface of the lower panel at the transverse joint, all to a tolerance of 6.4 mm (0.25 in).

The chair design shall hold the dowel bar tight inplace during placement of the grouting material.

- Immediately prior to placement of the dowel bar and filler material, the Contractor shall caulk the existing transverse joint crack as shown in the Plans. The caulking shall also be placed to provide a smooth level surface and tight fit for the foam core board filler material to the bottom of the slot. The transverse joint crack shall be caulked sufficiently to satisfy the about requirements and to prevent any of the grouting material from entering the joint crack at the bottom or the sides of the slot.
- A 6.4-mm (0.25-in) thick foam core board filler material, as approved by the Engineer, shall be placed at the middle of the dowel to maintain the transverse joint as shown in the plans. The filler material shall fit tight around the dowel and to the bottom and edges of the slot. The filler material shall be capable of remaining in a vertical position and tight to all edges during placement of the grout. If for any reason the filler material shifts during placement of the grout, the work shall be rejected and redone at the Contractor's expense.

APPENDIX D

IFT/DRIVE FILE GENERATION LOG

APPENDIX D

ITF/Drive File Generation Log

Slab 1

Shake Down #1 (Day 1):

The rocker beam was vibrated through 61 program segments in order to find the optimum drive frequencies and generate an ITF. The rocker beam was then rocked through 179 segments without saving the drive function (while the system attempted to converge on the generation of the desired drive file), and a total of 130 segments while saving the file. Therefore, the slab went through a total of 309 rocking segments during this first system shakedown. However, the response produced by the command signals following the first shakedown were judged to be inadequate for long-term specimen testing, so the process was repeated.

Shake Down #2 (Day 1):

The rocker beam was vibrated through 63 program segments in order to find the optimum drive frequencies and generate an ITF. The slab then underwent an additional 181 program segments without saving the drive file and 159 segments while saving the file, for a total of 340 rocking segments during the second shakedown. This drive file was deemed satisfactory, and was used for subsequent testing of the slab.

Slab 2

The TestStar hardware and computer software (TestWare) were upgraded prior to the testing of slab 2. This resulted in great difficulties during the creation of the ITF and drive files for this test slab.

The following log summarizes the approximate loads and numbers of cycles Slab 2 was subjected to during the various attempts to created an adequate ITF file. The days recorded represent the number of days *after* the dowel bars were retrofitted into the slots. Note that after each attempt to create the ITF file, adjustments were made to either TestStar and/or TestStar configuration file and to the TestWare program in attempt to solve the loading/stroke problem encountered.

13 Days:

- 64 vibrations to find optimum frequencies.
- 15 rocking cycles at approximately 9,000 lbs prior to the system going unstable.

- no compensation.
- 282 rocking cycles at approximately 9,000 lbs prior to the system going unstable.

15 Days:

- 64 vibrations to find optimum frequencies.
- 22 rocking cycles at approximately 15,000 lbs.

16 Days:

- 57 vibrations to find optimum frequencies.
- 60 rocking cycles at approximately 9,000 lbs prior to the system going unstable.

- 67 vibrations to find optimum frequencies.
- 20 rocking cycles at approximately 22,000 lbs.

- 73 vibrations to find optimum frequencies.
- 17 rocking cycles at approximately 22,000 lbs.

- 58 vibrations to find optimum frequencies.
- 27 rocking cycles at approximately 8,000 lbs.

21 Days:

- Adjustments made to the tuning (PIDF)
- 181 rocking cycles at approximately 15,000 lbs. running under no compensation
- 264 rocking cycles at approximately 10,000 lbs. running under no compensation

- 60 vibrations to find optimum frequencies.
- 28 rocking cycles at approximately 15,000 lbs.

- 15 Hz cutoff freq., P=7.04 Hz
- 60 vibrations to find optimum frequencies.
- 34 rocking cycles at approximately 10,000 lbs. prior to the system going unstable.

- 61 vibrations
- 8 rocking cycles at approximately 10,000 lbs. prior to the system going unstable.

<u>22 Days:</u>

- ran original shape file at 1 Hz
- 30 vibrations to find optimum frequencies.
- 109 rocking cycles at approximately 10,000 lbs. prior to the system going unstable.

- Set convergence rate to .05
- 58 vibrations to find optimum frequencies.
- 62 rocking cycles at approximately 9,000 lbs. prior to the system going unstable.

<u>23 Days:</u>

- New processor board 490.50B update rate 2049
- 50 vibrations to find optimum frequencies.
- 61 rocking cycles at approximately 9,000 lbs. prior to the system going unstable.

