FACTORS AFFECTING THE DETERIORATION OF TRANSVERSE CRACKS IN JRCP

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DISCLAIMER

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ABSTRACT

Jointed Reinforced Concrete Pavement (JRCP) develops transverse cracks as the drying and thermal shrinkage of the concrete is resisted by friction with the supporting layers. These cracks deteriorate with time and traffic due to loss of aggregate interlock load transfer capacity. However, unusually rapid deterioration of these cracks has been observed on some recently-constructed projects in Michigan. This rapid crack deterioration leads to accelerated maintenance requirements and shortened service lives. This research report describes the development, conduct and results of a laboratory investigation to determine the relative effects of selected factors on the deterioration of transverse cracks in JRCP.

A large-scale pavement test stand was developed to allow the rapid application of simulated heavy vehicle loads to concrete pavement slabs. The study involved the collection and analysis of load transfer data taken at the transverse crack induced in each slab. The data was used to help determine the impact of several pavement materials and design features on the rate of crack deterioration. These factors include aggregate type, treatment and gradation, foundation support, reinforcement type and quantity and slab tension.

The results of this test program suggest that reductions in transverse crack deterioration in JRCP can be achieved by using combinations of materials and structural designs that provide tight cracks, good long-term aggregate interlock, minimize differential deflections across joints and cracks, and reduce total deflections at any location in the pavement structure. For example, the provision of strong foundation support was especially effective because of reductions in the magnitude of the peak and differential displacements between the slab faces. Test results also suggest that the use of slab reinforcing designs that hold the cracks more tightly closed will

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provide improved crack performance. Such reinforcing designs may include the use of deformed steel or larger quantities of steel. Reducing the tensile stresses in the steel by shortening the joint spacing or using a subbase material which minimizes the friction between the slab and the subbase layer also helps to keep cracks tight.

Based on the results of these tests, it is recommended that pavements made with concrete derived from recycled concrete aggregate or slag should feature structural designs that minimize reliance on aggregate interlock in any area of the design (i.e., at joints or cracks). Appropriate structural designs might include jointed plain concrete pavements with doweled joints and panel lengths less than 5 m (16.4 ft), or pavements that use the "hinged joint" design. In addition, at least one state has been successful at using recycled concrete in CRCP. The use of blended aggregates (recycled concrete or slag combined with suitable natural aggregates) may be useful to provide additional design reliability, but is probably not necessary for the types of designs described above.

Transverse crack deterioration appeared to be strongly correlated with concrete strength (presumably due to reductions in pavement stiffness and abrasion resistance that probably accompany the use of weak concrete). Thus, pavements made with concrete that includes relatively weak aggregate particles, such as slag and recycled concrete, should: a) use mix designs that provide concrete strengths that are comparable to those of concrete made with virgin aggregates; b) use structural designs that reduce pavement stresses to levels that are appropriate for the strength that will be obtained; or c) do both. Inspection of batching, placement and curing must ensure that all mixes develop the required strengths.

It is the opinion of the authors that any of the aggregates included in this study could be used in Michigan concrete paving operations if appropriate concrete mixture proportioning and structural design modifications are made. It must be remembered that the use of different concrete aggregates results in the production of concrete with widely varying physical and mechanical properties; the use a "standard" structural or concrete mixture design for all materials can not be expected to produce pavements with comparable performance.

1.0 INTRODUCTION

Jointed reinforced concrete pavements (JRCP) typically develop transverse cracks over the first several years of their service lives due to one or more of the following mechanisms:

- Contractions of the slab (caused by combinations of uniform drying and thermal shrinkage) are restrained by friction between the slab and supporting layers.
- Stresses induced by temperature gradients through the pavement thickness (i.e., "curling" stresses).
- Stresses induced by moisture gradients through the pavement thickness (i.e., "warping" stresses).
- Stresses induced by vehicle loads.

Most JRCP designs rely on aggregate or grain interlock to transfer shear loads across these cracks. The loss of aggregate interlock due to opening of these cracks permits increased slab deflections, and may be accompanied by the infiltration of water and the intrusion of incompressibles into the cracks. These, in turn, lead to pumping and crack deterioration through faulting and spalling. Continued pumping eventually leads to a loss of slab support, which greatly increases load-related stresses in the slab and can result in fatigue cracking. Thus, transverse cracks must exhibit good long-term load transfer characteristics to minimize the development and severity of the distresses described above.

This research report describes a laboratory investigation to determine the effects of several materials and structural design parameters on JRCP transverse crack performance. The research program included the development and execution of a laboratory test program to collect and analyze load transfer data from a series of large-scale concrete pavement specimens that were subjected to repeated applications of loads simulating the passage of heavy truck traffic while being placed in varying degrees of tension to simulate the effects of uniform changes in slab length due to thermal and drying shrinkage.

1.1 BACKGROUND

The Michigan Department of Transportation (MDOT) has reconstructed several major Interstate projects since 1983, often using recycled concrete aggregate (RCA) in the new concrete pavement surface [McCarthy and MacCreery, 1983; Vandenbossche and Snyder, 1993]. Before recycling, many of these pavements had exhibited D-cracking; tests indicated that the gravel used in the original concrete pavement surface had poor resistance to freezing and thawing. In order to improve the durability of the new pavement sections, the coarse aggregate produced by recycling old pavement sections was crushed to a smaller top size (1.0 in. [25 mm] or less, in accordance with MDOT gradation 17A). This course of action was chosen in accordance with the concept that aggregate susceptibility to freeze-thaw damage (i.e., D-cracking) is diminished by reducing its average particle size.

It was recently observed that the transverse cracks on some of these newer JRCP were deteriorating rapidly through spalling and faulting, which would lead to increased maintenance requirements and shortened service lives. A preliminary evaluation of the causes of deterioration of these cracks suggested that the use of small-sized recycled concrete aggregates might be a major contributor to the crack deterioration as the reduced coarse aggregate top size could adversely affect the aggregate interlock load transfer characteristics of the crack faces [Darter, 1989]. In addition, the coarse aggregates produced by recycling concrete are composed of both relatively hard aggregate and relatively soft mortar, which may be weakened by the recycling (crushing and handling) processes. They may also have different abrasion resistance characteristics than traditional natural aggregates. Thus, these aggregates may fracture differently (and more readily) than virgin aggregates, producing unusual crack face textures. This theory may help explain why cores taken by MDOT at cracks along some of the prematurely-damaged projects described previously have exhibited very straight vertical crack faces with very little roughness or meander.

Darter (1989) and others also pointed out that there were several other factors that might be contributing to the poor performance of some of the Michigan recycled concrete pavement sections, including:

- The use of JRCP mainline pavement with jointed plain concrete pavement (JPCP) shoulders and shoulder joint panel lengths that were only about 1/3 that of the mainline panels, which may partly explain the fact that the mainline pavement cracks tended be aligned almost perfectly with the shoulder joints.
- The use of a rounded, pea stone base and sand subbase without a seperation layer between them, which may have created an unstable foundation that was prone to settlement.
- Failure of the contractor to cut the wires that hold the dowel baskets together during shipping (as indicated by some cores that were retrieved from the joints for post mortem analyses), which may have restricted movement at the joints and encouraged movement at the cracks.

It should also be pointed out that, while several Michigan recycled concrete pavements did exhibit exceptionally poor performance, most Michigan pavements that have been constructed using recycled concrete aggregate (RCA) have performed acceptably.

While the use of small-sized coarse aggregate and recycled concrete aggregate in the concrete surface course have been identified as possible contributors to the poor performance that was observed in Michigan, several other factors have been identified which may also have a significant impact on the rate of deterioration of transverse cracks in JRCP [Snyder, 1989; Raja and Snyder, 1991]. These factors include the following pavement materials and structural design features:

- Aggregate type, treatment, and gradation.
- Foundation support.
- Reinforcement type and quantity.
- Slab tension (due to friction with the pavement foundation).

This project was developed and conducted to examine the relative effects of all of the above factors on the load transfer endurance and deterioration of transverse cracks in JRCP.

2.0 DESCRIPTION OF THE TEST PROGRAM

2.1 EQUIPMENT

2.1.1 Test Stand

For this research it was necessary to develop equipment to apply loads that would simulate the effects of heavy vehicles moving across a transverse crack in a manner closely simulating field loading conditions. A test stand was developed similar to the apparatus used in the joint load transfer research conducted by Teller and Cashell [1959] in the 1950's, Colley and Humphrey [1967] in the 1960's, Ball and Childs [1975] and Ciolko and Colley [1979] in the 1970's. A schematic of the test stand that was developed for this project is shown in figure 1. A photo of the test stand is presented in figure 2.

This test stand uses independently-controlled hydraulic actuators to apply simulated moving wheel loads to normal thickness concrete slabs measuring up to 1.4 m (4.5 ft) wide by 3.0 m (10 ft) in length. Each specimen is uniformly supported through the use of an artificial foundation material (neoprene vibration isolation padding) mounted on a steel plate which is supported by structural steel sections. The steel sections are connected to the reaction frame in such a way that the test frame absorbs the simulated truck loadings in tension.

Test specimen casting frames, a handling frame (for transporting the large slabs in the laboratory), and a cracking frame (for inducing transverse cracks in the specimens) were designed, fabricated, and erected for this research work. Details concerning the form and function of key aspects of the stand are provided in the following sections.

2.1.2 Foundation

Since it is difficult to reproduce foundation properties accurately and consistently using natural materials, an artificial foundation material was selected to minimize variability of test results due to variances in foundation stiffness. The artificial foundation material selected for use was FABCEL-25, a high quality neoprene material that is supplied in scientifically-designed pads measuring approximately 46 cm x 46 cm x 3/4 cm [18 in x 18 in x 5/16 in]. The pad surfaces include molded and recessed cells that allow the neoprene to deform under load while maintaining lateral stability. Desired levels of foundation support are achieved by using combinations of various types and thicknesses of these pads. For this application, three layers of FABCEL-25 were used to provide a foundation with a simulated modulus of subgrade reaction of approximately 27 kPa/mm [100 psi/in], and two layers were used to simulate a foundation stiffness of approximately 68 kPa/mm [250 psi/in].

It should be noted that this material was useful in providing a uniform level of foundation support, but that the artificial nature of the material precluded the study of pumping and foundation erosion effects. It should also be noted that there was a modest deterioration in the stiffness of the pads over the duration of the test program (see section 2.5 below). However, it is believed that the overwhelming relative stiffness of the test slabs renders the effects of any minor changes

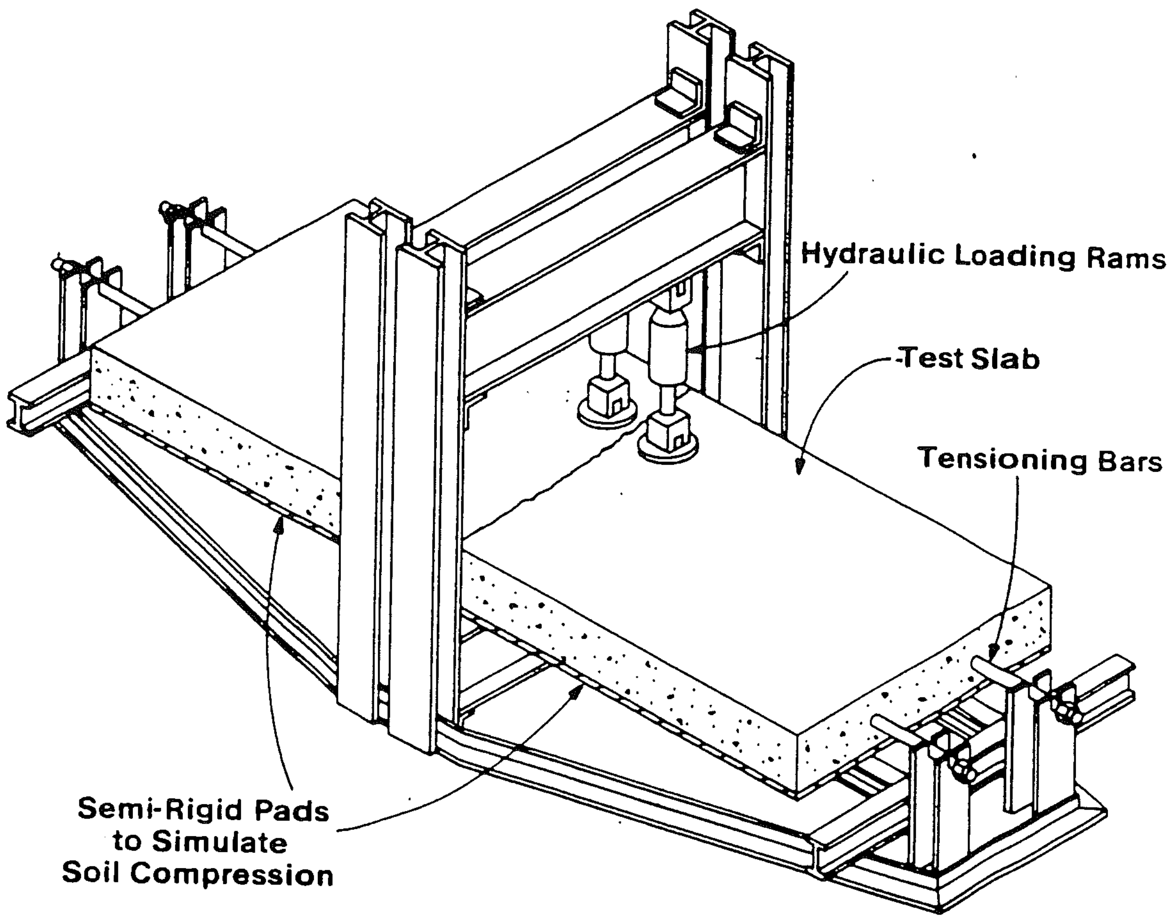


Figure 1. Isometric sketch of the accelerated pavement load test frame.

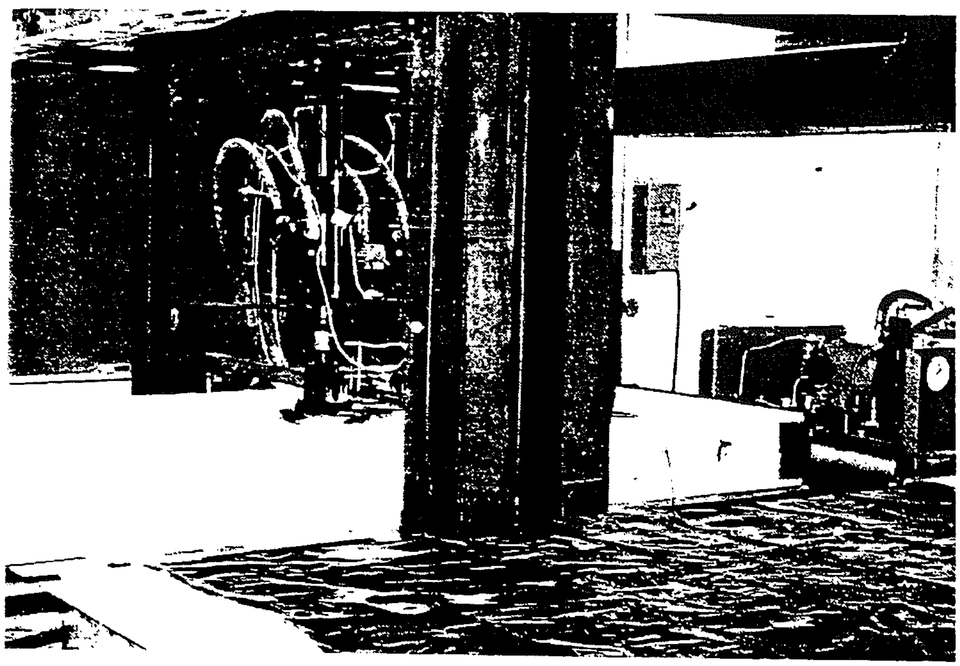


Figure 2. Photo of accelerated pavement load test frame.

in foundation stiffness irrelevant. This is intuitively apparent when one considers that the thickness design of a typical concrete pavement will decrease by only a few centimeters when the foundation stiffness increases by an order of magnitude.

2.1.3 Simulation of "Subgrade Drag" Forces

The test stand included tensioning braces (end columns) to allow the slabs to be placed in tension prior to and during testing to simulate the effects of resistance to thermal contraction and drying shrinkage caused by friction between the slab and foundation. Two steel reinforcing bars (No. 6 [19-mm (3/4-in) diameter]) were embedded in each end of each test specimen at the time of casting. The exposed ends of these rods were threaded and anchored through cross plates at the end columns by threaded couplers, washers and hexagonal nuts (see figure 3). Tightening the nuts placed the slab in tension; the magnitude of tension was monitored through strain gages that were attached to the reinforcing bars. Various magnitudes of tension could be imparted to simulate the effects of varying subgrade friction, slab length or both. Side benefits of this design were to reduce lateral movement of the slabs under dynamic loads and to simulate the slab continuity that would be associated with longer slabs in the field.

2.1.4 Simulation of Heavy Vehicle Loads

The test stand was designed to repeatedly apply load profiles through a pair of hydraulic actuators to simulate the passage of a 40-kN (9000-lb) wheel load at 88 kph (55 mph). A minimum load of 2.2 kN [500 lbs] was maintained on each actuator at all times to maintain contact between the load plates and test slabs, thereby eliminating unintentional impact loads. The load profile that was adopted for the test program is shown in figure 4. Note that one full load cycle, including a short period of inactivity in each cycle, required 0.2 sec, resulting in a load application frequency of 5 Hz. This allowed the simulation of up to 432,000 load cycles per day.

The hydraulic actuators (50-kN, [11,000-lb] capacity) were supplied by a 38-liter (10-gallon) per minute pump and were operated independently in load using an MTS T/RAC system and personal computer (80386 with math co-processor). The personal computer was also used for data acquisition, as described below. A schematic of the load control and application system is presented in figure 5. Loads were transmitted to the test specimens through a pair of 30-cm [12-in] diameter, 2.5-cm [1.0-in] thick steel plates, similar to those used on typical falling weight deflectometers (FWDs) used for pavement testing in the field. A 6-mm [1/4-in] hard rubber pad (of the same type found on FWDs) was cemented to the bottom of each load plate. The plates were positioned on either side of each transverse crack with their centers approximately 18 cm [7 in] from the crack and 46 cm [18 in] from the slab edge.

2.2 TEST SPECIMEN MATERIALS

2.2.1 Portland Cement Concrete Slabs

The test specimens were PCC slabs measuring approximately 3 m [10 ft] in length, 1.4 m [4.5 ft] wide and 250 mm [10 in] thick. These specimens were cast in molds constructed of 250-mm [10-

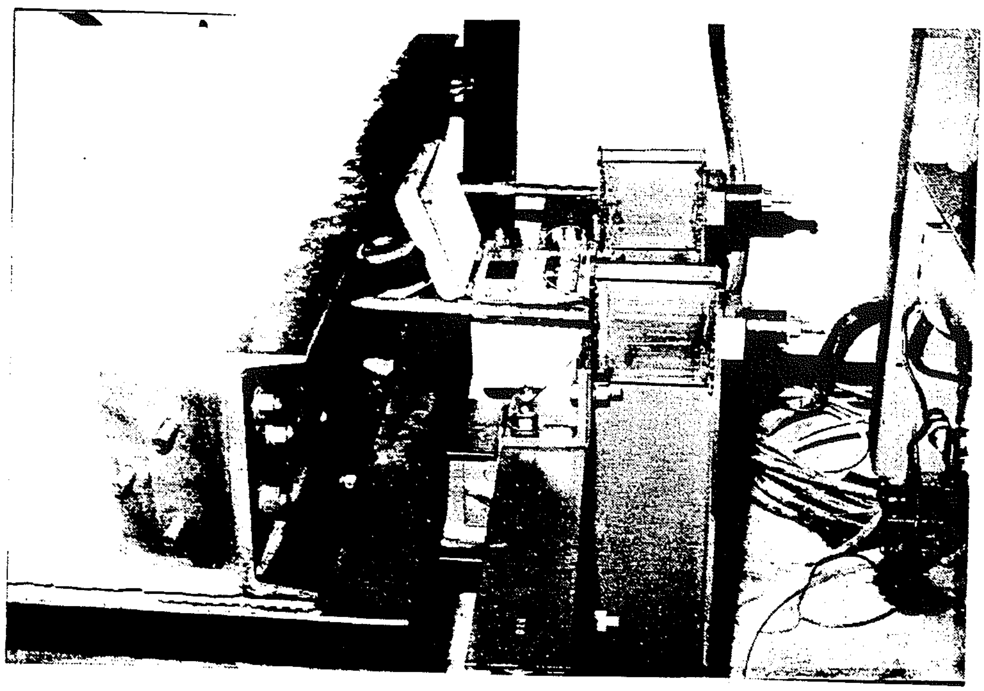
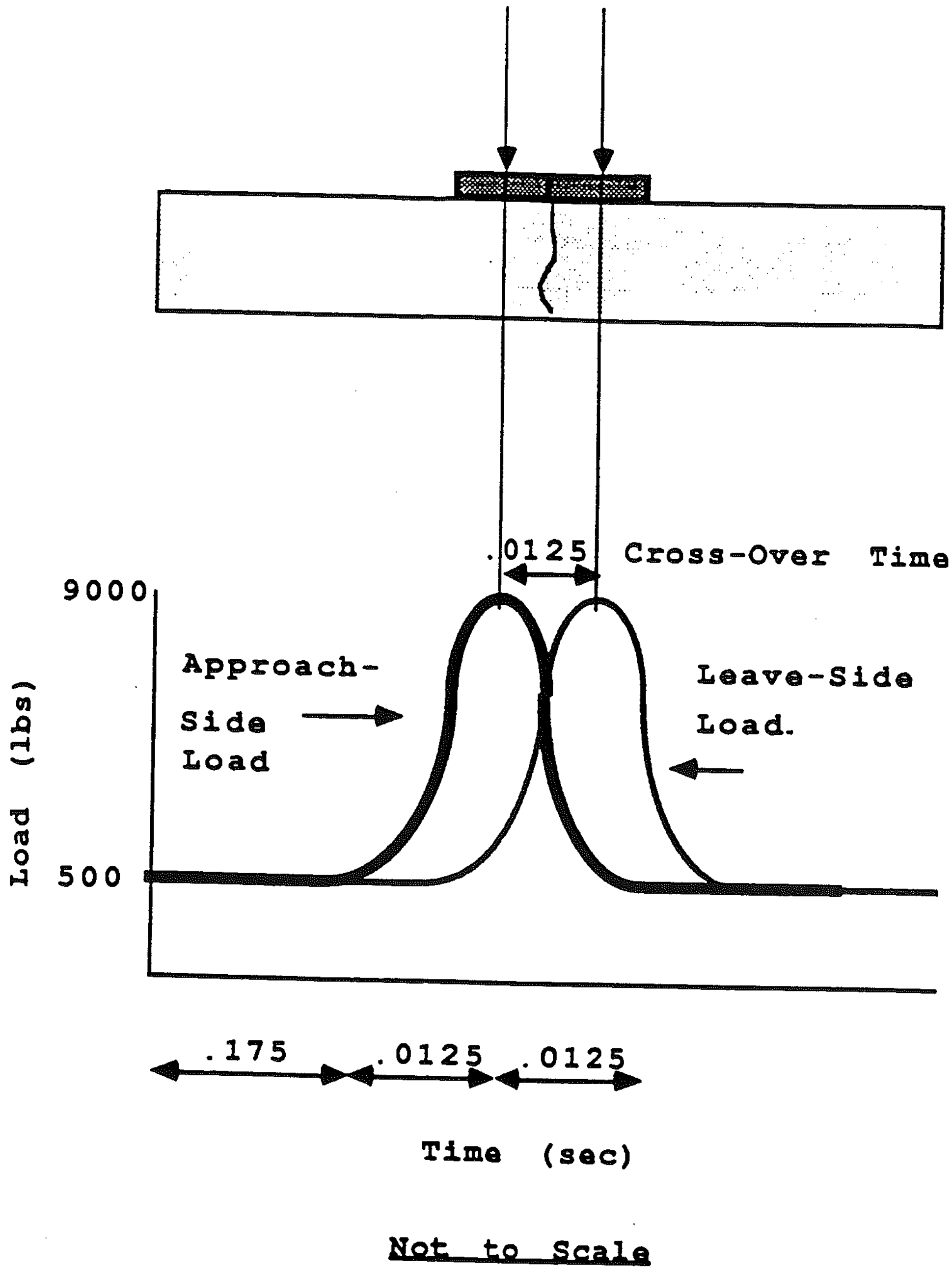


Figure 3. Photo of slab tensioning system.



Note: 1 lb = 0.4536 kg

Figure 4. Actuator load profile used to simulate 40-kN (9000-lb) wheel moving at 88 kph (55 mph).

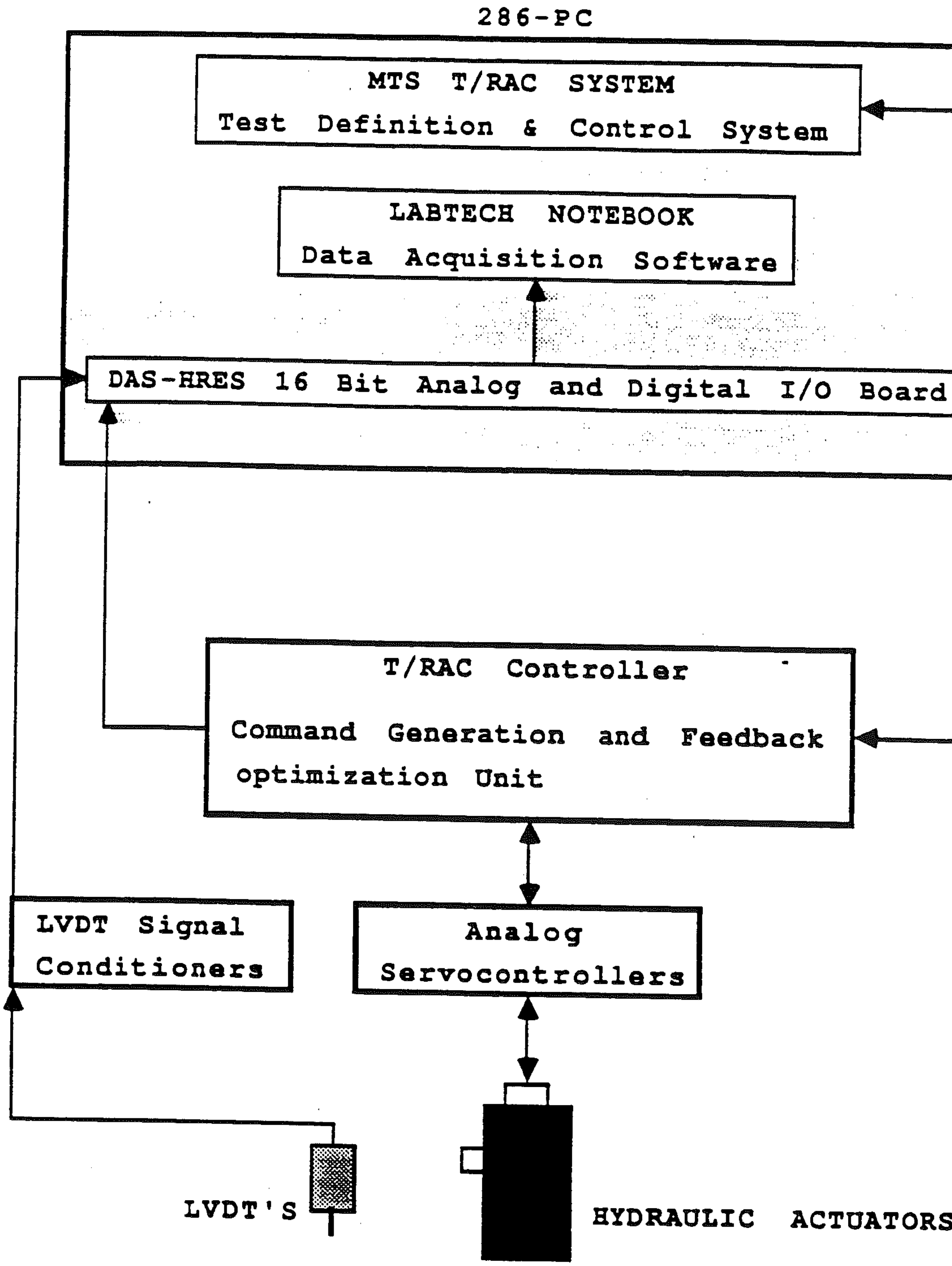


Figure 5. Schematic of load control and application system.

in] steel channels. The channel-molds included studs along the interior perimeter and a hinge near the midpanel location.

Basic concrete mix designs were concrete paving mixes provided by MDOT (based on the mortar-voids method of mixture proportioning) for each aggregate source and treatment used. These mix designs were modified slightly in the lab to try to hold the total volumetric proportion of coarse aggregate approximately constant for all mixtures while targetting a slump of 50 - 75 mm [2 - 3 in] and a total air content of 6 - 7 percent. Type I (normal) portland cement was used for all test specimens and Microair (by Master Builders) air-entraining agent was used for most specimens (Dairvair by W.R. Grace was used for some specimens). Table 1 presents a summary of the mix proportions and fresh mix characteristics for each of the specimens tested. Table 2 presents compression and flexural strength data for companion specimens that were cast at the same time as the test slabs.

Transverse cracks (plane-of-weakness type) were induced by including a 25-mm [1-in] deep by 6-mm [1/4-in] thick metal joint insert at the bottom of the slab near the center of the panel, resulting in a 230-mm [9-in] deep crack face. Cracking was induced 18 hours after casting by holding one half of the frame down while jacking the other half a short distance off of the floor. This slight flexural exercise of the specimen while it was relatively weak ensured that the shrinkage crack formed near mid-panel.

2.2.2 Slab Reinforcement

Most specimens contained a 2.8-m by 1.2-m [8-ft by 4-ft] sheet of wire mesh reinforcing (style 612-00/4), which was centered in the casting mold on chairs such that the steel was approximately 7.6 cm [3 in] below the slab surface. This mesh, MDOT's current standard for JRCP design, provided 0.16% steel reinforcement (by area of concrete) in the longitudinal direction using wires that were approximately 8.4 mm [0.33 in] in diameter and spaced 150 mm [6 in] apart. Variations in the steel reinforcing design are described in section 3.1 of this report.

2.3 INSTRUMENTATION AND DATA ACQUISITION

Each test specimen was instrumented for measurement of deflections on either side of the crack, changes in crack width and tension in the slab. Linearly varying deflection transducers (LVDT's) were used for measuring deflections on either side of the crack. Gage plugs and a hand-held vernier caliper were used to monitor crack opening. General purpose CEA-series strain gages were used to measure strain in the tensioning bars, thereby monitoring the amount of tension in the specimen. In addition, the load applied through each actuator was also monitored continuously and measured at the same time as the crack deflections. A typical instrumentation layout is presented in figure 6.

All testing and data collection operations in phase I of the study (the first 6 test slabs) were controlled using a 286-based personal computer equipped with a data acquisition system (Metrabyte DAS-HRES A/D board and Labtech Notebook software). At the beginning of the

Table 1. Summary of concrete batch proportions for test specimens.

		Mix Design				
Slab No.	Slab Description	CA	FA	Cement	Water	Air Cont.
1	6A Virgin Gravel #1	1966	1079	554	235	6.4
2	6A Virgin Limestone #1	1817	1240	560	245	5.4
3	6A Virgin Slag	1808	1297	744	305	6.7
4	17A Virgin Gravel #1	1878	1163	548	283	6.0
5	6A Recycled Gravel #1	1559	1209	523	263	6.7
6	50-50% 6A Recycled Gravel #1/4A Virgin Limestone	1682	1137	545	272	6.7
7	6A Virgin Gravel #1 (Replicate)	1814	1264	570	221	6.5
8	6A Virgin Gravel #1 (Replicate)	1832	1238	549	217	7.4
9	Stiff Foundation (k=250 psi/in)	1791	1250	563	234	6.8
10	High Tension (T=7000 lb/ft width)	1800	1238	563	234	6.8
11	6A Virgin Limestone #2	2044	1132	509	198	6.4
12	Large Wire, Stiff Foundation (k=250 psi/in, steel=0.23%)	2000	1098	566	263	6.5
13	6A Virgin Gravel #3	1972	1145	593	235	5.0
14	Large Wire (steel=0.23%)	2020	1123	571	216	6.0
15	6A Virgin Gravel #3 (Replicate)	2094	1211	436	184	6.8
16	Deformed Wire	1992	1056	598	239	6.3
17	Unreinforced	2003	1201	550	216	5.0
18	High Tension, Stiff Foundation (T=7000 lb/ft width, k=250 psi/in)	1963	1223	528	205	6.5
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone	1967	1295	416	156	6.3
20	6A Virgin Gravel #3 (with strain gages)	2174	1232	384	177	6.0
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone	2004	1182	548	224	5.0
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone	1860	1143	531	219	6.8

Table 1 (continued). Summary of concrete batch proportions for test specimens.

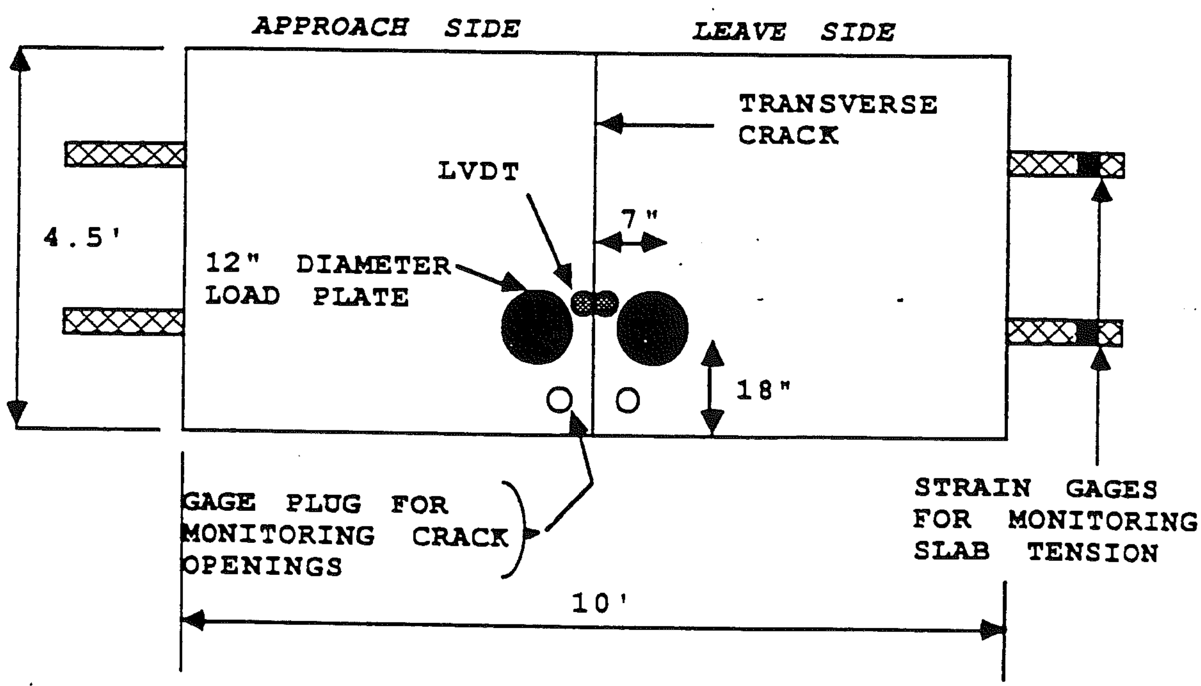
		Mix Design				
Slab No.	Slab Description	CA	FA	Cement	Water	Air Cont.
23	40-60% 6A Recycled Gravel #3 /4A Virgin Limestone	1922	1099	581	229	5.4
24	6A Recycled Limestone	1786	1064	590	221	6.2
25	Deformed Wire, High Tension (T=7000 lb/ft width)	1989	1110	539	234	5.5
26	6A Slag, Stiff Foundation (k=250 psi/in)	1758	1155	539	247	5.3
27	"Hinged" Joint	1976	1104	534	233	6.1
28	6A Recycled Gravel #3, Stiff Foundation (k=250 psi/in)	1747	1256	549	189	5.7
29	6A Recycled Gravel #3, Reduced Tension (T=2305 lb/ft width)	1776	1259	553	180	5.3
30	6A Recycled Gravel #3	1788	1249	518	212	6.2
31	6A Gravel #3, Selective Grading	1975	1076	532	248	5.4
32	4A Limestone #2	1922	1155	547	230	5.2
33	6A Slag, Large Wire (steel = 0.23%)	1708	1167	493	233	6.4
34	6A Recycled Gravel #3, Large Wire (steel = 0.23%)	1687	1133	533	222	6.8
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (steel=0.23%, k=250 psi/in)	1700	1132	537	224	6.3

Table 2. Summary of concrete strength data for test specimens.

		Average Strength					
		Compressive (psi)			Flexural (psi)		
Slab No.	Slab Description	3 day	7 day	28 day	18 hour	7 day	28 day
1	6A Virgin Gravel #1	N/A	N/A	5681	N/A	N/A	N/A
2	6A Virgin Limestone #1	N/A	N/A	5295	N/A	N/A	N/A
3	6A Virgin Slag	N/A	N/A	5954	N/A	N/A	N/A
4	17A Virgin Gravel #1	N/A	N/A	4294	N/A	N/A	N/A
5	6A Recycled Gravel #1	N/A	N/A	4780	N/A	N/A	N/A
6	50-50% 6A Recycled Gravel #1/4A Virgin Limestone	N/A	N/A	5352	N/A	N/A	N/A
7	6A Virgin Gravel #1 (Replicate)	N/A	N/A	4125	N/A	N/A	N/A
8	6A Virgin Gravel #1 (Replicate)	N/A	N/A	3645	N/A	N/A	N/A
9	Stiff Foundation (k=250 psi/in)	N/A	N/A	5837	N/A	N/A	N/A
10	High Tension (T=7000 lb/ft width)	N/A	N/A	4178	N/A	N/A	N/A
11	6A Virgin Limestone #2	4637	5464	6518	N/A	422	873
12	Large Wire, Stiff Foundation (k=250 psi/in, steel=0.23%)	N/A	3876	4723	N/A	N/A	N/A
13	6A Virgin Gravel #3	3847	5215	5466	364	760	796
14	Large Wire (steel=0.23%)	3262	3950	4212	312	678	706
15	6A Virgin Gravel #3 (Replicate)	N/A	4535	4800	362	684	738
16	Deformed Wire	3306	4145	5028	256	643	712
17	Unreinforced	3375	4128	5026	293	564	799
18	High Tension, Stiff Foundation (T=7000 lb/ft width, k=250 psi/in)	3776	N/A	4181	328	578	619
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone	4202	4304	4475	330	651	868
20	6A Virgin Gravel #3 (with strain gages)	2263	2882	3548	302	525	654
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone	3690	3330	N/A	346	707	683

Table 2 (continued). Summary of concrete strength data for test specimens.

		Average Strength					
		Compressive (psi)			Flexural (psi)		
Slab No.	Slab Description	3 day	7 day	28 day	18 hour	7 day	28 day
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone	3439	3900	4176	530	717	670
23	40-60% 6A Recycled Gravel #3 /4A Virgin Limestone	3816	4987	4677	480	818	738
24	6A Recycled Limestone	2507	2953	3950	257	526	608
25	Deformed Wire, High Tension (T=7000 lb/ft width)	3004	3516	4265	241	483	607
26	6A Slag, Stiff Foundation (k=250 psi/in)	2808	3163	4568	217	518	625
27	"Hinged" Joint	2798	3282	2773	388	246	596
28	6A Recycled Gravel #3, Stiff Foundation (k=250 psi/in)	2114	2621	3027	164	570	625
29	6A Recycled Gravel #3, Reduced Tension (T=2305 lb/ft width)	2258	3652	4124	269	845	960
30	6A Recycled Gravel #3	1858	2347	2868	247	562	653
31	6A Gravel #3, Selective Grading	2846	3582	4072	300	516	713
32	4A Limestone #1	2068	2638	2972	219	414	480
33	6A Slag, Large Wire (steel = 0.23%)	2072	2575	3247	189	387	489
34	6A Recycled Gravel #3, Large Wire (steel=0.23%)	3039	4207	5015	368	660	744
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (steel=0.23%, k=250 psi/in)	3514	3775	4842	391	665	712



Note: 1 in = 2.54 cm

1 ft = 0.3048 m

Figure 6. Typical instrumentation layout for test specimens.

second year of testing, the control system was upgraded to a 386-based personal computer. This system was connected directly to the hydraulic actuator control panel (MTS T/RAC controller) and signal conditioners. The arrangement, shown previously in figure 5, allowed the coordinated control of both hydraulic actuators and and the acquisition of load data from their load cells together with deflection data from the two external LVDT's.

Load and deflection data were collected following the completion of 1, 2000, 5000, 10000, 20000, 50000, 100000, 300000, 600000, 900000, and 1200000 load applications. Additional data (for the more long-lived specimens) was obtained after every 300000 to 600000 load cycles. Data was also collected more frequently as specimens began to exhibit signs of failure. Each data collection channel was sampled 250 times per load cycle or 1250 times per second (about 1 sample per channel every 0.0008 seconds). Each data collection sample lasted for one second (5 load cycles). This sampling rate and duration provided sufficiently close data points for plotting smooth load and deflection histories, which were used to identify peak loads and deflections. In this report, unless otherwise noted, all data pertaining to load and deflection measurments are based on the average of 5 sets of measurements.

2.4 STANDARD LABORATORY PROCEDURES

2.4.1 Preparation of Test Specimens

All aspects of the concrete batching and mixing process were carefully controlled to ensure maximum uniformity of materials and consistency in their handling. First, the coarse aggregates were sieved into their component size fractions and reblended to meet the center of the design grading specification. The coarse and fine aggregates were then both spread on the laboratory floor and left to dry for several hours prior to batching. Tests were run in accordance with applicable ASTM standards to determine the absorption capacities, unit weights and moisture contents of each aggregate batching stockpile just prior to batching and batch quantities were adjusted appropriately to take these factors into account. Trial batches were made to ensure the workability and air content of each mix prior to each cast.

The size of the test specimens and the capacity of the available drum mixers (0.07 m^3 [2.5 ft^3] batch capacity) made it necessary to mix the concrete in a continuous stream of small batches to prevent the formation of cold joints. For each batch, one-half of the coarse aggregates and one-half of the fine aggregate were first blended with one-half of the mix water. The drum was started and the cement was added, followed by the remaining half of the water (with air-entraining admixture), then the remaining coarse aggregates and fine aggregates. The mixer was operated for five minutes after the addition of the final component. This batch process was adopted to facilitate mixing of the mixes, which were often quite stiff and unworkable when more traditional batch procedures were followed.

Concrete was hauled to the steel channel specimen form in wheel barrows, where it was consolidated with a shaft-type vibrator. Each specimen was cast according to a schedule that generally allowed testing to begin after 28 days of curing. Specimens were cured in the laboratory under polyethelene sheets. Companion compression cylinders (150-mm x 300-mm [6-

in x 12-in]) and flexural beams (150-mm x 150-mm x 915-mm [6-in x 6-in x 36-in]) were cast at the same time as the load test specimens. Strength test data are provided in tables 1 and 2, as described previously.

The transverse crack was induced near midslab after approximately 18 hours of curing. A removable metal joint insert was cast into the bottom of each specimen to form a plane-of-weakness, as described previously. The slab was cracked full-depth along the weakened plane by jacking one-half of the slab and frame while clamping the other half to the cracking frame. A hinge mounted on top of the casting frame assured a tensile mode of fracture.

2.4.2 Testing the Specimens

After 28 days of curing, each test specimen was moved to the test stand while still in the structural channel casting form, which was equipped with lifting loops. The slabs were held securely in the form during cracking and transportation by short steel studs, which were welded to the insides of the form around its perimeter. After each specimen was placed and centered on the test stand, the casting form was removed. Transporting, placing and adjusting the specimen while it was still in the form helped to ensure that the reinforcing steel was not subjected to any significant stresses (other than during cracking) until the tension and simulated vehicle loads were applied.

After the specimen was set in place, tension was induced in the specimens as described previously. The LVDT's were then set to zero, the data acquisition system was initialized, and the cyclic load program was begun. Load, deflection and crack width data were collected at the intervals described earlier. The applied load profile and the slab tension were continually monitored and adjusted (as necessary) during the test program. The load program was generally continued until the reinforcing steel ruptured (see figure 7), as indicated by rapid increases in crack width and the operators inability to adequately tension the specimen.

Load transfer histories were compiled and plotted to determine the number of load cycles to failure and the load transfer efficiency at failure. Failure was defined as the number of cycles corresponding to the point on the load transfer history curve where a 45-degree line could be constructed tangent to the curve (see figure 8). The average load transfer efficiency at failure was approximately 76%.

2.5 VARIATIONS IN TEST EQUIPMENT AND PROCEDURES OVER TIME

2.5.1 Modifications to Test Equipment

The load testing frame was stiffened during the second year of the study in order to reduce testing noise and vibration and to improve specimen handling procedures. Metal shims were added and welds were inspected in the slab support system to reduce vibration during testing. These modifications resulted in a stiffer slab support system. As a result, specimens that were tested after these modifications (specimens 7 through 35) endured more load cycles to failure than those

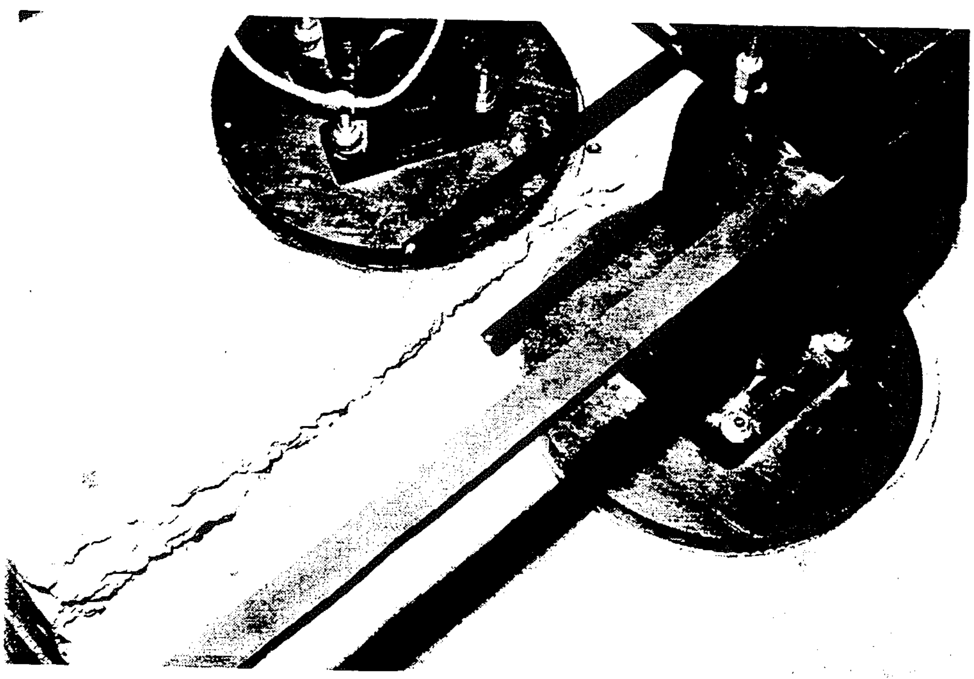


Figure 7. Photo of failed specimen and ruptured steel.

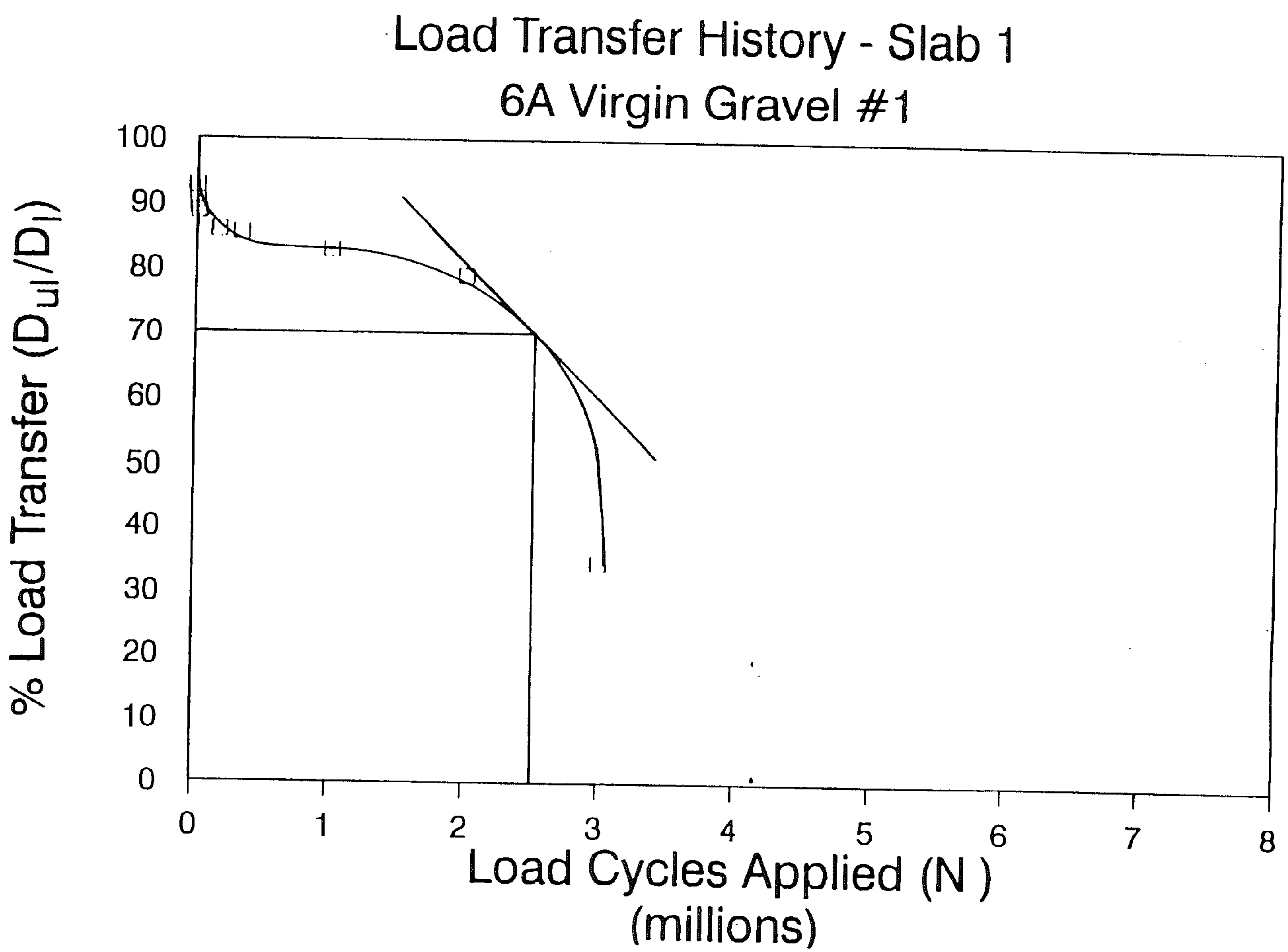


Figure 8. Determination of number of load cycles to failure from load history graph.

tested in the first year. Some replicates of first year specimens were tested during the second and third years of testing. The results of these tests were used to develop "calibration" factors so that all specimens (before and after the stand modifcation) could be compared more accurately.

Minor modifications were made to the slab tensioning system at several points during the test program. These modifications included: stiffening the four end columns (which were used to place the slabs in tension) and the fabrication of steel wedges which were inserted between the steel columns and the tensioning rods to ensure the elimination of any vertical component of stress in the slab tensioning system. The modification to the tension beams was necessary following partial failure during the testing of one specimen, which resulted in its loss. It is not believed that these modifications to the tensioning system had any significant effects on test results.

General system maintenance was also necessary, including the addition of hydraulic fluid, repair of small leaks, and the replacement of hydraulic and electrical fittings.

2.5.2 Variations in Foundation Stiffness over Time

When the project had been completed, modified plate bearing tests (k-value tests) were performed on the FABCEL-25 pads to determine whether the foundation stiffness had changed significantly over the 3-year test program. The plate bearing test generally followed the procedures described in the AASHTO test designation T 222-81 except that the test procedures were modified to use the hydraulic actuator from the test program to apply the necessary loads. The results of the testing suggest that some deterioration occurred as a results of repeated cyclic loads, age or both (see tables 3 and 4 and figures 9 and 10). These results suggest that the foundation support stiffness increased slightly over time. However, as discussed previously, it is not belived that these relatively small changes in foundation support significantly influcenced the project test results because the performance of concrete pavements is notoriously insensitive to even moderate changes in foundation stiffness.

Table 3. Artificial foundation stiffness (3 layers) at completion of test program, psi/in.

	k @ 10 psi	k (linear)
Group 1	131.48	164.31
Group 2	113.08	135.06
Average	122.28	149.69
New Pads	72.94	138.89

Note: 100 psi/in = 27 kPa/mm

Table 4. Artificial foundation stiffness (2 layers) at completion of test program, psi/in.

	k @ 10 psi	k (linear)
Group 1	115.93	232.45
Group 2	123.40	232.99
Average	119.66	232.72
New Pads	91.74	198.02

Note: 100 psi/in = 27 kPa/mm

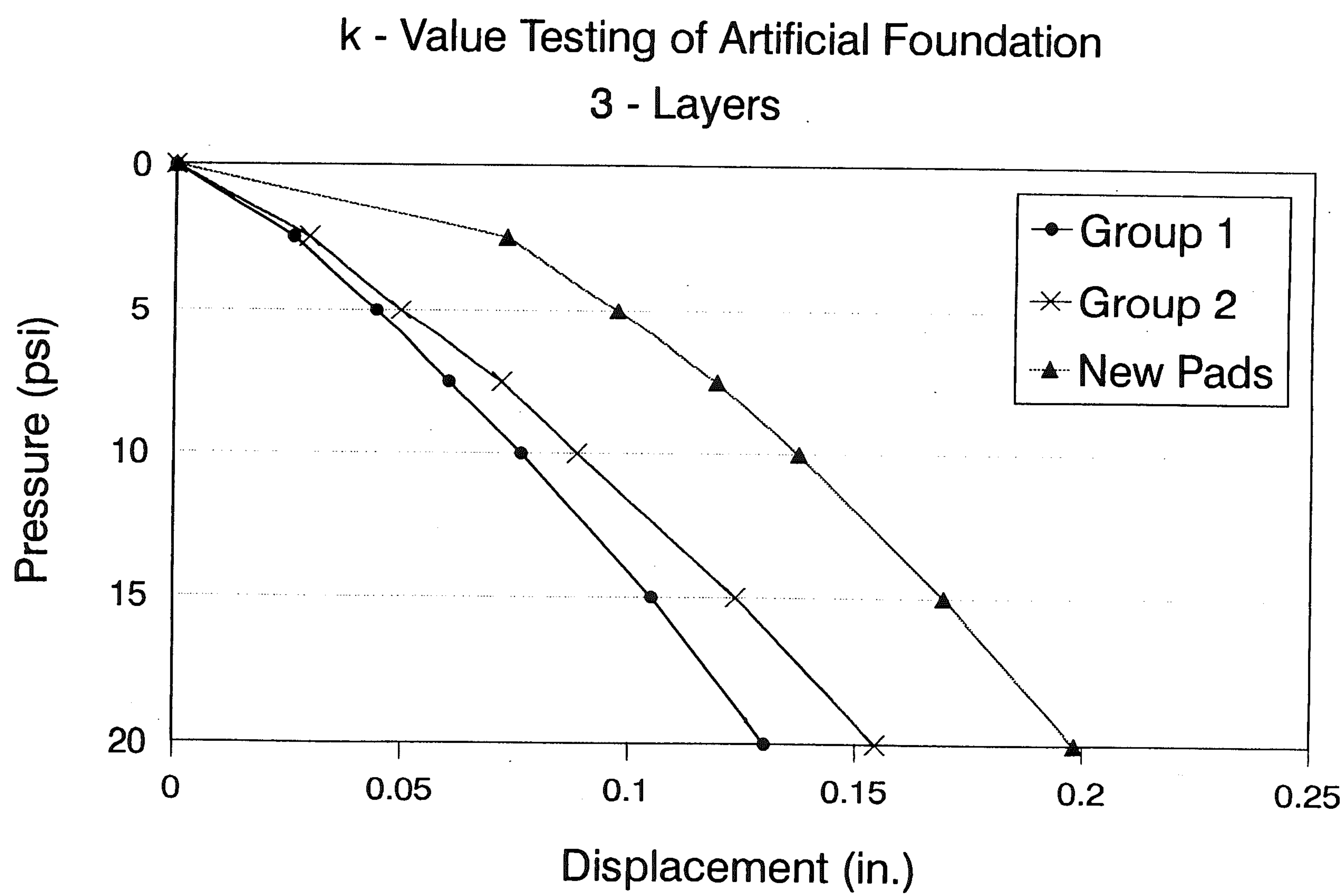


Figure 9. Static plate load test for 3 layers of artificial foundation material.

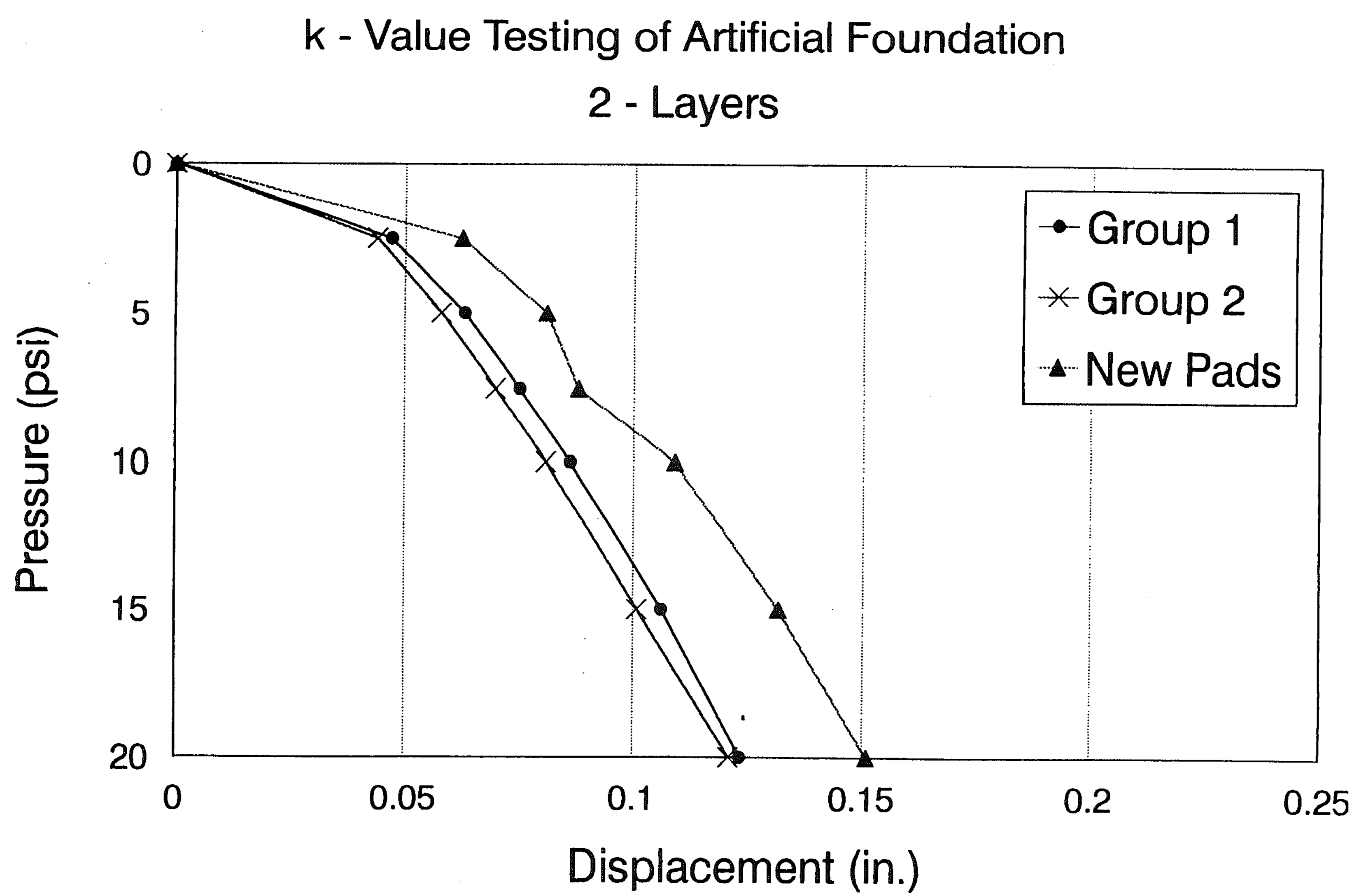


Figure 10. Static plate load test for 2 layers of artificial foundation material.

3.0 EXPERIMENTAL DESIGN

3.1 TEST VARIABLES AND VALUES

The following test variables and values were selected for consideration in this study:

Aggregate type:	gravel, limestone, slag and RCA
Aggregate treatment:	100% RCA and various weight-based blends of 6A RCA and 4A or 6A virgin limestone
Aggregate gradation:	MDOT 6A, MDOT 17A or blends of MDOT 6A and MDOT 4A or 6A
Reinforcing type:	smooth wire mesh, deformed wire mesh, deformed reinforcing bars and none
Mesh reinforcing quantity:	0.16% by area of concrete and 0.23% by area of concrete

Tables B-1 through B-3 in Appendix B provide descriptions of all of the specimens (and combinations of study variable values) that were tested under this research program. Descriptions of the variables and selected test values are presented below.

3.2 INITIAL TEST PROGRAM (YEAR 1)

The initial research program included only 6 test specimens and the scope of research was restricted to an examination of coarse aggregate type (limestone, gravel, slag or RCA), coarse aggregate gradation (MDOT 6A vs. MDOT 17A) and RCA treatment (100% 6A RCA vs. a 50-50 blend (by weight) of 6A RCA and 4A limestone). These coarse aggregates are hereafter referred to as "limestone #1," "gravel #1," "slag," and "RCA #1." Details concerning the sources and physical characteristics (i.e., absorption capacity, gradation, etc.) of these aggregates are summarized in tables A-2 through A-7 in Appendix A. It should be noted that these materials were used "as received" from the vendors without regrading; the aggregates used in later slabs were sieved and reblended to more nearly approach the center of the applicable MDOT grading specification. It is also worth mentioning that the recycled concrete aggregate was obtained by crushing and grading the slab composed of 6A gravel.

A full-factorial, replicated experimental design using these variables and test levels would have required the preparation of many more test specimens than resources allowed. Since the initial project goals were to demonstrate the capabilities of the test stand and to obtain preliminary measures of the main effects of these variables as quickly as possible, an unreplicated comparative experiment was selected. The first 6 specimens listed in table B-1 were the only ones tested under this phase of the research effort.

Standard conditions for these tests included:

- k = 27 kPa/mm (100 psi/in);
- 0.16 % (by area of concrete) smooth wire mesh longitudinal reinforcement;

- Slab tension, T, of 51 kN/m (3500 lbs/ft width), the theoretical tension that would develop in a fully-mobilized 41-ft panel being pulled (due to shrinkage) across a foundation interface with a frictional coefficient of 1.5.

The amount of tension used was computed from subgrade drag theory for an assumed slab-subbase friction coefficient of 1.5 and a 230-mm [9-in] slab. Tension was induced in the test specimens by adjusting the two tensioning bars embedded in each test specimen and monitoring tension bar strain with the strain gages, as described previously.

3.3 EXPANDED TEST PROGRAM (YEARS 1 AND 2)

The results of the first six tests provided some preliminary indications of the effects of aggregate type, grading and treatment on the deterioration of transverse cracks in JRCP (Raja, 1991; Raja and Snyder, 1991; see also section 4 of this report). They also validated the usefulness of the test frame as a tool for rapidly evaluating the effects of various materials and structural design parameters on the deterioration of these cracks. Thus, a second phase of testing was planned. Before this second phase of testing was begun, the test frame was modified as described previously to reduce frame vibration and bouncing of the slab. .

One goal of the second phase of testing was to determine the variability of results obtained from the accelerated loading test stand. Thus, specimens 7 and 8 were replicates of the first specimen. While the results of testing these two specimens were very close to each other, they were significantly different from the results of the first specimen, indicating that the test stand modifications had significantly altered the test conditions. It was believed that the modified stand was a better (more realistic) test, so the modifications were left in place and the results of the replicate tests were used to estimate a scaling factor for use in adjusting the results of the first six test specimens.

Additional goals of the second phase of testing were to:

- evaluate the effects of using different sources of natural and recycled aggregate;
- evaluate the effects of increasing the foundation support to limit vertical movements at the crack;
- evaluate the effects of varying the slab length or slab-subbase frictional coefficient to change the tension in the slab;
- evaluate the effects of using greater quantities of reinforcing to hold the cracks more tightly;
- evaluate the use of deformed wire mesh (rather than smooth wire mesh) to hold the cracks more tightly;
- evaluate the effects of blends of RCA with various quantities and sizes of virgin material; instrument the reinforcing in one slab to measure strains in the reinforcing steel in an attempt to verify the mode of rupture; and
- evaluate the effects of various combinations of the variables described above.

An additional source of limestone (referred to as "Limestone #2) and an additional source of gravel (referred to as "Gravel #3) were added to the test program. In addition, all subsequent

RCA was prepared from test specimens containing the new gravel #3. Details concerning the sources and physical properties of these aggregates are presented in tables A-2 through A-4 in Appendix A.

Foundation stiffness effects were evaluated by using a reduced thickness of the FABCEL-25 vibration isolation padding, as described earlier. The stiffer foundation was estimated to have a k-value of 68 kPa/mm (250 psi/in).

Slab tension was doubled to approximately 102 kN/m width [7000 lbs/ft width], which represents the effects of either doubling the foundation interface friction (from 1.5 to 3.0) or doubling the slab length (from 12.5 m [41 ft] to 25 m [82 ft]). It should be pointed out that any value of slab tension can be interpreted as an infinite number of combinations of slab length and interface friction, as illustrated in figure 11.

Changes in longitudinal steel quantities were achieved by increasing the size of the longitudinal wires while holding the transverse spacing of the wires constant. Deformed wire mesh was selected such that it had an identical longitudinal wire spacing and a nominal longitudinal wire size as close to that of the smooth longitudinal wires as possible. The sizes and strengths of the longitudinal wires that were used in various test specimens are presented in table A-1 of Appendix A.

Test specimens incorporating recycled concrete were produced by breaking and crushing slabs cast using 6A gravel (or 6A limestone in the case of test specimen 24) in commercial crushers. This crushed concrete was then sieved into component size fractions and reblended to approach the center of the applicable MDOT grading specification. Test specimens containing 100% recycled crushed concrete were graded to meet MDOT specification 6A. Two recycled blend specimens contained coarse aggregate composed of a blend of 50% (by weight) recycled gravel #3 concrete graded to meet MDOT specification 6A and 50% virgin crushed limestone #1 graded to meet MDOT specification 4A. Another recycled blend specimen contained coarse aggregate composed of a blend of 50% recycled gravel #3 concrete and 50% virgin crushed limestone #1, both graded to meet MDOT specification 6A. The 40-60% recycled blend specimen contained coarse aggregate composed of a blend of 40% recycled gravel #3 concrete graded to meet MDOT specification 6A and 60% virgin crushed limestone #1 graded to meet MDOT specification 4A. In addition, a 50-50% blend specimen was cast using coarse aggregate composed of a blend of 50% virgin gravel #3 graded to meet MDOT specification 6A and 50% virgin crushed limestone #1 graded to meet MDOT specification 4A; the purpose of this specimen was to determine whether the unexpectedly poor performance of many of the recycled blend specimens was due to the gradation of the blend or the use of recycled concrete products.