- Changed update rate to 3790
- 54 vibrations to find optimum frequencies.
- 36 rocking cycles at approximately 9,000 lbs.

Twenty-three days after the dowel bars were retrofitted the final shakedown was run. The 490.50B processor board along with a different firmware program solved all the previous instability and loading problems. The rocker beam was vibrated through 58 segments in order to find the optimum frequencies. The rocker beam was then rocked through 264 segments without saving and 175 segments saving the file. The slab went through a total of 439 rocking segments during this shakedown. The cylinders for the backfill material were broken immediately after the successful shakedown in order to determine the strength of the 3U18 material at the time testing commenced.

Slab 3

Shake Down #1 (Day 1):

The rocker beam was vibrated through 57 segments in order to find the optimum frequencies. The rocker beam was then rocked through 82 segments without saving the file. The response produced by the command signals for this first shakedown did not appear to be adequate for testing. It appeared that the guide plate at the one end of the rocker beam was pushing the beam out of plumb, therefore, the guide plate was readjusted and a second shakedown was performed.

Shake Down #2 (Day 1):

The rocker beam was vibrated through 57 segments in order to find the optimum frequencies. The slab underwent an additional 207 segments without saving the drive file and 173 segments saving the drive file.

Slab 4

Shake Down #1 (Day 1):

The rocker beam was vibrated through 57 segments in order to find the optimum frequencies. The rocker beam was then rocked through 162 segments without saving the drive file. The system went unstable and shut down due to low oil interlock.

Shake Down #2 (Day 1):

The rocker beam was vibrated through 58 segments in order to find the optimum frequencies. The slab underwent an additional 222 segments without saving the drive file and 107 segments saving the drive file.

Shake Down #3 (Day 2):

The load profile had numerous loading spikes due to the misalignment of the bearing plate attached to the west horizontal girder. The bearing plate was readjusted and the rocker beam was moved accordingly. During the third shake down the rocker beam was vibrated through 61 segments in order to find the optimum frequencies. The slab underwent an additional 338 segments without saving the drive file and 278 segments saving the drive file.

Shake Down #4 (Day 87):

Manifold on horizontal actuator (Actuator B) cracked and swivel on vertical actuator (Actuator A) began to fail, therefore, switched horizontal actuator with vertical actuator and vice versa. LVDTs required removal to clean up oil spill. During the forth shake down the rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 257 segments without saving the drive file and 190 segments saving the drive file. The drive file did not save correctly due to the system going unstable, therefore, the slab went an additional 190 segments to save the drive file.

Slab 5

Shake Down #1 (Day 1):

The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 286 segments without saving the drive file and 167 segments saving the drive file.

Shake Down #2 (Day 34):

The load profile had numerous loading spikes. During the second shake down the rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 272 segments without saving the drive file and 173 segments saving the drive file.

Slab 6

Shake Down #1 (Day 1):

The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 305 segments without saving the drive file and 139 segments saving the drive file. The system went unstable preventing the remaining 48 cycles to be compensated and saved.

Shake Down #2 (Day 1):

The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 160 segments without saving the drive file. The system went unstable prior to a good load profile being generated during compensation.

Shake Down #3 (Day 1):

The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. After the 63 segments of white noise the main hydraulic pump was tripped due to low oil fluid.

Shake Down #4 (Day 1):

The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 337 segments without saving the drive file and 214 segments saving the drive file.

Shake Down #5 (Day 2):

The load profile had some noise, therefore, tried clean it up by creating a new drive file. During the fifth shake down the rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 306 segments without saving the drive file. Was unsuccessful in creating a load profile without noise.

Shake Down #6 (Day 35):

Swivel on horizontal actuator was replaced with new one. The load profile was not responding to the new swivel, therefore, a new drive file needed to be created. During the sixth shake down the rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 423 segments without saving the drive file. Was unsuccessful in creating a load profile without noise.

Shake Down #7 (Day 35):
The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 324 segments without saving the drive file and 198 cycles saving the drive file. The system went unstable before the drive file could be created.

Shake Down #8 (Day 37):
Tried to create a drive file without load spikes, since unsuccessful on Day 35. The rocker beam was vibrated through 63 segments in order to find the optimum frequencies. The slab underwent an additional 371 segments without saving the drive file. The system went unstable before the drive file could be created.

Shake Down #9 (Day 66):
Tried to create a drive file without load spikes, since unsuccessful on Day 37. The rocker beam was vibrated through 39 segments in order to find the optimum frequencies. The slab underwent an additional 245 segments without saving the drive file and 194 segments saving the drive file (drive file created at 1.5 hertz).

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