Specimen 20 was instrumented with strain gages on several of the reinforcing wires in an effort to better quantify the stress state of the steel when subjected to the combined tension, shear and bending loads being induced by slab tension and simulated vehicle loads. Unfortunately, this test was not successful because the crack formed with a large horizontal deviation (rather than a reasonably straight vertical formation) and completely bypassed the strain gages. In addition, the unusual crack shape also produced atypical crack deterioration performance and the results of this

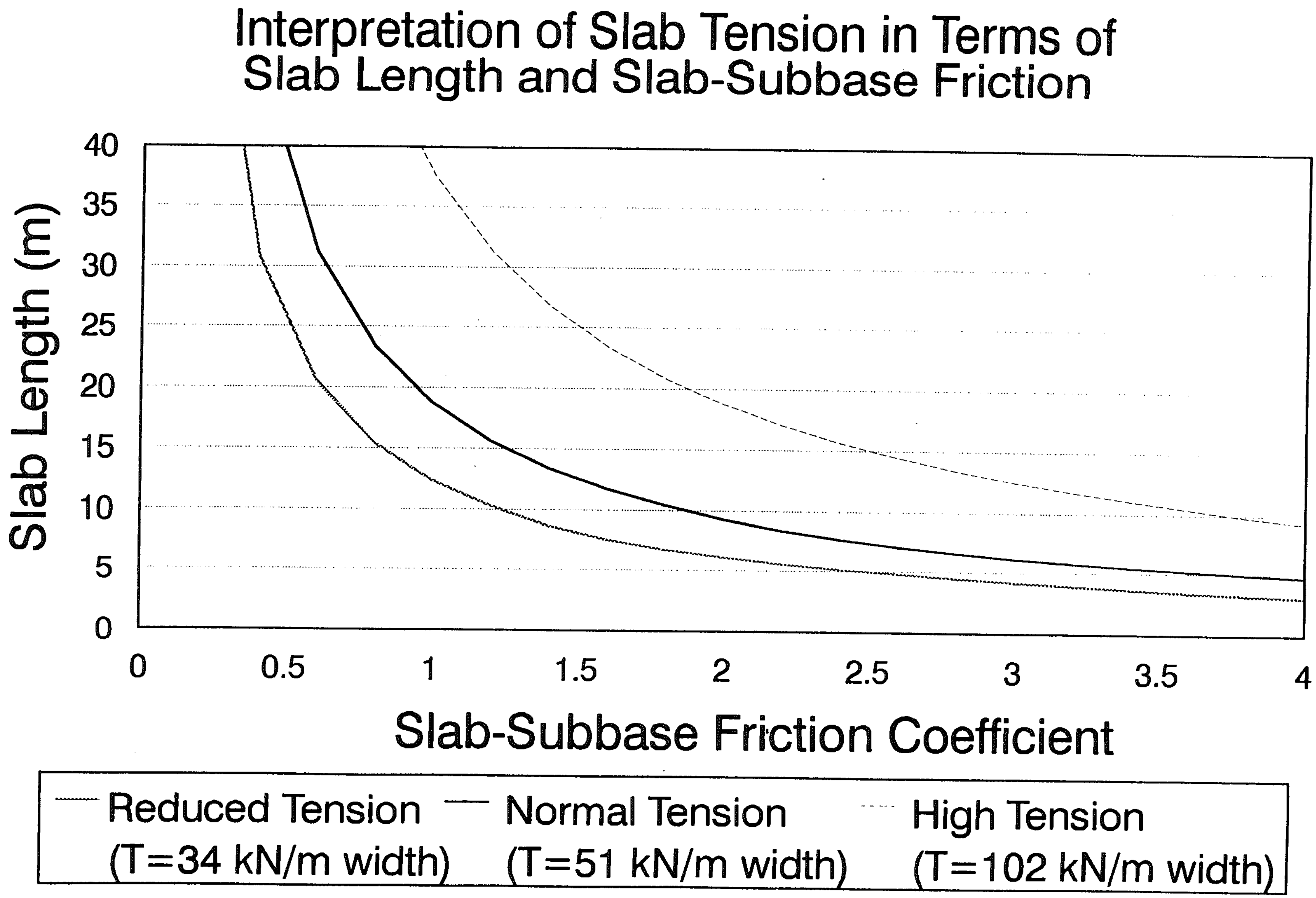


Figure 11. Interpretation of slab tension in terms of slab length and slab-subbase friction.

specimen were considered unrepresentative; they are not included in any of the analyses and discussions presented in section 4 of this report.

Specimens 7 through 10 were tested during the first calendar year of testing, even though they belonged to the second phase of the research project. After the tenth specimen had been tested, the original operator graduated and a second operator took over (and stayed through all but the last few test specimens).

3.4 FINAL SUPPLEMENTAL TESTS (YEARS 3 and 4)

The results of the first twenty-five tests provided answers to many of the questions that had been raised concerning the effects of foundation stiffness, reinforcement type and quantity, and coarse aggregate parameters on the deterioration of transverse cracks in JRCP (see section 4 of this report). In some cases, however, the insight provided by the test results lead to additional questions that could only be answered by performing additional tests. Thus, a third and final phase of testing was planned and (after contractual delays of nearly a year) performed.

Goals of this final phase of testing were to:

- determine whether the improvements in RCA pavement crack performance associated with the use of stiffer foundations or increased steel quantities would also reduce the rate of deterioration of concrete pavements prepared using slag aggregate;
- evaluate the effectiveness of the "hinged joint" design in mitigating crack deterioration by limiting both vertical and horizontal movements at formed cracks (dummy joints);
- evaluate the effectiveness of providing both increased foundation support and increased steel quantities in RCA concrete pavements on reducing crack deterioration;
- evaluate the effectiveness of reducing slab length or foundation interface friction (i.e., reduced slab tension) on the rate of deterioration of cracks in RCA concrete pavement;
- investigate the effects of aggregate grading on the differences in performance observed between virgin gravel #1 and #3;
- test a specimen cast using 100% recycled gravel #3 and standard test parameters to compare with the results of the specimen prepared using 100% recycled gravel #1; and
- determine the potential merit of using all large aggregate by testing a specimen prepared using 100% virgin limestone graded to MDOT specification 4A.

Ten additional test specimens were prepared to address these issues, as described in table B-3 of Appendix B. The test values for foundation stiffness and steel quantity were achieved as described previously. In addition, reduced tension of approximately 34 kN/m width [2305 lbs/ft width]) was used to represent the effects of reducing slab length from 12.5 m [41 ft] to 8.3 m [27 ft] while maintaining foundation interface friction at 1.5. The "hinged joint" reinforcement consisted of 750-mm [30-in] long 19-mm [#6] deformed reinforcing bars placed at 460 mm [18 in] spacing across a formed joint. These larger bars also resulted in an increase in the quantity of steel at the joint/crack face (approximately 0.27% by area of concrete).

4.0 ANALYSIS AND DISCUSSION OF TEST RESULTS

4.1 GENERAL

The principal sources of data for the evaluation of the relative performance of the test specimens were the deflection and load data that were obtained periodically throughout the test program. These data were considered in several ways, including the computation of load transfer efficiency values, determination of peak deflections on either side of the crack, and the determination of maximum differential deflections across the crack. In addition, the width of the crack was monitored as each specimen was prepared and tested.

The following subsections address the determination and use of these performance indicators.

4.1.1 Load Transfer Efficiency

The ability of transverse cracks to transfer load is a major factor in the structural performance of the crack and the surrounding slab fragments. In this study, the ability to transfer load was evaluated by comparing the deflections of the two slab fragments using the following commonly-adopted definition:

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

where

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

This formula was used in this study because of its conceptual simplicity, ease of application and broad experience base in research and practice. With this definition of load transfer efficiency, the theoretical maximum load transfer that can be achieved is 100%, which is obtained when the two sides of the joint or crack deflect equally under an applied load. At the other extreme, the load transfer efficiency is zero if the two sides move with complete independence.

Data was collected to allow the evaluation of %LTE with either side of the crack being loaded. The load transfer efficiency data tabulated in this report are based on the "approach" or "upstream" side of the crack being loaded and the "leave" or "downstream" side being unloaded.

While load transfer efficiency of joints and cracks is always considered important in terms of reducing load-related stresses in concrete pavements, it is also an important indicator of the potential for deterioration of undoweled joints and cracks. Loss of load transfer can produce increased load-related stresses, abrasion of the crack face (resulting in further losses of load transfer), joint or crack seal damage, spalling, and can contribute to foundation erosion and pumping by providing a mechanism for transporting water-borne fines. Therefore, one primary technique for measuring the performance potential of a transverse crack in concrete pavement is to evaluate the load transfer efficiency at any point in time.

The relative performance of cracks in pavements composed of various materials and featuring different structural designs can be measured by considering the load transfer performance of a crack over a period of time. In this study, an endurance index was developed to represent the cumulative load transfer performance of a crack over the number of load applications that had been applied. Several methods of computing endurance index were considered and evaluated for their ability to provide an accurate comparison of the relative performances of different test specimens. Three of these are tabulated in table B-4 in Appendix B. The first is based on the area under the curve representing load transfer vs. the log of the number of load applications. The index is computed as the percentage proportion of area under that curve compared with the area that would be bounded by load transfer limits of 0 and 100% and logarithmic limits of 0 and 8 (i.e., 0 to 100,000,000 load applications). The second is computed identically except that the load application scale is linear rather than logarithmic and the load cycle limit is 1,000,000. The third index is identical to the second except that the load cycle limit is 10,000,000. It is this third index that was found to provide the best relative measure of the crack performance and was adopted for use in this analysis. Figure 12 illustrates the determination of the endurance index for a specific load transfer history curve.

The load transfer history curve was also used to estimate the number of load cycles to failure for each specimen. This process was described in detail in section 2.4.2 and illustrated in figure 8.

4.1.2 Peak Deflection

Peak deflections were determined by examining the LVDT data and identifying the maximum deflections under the applied loads. Since all slabs were of the same thickness, peak deflections should be directly related to the degree of load transfer being provided by grain interlock and shear in the reinforcing steel. Large increases in peak deflection were observed just prior to failure of most specimens.

4.1.3 Differential Deflection

Differential deflections were determined by examining pairs of LVDT data points and identifying the pairs with the maximum difference. This data was used to determine load transfer efficiency, but also provides an added input to the analysis. For example, slab A could have $d_L = 200$ microns and $d_{UL} = 150$ microns, while slab B might have $d_L = 100$ microns and $d_{UL} = 75$ microns. Slab A would have a load transfer efficiency of 75% and a differential deflection of 50 microns while slab B would also have a load transfer efficiency of 75% with a differential deflection of only 25 microns. Evaluation of load transfer efficiency alone would not predict a difference in the performance of the two slabs. Similarly, consideration of only peak deflection would suggest higher slab stresses or poor foundation support (or both). Differential vertical deflection would be necessary to predict continued deterioration of the crack due to fatigue of the reinforcing steel and abrasion of the crack face under the repeated differential movement.

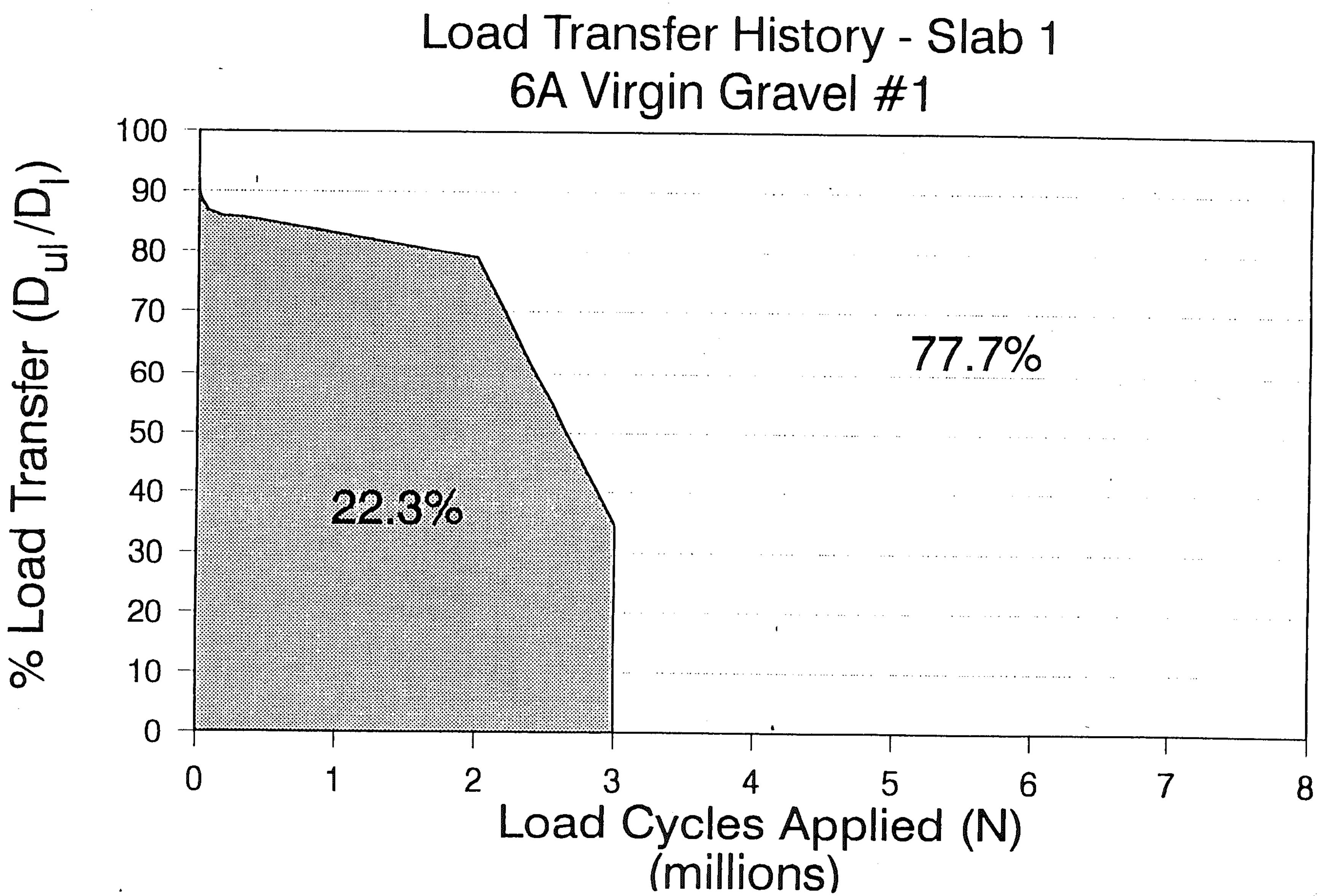


Figure 12. Determination of endurance index.

4.1.4 Crack Width

Crack widths were measured using hand-held calipers and gage plugs that were embedded in the slabs (on either side of the expected crack location) when they were cast. Gage distance measurements were taken soon after casting, just prior to cracking, just after cracking, when the slab was placed in the test stand, and at every interval where load and deflection data were taken. Crack widths were computed as the difference between any given gage distance and the distance recorded just prior to cracking the slab. Crack widths were strongly correlated with load transfer, crack endurance and crack performance.

4.2 EFFECT OF COARSE AGGREGATE TYPE

The effect of coarse aggregate type was evaluated by comparing the results of tests of several sets of data, including specimens 1, 2, 3 and 5 from the first phase of testing, specimens 11, 13/15, 24, 30 and 31 or specimens 14, 33 and 34. A summary of performance measures for these test slabs is presented in table 5.

Specimens 1, 2, 3 and 5 were prepared using virgin gravel #1, virgin limestone #1, slag and recycled gravel #1 concrete, respectively. Each specimen was prepared using MDOT 6A coarse aggregate gradation, 0.16% smooth wire mesh reinforcement, an induced slab tension of 51 kN/m [3500 lb/ft width], and a foundation stiffness of 27 kPa/mm [100 psi/in]. Figure 13 shows a comparison of the load transfer histories of these specimens.

The results show that the specimens containing crushed limestone and gravel coarse aggregates started with and retained higher load transfer efficiencies than the specimens prepared using slag or recycled concrete as coarse aggregate. This is also reflected in the lower measured differential deflections for the natural aggregates. These performance characteristics are probably attributable to the smoother crack face textures associated with the manufactured aggregates, which are usually assumed to be somewhat weaker than the natural aggregates. It appeared that the slag and recycled concrete aggregates often fractured at the same time the crack was formed while the limestone and gravel aggregates pulled out of the mortar (instead of fracturing), resulting in rougher crack faces. It is also possible that these results were influenced by differences in the coarse aggregate gradations, which all met the requirements of MDOT gradation specification 6A, but still varied somewhat with the slag being significantly finer than either the other aggregates (aggregate sources used as-delivered during the first year of testing; sieving and blending to achieve consistent gradations was not performed until the second year of testing).

Increased differential deflections and reduced initial load transfer translate into significantly fewer load cycles to failure (and a greatly reduced endurance index) for the slag and recycled concrete specimens for these design and test conditions, as shown in table 5. This is because the differential movements of the crack result in further abrasion of the crack face texture (and further reduction in grain interlock) as well as more rapid fatigue of the mesh reinforcing in the vicinity of the crack.

Table 5. Performance summary for the effect of coarse aggregate type.

Slab	Brief Description	Cycles to Failure (Unadjusted)*	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
1	6A Virgin Gravel #1	2,516,000 (755,000)	22.3	144	1195	N/A
2	6A Virgin Limestone #1	4,630,000 (1,336,000)	35.9	167	1092	N/A
3	6A Virgin Slag	540,000 (180,000)	4.0	276	1087	N/A
5	6A Recycled Gravel #1	710,000 (237,000)	5.6	228	1461	N/A
11	6A Virgin Limestone #2	1,600,000+	N/A(16.6)	27	1442	279
13	6A Virgin Gravel #3	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3	225,000	1.8	84	1414	1626
24	6A Recycled Limestone #1	95,000	1.0	367	1842	762
30	6A Recycled Gravel #3	210,000	2.0	145	1675	203
31	6A Virgin Gravel #3, Selective Grading	145,000	1.6	194	1680	508
14	6A Virgin Gravel #3, 0.23% Steel	1,430,000	13.3	18	944	305
33	6A Virgin Slag, 0.23% Steel	445,000	3.6	231	1747	330
34	6A Recycled Gravel #3, 0.23% Steel	670,000	7.0	156	1491	457

NOTE:* Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

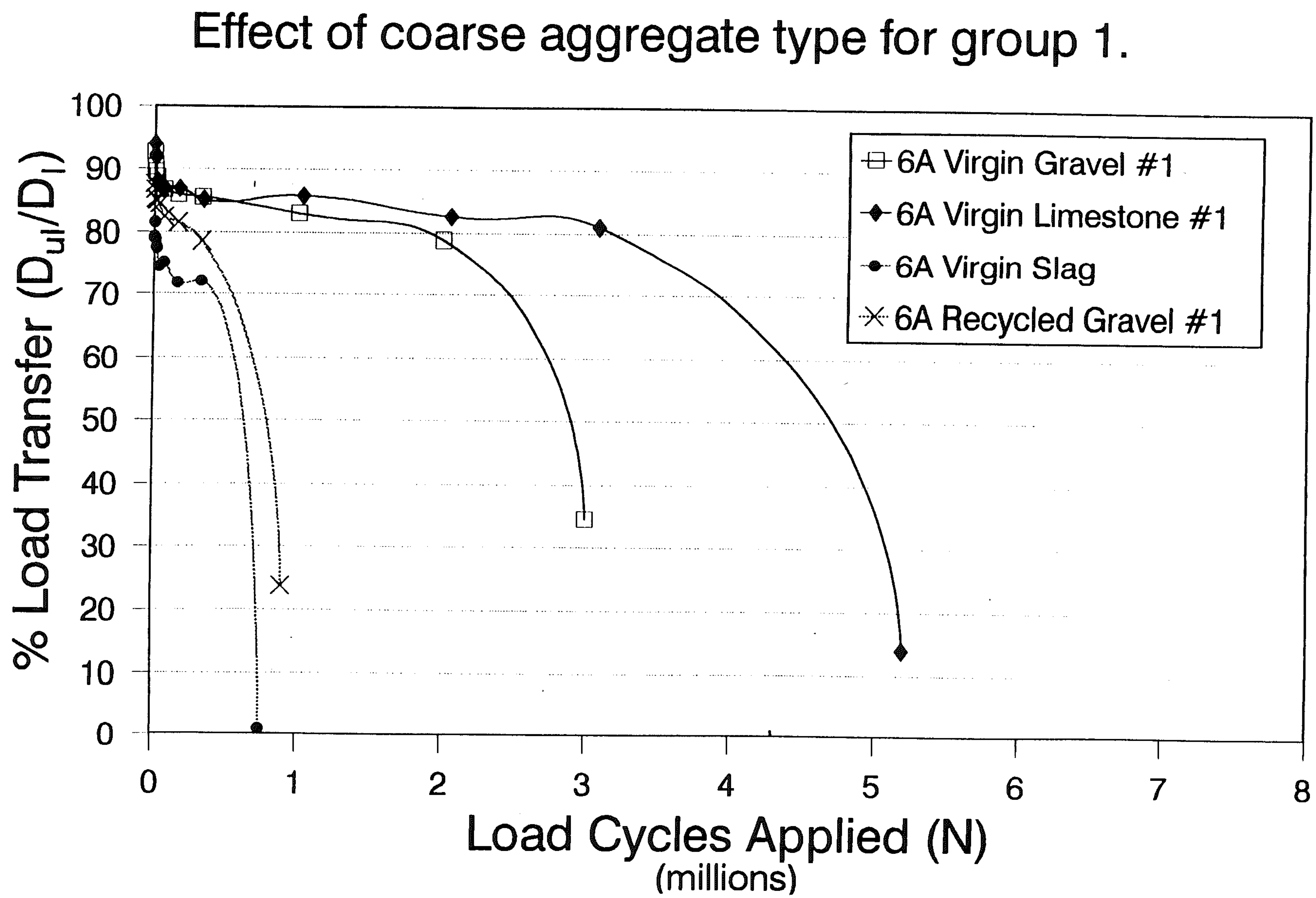


Figure 13. Effect of coarse aggregate type for group 1.

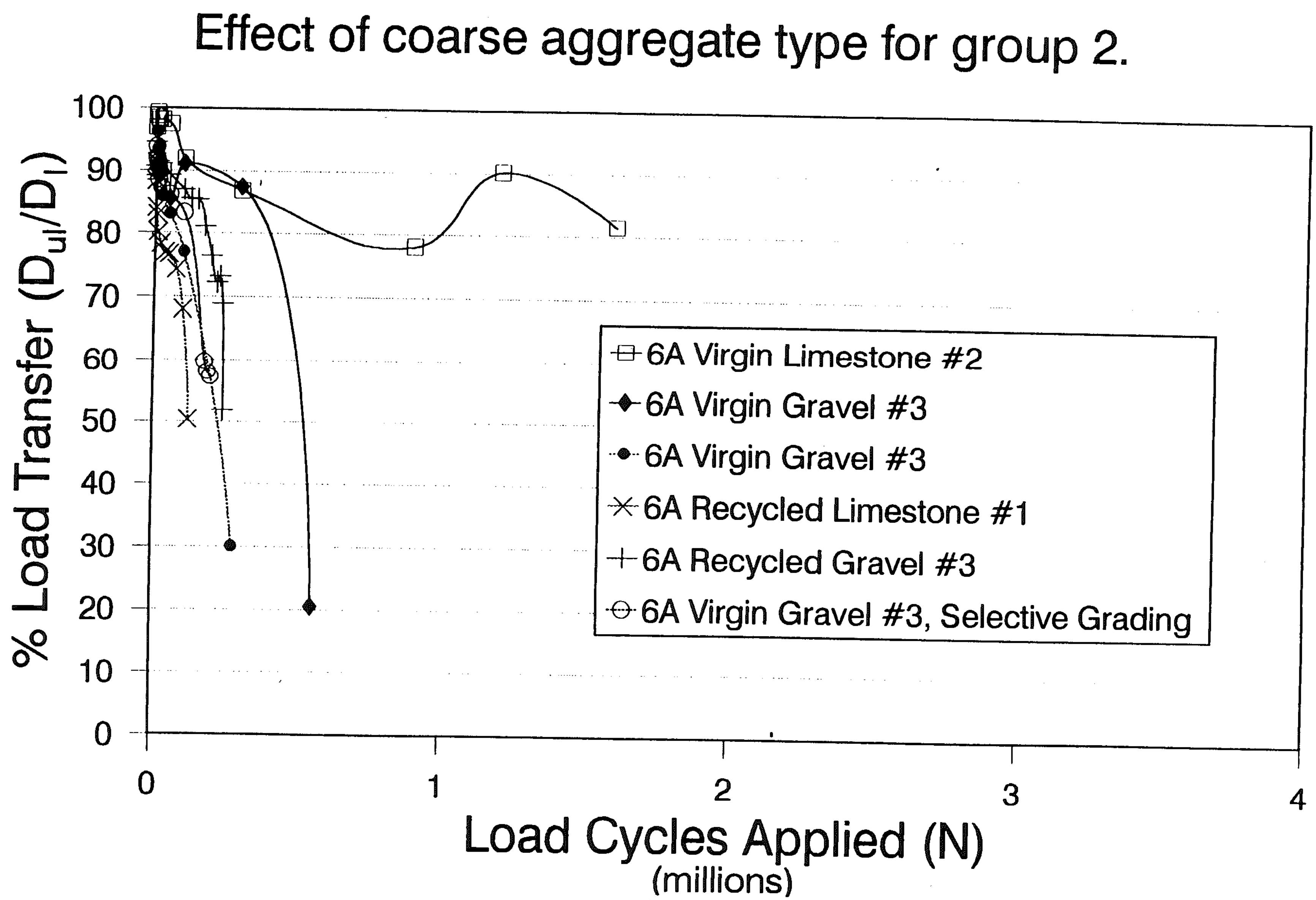


Figure 14. Effect of coarse aggregate type for group 2.

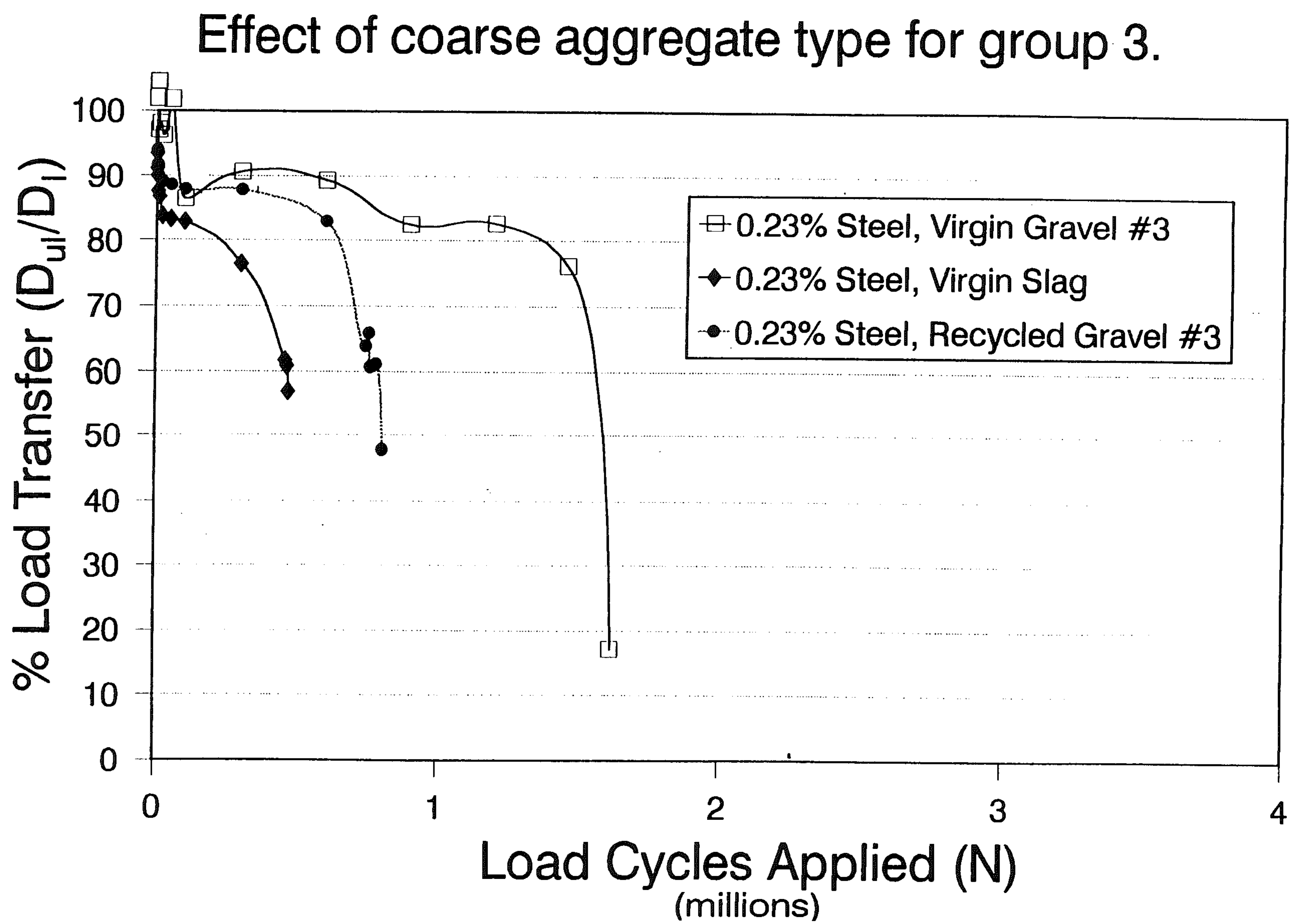


Figure 15. Effect of coarse aggregate type for group 3.

Consideration of the second section of table 5 (specimens 11, 13, 15, 24, 30 and 31) and figure 14 does not yield results that are as clearly cut as in the previous discussion. Performance varies widely, but some of this can be explained as a function of the crack width, which also varies widely between specimens. Although efforts were made to handle each test specimen identically, there was sometimes difficulty in verifying the formation of cracks and controlling their movement during handling of the untested specimens. It is likely that these variations in initial crack width mask the effects of aggregate type at least partially.

A reasonable comparison is possible between specimens 11 and 30 (virgin limestone #2 and recycled gravel #3, respectively), which exhibited comparable initial crack widths. As in the previous comparison, the natural material exhibited much lower differential vertical deflections than the recycled concrete specimen and performed acceptably for nearly 10 times as many load applications (testing was terminated before failure for scheduling reasons).

Considering the third section of table 5 (specimens 14, 33 and 34) and figure 15 provides a good comparison of specimens containing gravel #3 and a greater amount of steel reinforcing (0.23% by area of concrete). All three specimens exhibited comparable crack widths, but the slag and recycled concrete exhibited much higher peak and differential deflections, with the slag having the highest deflections. These differences in deflections are most probably attributable to differences in crack face texture, which was observed to be minimal for the slag concrete, slightly greater for the recycled aggregate concrete, and greatest for the virgin gravel. The initial deflection measurements were very well correlated with performance, as the slag concrete performed worst (27% of virgin aggregate endurance), followed by the recycled aggregate concrete (53% of the virgin aggregate endurance).

It is worth noting that the use of increased steel quantities generally improved the performance of each specimen when compared to specimens prepared with similar materials and standard reinforcing quantities.

4.3 EFFECT OF COARSE AGGREGATE SOURCE

Indications of the effect of varying coarse aggregate source are provided by considering the performance of specimens 7, 8, 13, 15 and 31 or specimens 2 and 11.

The first group of specimens (7 ... 31) were prepared using coarse aggregate meeting the MDOT 6A gradation specification; 7 and 8 included gravel #1, and 13, 15 and 31 included gravel #3, with specimen 31 being specially graded to provide the same particle size distribution as was used in the gravel #1 specimens after it was found that the as-delivered gradation of gravel #3 was significantly finer than that of gravel #1 (see table A-4, Appendix A). Selective grading was accomplished by sieving the aggregate into component size fractions and reblending the material to produce the desired gradation. Table 6 presents a summary of the performance results for these test specimens; figure 16 provides a graph that compares the load transfer histories of the five specimens.

Table 6. Performance summary for the effect of coarse aggregate source.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
7	6A Virgin Gravel #1	2,500,000	24.1	168	1651	N/A
8	6A Virgin Gravel #1	2,930,000	30.1	127	1575	N/A
13	6A Virgin Gravel #3	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3	225,000	1.8	84	1414	1626
31	6A Virgin Gravel #3, Selective Grading	145,000	1.6	194	1680	508
2	6A Virgin Limestone #1	4,630,000*	35.9	137	1092	N/A
11	6A Virgin Limestone #2	1,600,000+	N/A (16.6)	27	1442	279

NOTE:* Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

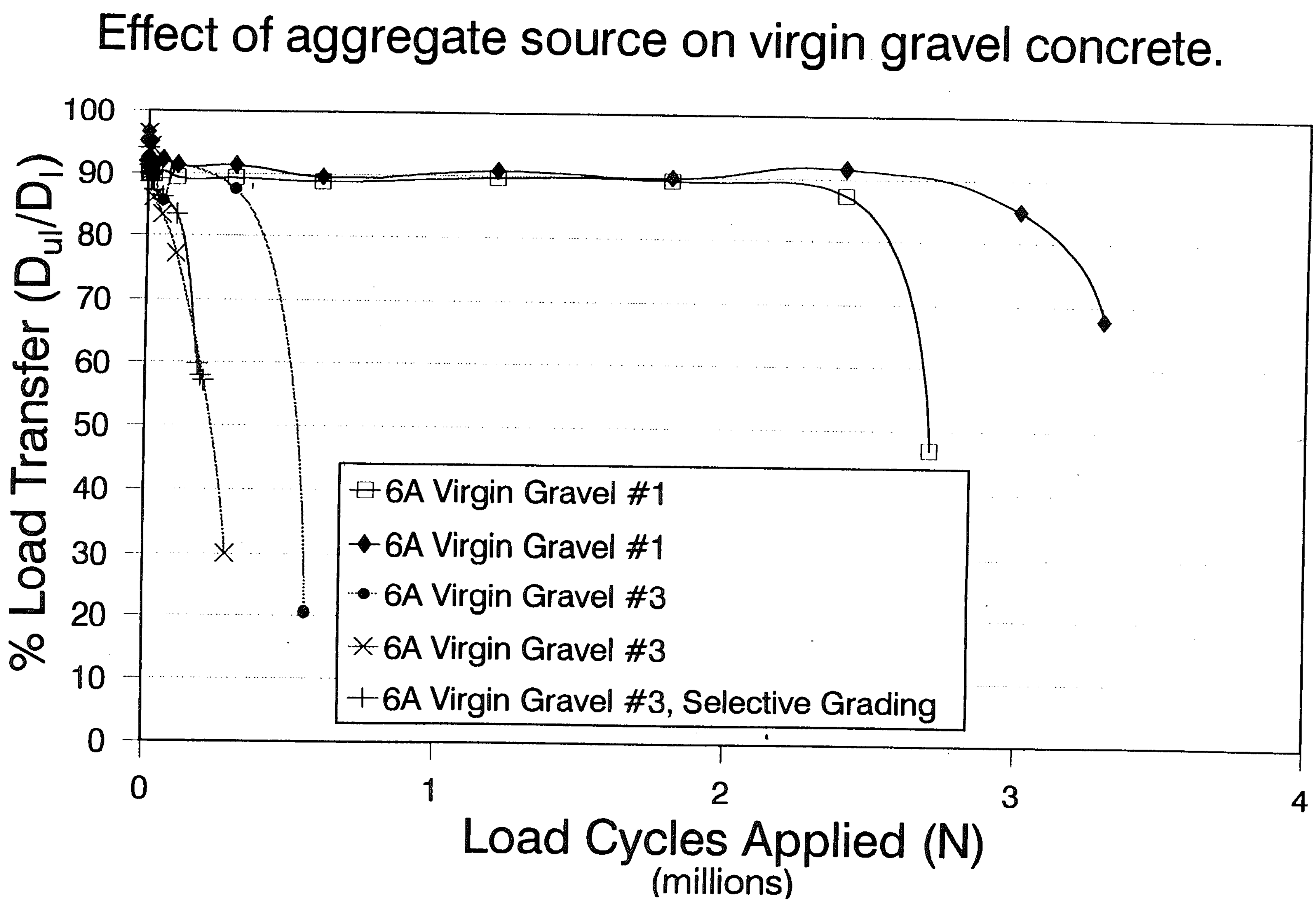


Figure 16. Effect of aggregate source on virgin gravel concrete.

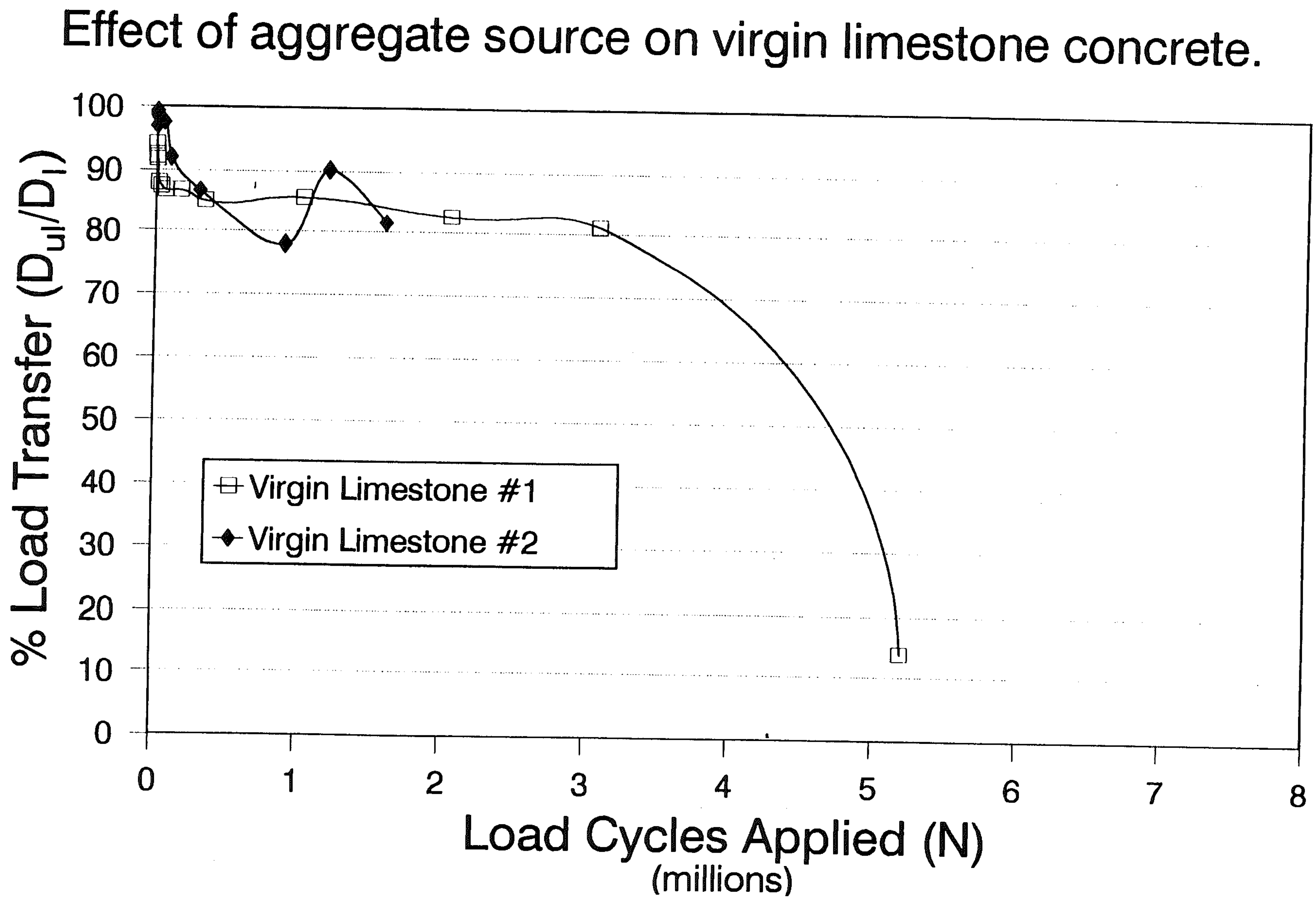


Figure 17. Effect of aggregate source on virgin limestone concrete.

Table 6 and figure 16 show a clearly superior performance for the two gravel #1 specimens (which exhibited extremely good agreement using any of the tabulated performance measures). Variations in the performance of the gravel #3 specimens are probably best explained by differences in the initial crack widths and differential deflections. For example, specimen 13 performed best and had the narrowest initial crack width while specimen 15 had an extremely wide crack and endured less than half as many load cycles before failure. Specimen 31 also had a wider crack than specimen 13 (not nearly so wide as specimen 15), greater peak and differential deflections than either specimens 13 or 15 and performed worst, in spite of the presence of more large particles in the mix.

There are no easy explanations for the striking difference in performance between the two gravel sources. It was noted during the disposal of the gravel #3 specimens that many of the aggregate particles were found to have no mortar adhering to the particles, suggesting that the poor performance may be due, at least in part, to poor strength due to debonding. However, tables A-7 and A-8 indicate that all of the specimens had comparable mix designs and measured strengths. Another possibility is that the initial crack widths of the gravel #1 specimens were narrower than for the gravel #3 specimens (crack width measurements were not measured until the second year of testing). It is also possible that gravel #3 contained a number of significantly weaker aggregate particles; this is considered somewhat unlikely, however, because gravel #3 was selected as a replacement for gravel #1 (after the source for #1 closed) because of their close proximity to each other and the presumption that they were both drawn from the same glacial deposit. The possibility of a systematic degradation in test stand characteristics is also rejected on the basis of the results of other test specimens that exhibited long test runs.

Results of the tests performed on specimens 2 and 11 are summarized in table 6 and figure 17. It should be noted that the number of load cycles to failure and the endurance index shown for specimen 2 are adjusted upward from the actual test results (using the actual test results of specimens 1, 7 and 8 to develop an adjustment factor) so that the effects of changes to the load frame between years 1 and 2 can be taken into consideration. The specimen prepared using 6A virgin limestone #1 endured approximately 4,630,000 load repetitions before failure, and the endurance index was 36. The specimen prepared using 6A virgin limestone #2 was not tested to failure, but endured 1,600,000 load repetitions before testing was discontinued because of test scheduling problems. The 6A virgin limestone #1 specimen displayed higher differential and peak deflections than the 6A virgin limestone #2 specimen, but this was attributed to the stiffening of the test frame after the first year of testing and is correlated with the endurance index and cycle-to-failure data shown, which have been adjusted as described previously. Since the 6A virgin limestone #2 specimen was not tested to failure, it can not be accurately determined whether the difference in aggregate source was significant for the limestone specimens. However, test logs indicate that specimen 11 was not exhibiting signs of imminent failure when testing was terminated. Thus, it appears that both specimens may have performed comparably.

In summary, the test results do not conclusively show a difference in performance that can be attributed solely to aggregate source. There was a significant difference in the performance of specimens prepared using gravel sources, but crack width data was not obtained early in the test program to determine whether the performance differences were due to aggregate source or other factors.

4.4 EFFECT OF COARSE AGGREGATE GRADATION

Indications of the effect of varying coarse aggregate source are provided by considering the performance of specimens 1 and 4 or 2 and 32. A summary of the performance data associated with these specimens in presented in table 7.

Specimens 1 and 4 were tested in the first year of the project. Specimen 1 was prepared using gravel meeting the requirements of MDOT specification 6A (37.5-mm [1.5-in] top size, coarser gradation), and specimen 4 was prepared using gravel meeting the requirements of MDOT specification 17A coarse aggregate (25-mm [1.0-in] top size, finer gradation). The common test parameters were: virgin gravel #1, 0.16% smooth wire mesh reinforcing, foundation stiffness of 27 kPa/mm [100 psi/in], and an induced slab tension of 51 kN/m width [3500 lb/ft width]. Load transfer histories for these two specimens are presented in figure 18. Although crack width data was not collected for these specimens, the initial deflection data appear to be comparable, which suggest that any differences in crack width were probably not great.

The 6A gradation specimen endured approximately 2,516,000 load repetitions (scaled) before failure, and the endurance index was 22. The 17A gradation specimen endured approximately 1,916,000 load repetitions (scaled) before failure, and the endurance index was 17. The two specimens performed nearly equivalently through approximately 160,000 load repetitions. The 17A gradation specimen then shows signs of losing load transfer. This may be due to the relatively small size of coarse aggregate which, after initial abrasion of the crack faces, requires a larger vertical displacement of the two slab fragments to make contact and transfer load.

Figure 19 shows a similar comparison of the load transfer histories for specimens prepared using virgin limestone aggregate (specimens 2 and 32). Specimen 2 was prepared using limestone #1 meeting the requirements of MDOT specification 6A (37.5-mm [1.5-in] top size), and specimen 32 was prepared using limestone #1 meeting the requirements of MDOT specification 4A coarse aggregate (62-mm [2.5-in] top size). The common test parameters were: 0.16% smooth wire mesh reinforcing, foundation stiffness of 27 kPa/mm [100 psi/in], and an induced slab tension of 51 kN/m width [3500 lb/ft width]. The 6A gradation specimen endured approximately 4,630,000 load repetitions (scaled) before failure, and the endurance index was 36. The 4A gradation specimen endured approximately 310,000 load repetitions before failure, and the endurance index was only 3.

The poor performance of the concrete made using the large aggregate (specimen 32) was unexpected, but may be explained (in part) by the rather wide crack width and resulting high deflections that were associated with the large aggregate specimen. It was also observed that the concrete produced using the 4A limestone was relatively weak when compared with that of the 6A limestone (21 MPa [3000 psi] vs. 37 MPa [5300 psi] compressive strength at 28 days; see table A-8, Appendix A), which may have been due to the poor particle size distribution in this mix and the reduced bond strength that often accompanies the use of larger aggregate particles. The

Table 7. Performance summary for the effect of aggregate gradation.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
1	6A Virgin Gravel #1	2,516,000*	22.3	144	1195	N/A
4	17A Virgin Gravel #1	1,916,000*	17.5	174	1275	N/A
2	6A Virgin Limestone #1	4,630,000*	35.9	137	1092	N/A
32	4A Virgin Limestone #1	310,000	2.6	207	1763	838

NOTE: * Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

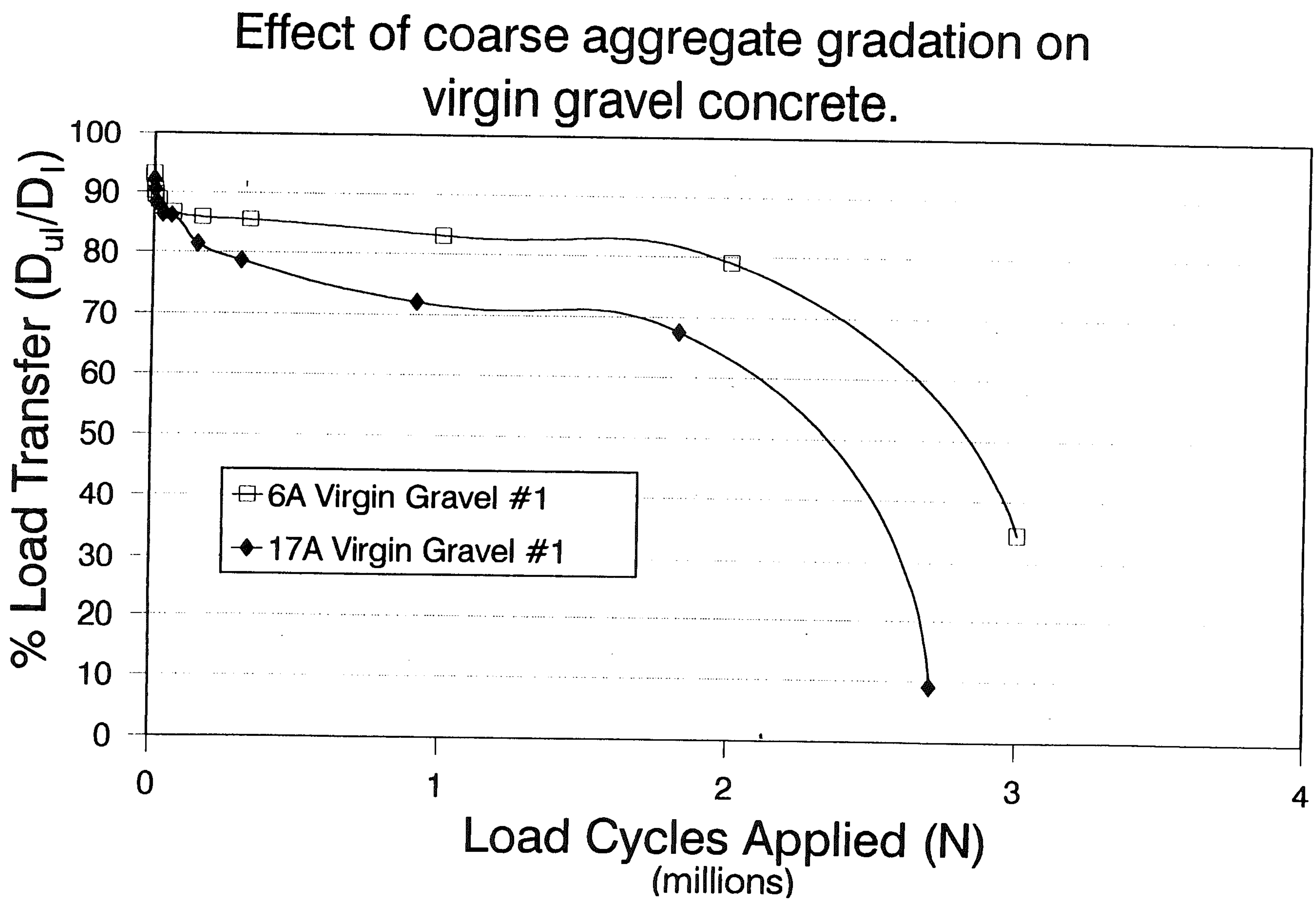


Figure 18. Effect of coarse aggregate gradation on virgin gravel concrete.

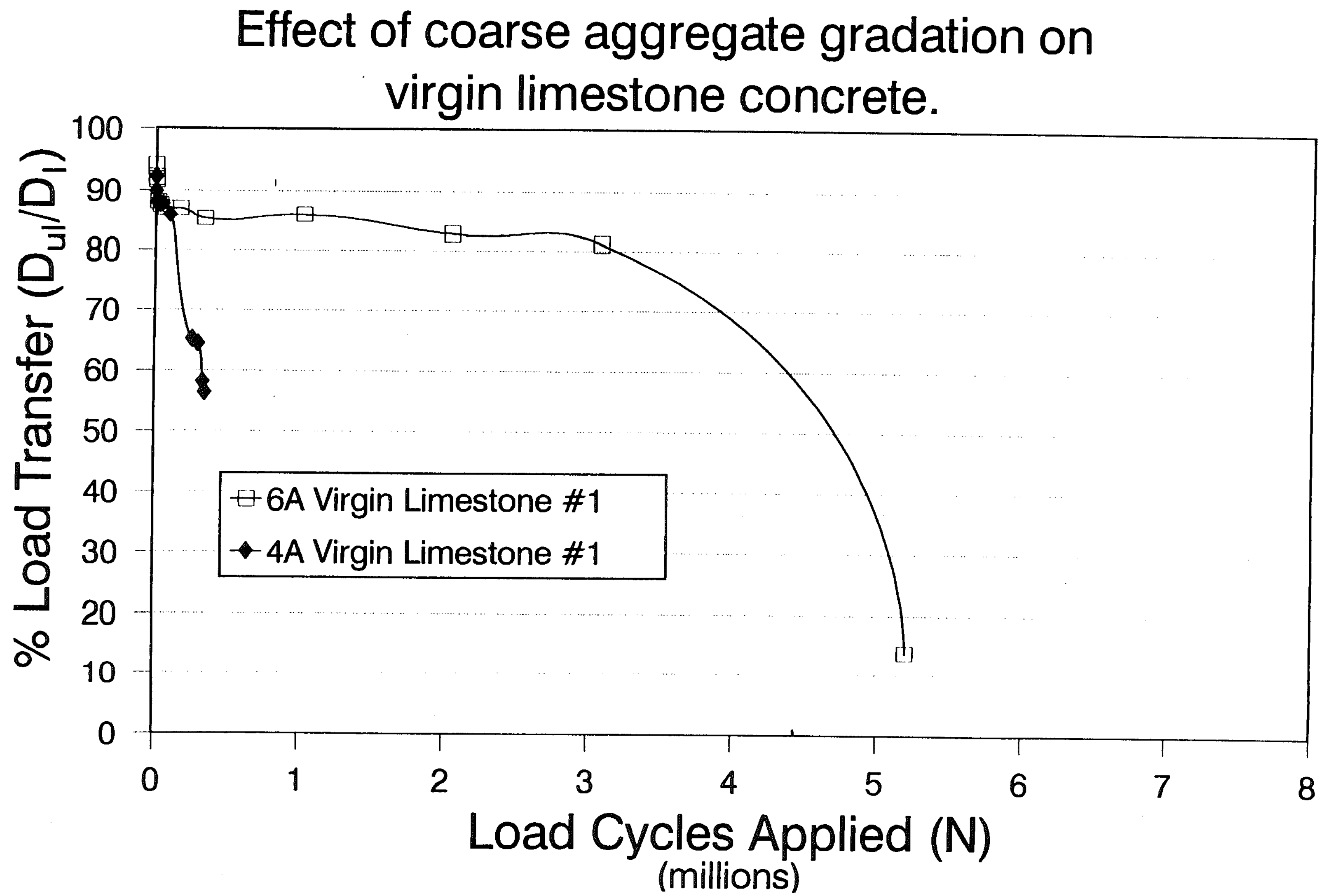


Figure 19. Effect of coarse aggregate gradation on virgin limestone concrete.

concrete was easily fractured during disposal and the aggregate particles near the crack face showed signs of being fractured into many pieces. Finally, the use of equivalent weights of coarse aggregate results in a drastic reduction in the *number* of particles present in the mix and at the crack face, which results in deeper grain interlock but at fewer locations. It is the opinion of the authors of this report that the combined effects of a large crack width and relatively few particles probably contributed most to the poor performance of specimen 32.

In summary, it seems likely that the inclusion of larger particles in the concrete mixture is effective in providing some improved grain interlock at transverse joints and cracks when all other factors (e.g., crack width, particle strength, concrete strength, etc.) are held constant. In addition, it can be shown geometrically that larger aggregate particles allow less differential vertical movement across a joint or crack for any given crack width.

4.5 EFFECT OF COARSE AGGREGATE TREATMENT

Indications of the effect of varying coarse aggregate treatment (i.e., virgin aggregate vs. recycled aggregate vs. a blend of virgin and recycled materials) are provided by considering the performance data presented in table 8.

The first comparison was performed using specimens 1, 5 and 6, which were all prepared and tested during the first year of the project. The first specimen was prepared using gravel #1 graded to MDOT specification 6A, the second specimen was prepared using 100% recycled gravel #1 concrete graded in accordance with MDOT specification 6A, and the third was prepared using a 50-50% blend (by weight) of 6A recycled gravel #1 concrete and 4A virgin limestone #1. The virgin aggregate specimen endured approximately 2,516,000 load repetitions (scaled) before failure, and the endurance index was 22. The 100% recycled gravel specimen endured approximately 710,000 load repetitions (scaled) before failure, and the endurance index was 5.6. The 50-50% blend specimen endured approximately 1,000,000 load repetitions (scaled) before failure, and the endurance index was 7.6. Figure 20 provides a comparison of the load transfer histories of these three specimens.

Test results show that the specimen prepared using virgin aggregate performed considerably better than the other two specimens. Although all measures of initial peak and differential deflections were approximately equal for all three specimens, they were lowest for the virgin aggregate (which performed best by far) and highest for the recycled aggregate concrete (which performed worst).

The use of 100% recycled material resulted in approximately 72% fewer loads to failure and a 75% reduction in endurance index when compared to the virgin aggregate specimen. Blending the recycled aggregate with an equal weight of large virgin limestone improved the performance somewhat (35% higher endurance index than for straight recycled material), but still did not approach that of the virgin aggregate specimen (a 66% reduction in endurance index when compared to specimen 1). Assuming that crack widths were comparable for the three specimens (crack width measurements were not obtained during the first year of testing), these results

Table 8. Performance summary for the effect of aggregate treatment.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
1	6A Virgin Gravel #1	2,516,000*	22.3	144	1195	N/A
5	6A Recycled Gravel #1	710,000*	5.6	228	1461	N/A
6	50-50% 6A Recycled Gravel #1 / 4A Virgin Limestone #1	1,000,000*	7.6	179	1397	N/A
2	6A Virgin Limestone #1	4,630,000*	35.9	137	1092	N/A
24	6A Recycled Limestone #1	95,000	1.0	367	1842	762
13	6A Virgin Gravel #3	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3	225,000	1.8	84	1414	1626
30	6A Recycled Gravel #3	210,000	2.0	145	1675	203
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	710,000	7.8	32	1710	686
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone #1	6,776,000+	N/A(62.2)	17	1478	0
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone #1	480,000	4.3	122	1386	0†
23	40-60% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	520,000	5.1	88	1580	483

NOTE: * Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.
† Crack width was attributed to compression of the top of the slab during 18-hour cracking.

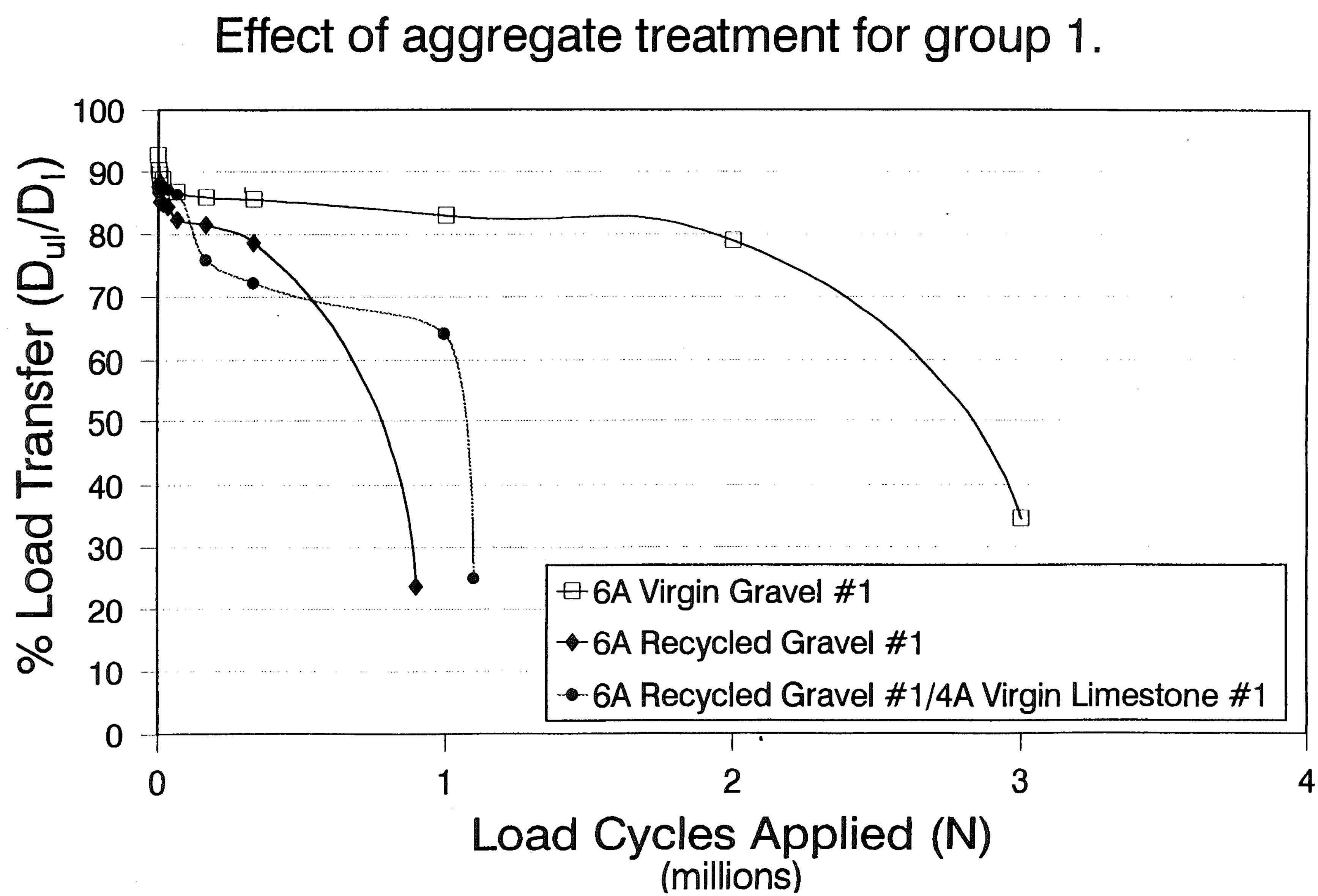


Figure 20. Effect of aggregate treatment for group 1.

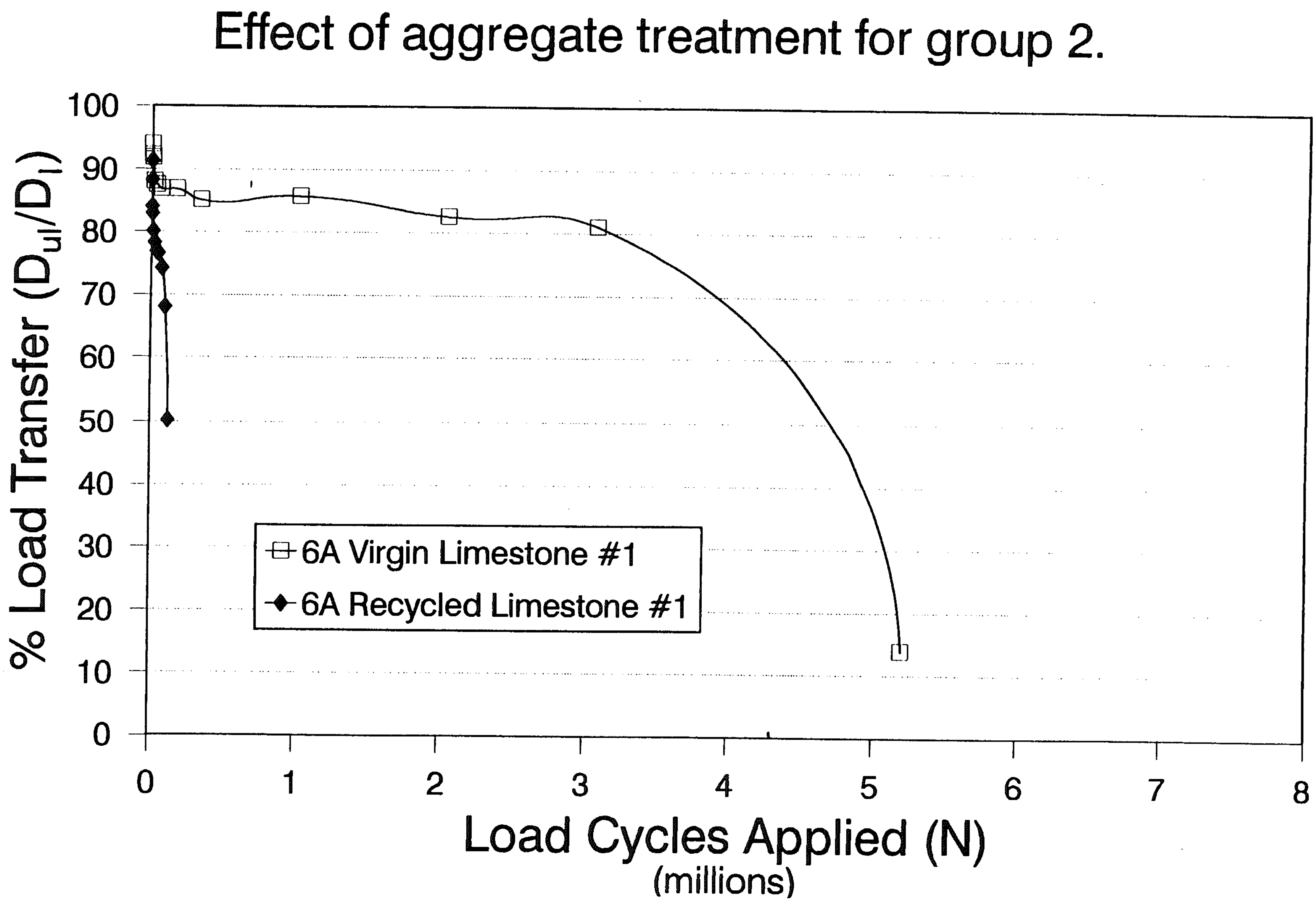


Figure 21. Effect of aggregate treatment for group 2.

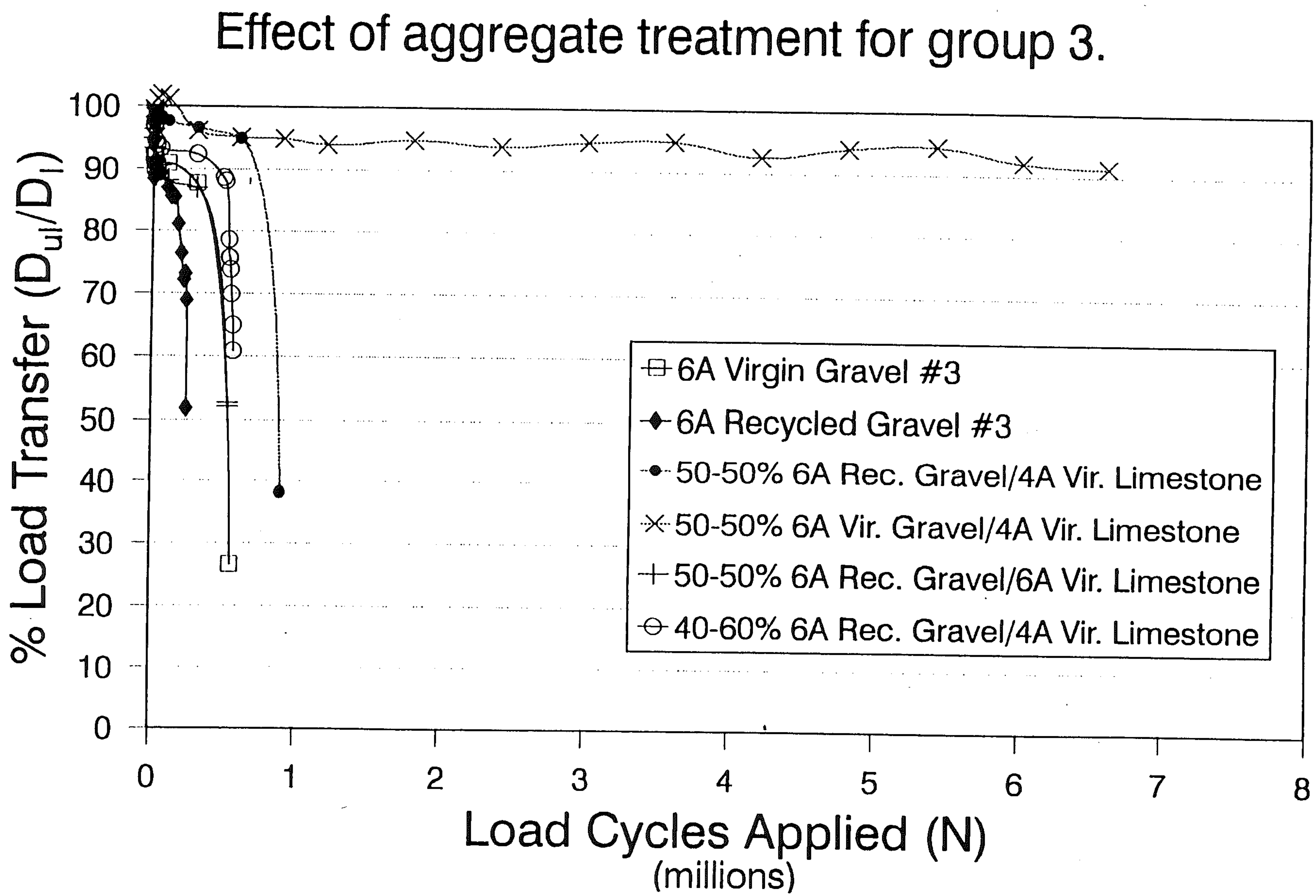


Figure 22. Effect of aggregate treatment for group 3.

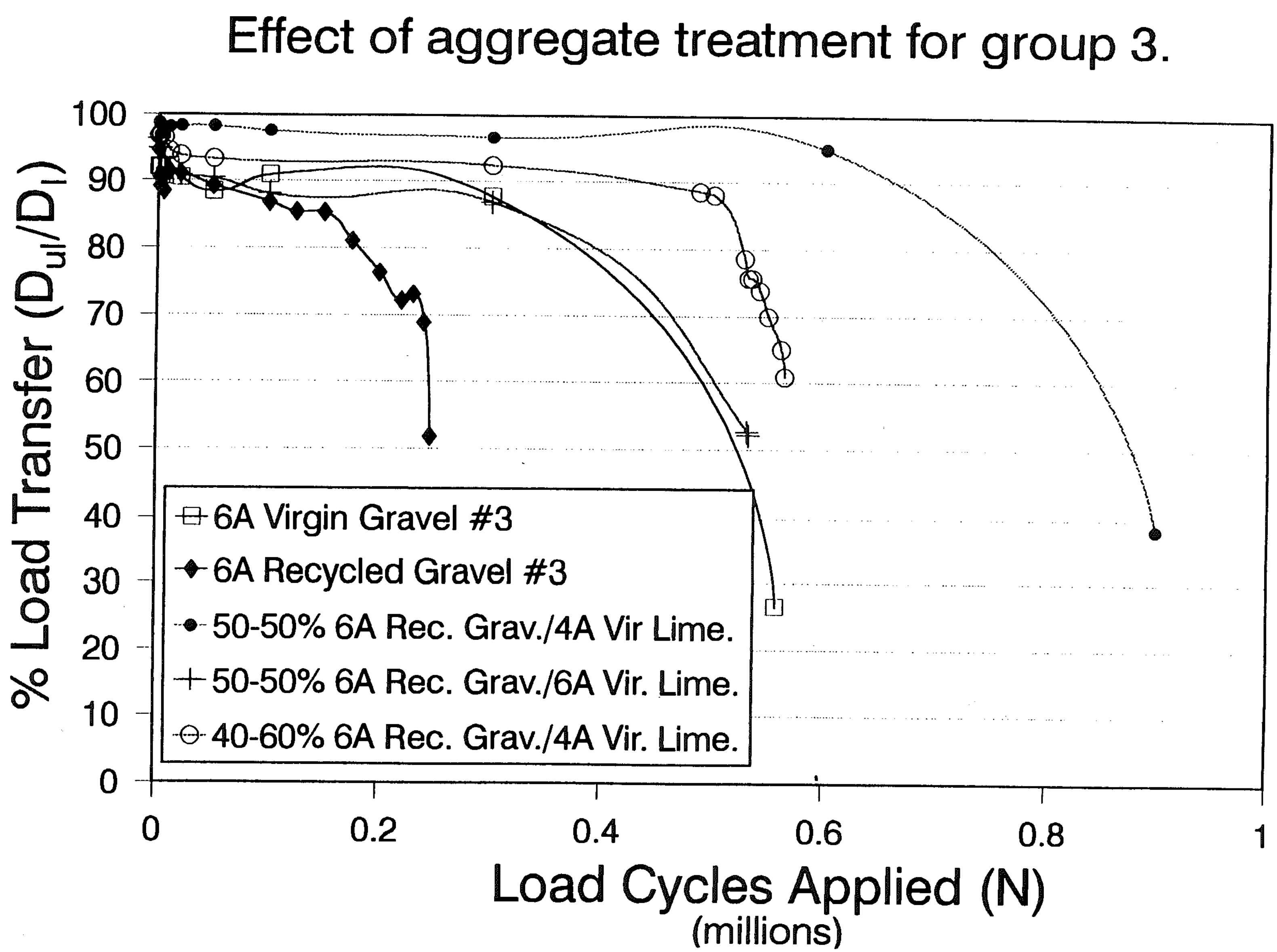


Figure 23. Effect of aggregate treatment for group 3.

suggest that the recycled aggregate concrete had the smoothest crack face texture, which was verified by visual inspection when the tests were completed.

It should be also noted that during transportation of the blended aggregate specimen from the cracking frame to the test stand, one of the lifting ropes broke, causing one end of the specimen to drop a distance of about 5 cm [2 in]. This may have affected the overall performance of this specimen, but it is believed that this specimen would have still performed more poorly than the virgin aggregate specimen. Nevertheless, additional testing of blended specimens was performed (as described below) to ensure that the mishandling did not adversely affect the test results.

Figure 21 shows the effect of aggregate treatment on limestone coarse aggregate specimens. Specimen 2 was prepared using virgin limestone #1 graded to MDOT specification 6A; specimen 24 was prepared using 100% recycled limestone #1 concrete graded to MDOT specification 6A. The virgin limestone specimen endured approximately 4,630,000 load repetitions (scaled) before failure, and the endurance index was 36. The specimen composed of recycled limestone coarse aggregate endured only 95,000 load repetitions before failure, and had an endurance index of 1.0.

These two specimens probably are not useful for this comparative analysis. Performance trends observed with other coarse aggregates suggest that the virgin limestone should have outperformed the recycled material due to a reduction in crack face texture for the recycled slab. However, the differences observed here are extreme and are probably attributable to other factors.

For example, a visual inspection of the 100% recycled limestone #1 specimen before testing showed that the crack width at the bottom of the slab was significantly greater than that at the top of the slab (where the measurements were taken). This increased width at the bottom of the slab was attributed to problems that developed during the cracking of the specimen, when it was necessary to repeatedly raise and lower the specimen in order to form a crack through the entire slab thickness. This action may have fatigued or hardened the reinforcing steel, and may have also caused some loose concrete particles to become wedged into the crack (resulting in the rather large initial crack width that was observed). In any event, there are enough questions about the handling and performance of specimen 24 that it probably should not be considered an accurate indicator of the relative performance potential of recycled limestone concrete.

An additional measure of the effects of aggregate treatment on the load transfer of transverse cracks can be obtained by considering the results of tests on specimens 13, 15, 30, 19, 21, 22 and 23. The bottom section of table 8 contains a summary of the performance indicators for these specimens and figures 22 and 23 provide graphs of the load transfer histories of the specimens. The coarse aggregate treatments that were tested included:

- virgin gravel #3 (graded to meet MDOT specification 6A) in specimens 13 and 15,
- recycled gravel #3 concrete (also graded to meet MDOT specification 6A) in specimen 30,
- a 50-50% blend of 6A recycled gravel #3 concrete and 4A virgin limestone #1 in specimen 19,
- a 50-50% blend of 6A recycled gravel #3 concrete and 6A virgin limestone #1 in specimen 21,
- a 40-60% blend of 6A recycled gravel #3 concrete and 4A virgin limestone #1 in specimen 22, and

- a 50-50% blend of 6A virgin gravel #3 and 4A virgin limestone #1 in specimen 23, which was tested to study the difference between virgin aggregate blends and recycled aggregate blends.

The results show that the specimen containing the 50-50% blend of 6A virgin gravel and 4A virgin limestone performed far better than the other test specimens. This specimen was not tested to failure, but endured 6,776,000 load repetitions, considerably more than the other test specimens, and the endurance index was greater than 62. This specimen had an initial differential deflection of only 17 microns [.00065 in] and an initial crack width that was too tight to measure (essentially zero). In addition to a narrow crack, this specimen seemed to exhibit excellent grain interlock due to the excellent distribution of particles at the crack face.

The performance of the 100% recycled gravel #3 (specimen number 30) should be compared with that of the virgin gravel #3 specimen number 13, since both had approximately the same initial crack width. The recycled crack width was actually slightly wider, and the peak and differential deflections were higher, resulting in a 51% reduction in performance. The performance reduction is probably due to both the increased crack width and the reduced crack face texture.

It should be noted that the reinforcement wire of specimen 30 (recycled gravel #3) appeared to have a slight manufacturing-related flaw: the third wire from the loaded edge of the specimen seemed to have a distinct differentiation of material (i.e. an inner core material varying from the outside material). The fourth and fifth wires also exhibited abnormal fracture patterns, suggesting that the entire wire mesh may have been defective. If this is true, the performance of this specimen may not have been representative of the performance potential for this material selection.

It is difficult to directly compare any of the recycled blend results to those of the virgin gravel specimens (numbers 13 and 15) or recycled gravel concrete specimen (number 30) because the initial crack widths are so much different in each case. However, the performance data and load transfer graphs (table 8 and figure 23) generally seem to suggest that blending the recycled material with a quantity of virgin stone of equal or larger size can be effective in improving the load transfer performance of the transverse cracks. The data also seem to suggest that blending virgin limestone with either the virgin gravel or the recycled gravel concrete provided performance improvements over the use of this particular virgin gravel source (which was not found to be true for gravel #1). The relative benefits of using more than 50% virgin material or varying the gradation of the virgin aggregate are not clearly indicated by these test results.

4.6 EFFECT OF FOUNDATION SUPPORT

The effects of foundation stiffness on the load transfer of transverse joints was evaluated by comparing the results of tests performed on specimens prepared using virgin gravel #3, recycled gravel #3, and virgin slag coarse aggregate. For each material, performance measures were obtained for tests performed on two values of foundation stiffness: approximately 27 kPa/mm [100 psi/in] and approximately 68 kPa/mm [250 psi/in]. A summary of the performance indicators for these tests is presented in table 9 and figures 24, 25 and 26 are graphs of the load transfer histories for these specimens.

The results of the virgin gravel aggregate specimens (specimens 9, 13, and 15) show that the specimen placed on the stiffer foundation had a higher load transfer efficiency initially and retained a higher load transfer efficiency than the specimens placed on the standard (relatively softer) foundation. The stiff foundation specimen endured approximately 6,100,000 load repetitions before failure, and the endurance index was 59. The two standard foundation specimens endured an average of only 350,000 load repetitions before failure, and the average endurance index was 2.5. Although crack width data were not obtained for specimen 9 (stiff foundation), it is believed that most of the improved performance of that specimen can be attributed to reductions in both peak deflections (30 - 40 % reduction) and differential deflections (50 - 60% reduction) over the entire performance life of the specimen, as brought about by the increase in foundation support. Increased foundation support allows a greater portion of the load to be transferred into the foundation, thereby reducing the amount of load being carried by the grain interlock and reinforcing steel across the crack.

Test results obtained from the recycled gravel #3 concrete specimens (specimens 28 and 30) show that the load transfer efficiency of the specimen tested on the stiffer foundation actually started out lower than that of the specimen on the standard (relatively softer) foundation specimen. This can be attributed to the differences in crack width; specimen 28 had an initial crack width that was more than twice as wide as that of specimen 30. In spite of this initially wide crack and greater differential deflections, the specimen tested on the stiff foundation still outperformed the specimen that was tested on the standard foundation. The stiff foundation specimen endured approximately 770,000 load repetitions before failure, and the endurance index was 6.4. The standard foundation specimen endured approximately 210,000 load repetitions before failure, and the endurance index was 2.0. In this case, the stiff foundation specimen was able to endure 260% more load repetitions than the standard foundation specimen.

In this comparison, the differential deflections of the stiff foundation specimen are, in fact, greater than the differential deflections of the standard foundation specimen, presumably due to the larger crack width. However, the peak deflection for the stiff foundation specimen at 10,000 cycles was 1501 microns [59.11 mils] while the peak deflection for the standard foundation specimen was 1675 microns [65.93 mils], which demonstrates that the stiff foundation reduces the magnitude of the overall deflection of the slab, as observed in the previous comparison.

It should be noted again that the reinforcement wire of specimen 30 (recycled gravel #3) appeared to have a slight manufacturing-related flaw, as described previously. If the wire mesh was defective, the performance of this specimen may not have been representative of the performance potential for this material selection and the performance comparison above may not be accurate. It should also be noted that debonding was evident around the reinforcement of the stiff foundation specimen. A continuous crack along the top of the reinforcement bars was found upon inspection of the crack face. Honeycombing was also evident at the crack face. The debonding and honeycombing may explain the relative difference in performance of the recycled gravel, stiff foundation specimen and the virgin gravel, stiff foundation specimen.

Table 9. Performance summary for the effect of foundation stiffness.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
13	6A Virgin Gravel, k = 27 kPa/mm	480,000	4.1	110	1393	127
15	6A Virgin Gravel, k = 27 kPa/mm	225,000	1.8	84	1414	1626
9	6A Virgin Gravel, k = 68 kPa/mm	6,100,000	59.2	42	1019	N/A
30	6A Recycled Gravel, k = 27 kPa/mm	210,000	2.0	145	1675	203
28	6A Recycled Gravel, k = 68 kPa/mm	770,000	6.4	271	1501	432
3	6A Virgin Slag, k = 27 kPa/mm	540,000*	4.0	276	1087	N/A
26	6A Virgin Slag, k = 68 kPa/mm	260,000	2.0	342	1830	483

NOTE:* Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

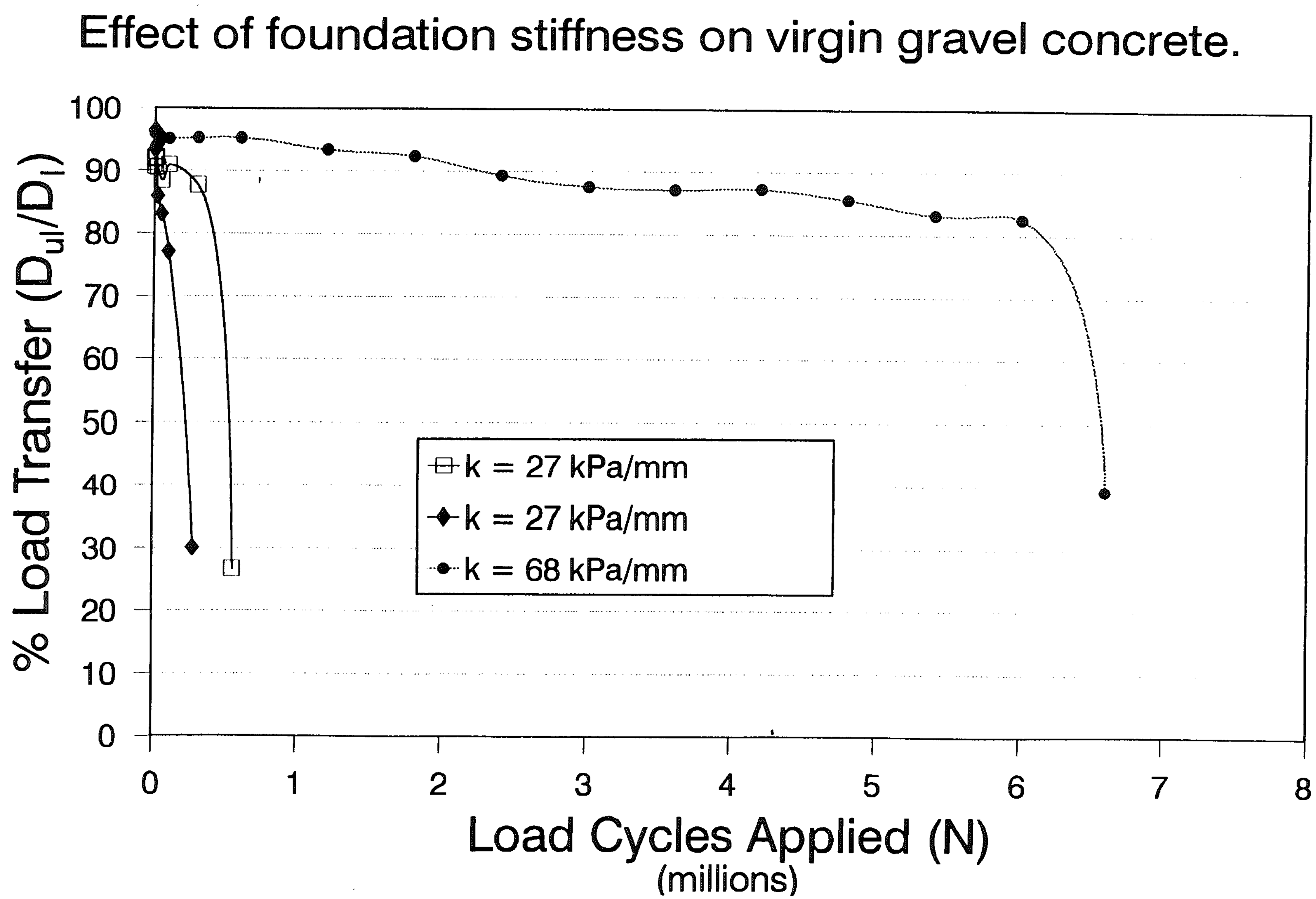


Figure 24. Effect of foundation stiffness on virgin gravel concrete.

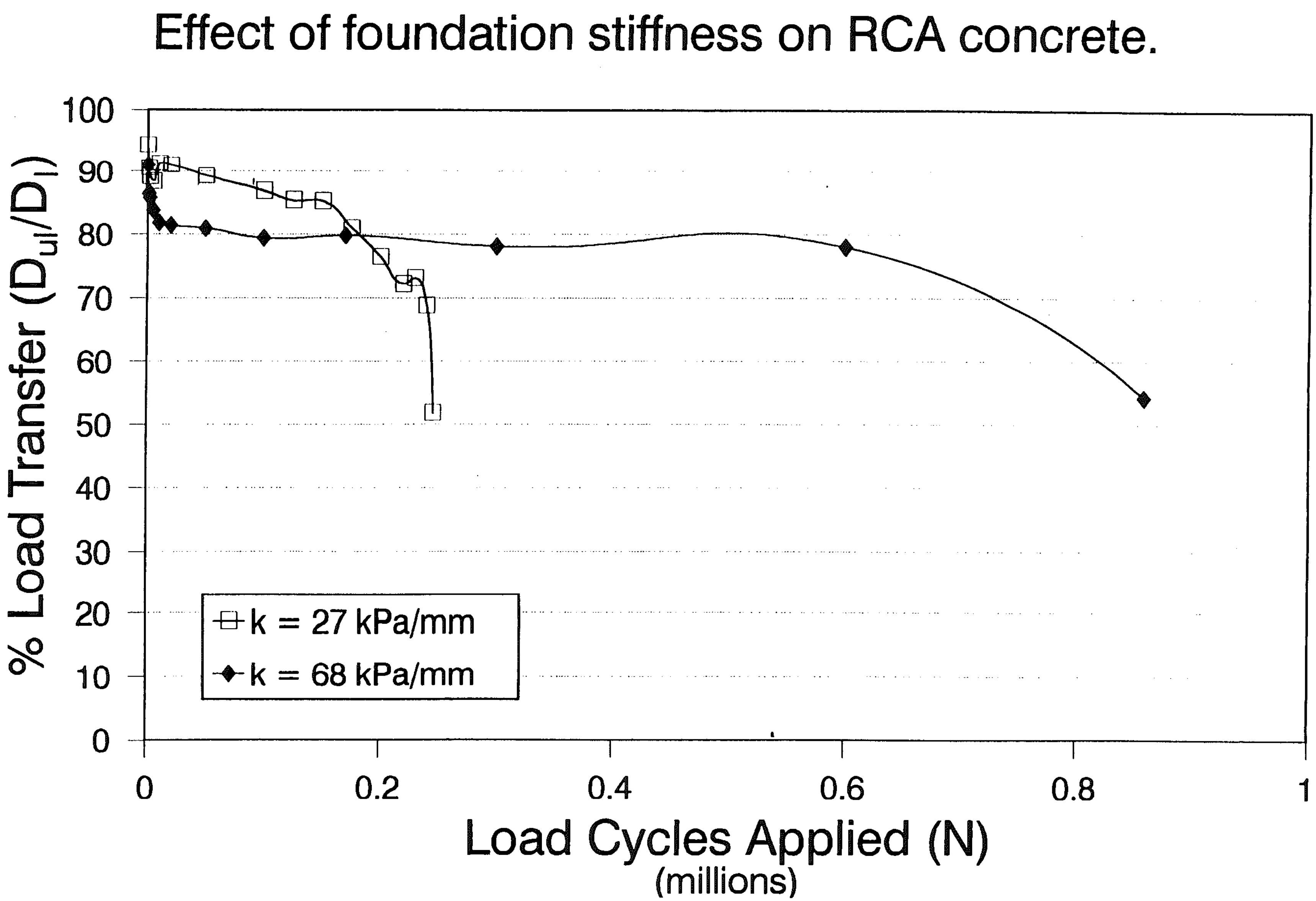


Figure 25. Effect of foundation stiffness on RCA concrete.

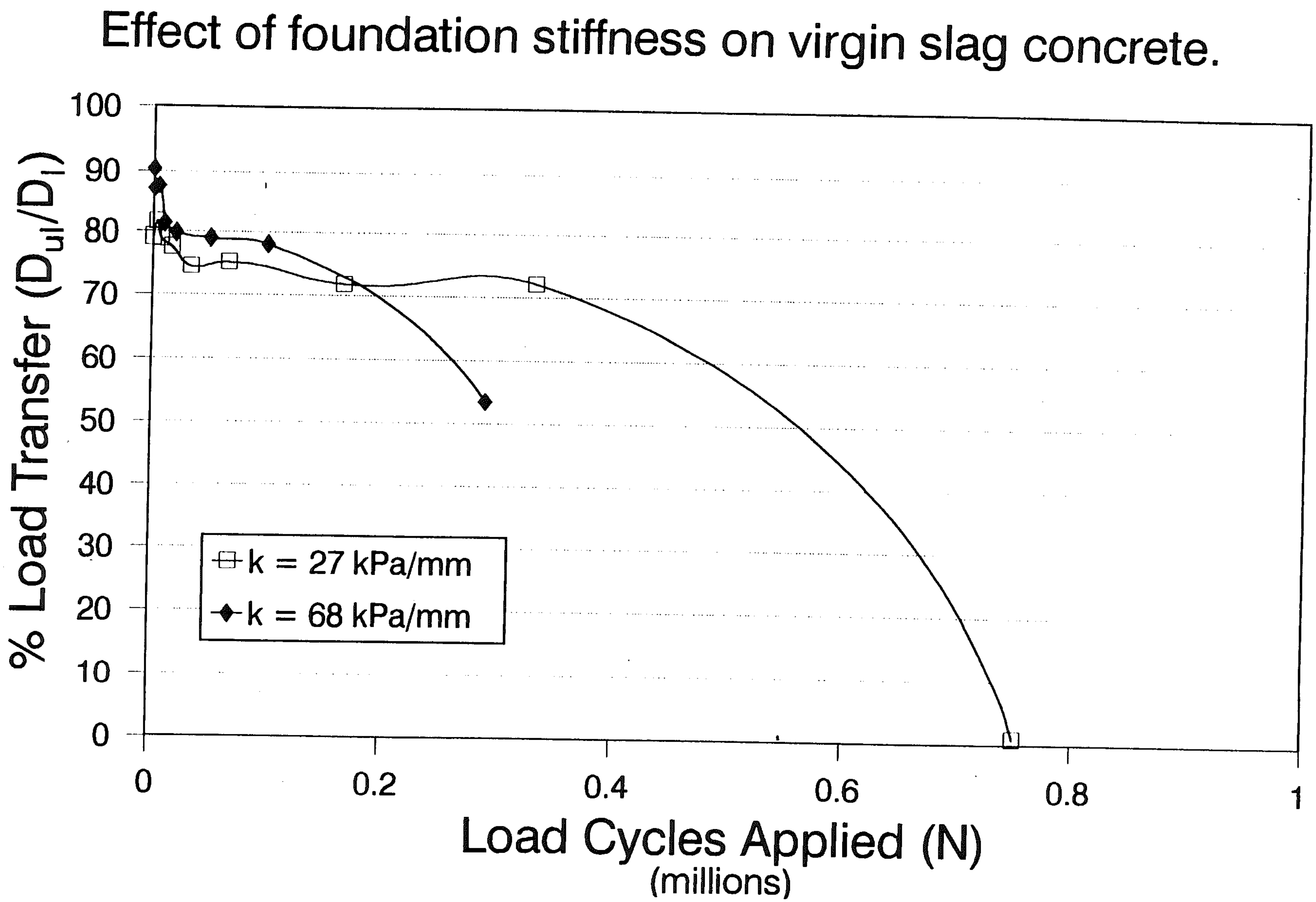


Figure 26. Effect of foundation stiffness on virgin slag concrete.

A comparison of the performances of the slag specimens on varying foundation stiffnesses could be used to draw conclusions that are contrary to those given above. The specimen tested using a foundation stiffness of 27 kPa/mm [100 psi/in] endured approximately 540,000 load cycles before failure, and the endurance index was 4.0. The differential deflection at 10,000 load repetitions was 276 microns [10.85 mils]. The specimen tested using a foundation stiffness of 68 kPa/mm [250 psi/in] endured approximately 260,000 load repetitions before failure, and the endurance index was 2.0. The differential deflection of this specimen at 10,000 load repetitions was 342 microns [13.45 mils].

The apparent reduction in performance of the specimen tested on the stiff foundation is probably attributable to a problem that developed during the cracking of the specimen. The original crack (induced after 18 hours of curing) did not appear to propagate to the surface by the time testing was to begin (after 28 days of curing). An attempt was made to induce the crack just prior to testing the specimen, which apparently resulted in significantly more fracture through the aggregate particles rather than around them (due to increased strength of the bond between the aggregate and mortar after 28 days). Visual inspection of the crack face after failure revealed that the top 1 to 2 inches of the crack face exhibited a smoother crack texture than the rest of the face. In addition, the second effort at cracking the specimen probably resulted in a wider crack (although the crack width of the other specimen was not measured). Finally, it is possible that a different adjustment factor should have been used for the slag aggregate specimen from year 1 (specimen 3); the adjustment factor used for the gravel specimens and applied here to the slag specimen may not be appropriate. If this were the case, it is possible that the *actual* number of load cycles to failure for specimen 3 is less than that of specimen 26.

In summary, it appears (based on the results of tests performed on virgin gravel and recycled gravel concrete specimens) that increases in the foundation stiffness can produce tremendous improvements in the load transfer endurance of transverse cracks in JRCP. This is because the increased foundation support reduces peak and differential deflections and reduces the shear load that must be carried by the crack face and reinforcing steel. Comparisons of the performance of the slag specimens are probably not appropriate for the reasons stated previously.

4.7 EFFECT OF SLAB TENSION

The effect of slab tension (i.e., slab length and slab-subbase friction factor) was evaluated by comparing the results of two sets of data. The first comparison was of specimens prepared using virgin gravel #3 graded to MDOT specification 6A (specimens 10, 13, and 15). Specimens 13 and 15 were tested with a standard slab tension of 51 kN/m width [3500 lb/ft width] and specimen 10 was tested with a higher slab tension of 102 kN/m width [7000 lb/ft width]. Table 10 presents a summary of the performance indicators associated with these tests, and figure 27 provides a load transfer history comparison for the three specimens.

Since the crack width of specimen 15 was exceedingly large, the discussion that follows compares only the results of specimens 13 and 10. Specimen 13 endured 480,000 load repetitions before failure, and had an endurance index of 4.1. This specimen had a differential deflection of 110 microns [3.83 mils] and a crack width of 127 microns [35.0 mils] after 10,000 load cycles. The

Table 10. Performance summary for the effect of slab tension.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
13	6A Virgin Gravel #3, T = 51 kN/m	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3, T = 51 kN/m	225,000	1.8	84	1414	1626
10	6A Virgin Gravel #3, T = 102 kN/m	188,000	2.2	191	1334	N/A
29	6A Recycled Gravel #3, T = 34 kN/m	830,000	8.1	203	1775	51
30	6A Recycled Gravel #3, T = 51 kN/m	210,000	2.0	145	1675	203

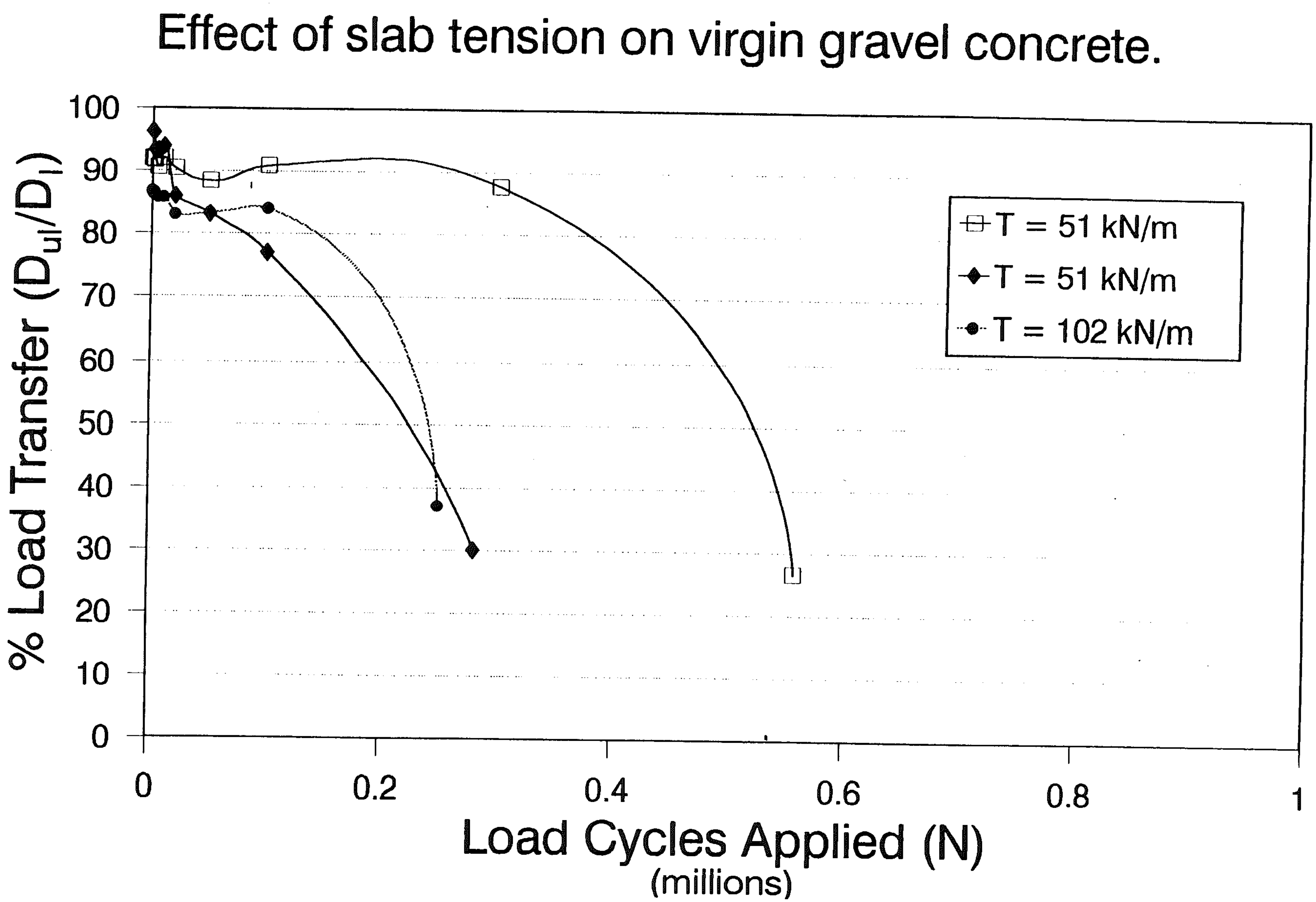


Figure 27. Effect of slab tension on virgin gravel concrete.

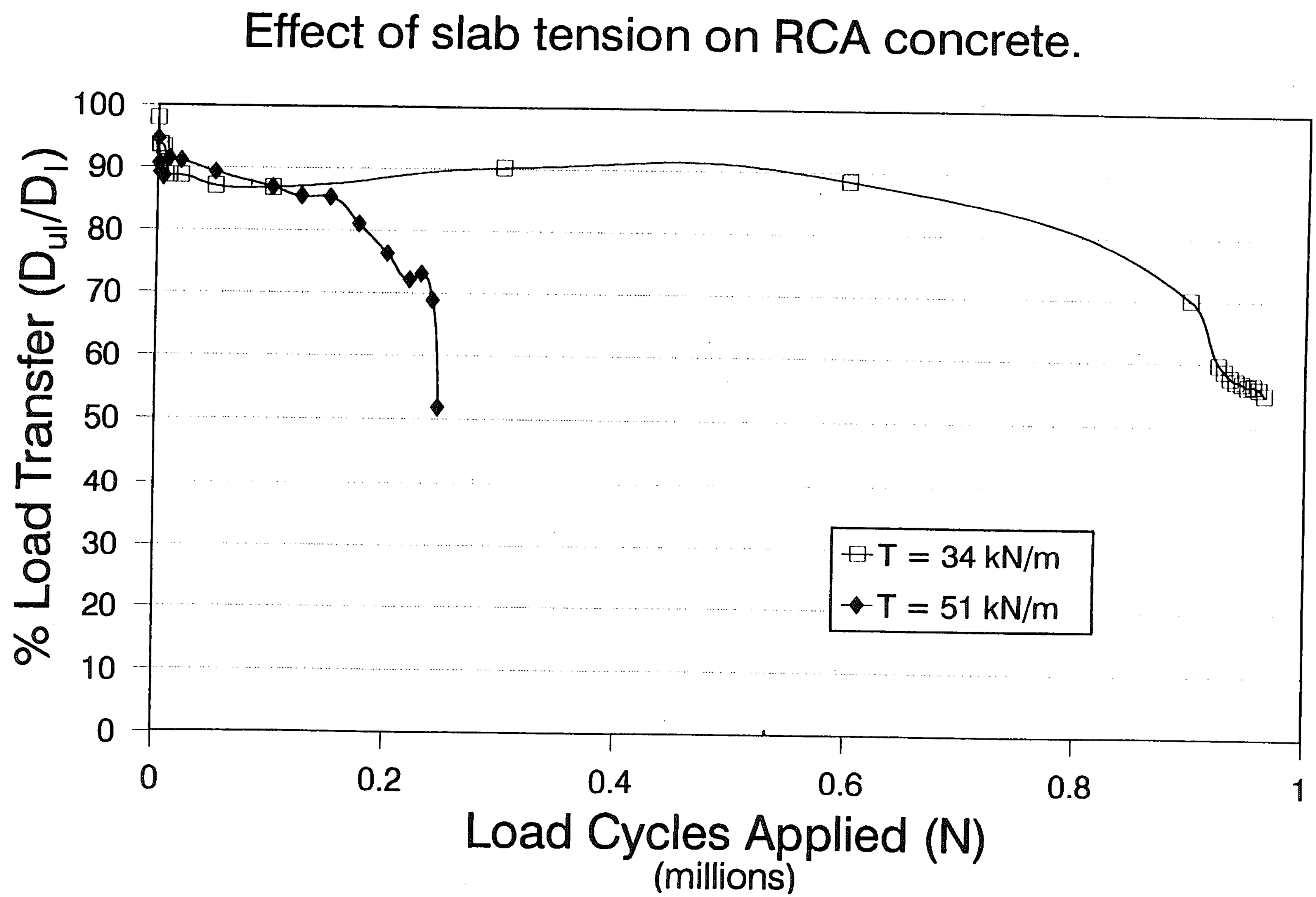


Figure 28. Effect of slab tension on RCA concrete.

high tension specimen endured only 188,000 load repetitions before failure, and the endurance index was 2.2. This specimen had a differential deflection of 191 microns [7.50 mils] after 10,000 load cycles. Crack widths were not measured for this specimen.

For this comparison, the results show that the high tension specimen performed worse than the specimen that was subjected to lower tension. This reduction in performance is probably due primarily to an increase in crack width (due to higher strain in the reinforcing steel). This increase in crack width reduces the contact between the two crack faces, thereby reducing load transfer and increasing abrasion of the crack face. This reduction in load transfer capacity is evident at all points in the test history of slab 10 (see figure 27).

Furthermore, the increase in crack width means that the longitudinal steel is forced to carry a greater portion of the shear load, possibly producing a greater propensity for fatigue failures due to critical combinations of shear, tension and bending stresses in the steel. In fact, the steel in this specimen (as well as many others) was observed to have failed due to a reduction in cross-sectional area that appeared to be the result of the formation of fatigue cracks. There was also a slight reduction in the diameter of the steel at the crack for the high tension specimen, suggesting that tensile failure was the ultimate mode of steel failure after the fatigue crack was initiated. These findings suggest that current reinforcing steel design procedures fail to adequately consider all of the stresses in JRCP reinforcing steel, resulting in a contribution to the premature failures of some JRCP sections.

The second comparison of slab tension effects studies specimens prepared using recycled gravel #3 concrete graded to MDOT specification 6A (specimens 29 and 30). Specimen 29 was subjected to the standard slab tension of 51 kN/m width [3500 lb/ft width] and specimen 30 was subjected to a reduced slab tension of 34 kN/m width [2305 lb/ft width]. Figure 28 shows the load transfer histories for these specimens and the performance indicators are presented in table 10.

The standard slab tension specimen endured 210,000 load repetitions with an endurance index of 2.0. The differential deflection for this specimen was 145 microns [5.70 mils] at 10,000 cycles, and the crack width was 203 microns [8.00 mils] at the same point. The reduced slab tension specimen endured approximately 830,000 load repetitions before failure and the endurance index was 8.1. The differential deflection for this specimen was 203 microns [8.00 mils] at 10,000 cycles, and the crack width was 51 microns [2.00 mils] at the same point.

In this comparison, the specimen with the reduced slab tension endured nearly 300% more load repetitions before failure in spite of having somewhat higher initial peak and differential vertical deflections. Again, the improvement in performance was attributed to the reduction of the initial crack width (brought about, at least in part, by the reduction in steel strain at the crack), and consequently the reduction in the shear, tension and bending stresses in the reinforcing steel.

In summary, reductions in slab tension (brought about by the use of shorter panel lengths and/or reductions in slab-foundation interface friction) appear to be highly effective in reducing the rate of deterioration of transverse cracks in JRCP made using any coarse aggregate source. This is brought about by a the reduction in crack width and steel stresses that are associated with reduced tension, allowing the grain interlock load transfer to remain high with reduced abrasion.

4.8 EFFECT OF REINFORCEMENT QUANTITY

The effect of the quantity of reinforcement on the load transfer performance of transverse joints was studied by comparing the performance of several sets of specimens prepared using different types of coarse aggregate. Specimens 13, 15 and 14 were prepared using gravel #3 graded to MDOT specification 6A; specimens 13 and 15 contained 0.16% longitudinal steel while specimen 14 contained 0.23% steel. A summary of the performance measures for these slabs is presented in table 11; load transfer histories are presented in figure 29.

The results show that the specimen containing 0.23% longitudinal steel performed much better than the specimens containing 0.16% longitudinal steel. Specimen 14 (0.23% steel) endured approximately 1,430,000 cycles before failure, and the endurance index was 13.3. This specimen had a differential deflection of 18 microns [0.70 mils] and a crack width of 305 microns [12.0 mils] at 10,000 cycles. If specimen 15 is dropped from the comparison (because of its unusually wide crack), specimen 13 can be assumed to represent the 0.16% case with a performance of 480,000 cycles to failure, an endurance index of 4.1, a differential deflection of 110 microns [4.33 mils] and a crack width of 127 microns [5.0 mils].

The use of larger wires and a greater quantity of reinforcing steel resulted in an increase in load cycles to failure of nearly 200%, in spite of a wider initial crack width for the specimen with more steel. The improvement in performance can be attributed to the ability of the increased steel quantity to reduce strain in the steel, thereby reducing the tendency for the crack to open as the test program proceeded. In this way, grain interlock was maintained for a longer period of time, reducing differential deflections and the resulting steel fatigue and abrasion of the crack face texture.

Figure 30 shows a similar comparison of two specimens prepared using recycled gravel #3 concrete and smooth wire mesh reinforcement (specimens 30 and 34). This comparison confirms the improved performance of the specimen which contains 0.23% longitudinal steel, again in spite of a greater crack width. The specimen containing 0.23% longitudinal steel endured approximately 670,000 load repetitions before failure, and the endurance index was 7.0. This specimen had a differential deflection of 156 microns [6.15 mils] and a crack width of 457 microns [18.0 mils] at 10,000 load cycles. The specimen containing 0.16% longitudinal steel endured approximately 210,000 load repetitions before failure, and the endurance index was 2.0. This specimen had a differential deflection of 145 microns [5.70 mils] and a crack width of 203 microns [8.00 mils] at 10,000 cycles. Again, the use of larger wires and a greater amount of reinforcing steel resulted in a net increase in load cycles to failure (approximately 220%) and a comparable increase in the endurance index.

Figure 31 shows the load transfer histories for the effect of the quantity of reinforcement on specimens prepared using 6A virgin slag (specimens 3 and 33). The first specimen contained 0.16% smooth wire mesh reinforcing, and the second specimen contained 0.23% smooth wire

Table 11. Performance summary for the effect of reinforcement quantity.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
13	6A Virgin Gravel, 0.16% Steel	480,000	4.1	110	1393	127
15	6A Virgin Gravel, 0.16% Steel	225,000	1.8	84	1414	1626
14	6A Virgin Gravel, 0.23% Steel	1,430,000	13.3	18	944	305
30	6A Recycled Gravel, 0.16% Steel	210,000	2.0	145	1675	203
34	6A Recycled Gravel, 0.23% Steel	670,000	7.0	156	1491	457
3	6A Virgin Slag, 0.16% Steel	540,000*	4.0	276	1087	N/A
33	6A Virgin Slag, 0.23% Steel	670,000	3.6	231	1747	330

NOTE:* Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

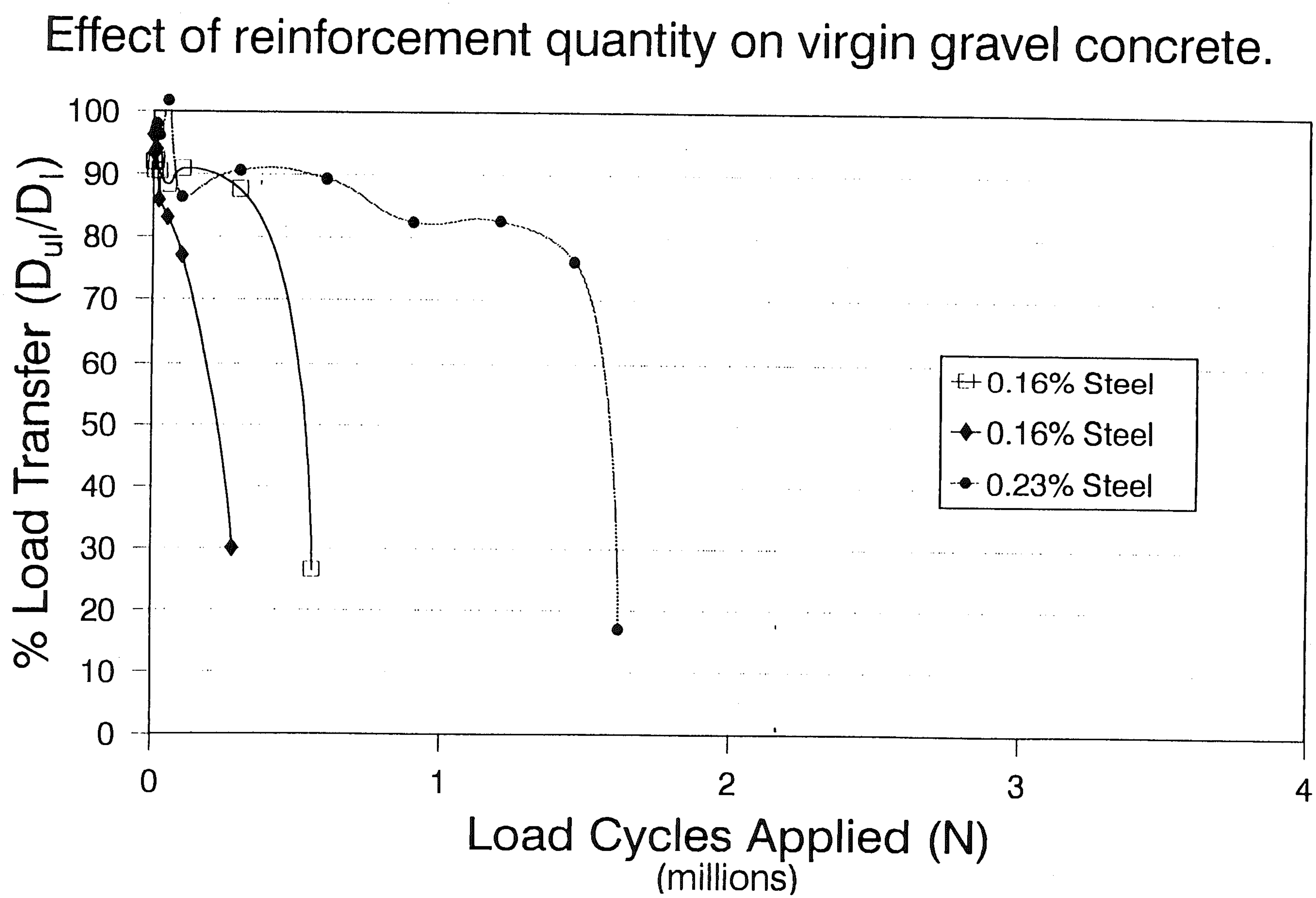


Figure 29. Effect of reinforcement quantity on virgin gravel concrete.

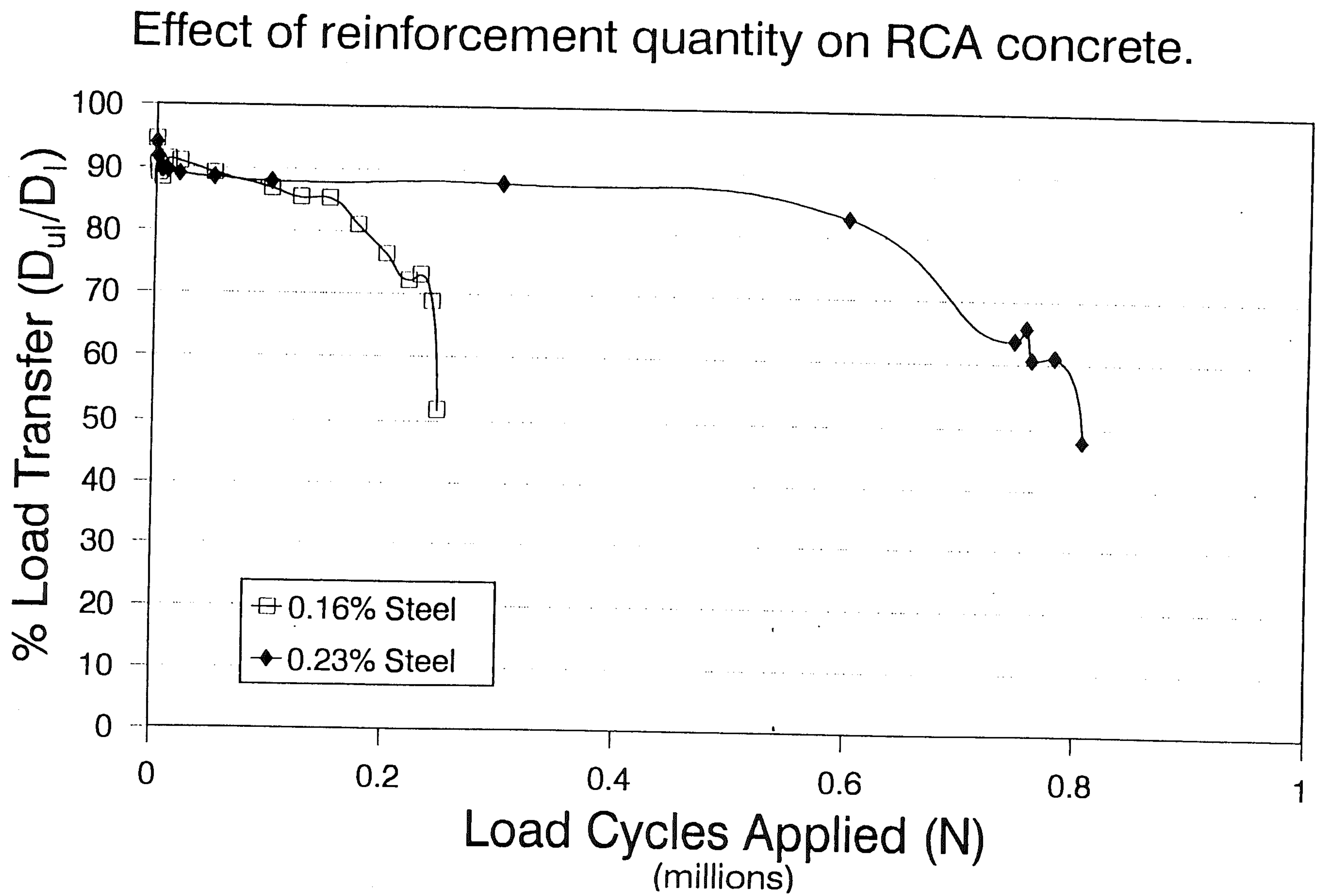


Figure 30. Effect of reinforcement quantity on RCA concrete.

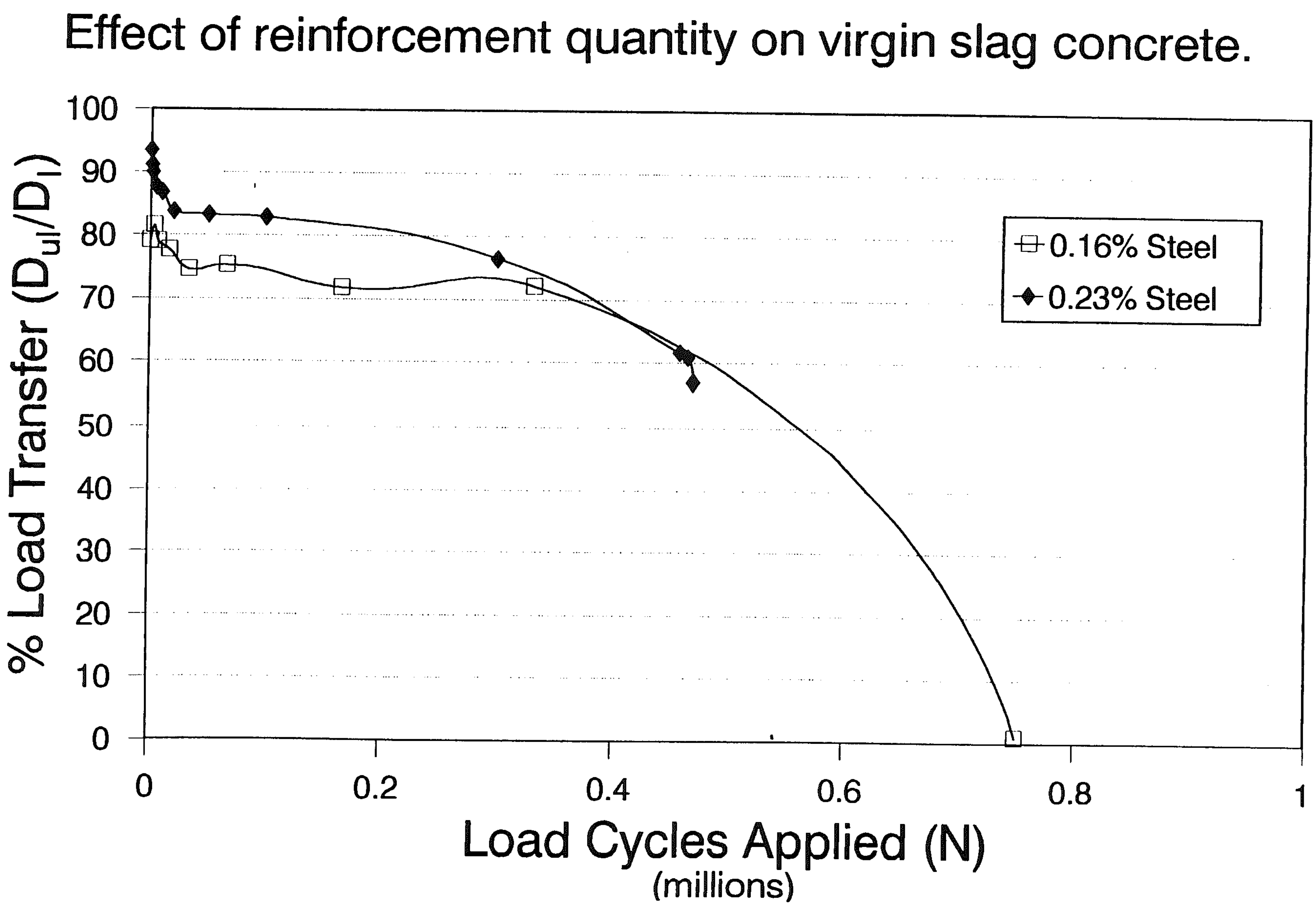


Figure 31. Effect of reinforcement quantity on virgin slag concrete.

mesh reinforcing. The first specimen endured approximately 540,000 load cycles before failure, and the differential deflection at 10,000 cycles for this specimen was 276 microns [10.85 mils]. The large wire specimen endured approximately 670,000 cycles before failure, and the differential deflection at 10,000 cycles was 231 microns [9.10 mils]. Thus, the slag specimen prepared using 0.23% smooth wire mesh reinforcing endured approximately 24% more load repetitions before failure. The mechanism for this improved performance is the same as was described above, although the results are not quite as dramatic given the lack of crack surface texture presented by the slag aggregate concrete.

In summary, the use of 50% more smooth longitudinal reinforcing produced performance improvements that ranged from 20% to more than 200%. This is a direct result of the ability of the increased steel to hold the cracks more tightly (due to reduced strain at any given level of tension) and reductions in steel stress and fatigue. These performance improvements were often observed, even when the initial crack width of the more highly reinforced specimen was somewhat wider than that of the normally reinforced specimen (which probably happened because it was more difficult to crack these specimens and more force was used to produce the cracks). In spite of the wider crack, the highly reinforced specimens generally exhibited comparable deflections and better overall performance than the normally reinforced specimens. It is believed that this phenomena can be attributed to the reductions in shear and bending fatigue that can be associated with the use of larger wires. If these assumptions are true, they suggest the need to reconsider traditional mesh reinforcing design procedures, which have considered only tension or "subgrade drag" forces, ignoring the effects of traffic loads on reinforcement performance.

4.9 EFFECT OF REINFORCEMENT TYPE

The effect of the variation of reinforcement type was studied by comparing the performance of four specimens prepared using virgin gravel #3 (graded to MDOT specification 6A) and 0.16% longitudinal steel (specimens 13, 15, 16, and 27). The first two specimens contained smooth wire mesh reinforcement, the third specimen contained deformed wire mesh reinforcement, and the fourth specimen contained 760-mm [30-in] long, 19-mm diameter [#6] deformed reinforcing bars on 460-mm [18-in] centers (the so-called "hinged" joint design, which represents 0.27% deformed steel reinforcement). Table 12 summarizes the performance indicators for these specimens and figures 32 and 33 show the load transfer history of these test specimens.

The specimen prepared using the “hinged” joint design endured 7,800,000 load repetitions without failing, for an endurance index of more than 71. The differential deflection for this specimen at 10,000 load repetitions was only 66 microns [2.61 mils], and the crack width was essentially nonexistent. The deformed wire specimen endured approximately 600,000 cycles before failure, with an endurance index of 6.2, a differential deflection of 104 microns [4.09 mils] and a crack width of 51 microns [2.00 mils] after 10,000 load repetitions. Specimen 15 was dropped from the comparison because of its unusually wide crack, which was believed to make the comparison invalid. The remaining smooth wire specimen, number 13, endured 480,000 cycles to failure, with an endurance index of 4.1, a differential deflection of 110 microns [4.33 mils] and a crack width of 127 microns [5.0 mils] after 10,000 load cycles.

Table 12. Performance summary for the effect of reinforcement type.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
13	6A Virgin Gravel, Smooth wire mesh	480,000	4.1	110	1393	127
15	6A Virgin Gravel, Smooth wire mesh	225,000	1.8	84	1414	1626
16	6A Virgin Gravel, Deformed wire mesh	660,000	6.2	104	1448	51
27	6A Virgin Gravel, Deformed bar ("Hinged" Joint)	7,800,000+	N/A (71.4)	66	1816	0

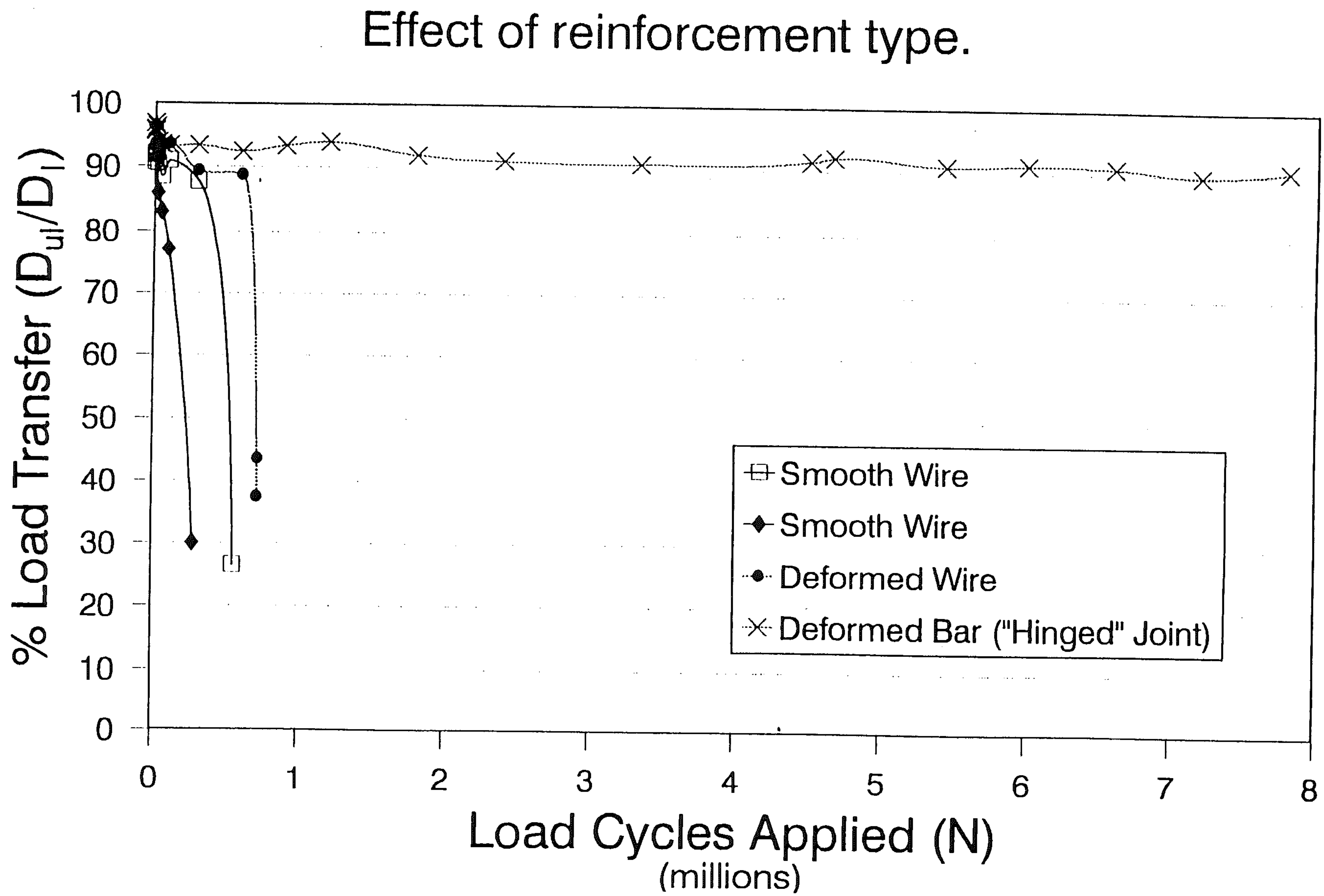


Figure 32. Effect of reinforcement type.

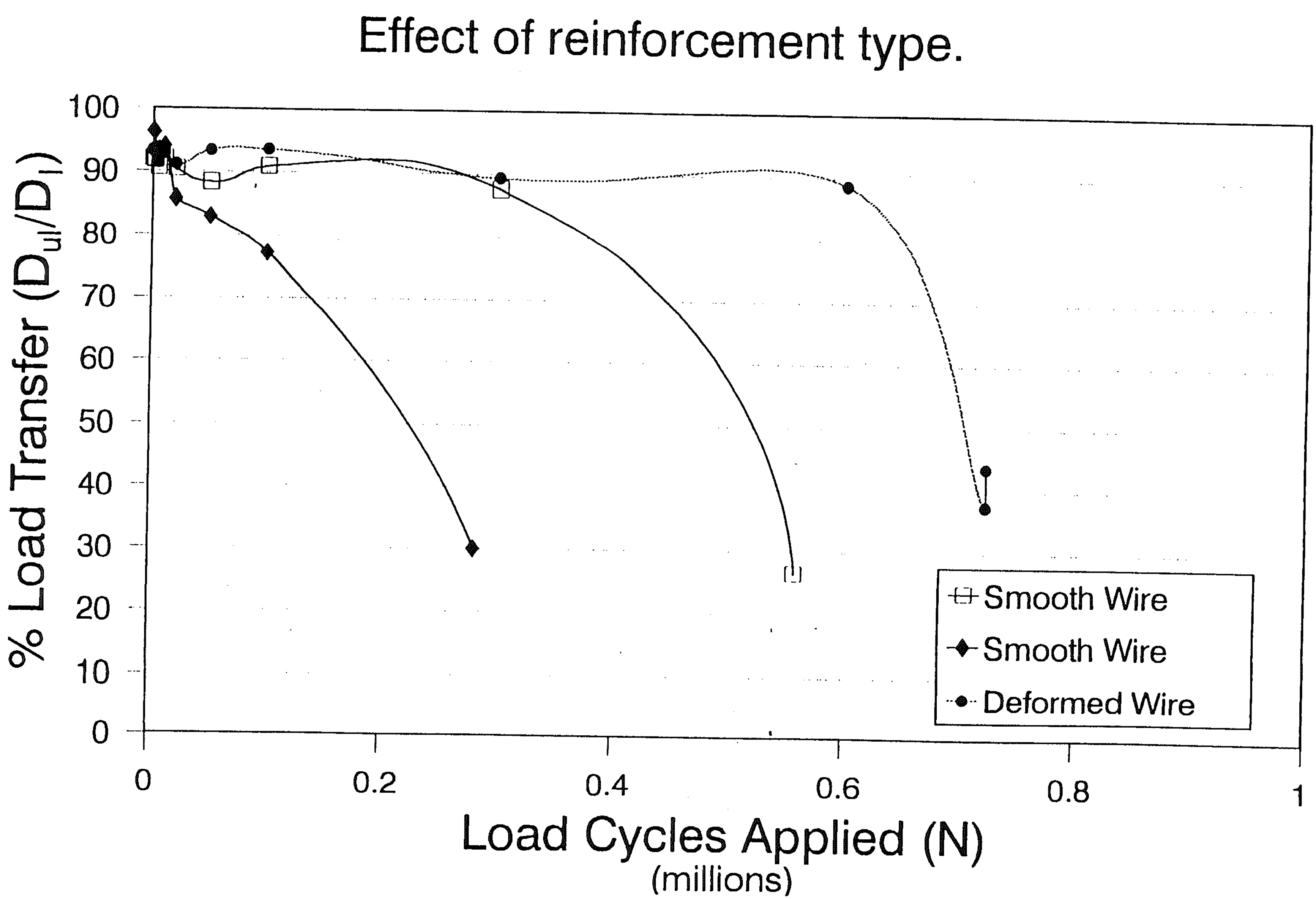


Figure 33. Effect of reinforcement type.

The use of deformed wire mesh provided a 38% increase in the load cycles to failure. This is attributed mainly to the fact that the crack width of the deformed wire specimen was consistently less than that of the normally reinforced specimens, presumably due to the bond between the concrete and deformed steel and accompanying restriction of strain to the vicinity of the crack. The deflection characteristics of the deformed wire specimen were not significantly different from those of the normally reinforced specimens. This suggests that most of the performance improvement may be due to increases in shear and bending fatigue resistance that accompany the use of slightly larger and harder wire (see table A-1, Appendix A).

The "hinged" joint design, which incorporated large deformed bars into the joint design, provided excellent performance, withstanding more than 7,800,000 load cycles without exhibiting any appreciable increases in crack width or deflection measurements. The crack was held tight because of the exceptional bond or interlock between the deformed bars and the surrounding concrete. This, in turn, resulted in reduced differential deflections and reduced shear and bending in the steel. The use of large deformed bars, rather than smaller wires, provided greatly improved resistance to shear and bending fatigue. The excellent performance of this specimen, together with the extended corrosion life that would be expected of larger bars, suggest that this design would provide excellent field performance.

4.10 COMBINED EFFECTS OF FOUNDATION STIFFNESS AND REINFORCEMENT DESIGN

The combined effects of reinforcement design and foundation stiffness were evaluated by comparing the performance parameters and load transfer histories two sets of specimens. The first set (specimens 13, 15, 9, 14 and 12) was prepared with virgin gravel #3 coarse aggregate graded to MDOT specification 6A and a slab tension of 51 kN/m width [3500 lb/ft width]. The first two specimens (control specimens 13 and 15) were prepared using 0.16% smooth wire mesh reinforcing and were tested on a foundation stiffness of 27 kPa/mm [100 psi/in]. Specimen 9 contained the same amount of wire mesh reinforcement, but was tested on a stiff foundation of 68 kPa/mm [250 psi/in]. Specimen 14 was prepared on the standard foundation using 0.23% smooth wire mesh reinforcement. The fifth specimen (specimen 12) was prepared with the increased steel quantity (0.23%) and was tested on the stiff foundation (68 kPa/mm [250 psi/in]). Table 13 presents a summary of the performance indicators for these specimens and figure 34 presents their load transfer histories.

The results of the first four specimens have been discussed previously in sections describing the effects of either foundation stiffness or reinforcement quantity. Specimen 12 (large wire, stiff foundation) showed much better performance than the control specimen (number 13), with an eight-fold increase in load repetitions to failure. It also provided a 180% improvement in performance over specimen 14, which included an increase in steel quantities but no increase in foundation support. However, it performed no better than (and actually slightly poorer than) specimen 9, which featured an improved foundation without any additional steel. This suggests that the major source of improvement in crack performance for gravel specimens (or other materials that produce an average or better crack face texture) may come from limiting differential slab fragment deflections, which is better achieved through the use of a strong foundation than

Table 13. Performance summary for the combined effects of foundation stiffness and reinforcement design.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
13	6A Virgin Gravel #3, k = 27 kPa/mm, 0.16% Steel	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3, k = 27 kPa/mm, 0.16% Steel	225,000	1.8	84	1414	1626
9	6A Virgin Gravel #3, k = 68 kPa/mm, 0.16% Steel	6,100,000	59.2	42	1019	N/A
14	6A Virgin Gravel #3, k = 27 kPa/mm, 0.23% Steel	1,430,000	13.3	18	944	305
12	6A Virgin Gravel #3, k = 68 kPa/mm, 0.23% Steel	4,000,000	39.2	68	886	203
30	6A Recycled Gravel #3, k = 27 kPa/mm, 0.16% Steel	210,000	2.0	145	1675	203
28	6A Recycled Gravel #3, k = 68 kPa/mm, 0.16% Steel	770,000	6.4	272	1501	432
34	6A Recycled Gravel #3, k = 27 kPa/mm, 0.23% Steel	670,000	7.0	156	1491	457
35	6A Recycled Gravel #3, k = 68 kPa/mm, 0.23% Steel	2,330,000	15.8	138	1338	0†

NOTE: † Crack width was attributed to compression of the top of the slab during 18-hour cracking.

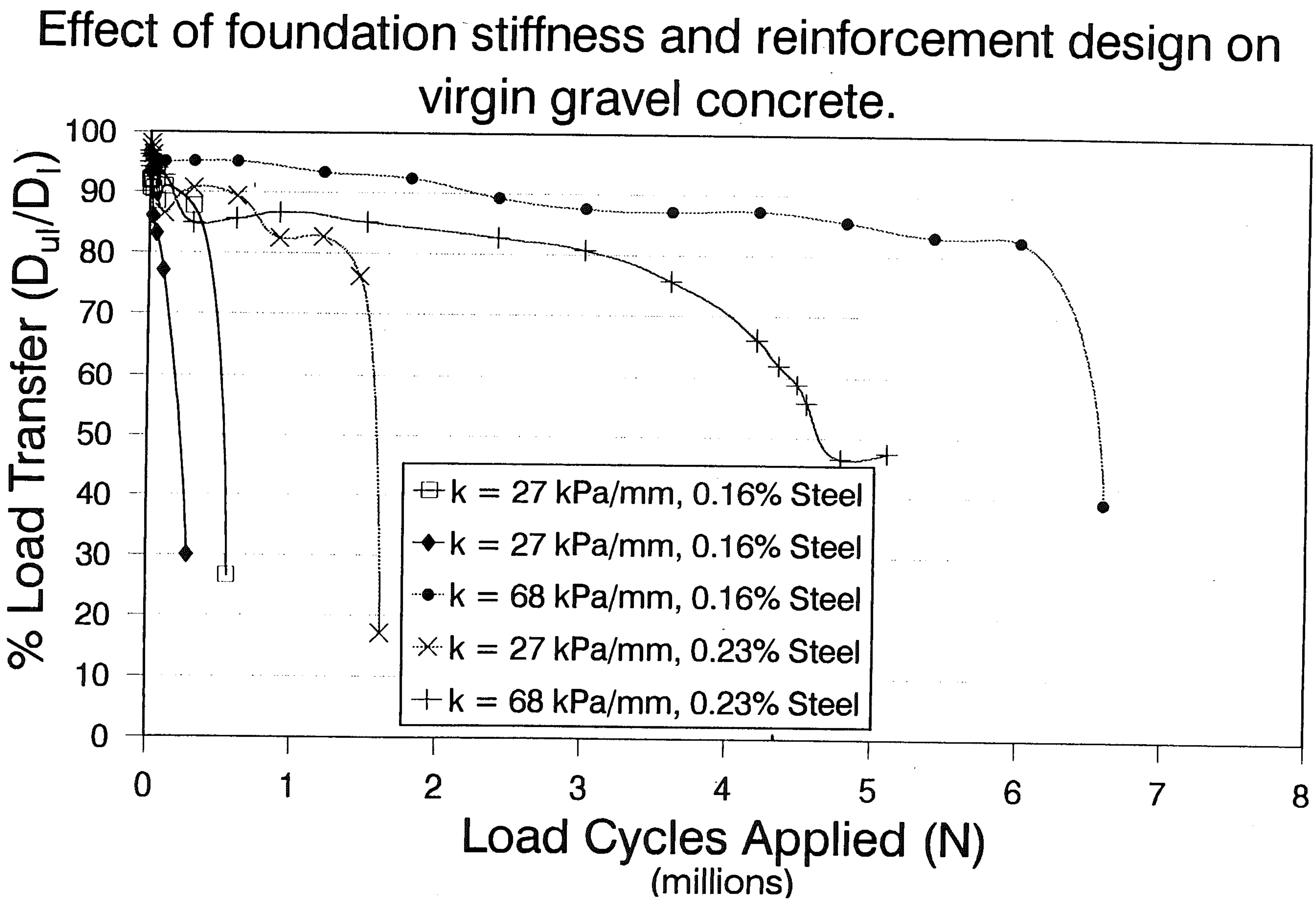


Figure 34. Effect of foundation stiffness and reinforcement design on virgin gravel concrete.

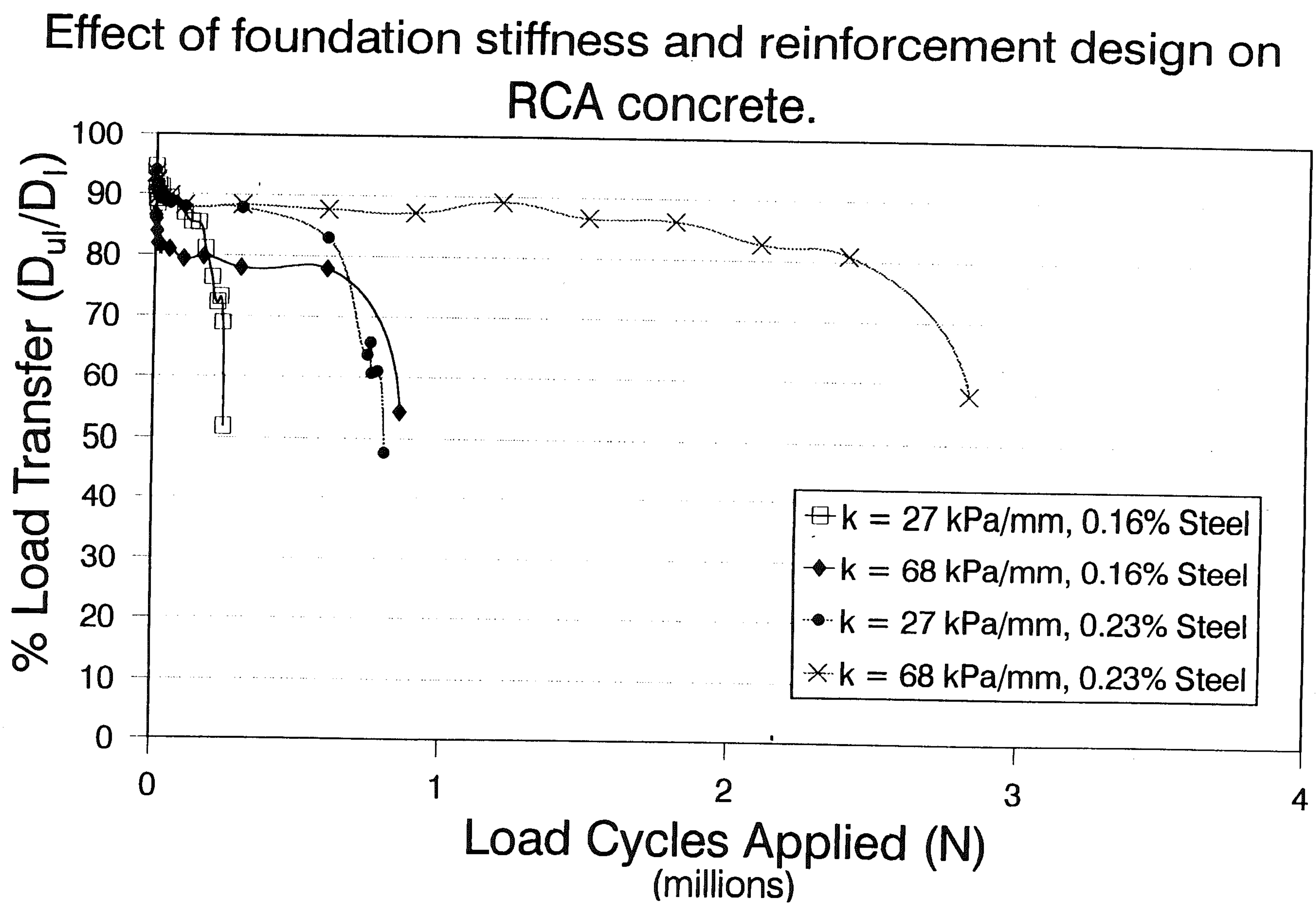


Figure 35. Effect of foundation stiffness and reinforcement design on RCA concrete.

with larger mesh reinforcing wires. Large wires were shown to provide good initial resistance to differential slab fragment deflections, but would often eventually fail in fatigue or would loosen due to bearing failure of the surrounding concrete. The deformed bars used in specimen 27 also provided excellent performance because they were large enough to hold the crack tight (limiting differential vertical movement) and did not fatigue appreciably or develop significant concrete bearing failures.

The second set of specimens used to evaluate the combined effects of foundation stiffness and reinforcement design are specimens 30, 28, 34 and 35, which were prepared using recycled gravel #3 concrete coarse aggregate (graded according to MDOT specification 6A) and a slab tension of 51 kN/m width [3500 lb/ft width]. Specimen 30 contain 0.16% smooth wire reinforcement and was tested on a foundation stiffness of 27 kPa/mm [100 psi/in]. Specimen 34 was prepared using 0.23% smooth wire mesh reinforcement and was tested on the same standard foundation. The third specimen (specimen 28) was prepared using 0.16% smooth wire mesh reinforcement (similar to specimen 30) and was tested on a stiffer foundation (68 kPa/mm [250 psi/in]). The fourth specimen (specimen 35) was prepared using more steel (0.23% smooth wire mesh reinforcement, similar to specimen 34) and was tested on the stiffer foundation. Table 13 presents a summary of the performance indicators for these specimens and figure 35 presents their load transfer histories.

The performance of the first three specimens has already been discussed in previous sections concerning the individual effects of foundation stiffness and reinforcement design. In this comparison, it is apparent that the effects of either the increase in foundation stiffness or the increase in steel quantities produced large and comparable improvements in crack performance. Consideration of the fourth specimen performance shows that when both design improvements were incorporated together, a further increase in performance obtained. Specimen 35 endured approximately 2,330,000 cycles before failure (more than 10 times as many as for the control specimen and 3 to 4 times as many as for the other two specimens), and had an endurance index of 15.8, even though it exhibited initial and peak deflections that were only slightly lower than those of the comparison specimens. The exceptionally good performance of specimen 35 was probably due in part to the slightly reduced deflections that it exhibited (and maintained, due to the stiff foundation), but more to the very tight crack that was present initially (and maintained, due to the use of increased steel quantities). It is expected that, with an initial crack width more comparable to that of the other three specimens, the improvement in endurance would not have been as great.

4.11 COMBINED EFFECTS OF FOUNDATION STIFFNESS AND SLAB TENSION

The combined effects of foundation stiffness and slab tension were evaluated by comparing the results of tests on specimens 10, 18, 13, 15 and 9, which are summarized in table 14. All five of these specimens were prepared using virgin gravel #3 coarse aggregate (graded to meet MDOT specification 6A) and 0.16% smooth wire mesh reinforcement. Specimen 10 was tested using a slab tension of 102 kN/m width [7000 lb/ft width] and a foundation stiffness of 27 kPa/mm [100 psi/in]. Specimen 18 was tested using the same high slab tension and a stiff foundation 68 kPa/mm [250 psi/in]. Specimens 13 and 15 were tested using the standard slab tension of 51 kN/m width [3500 lb/ft width] and the standard foundation stiffness of 27 kPa/mm [100 psi/in].

Table 14.. Performance summary for the combined effects of foundation stiffness and slab tension.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
10	6A Virgin Gravel #3, k = 27 kPa/mm, T = 102 kN/m	188,000	2.2	191	1334	N/A
18	6A Virgin Gravel #3, k = 68 kPa/mm, T = 102 kN/m	240,000	2.3	54	1156	559
13	6A Virgin Gravel #3, k = 27 kPa/mm, T = 51 kN/m	480,000	4.1	110	1393	127
15	6A Virgin Gravel #3, k = 27 kPa/mm, T = 51 kN/m	225,000	1.8	84	1414	1626
9	6A Virgin Gravel #3, k = 68 kPa/mm, T = 51 kN/m	6,100,000	59.2	42	1019	N/A

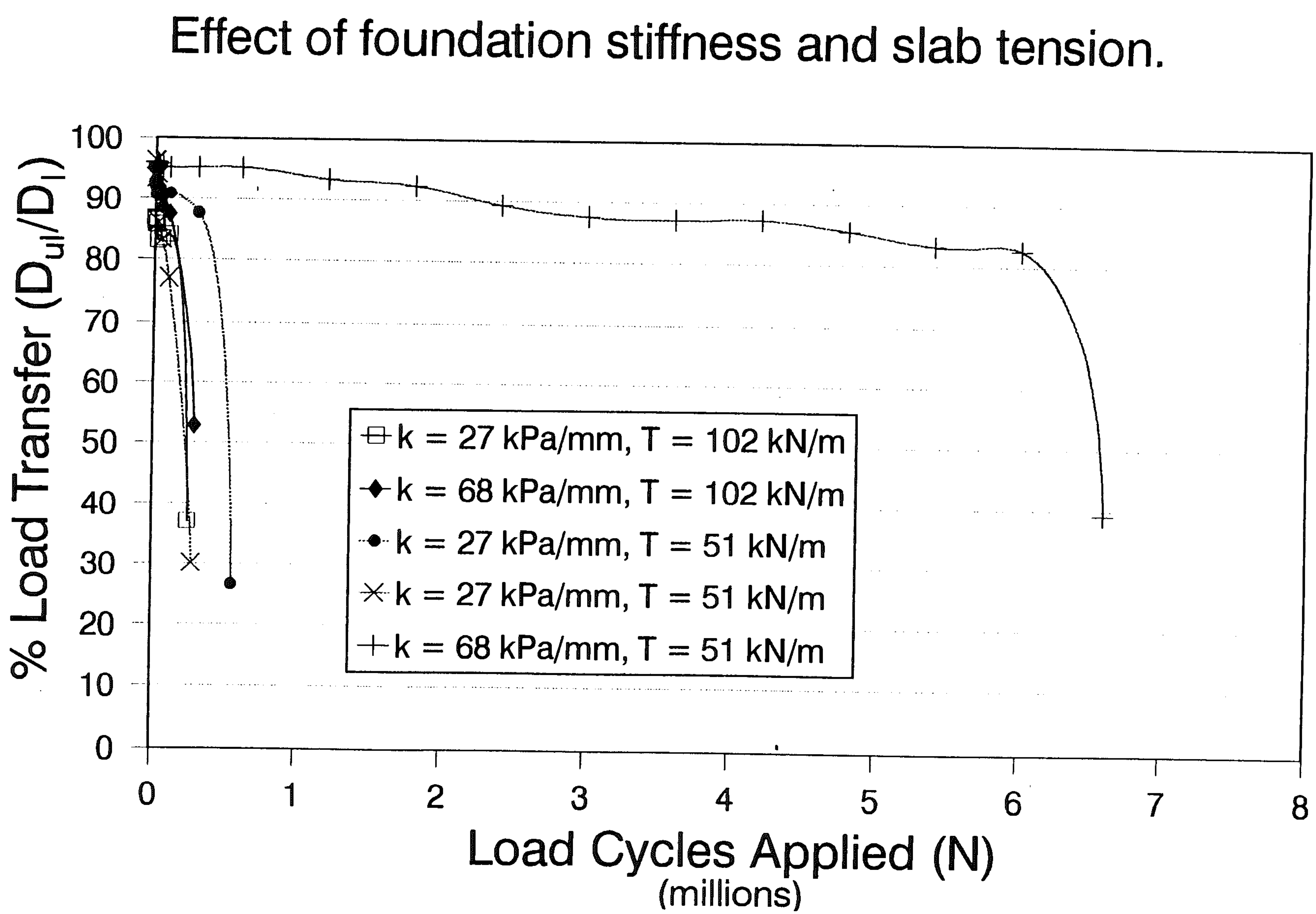


Figure 36. Effect of foundation stiffness and slab tension.

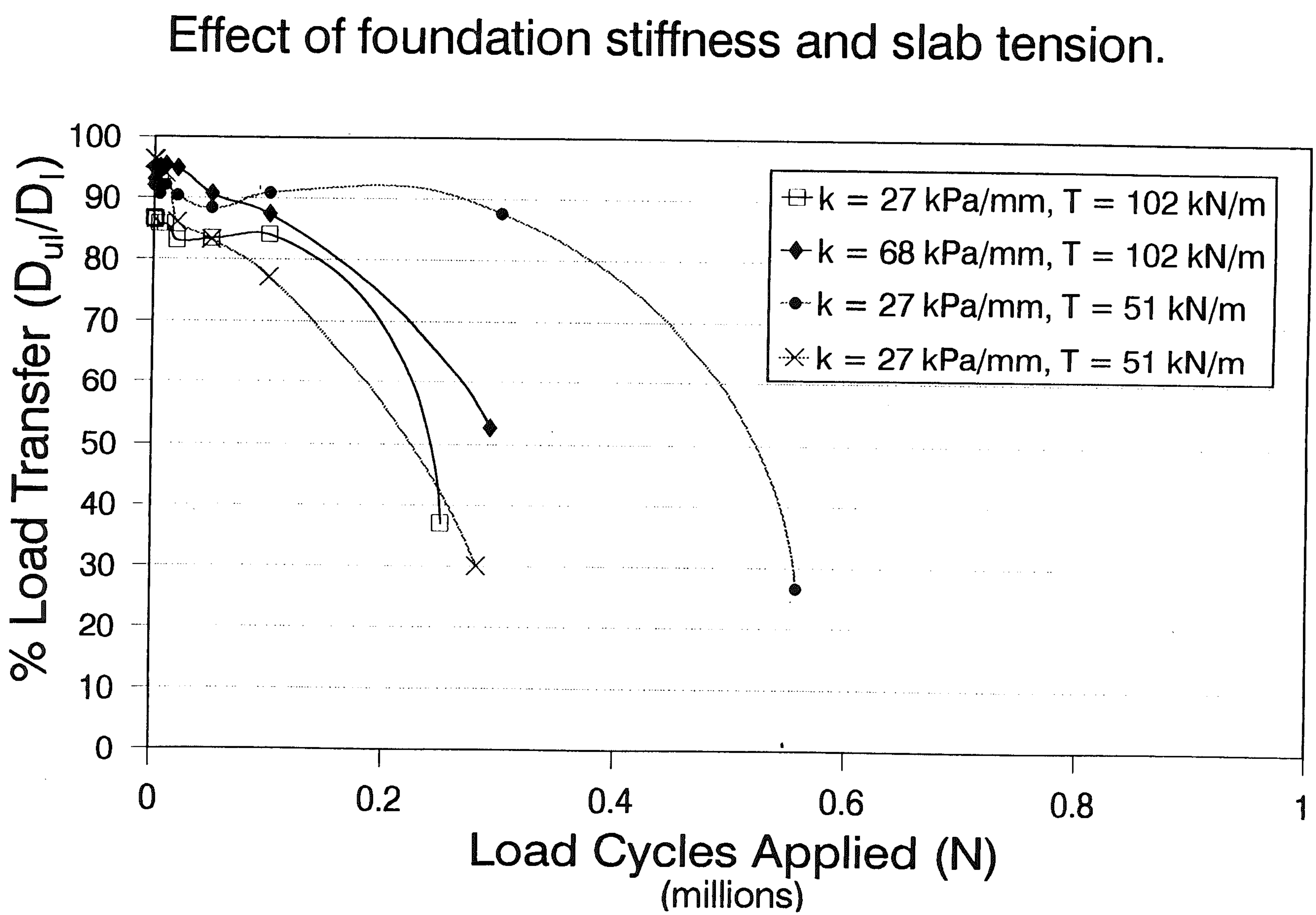


Figure 37. Effect of foundation stiffness and slab tension.

Specimen 9 was tested with the standard slab tension and the increased foundation stiffness. The load transfer histories of these specimens are plotted for comparison in figures 36 and 37.

The results show that specimen 9 (stiff foundation, standard tension) performed far better than the other test specimens. This specimen endured approximately 6,100,000 load cycles before failure, and the endurance index was 59. The differential deflection at 10,000 load repetitions was only 42 microns [1.64 mils], and peak deflections at the same time were also lower than those of the comparison specimens (1019 microns [40.1 mils]). Crack widths were not measured at that stage of the test program. The performance improvement of this specimen was attributed to the reduction in the differential slab fragment deflections which reduces the bending stress and fatigue of the reinforcing steel and decreases the potential for abrasion of the crack face, which reduces grain interlock.

As discussed in a previous section, the use of a stiffer foundation alone improved the performance of the high tension specimen compared to the performance of the high tension specimen tested on the relatively soft foundation. However, this performance improvement was an increase of only approximately 30% in the number of cycles endured to failure. This suggests that the combined effect of reducing slab tension and improving foundation support at the same time may be capable of producing improvements in crack performance that are greater than the sum of the individual effect improvements.

4.12 COMBINED EFFECT OF SLAB TENSION AND REINFORCEMENT TYPE

The combined effects of slab tension and reinforcing type were evaluated by comparing the results of tests on specimens 10, 13, 25 and 16, which are summarized in table 15. These specimens were all prepared using virgin gravel #3 (graded to meet MDOT specification 6A) and 0.16% longitudinal reinforcing steel. Specimen 10 included the use of smooth wire mesh and was tested at a high level of slab tension (102 kN/m width [7000 lb/ft width]). Slab 13 was identical to specimen 10 except that the standard level of slab tension was used (51 kN/m width [3500 lb/ft width]). Specimen 25 included deformed wire mesh and the high level of tension, while specimen 16 included deformed wire mesh and was tested at the standard level of tension. The load transfer histories for these specimens are presented in figure 38.

As discussed in previous sections the effects of increasing slab tension while holding other design and materials parameters constant generally reduced crack performance by 60 - 65%. This is presumed to be due to the increase in crack width (and accompanying decrease in grain interlock and increase in steel stress) that accompanies the increase in slab tension.

Similarly, the use of deformed wire mesh (rather than smooth mesh) generally improved performance by 25 - 38% when other design and materials parameters are held constant. This is attributed to the decrease in crack width that comes from the reduced length over which the steel strains at the crack, resulting in improved grain interlock and reduced steel stresses.

The results of the comparison suggest that the performance improvements that results from both reducing slab tension and using deformed wire may be roughly additive (within the range of

Table 15. Performance summary for the combined effects of slab tension and reinforcement type.

Slab	Brief Description	Cycles to Failure	Endurance Index	Differential Deflection @ 10,000 cycles (μm)	Peak Deflection @ 10,000 cycles (μm)	Crack width @ 10,000 cycles (μm)
10	6A Virgin Gravel #3, T = 102 kN/m, Smooth wire mesh	188,000	2.2	191	1334	N/A
13	6A Virgin Gravel #3, T = 51 kN/m, Smooth wire mesh	480,000	4.1	110	1393	127
25	6A Virgin Gravel #3, T = 102 kN/m, Deformed wire mesh	235,000	2.1	181	1685	787
16	6A Virgin Gravel #3, T = 51 kN/m, Deformed wire mesh	660,000	6.2	104	1448	51

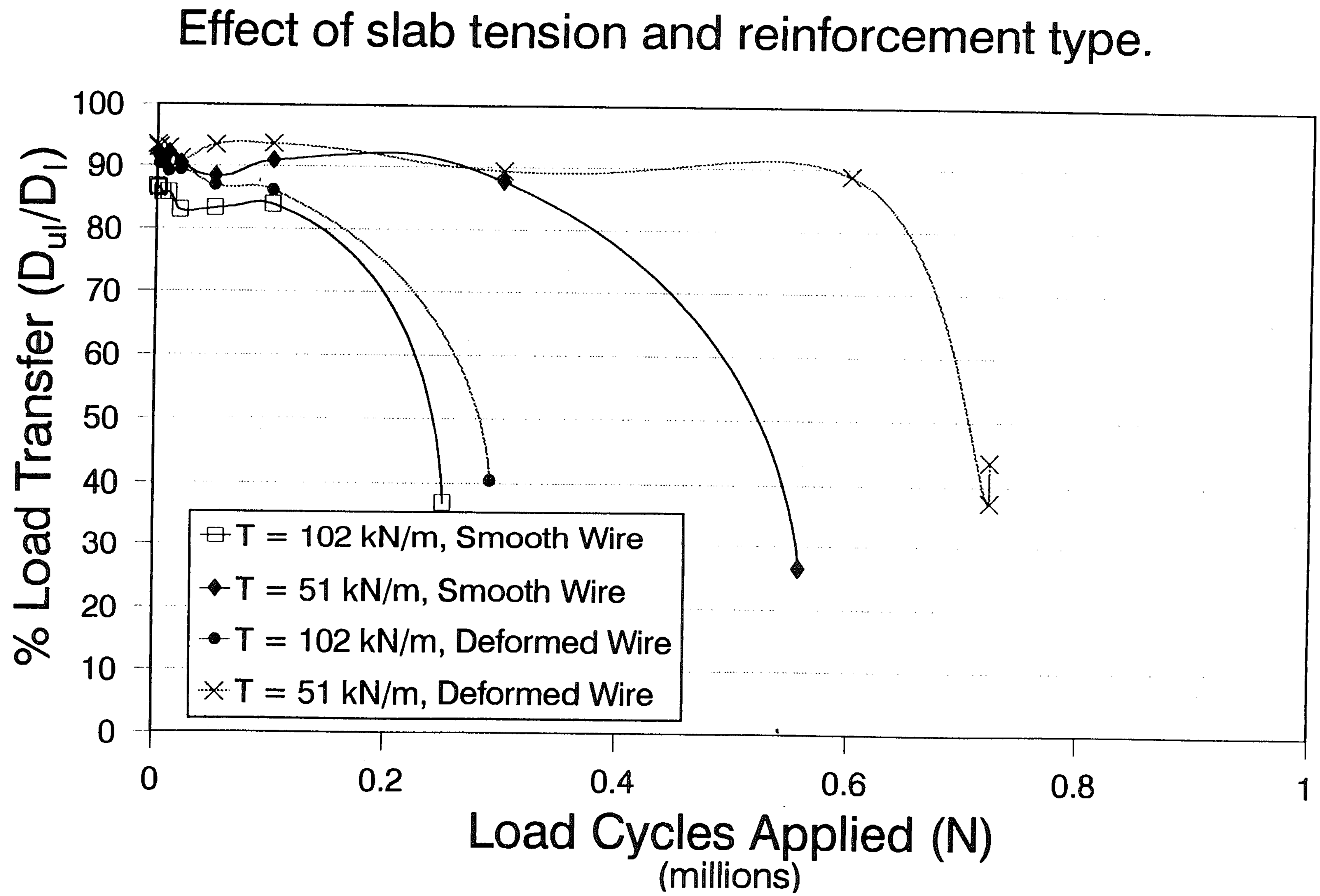


Figure 38. Effect of slab tension and reinforcement type.

variables tested in this program). This implies that the use of deformed wire and shorter joint spacings and/or reduced slab-foundation interface friction should provide reductions in transverse crack deterioration.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The following conclusions can be drawn from the results of the accelerated laboratory tests of large-scale concrete pavement slabs:

1. Concrete specimens prepared using the natural aggregate products (gravels and limestones) included in this study provided better crack deterioration performance under these test conditions than did concrete prepared using the manufactured aggregates (recycled concrete aggregates and slag aggregates) when all other factors were held constant. This can be attributed mainly to the reduction in crack face texture associated with the use of inherently weaker particles (e.g., slag and some recycled concrete products) and/or the reduced quantity of relatively harder natural aggregates at the crack face to provide grain interlock and improved abrasion resistance.

2. Concrete aggregate source was not conclusively determined to be responsible for differences in the performance of some test specimens. Significant differences were observed between the performance of concrete specimens prepared using gravels from two different sources, but insufficient data was available to determine whether other factors (i.e., crack width) were responsible.

3. The inclusion of larger coarse aggregate particles seemed to be effective in providing some improved grain interlock at transverse joints and cracks when all other factors are held constant and provided that the grading used does not adversely affect the strength of the concrete.

4. The performance of recycled concrete aggregate specimens seemed to be improved by blending them with quantities of virgin aggregate particles of equal or greater size. The additional benefit of blending with more than an equal weight of virgin particles was not clear. Furthermore, it appeared that even specimens prepared using virgin coarse aggregates could derive significant benefits from the inclusion of larger particles of natural aggregate.

5. Increases in foundation stiffness produced tremendous improvements in load transfer endurance for the natural and recycled concrete aggregate specimens that were tested. This increase in performance is attributed to the fact that increased foundation support reduces peak and differential vertical deflections (thereby reducing crack face abrasion and loss of grain interlock) and transfers more load to the foundation (thereby reducing the shear transferred through grain interlock and reducing the bending and shear stresses in the reinforcing steel).

6. Reductions in slab tension (brought about by the use of shorter panel lengths and/or reductions in slab-foundation interface friction) appear to be highly effective in reducing the rate of deterioration of transverse cracks in JRCP. This is accomplished through the reduction

in crack width and steel stresses that are associated with reduced tension, thereby allowing the grain interlock load transfer to remain high as crack face abrasion is reduced.

7. The use of additional longitudinal steel reinforcement produced performance improvements that ranged from 20% to more than 200%. This is a direct result of the ability of the increased quantity of steel to hold the cracks more tightly (due to reduced strain at any given level of tension) and due to the reductions in steel stress that are associated with having more steel present to resist relatively constant forces. These performance improvements were observed even when the initial crack widths were increased significantly, suggesting that smaller amounts of reinforcing may be failing due to combined shear, bending and tension in the reinforcing steel. If this is true, reinforcing design procedures that fail to address vehicle load-related stresses in the steel may be inadequate.

8. The use of deformed wire mesh reinforcing consistently produced improvements in crack performance by limiting crack width.

9. The "hinged joint" design provided excellent performance. Failure was never induced, even after several million repetitions of a critical loading. The success of this particular design can be attributed to both elimination of crack openings and increased quantities of steel (0.27% at the crack) and sizes of steel bars, which greatly reduce both environmental and load-related stresses in the steel.

10. Combining improvements in both foundation stiffness and reinforcement design provided exceptionally good performance when RCA was used in the pavement slab. However, the use of both design improvements provided little additional improvement in performance beyond that provided by the foundation stiffness alone in the case of the gravel test specimens. It is hypothesized that the mechanism of improved foundation stiffness (reduced deflections, crack face abrasion and steel stresses) benefits any of the slabs tested in this program, but that the mechanism of increased steel quantities (tighter cracks, lower steel stresses) benefits mainly those that have reduced crack face texture and reduced grain interlock initially (i.e., RCA and possibly slag).

11. Combining increases in foundation stiffness and reductions in slab tension appears to produce crack performance improvements that are significantly greater than those that result from either increases in foundation stiffness or reductions in slab tension alone. This was found to be true for specimens composed of natural gravel concrete, but is believed to be true for other types of concrete as well.

12. Combining decreases in slab tension with the use of deformed wire mesh appears to produce crack performance improvements that are significantly greater than those that result from either reductions in slab tension or the use of deformed wire reinforcing alone.

5.2 RECOMMENDATIONS

The results of this study suggest that improvements in JRCP performance (as measured by reductions in transverse crack deterioration) can be achieved by using combinations of materials and structural designs that provide tight cracks, good long-term aggregate interlock, minimize differential deflections across joints and cracks, and reduce total deflections at any location in the pavement structure. Specific recommendations that can be made based on the results of this study include:

- Pavements made with concrete derived from recycled concrete aggregate or slag should feature structural designs that minimize reliance on aggregate interlock in any area of the design (i.e., at joints or cracks). A conservative approach might be to avoid using recycled concrete or slag aggregate in JRCP pavements. Appropriate structural designs might include jointed plain concrete pavements with doweled joints and panel lengths less than 5 m (16.4 ft), or pavements that use the "hinged joint" design. In addition, at least one state has been successful at using recycled concrete in CRCP. The use of blended aggregates (recycled concrete or slag combined with suitable natural aggregates) may be useful to provide additional design reliability, but is probably not necessary for the types of designs described above. Drained bases are especially recommended for these types of pavements to reduce moisture-related damage and loss of support. However, stabilized bases must be used with care to avoid cracking due to combined curl/warp and traffic stresses; a sand or granular "cushion" interlayer may be necessary.

- The performance of JRCP would generally be improved by:
    - increasing steel quantities
    - using deformed wire mesh
    - increasing foundation support
    - decreasing slab length
    - reducing foundation interface friction
    - using well-graded, durable coarse aggregate with a larger top size than is currently provided in the MDOT 6A gradation.

  Improvements in foundation support and reductions in slab length would seem to have the potential of providing the most cost-effective and universal improvements in JRCP performance.

- Transverse crack deterioration appeared to be strongly correlated with concrete strength (presumably due to reductions in pavement stiffness and abrasion resistance that probably accompany the use of weak concrete). Thus, pavements made with concrete that includes relatively weak aggregate particles, such as slag and recycled concrete, should: a) use mix designs that provide concrete strengths that are comparable to those of concrete made with virgin aggregates; b) use structural designs that reduce pavement stresses to levels that are appropriate for the strength that will be obtained; or c) do both. Inspection of batching, placement and curing must ensure that all mixes develop the required strengths.

- It is the opinion of the authors that any of the aggregates included in this study could be used in Michigan concrete paving operations if appropriate concrete mixture proportioning and structural design modifications are made. It must be remembered that the use of different concrete aggregates results in the production of concrete with widely varying physical and mechanical properties; the use of a "standard" structural or concrete mixture design for all materials can not be expected to produce pavements with comparable performance.

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APPENDIX A

MATERIAL PROPERTIES AND DESIGN PARAMETERS

List of Conversions for Appendix A

1 in = 25.4 cm
1 lb = 0.453 kg
1 lb/in^2 (psi) = 6.89 x 10^3 Pa
1 lb/ft = 1.46 x 10^{-2} kN/m
1 psi/in = 0.27 kPa/mm

Table A-1. Sizes and strengths of reinforcing steel.

Specimen	Statistical Property	Diameter, in. (mm)	Ultimate Strength, psi (MPa)	Yield Strength, psi (MPa)
Standard Wire	Mean Value	0.330 (8.38)	87,411 (602.7)	68,603 (473.0)
	Standard Deviation	0.001 (0.02)	1584.6 (10.93)	--
	Number of Samples (n)	6	6	1
Deformed Wire	Mean Value	0.342 (8.69)	79,811 (550.3)	62,657 (432.0)
	Standard Deviation	0.002 (0.05)	4042.4 (27.87)	--
	Number of Samples (n)	6	6	1
Large Wire Shipment 1	Mean Value	0.395 (10.03)	90,930 (626.9)	71,650 (494.0)
	Standard Deviation	0.002 (0.04)	4283.1 (29.53)	--
	Number of Samples (n)	6	6	1
Large Wire Shipment 2	Mean Value	0.390 (9.90)	95,835 (660.8)	63,672 (439.0)
	Standard Deviation	0.001 (0.02)	977.4 (6.74)	--
	Number of Samples (n)	6	6	1

Table A-2. Aggregate sources.

Aggregate	Source
Gravel #1	Koenig Sand & Gravel, Pit No. 63-9
Gravel #3	Mickelson #3, Pit 63-88
Limestone #1	Inland Lime and Stone
Limestone #2	Holloway Sand & Gravel, Rockwood Stone Level 3, Pit No. 58-8
Slag	E. C. Levy - Dix Yard

Table A-3. Physical characteristics of aggregates.

Aggregate	Specific Gravity	Absorption Capacity	Comments
6A Gravel #1	2.61	0.90	
6A Limestone #1	2.60	0.66	
6A Gravel #3 Shipment 1	2.67	1.05	
6A Gravel #3 Shipment 2	2.62	0.41	Same source as shipment 1 but had a finer grading requiring sieving and reblending.
6A Limestone #2	2.62	2.20	Same source as 6A Limestone #1 but had finer grading.
4A Limestone #1	2.67	0.50	Same source as 6A Limestone #1.
6A Slag	2.39	2.34	
6A Recycled Gravel #1	2.28	4.60	6A Gravel #1 was recycled by Holloway Construction, sieved and reblended to specification.
6A Recycled Gravel #2	2.46	4.83	6A Gravel #3 was recycled by Holloway Construction, sieved and reblended to specification.
6A Recycled Gravel #3	2.39	4.54	6A Gravel #3 was recycled by Iafrate Construction, sieved and reblended to specification.
6A Recycled Limestone	2.34	4.46	6A Limestone #2 was recycled by Iafrate Construction, sieved and reblended to specification.

Table A-4. Coarse aggregate gradation for 6A material.

		Sieve Size			
		Total Percent Passing			
Slab No.	Aggregate / Slab Description	1- 1/2 in. (37.5 mm)	1 in. (25.0 mm)	1/2 in. (12.5 mm)	#4 (4.75 mm)
	6A Grading Specification	100	95-100	30-60	0-8
	6A Virgin Gravel #1 (Slab 1, 7, and 8)	100.00	98.00	38.00	4.00
	6A Virgin Gravel #3, Shipment 1 (Slab 9, 10, 12, 13, 14, 15 16, 17, 18, 20, 21, 25, 27)	100.00	98.97	49.56	0.89
	6A Virgin Gravel #3, Shipment 2	100.00	100.00	68.00	28.00
	6A Virgin Limestone #1 (Slab 2, 22)	100.00	98.00	38.00	2.00
	6A Virgin Limestone #2 (Slab 11)	100.00	98.30	44.27	1.53
	6A Recycled Gravel #1 (Slab 5, 6)	100.00	97.00	42.00	4.00
	6A Recycled Gravel #3 (Slab 19, 22, 23)	100.00	99.69	54.47	1.70
3	6A Virgin Slag	100.00	100.00	60.00	2.00
24	6A Recycled Limestone #1	100.00	97.38	43.60	1.12
26	6A Slag, Stiff Foundation (k=250 psi/in)	100.00	99.82	52.56	6.77
28	6A Recycled Gravel #3, Stiff Foundation (k=250 psi/in)	100.00	99.83	54.47	1.62
29	6A Recycled Gravel #3, Reduced Tension (T=2305 lb/ft width)	100.00	99.33	42.43	1.23
30	6A Recycled Gravel #3	100.00	98.00	51.40	4.65
31	6A Virgin Gravel #3, Selective Grading	100.00	97.60	44.20	0.70
33	6A Slag, Large Wire (steel = 0.23%)	100.00	99.51	49.73	6.26
34	6A Recycled Gravel #3, Large Wire (steel=0.23%)	100.00	96.70	40.00	3.75
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (steel=0.23%, k=250 psi/in)	100.00	97.86	42.70	1.56

Table A-5. Coarse aggregate gradation for 17A material.

		Sieve Size			
		Total Percent Passing			
Slab No.	Aggregate / Slab Description	1 - 1/2 in. (37.5 mm)	1 in. (25.0 mm)	1/2 in. (12.5 mm)	#4 (4.75 mm)
	17A Grading Specification	100	90-100	50-75	0-8
4	17A Virgin Gravel #1	100.00	100.00	56.00	6.00

Table A-6. Coarse aggregate gradation for 4A material.

		Sieve Size						
		Total Percent Passing						
Slab No.	Aggregate / Slab Description	2 - 1/2 in. (63.0 mm)	2 in. (50.0 mm)	1 - 1/2 in. (37.5 mm)	1 in. (25.0 mm)	1/2 in. (12.5 mm)	3/8 in. (9.5 mm)	#4 (4.75 mm)
	4A Grading Specification	100	95-100	65-90	10-40	0-20	0-5	--
	4A Limestone #1 (Slab 6, 19, 21, 23)	82.00	47.00	9.00	2.00	0.00	0.00	--
32	4A Virgin Limestone #1	--	94.96	58.70	9.09	0.59	--	0.59

Table A-7. Summary of concrete batch proportions for test specimens.

		Mix Design				
Slab No.	Slab Description	CA	FA	Cement	Water	Air Cont.
1	6A Virgin Gravel #1	1966	1079	554	235	6.4
2	6A Virgin Limestone #1	1817	1240	560	245	5.4
3	6A Virgin Slag	1808	1297	744	305	6.7
4	17A Virgin Gravel #1	1878	1163	548	283	6.0
5	6A Recycled Gravel #1	1559	1209	523	263	6.7
6	50-50% 6A Recycled Gravel #1/4A Virgin Limestone	1682	1137	545	272	6.7
7	6A Virgin Gravel #1 (Replicate)	1814	1264	570	221	6.5
8	6A Virgin Gravel #1 (Replicate)	1832	1238	549	217	7.4
9	Stiff Foundation (k=250 psi/in)	1791	1250	563	234	6.8
10	High Tension (T=7000 lb/ft width)	1800	1238	563	234	6.8
11	6A Virgin Limestone #2	2044	1132	509	198	6.4
12	Large Wire, Stiff Foundation (k=250 psi/in, steel=0.23%)	2000	1098	566	263	6.5
13	6A Virgin Gravel #3	1972	1145	593	235	5.0
14	Large Wire (steel=0.23%)	2020	1123	571	216	6.0
15	6A Virgin Gravel #3 (Replicate)	2094	1211	436	184	6.8
16	Deformed Wire	1992	1056	598	239	6.3
17	Unreinforced	2003	1201	550	216	5.0
18	High Tension, Stiff Foundation (T=7000 lb/ft width, k=250 psi/in)	1963	1223	528	205	6.5
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone	1967	1295	416	156	6.3
20	6A Virgin Gravel #3 (with strain gages)	2174	1232	384	177	6.0
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone	2004	1182	548	224	5.0
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone	1860	1143	531	219	6.8

Table A-7 (continued). Summary of concrete batch proportions for test specimens.

Slab No.	Slab Description	Mix Design				
		CA	FA	Cement	Water	Air Cont.
23	40-60% 6A Recycled Gravel #3 /4A Virgin Limestone	1922	1099	581	229	5.4
24	6A Recycled Limestone	1786	1064	590	221	6.2
25	Deformed Wire, High Tension (T=7000 lb/ft width)	1989	1110	539	234	5.5
26	6A Slag, Stiff Foundation (k=250 psi/in)	1758	1155	539	247	5.3
27	"Hinged" Joint	1976	1104	534	233	6.1
28	6A Recycled Gravel #3, Stiff Foundation (k=250 psi/in)	1747	1256	549	189	5.7
29	6A Recycled Gravel #3, Reduced Tension (T=2305 lb/ft width)	1776	1259	553	180	5.3
30	6A Recycled Gravel #3	1788	1249	518	212	6.2
31	6A Gravel #3, Selective Grading	1975	1076	532	248	5.4
32	4A Limestone #2	1922	1155	547	230	5.2
33	6A Slag, Large Wire (steel = 0.23%)	1708	1167	493	233	6.4
34	6A Recycled Gravel #3, Large Wire (steel = 0.23%)	1687	1133	533	222	6.8
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (steel=0.23%, k=250 psi/in)	1700	1132	537	224	6.3

Table A-8. Summary of concrete strength data for test specimens.

Slab No.	Slab Description	Average Strength					
		Compressive (psi)			Flexural (psi)		
		3 day	7 day	28 day	18 hour	7 day	28 day
1	6A Virgin Gravel #1	N/A	N/A	5681	N/A	N/A	N/A
2	6A Virgin Limestone #1	N/A	N/A	5295	N/A	N/A	N/A
3	6A Virgin Slag	N/A	N/A	5954	N/A	N/A	N/A
4	17A Virgin Gravel #1	N/A	N/A	4294	N/A	N/A	N/A
5	6A Recycled Gravel #1	N/A	N/A	4780	N/A	N/A	N/A
6	50-50% 6A Recycled Gravel #1/4A Virgin Limestone	N/A	N/A	5352	N/A	N/A	N/A
7	6A Virgin Gravel #1 (Replicate)	N/A	N/A	4125	N/A	N/A	N/A
8	6A Virgin Gravel #1 (Replicate)	N/A	N/A	3645	N/A	N/A	N/A
9	Stiff Foundation (k=250 psi/in)	N/A	N/A	5837	N/A	N/A	N/A
10	High Tension (T=7000 lb/ft width)	N/A	N/A	4178	N/A	N/A	N/A
11	6A Virgin Limestone #2	4637	5464	6518	N/A	422	873
12	Large Wire, Stiff Foundation (k=250 psi/in, steel=0.23%)	N/A	3876	4723	N/A	N/A	N/A
13	6A Virgin Gravel #3	3847	5215	5466	364	760	796
14	Large Wire (steel=0.23%)	3262	3950	4212	312	678	706
15	6A Virgin Gravel #3 (Replicate)	N/A	4535	4800	362	684	738
16	Deformed Wire	3306	4145	5028	256	643	712
17	Unreinforced	3375	4128	5026	293	564	799
18	High Tension, Stiff Foundation (T=7000 lb/ft width, k=250 psi/in)	3776	N/A	4181	328	578	619
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone	4202	4304	4475	330	651	868
20	6A Virgin Gravel #3 (with strain gages)	2263	2882	3548	302	525	654
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone	3690	3330	N/A	346	707	683

Table A-8 (continued). Summary of concrete strength data for test specimens.

Slab No.	Slab Description	Average Strength					
		Compressive (psi)			Flexural (psi)		
		3 day	7 day	28 day	18 hour	7 day	28 day
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone	3439	3900	4176	530	717	670
23	40-60% 6A Recycled Gravel #3 /4A Virgin Limestone	3816	4987	4677	480	818	738
24	6A Recycled Limestone	2507	2953	3950	257	526	608
25	Deformed Wire, High Tension (T=7000 lb/ft width)	3004	3516	4265	241	483	607
26	6A Slag, Stiff Foundation (k=250 psi/in)	2808	3163	4568	217	518	625
27	"Hinged" Joint	2798	3282	2773	388	246	596
28	6A Recycled Gravel #3, Stiff Foundation (k=250 psi/in)	2114	2621	3027	164	570	625
29	6A Recycled Gravel #3, Reduced Tension (T=2305 lb/ft width)	2258	3652	4124	269	845	960
30	6A Recycled Gravel #3	1858	2347	2868	247	562	653
31	6A Gravel #3, Selective Grading	2846	3582	4072	300	516	713
32	4A Limestone #1	2068	2638	2972	219	414	480
33	6A Slag, Large Wire (steel = 0.23%)	2072	2575	3247	189	387	489
34	6A Recycled Gravel #3, Large Wire (steel=0.23%)	3039	4207	5015	368	660	744
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (steel=0.23%, k=250 psi/in)	3514	3775	4842	391	665	712

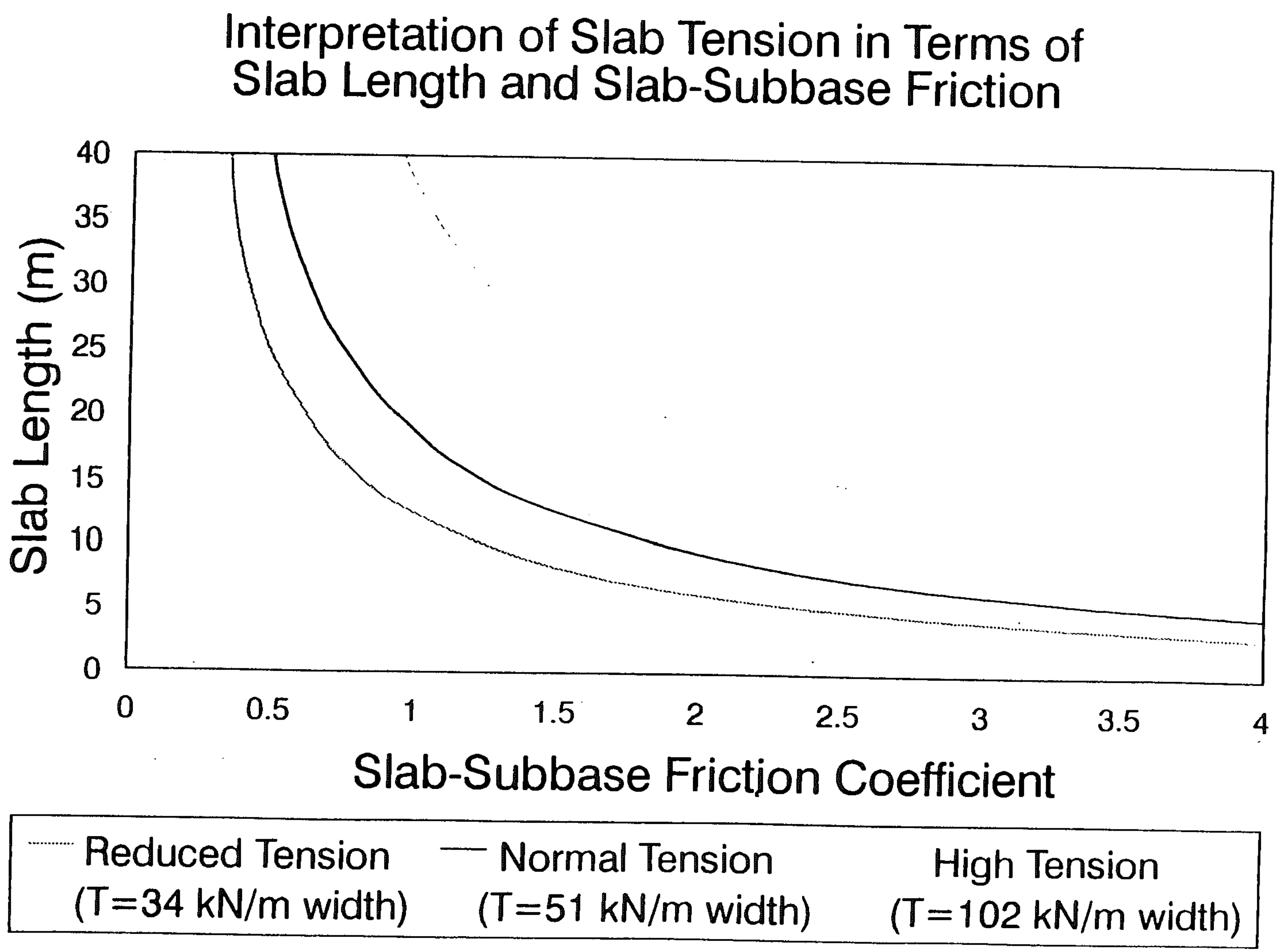

Figure A-1. Interpretation of slab tension in terms of slab length and slab-subbase friction.

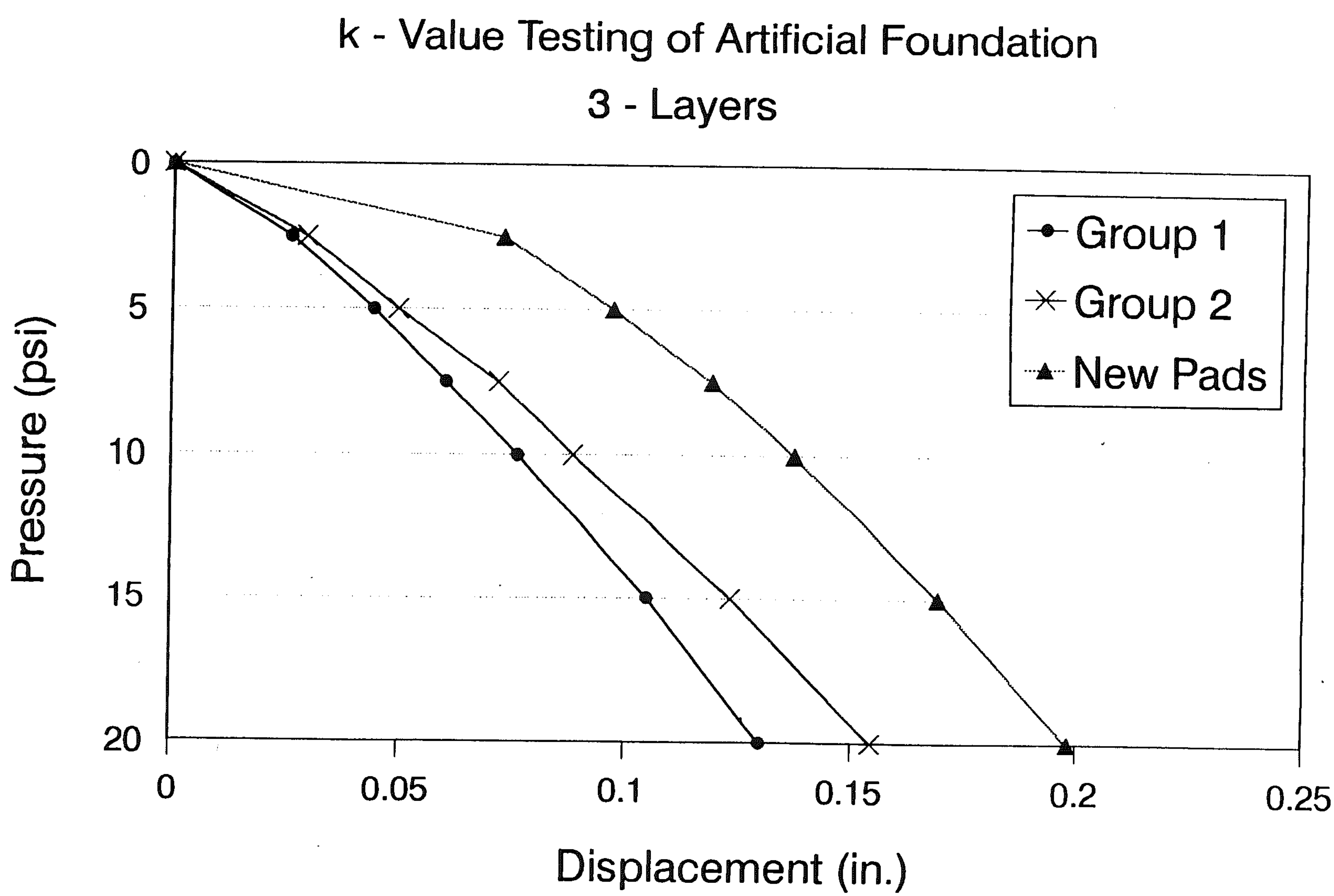

Figure A-2. Static plate load test for 3 layers of artificial foundation material.

k - Value Testing of Artificial Foundation

2 - Layers

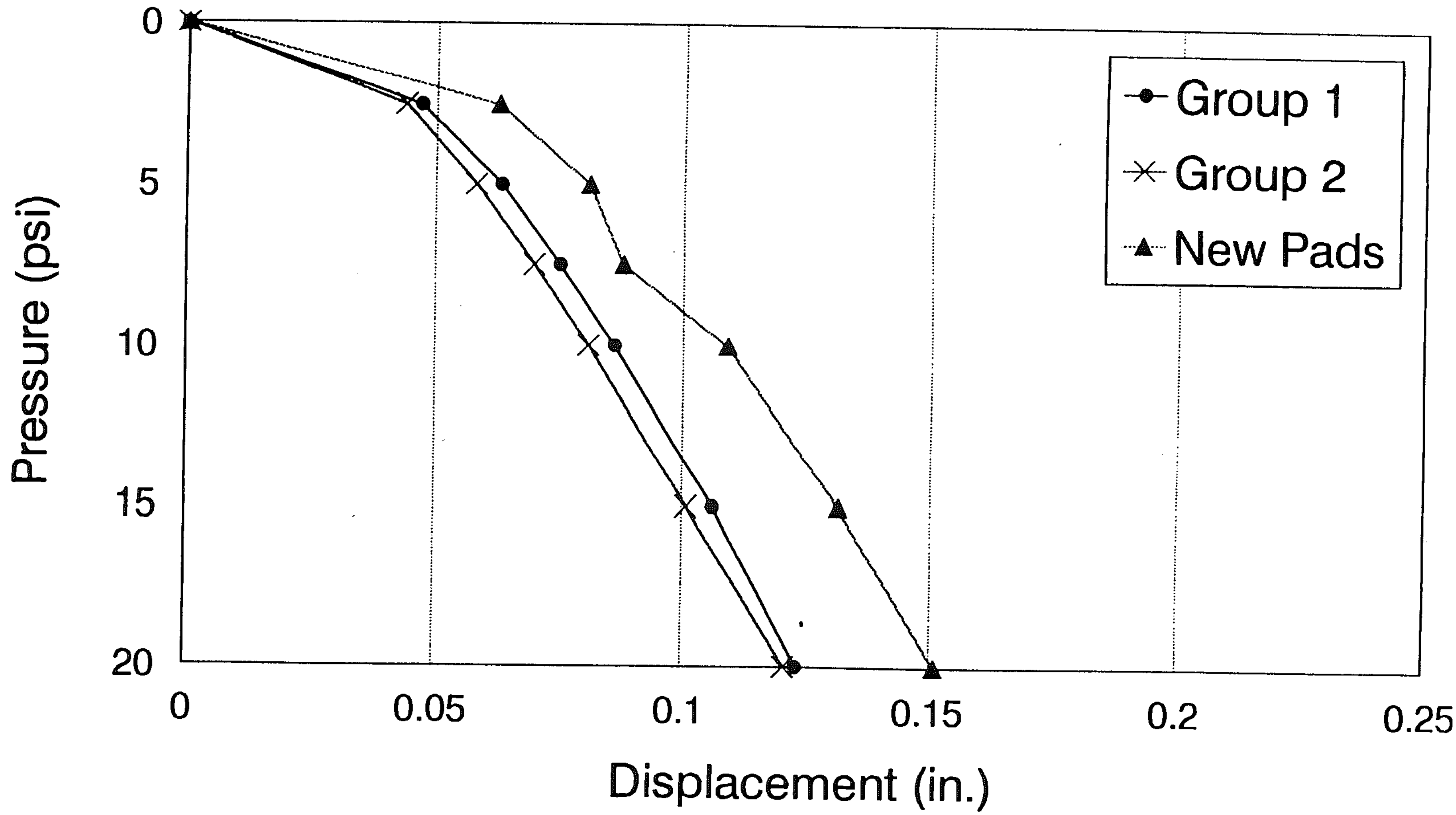

Figure A-3. Static plate load test for 2 layers of artificial foundation material.

APPENDIX B

TABULATED PERFORMANCE DATA

List of Conversions for Appendix

1 lb/ft = 1.46×10^{-2} kN/m
1 psi/in = 0.27 kPa/mm
1 in = 25.4 mm
1 mil = 25.4 μm

Table B-1. Test specimens from year 1.

Slab No.	Test Slab Description	Adjusted Cycles*	Actual Cycles	Cycles to Failure	LT @ Failure
1	6A Virgin Gravel #1	3,000,000	900,000	2,516,000	70
2	6A Virgin Limestone #1	5,200,000	1,500,000	4,630,000	67
3	6A Virgin Slag	750,000	250,000	540,000	65
4	17A Virgin Gravel #1	2,700,000	825,000	1,916,000	65
5	6A Recycled Gravel #1	900,000	300,000	710,000	67
6	50-50% 6A Recycled Gravel #1 / 4A Virgin Limestone #1	1,100,000	350,000	1,000,000	64
7	6A Virgin Gravel #1 (Replicate)	--	2,700,000	2,500,000	87
8	6A Virgin Gravel #1 (Replicate)	--	3,300,000	2,930,000	85
9	6A Virgin Gravel #3, Stiff Foundation (k = 250 psi/in)	--	6,600,000	6,100,000	82
10	6A Virgin Gravel #3, High Tension (T = 7000 lb/ft width)	--	250,000	188,000	70

NOTE: * Modification factors were used on year 1 data because the test frame was modified between year 1 and year 2 testing.

Table B-2. Test specimens from year 2.

Slab No.	Test Slab Description	Actual Cycles	Cycles to Failure	LT @ Failure
11	6A Virgin Limestone #2	1,599,000	**	N/A
12	6A Virgin Gravel #3, Large Wire, Stiff Foundation (0.23% Steel, k = 250 psi/in)	5,102,000	4,000,000	70
13	6A Virgin Gravel #3	558,000	480,000	74
14	6A Virgin Gravel #3, Large Wire (0.23% Steel)	1,622,000	1,430,000	78
15	6A Virgin Gravel #3 (Replicate)	275,000	225,000	66
16	6A Virgin Gravel #3, Deformed Wire	722,000	660,000	87
17	6A Virgin Gravel #3, Unreinforced	1,900,000	N/A***	N/A
18	6A Virgin Gravel #3, High Tension, Stiff Foundation (T = 7000 lb/ft width, k = 250 psi/in)	293,000	240,000	75
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	900,000	710,000	89
20	6A Virgin Gravel #3 (with strain gages)†	2,441,000	2,080,000	88
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Limestone #1	6,776,000+	**	N/A
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone #1	533,000	480,000	73
23	40-60% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	565,000	520,000	81
24	6A Recycled Limestone #1	125,000	95,000	71
25	6A Virgin Gravel #3, Deformed Wire, High Tension (T = 7000 lb/ft width)	291,000	235,000	74

NOTE: ** Slab not tested to failure.
*** Slab under compression (not tension).
† Unusual crack pattern, load cycle count is probably not representative.

Table B-3. Test specimens from year 3.

Slab No.	Test Slab Description	Actual Cycles	Cycles to Failure	LT @ Failure
26	6A Virgin Slag, Stiff Foundation (k = 250 psi/in)	290,000	260,000	66
27	6A Virgin Gravel #3, Deformed Bar ("Hinged" Joint)	7,800,000+	**	N/A
28	6A Recycled Gravel #3, Stiff Foundation (k = 250 psi/in)	859,000	770,000	71
29	6A Recycled Gravel #3, Reduced Tension (T = 2305 lb/ft width)	965,000	830,000	80
30	6A Recycled Gravel #3	246,000	210,000	77
31	6A Virgin Gravel #3, Selective Grading	200,000	145,000	78
32	4A Virgin Limestone #1	350,000	310,000	65
33	6A Virgin Slag, Large Wire (0.23% Steel)	470,000	445,000	67
34	6A Recycled Gravel #3, Large Wire (0.23% Steel)	805,000	670,000	78
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (0.23% Steel, k = 250 psi/in)	2,850,000	2,330,000	78

NOTE: ** Slab not tested to failure.

Table B-4. Comparison of endurance index calculations.

Slab No.	Slab Description	Endurance Index (Log)	Endurance Index (1000000)	Endurance Index (10000000)
1	6A Virgin Gravel #1	71	85	22.3
2	6A Virgin Limestone #1	74	88	35.9
3	6A Virgin Slag	55	40	4.0
4	17A Virgin Gravel #1	69	77	17.5
5	6A Recycled Gravel #1	61	56	5.6
6	50-50% 6A Recycled Gravel #1 / 4A Virgin Limestone #1	63	71	7.6
7	6A Virgin Gravel #1 (Replicate)	72	91	24.1
8	6A Virgin Gravel #1 (Replicate)	75	91	30.1
9	6A Virgin Gravel #3, Stiff Foundation (k = 250 psi/in)	80	96	59.2
10	6A Virgin Gravel #3, High Tension (T = 7000 lb/ft width)	56	22	2.2
11	6A Virgin Limestone #2	N/A (74)*	108	N/A (16.6)
12	6A Virgin Gravel #3, Large Wire, Stiff Foundation (0.23% Steel, k = 250 psi/in)	77	87	39.2
13	6A Virgin Gravel #3	64	41	4.1
14	6A Virgin Gravel #3, Large Wire (0.23% Steel)	69	88	13.3
15	6A Virgin Gravel #3 (Replicate)	61	18	1.8

Table B-4 (continued). Comparison of endurance index calculations.

Slab No.	Slab Description	Endurance Index (Log)	Endurance Index (1000000)	Endurance Index (10000000)
16	6A Virgin Gravel #3, Deformed Wire	67	62	6.2
17	6A Virgin Gravel #3, Unreinforced	N/A (67)**	N/A (69)	N/A (7.9)
18	6A Virgin Gravel #3, High Tension, Stiff Foundation (T = 7000 lb/ft width, k = 250 psi/in)	63	23	2.3
19	50-50% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	72	78	7.8
20	6A Virgin Gravel #3 (with strain gages)	71***	92	25.0
21	50-50% 6A Virgin Gravel #3 / 4A Virgin Gravel #1	N/A (76)*	97	N/A (62.2)
22	50-50% 6A Recycled Gravel #3 / 6A Virgin Limestone #1	66	43	4.3
23	40-60% 6A Recycled Gravel #3 / 4A Virgin Limestone #1	69	51	5.1
24	6A Recycled Limestone #1	54	9	1.0
25	6A Virgin Gravel #3, Deformed Wire, High Tension (T = 7000 lb/ft width)	60	21	2.1
26	6A Virgin Slag, Stiff Foundation (k = 250 psi/in)	58	21	2.0
27	6A Virgin Gravel #3, Deformed Bar ("Hinged" Joint)	N/A (82)*	93	N/A (71.4)
28	6A Recycled Gravel #3, Stiff Foundation (k = 250 psi/in)	63	65	6.4
29	6A Recycled Gravel #3, Reduced Tension (T = 2305 lb/ft width)	69	84	8.1

Table B-4 (continued). Comparison of endurance index calculations.

Slab No.	Slab Description	Endurance Index (Log)	Endurance Index (1000000)	Endurance Index (10000000)
30	6A Recycled Gravel #3	61	21	2.0
31	6A Virgin Gravel #3, Selective Grading	60	16	1.6
32	4A Virgin Limestone #1	61	26	2.6
33	6A Virgin Slag, Large Wire (0.23% Steel)	63	36	3.6
34	6A Recycled Gravel #3, Large Wire (0.23% Steel)	67	66	7.0
35	6A Recycled Gravel #3, Large Wire, Stiff Foundation (0.23% Steel, k = 250 psi/in)	73	88	15.8

NOTE: * Slab not tested to failure.
** Specimen was under compression (not tension).
*** Unusual crack pattern, load cycle count is probably not representative.

Table B-5. Percent load transfer efficiency measured on the approach side.

Load Transfer Efficiency (%)

Slab No. / Cycles	1	2	3	4	5	6	7	8	9
1	92.7	93.9	79.2	91.9	87.6	88.0	92.6	96.3	97.4
1000	90.3	92.2	81.5	90.4	86.7	87.1	92.8	94.8	96.3
2000	89.5	91.8	79.0	88.4	85.2	--	92.4	92.3	97.4
5000	88.8	88.0	77.7	88.0	85.0	87.4	92.3	91.5	98.5
10000	--	87.5	74.6	86.4	84.4	87.2	92.2	91.9	98.5
20000	86.7	86.9	75.4	86.2	82.4	86.4	92.4	91.3	99.5
50000	85.9	86.9	71.8	81.4	81.5	76.0	93.9	92.1	98.8
100000	85.6	85.1	72.2	78.7	78.7	72.2	92.3	91.3	96.4
250000	--	--	0.9	--	--	--	--	--	--
300000	83.0	85.8	--	72.1	23.7	64.1	92.3	91.3	96.5
350000	--	--	--	--	--	25.0	--	--	--
600000	79.0	82.6	--	67.6	--	--	89.9	89.6	96.5
825000	--	--	--	9.1	--	--	--	--	--
900000	34.5	81.1	--	--	--	--	--	--	--
1200000	--	--	--	--	--	--	91.3	90.8	94.1
1500000	--	13.9	--	--	--	--	--	--	--
1800000	--	--	--	--	--	--	88.9	89.8	94.7
2400000	--	--	--	--	--	--	86.2	95.4	89.9
2700000	--	--	--	--	--	--	80.5	--	--
3000000	--	--	--	--	--	--	--	97.7	88.9
3300000	--	--	--	--	--	--	--	67.8	--
3600000	--	--	--	--	--	--	--	--	87.9
4200000	--	--	--	--	--	--	--	--	89.4
4800000	--	--	--	--	--	--	--	--	87.2
5400000	--	--	--	--	--	--	--	--	84.2
6000000	--	--	--	--	--	--	--	--	84.2
6600000	--	--	--	--	--	--	--	--	82.7

Table B-5 (Continued). Percent load transfer efficiency measured on the approach side.

Load Transfer Efficiency (%)

Slab No.	10	11	12	13	14	15	16	17	18
Cycles									
1	86.7	96.9	96.7	--	101.9	96.3	93.4	92.2	93.0
1000	86.3	98.8	96.1	90.9	104.5	96.2	92.9	--	94.5
2000	86.5	99.3	94.9	90.9	104.6	93.4	93.0	90.8	94.8
5000	85.7	98.4	94.1	89.8	97.1	93.4	91.3	89.7	95.0
10000	85.7	98.1	92.3	91.1	98.1	94.0	92.8	88.6	95.3
20000	83.0	98.2	93.0	89.7	96.2	86.0	91.1	86.4	94.8
50000	83.3	97.5	93.8	85.5	101.8	83.2	93.4	89.4	90.7
100000	84.0	91.9	92.7	91.1	86.5	77.1	93.6	89.3	87.4
250000	37.0	--	--	--	--	--	--	--	--
281000	--	--	--	--	--	30.1	--	--	--
293000	--	--	--	--	--	--	--	--	52.8
300000	--	86.9	84.9	87.5	90.7	--	89.4	71.7	--
558000	--	--	--	20.5	--	--	--	--	--
600000	--	--	85.6	--	89.4	--	88.8	64.0	--
723000	--	--	--	--	--	--	43.7	--	--
900000	--	78.1	86.7	--	82.5	--	--	52.8	--
1200000	--	90.2	--	--	82.7	--	--	49.9	--
1500000	--	--	85.1	--	--	--	--	43.4	--
1599000	--	81.5	--	--	--	--	--	--	--
1622000	--	--	--	--	17.1	--	--	--	--
1800000	--	--	--	--	--	--	--	36.7	--
1900000	--	--	--	--	--	--	--	34.6	--
2400000	--	--	82.7	--	--	--	--	--	--
3000000	--	--	80.7	--	--	--	--	--	--
3600000	--	--	75.7	--	--	--	--	--	--
4200000	--	--	66.3	--	--	--	--	--	--
5102000	--	--	47.4	--	--	--	--	--	--

Table B-5 (Continued). Percent load transfer efficiency measured on the approach side.

Load Transfer Efficiency (%)

Slab No. / Cycles	19	20	21	22	23	24	25	26	27
1	98.8	--	99.5	96.6	96.2	91.3	92.0	90.4	96.8
1000	96.8	88.4	98.2	96.9	94.8	88.3	91.7	87.2	95.6
2000	97.0	--	98.7	96.9	94.7	84.1	90.4	87.4	95.9
5000	97.8	88.4	99.1	96.5	93.2	83.0	90.4	87.7	95.6
10000	98.1	88.3	98.9	94.4	91.2	80.1	89.2	81.3	96.3
20000	98.3	92.8	101.1	93.8	90.8	78.3	89.4	79.9	93.9
50000	98.3	92.0	101.8	93.3	90.4	76.7	87.0	79.1	93.5
100000	97.6	90.8	101.2	--	88.1	68.1	86.2	78.2	93.3
125000	--	--	--	--	--	50.3	--	--	--
290000	--	--	--	--	--	--	40.5	53.5	--
300000	96.6	91.0	96.1	92.4	86.9	--	--	--	93.4
533000	--	--	--	60.9	--	--	--	--	--
565000	--	--	--	--	52.1	--	--	--	--
600000	94.9	94.7	95.1	--	--	--	--	--	92.5
900000	38.3	91.1	94.9	--	--	--	--	--	93.4
1200000	--	93.5	94.1	--	--	--	--	--	94.0
1800000	--	--	94.8	--	--	--	--	--	92.0
2400000	--	--	93.9	--	--	--	--	--	91.2
2441000	--	85.7	--	--	--	--	--	--	--
3000000	--	--	94.7	--	--	--	--	--	--
3600000	--	--	94.9	--	--	--	--	--	--
4200000	--	--	92.7	--	--	--	--	--	--
4800000	--	--	94.1	--	--	--	--	--	--
5400000	--	--	94.5	--	--	--	--	--	--
6000000	--	--	92.1	--	--	--	--	--	91.3
6600000	--	--	91.4	--	--	--	--	--	91.0
7200000	--	--	--	--	--	--	--	--	89.8
7200000	--	--	--	--	--	--	--	--	90.7

Table B-5 (Continued). Percent load transfer efficiency measured on the approach side.

Load Transfer Efficiency (%)

Slab No.	28	29	30	31	32	32	34	35
Cycles								
1	91.1	97.9	94.5	93.8	92.3	93.5	94.0	93.4
1000	86.5	93.5	90.6	91.9	89.9	91.2	91.8	92.0
2000	85.9	93.6	89.1	91.7	89.9	90.0	91.7	92.4
5000	83.9	93.1	88.4	91.1	88.0	87.7	90.0	89.7
10000	81.9	88.7	91.4	88.5	88.3	86.8	89.5	89.7
20000	81.5	88.7	91.1	87.2	87.6	83.8	89.1	89.1
50000	81.0	87.1	89.3	86.1	87.7	83.3	88.6	89.8
100000	79.4	86.9	86.9	83.3	85.8	82.9	88.0	88.3
200000	--	--	--	57.1	--	--	--	--
246000	--	--	51.8	--	--	--	--	--
300000	78.1	90.2	--	--	64.5	76.5	87.9	88.4
350000	--	--	--	--	56.5	--	--	--
470000	--	--	--	--	--	57.1	--	--
600000	78.0	88.6	--	--	--	--	83.0	87.8
805000	--	--	--	--	--	--	47.8	--
859000	54.2	--	--	--	--	--	--	--
900000	--	70.2	--	--	--	--	--	87.3
965000	--	55.0	--	--	--	--	--	--
1200000	--	--	--	--	--	--	--	89.2
1500000	--	--	--	--	--	--	--	86.7
1800000	--	--	--	--	--	--	--	86.3
2100000	--	--	--	--	--	--	--	82.7
2400000	--	--	--	--	--	--	--	81.0
2825000	--	--	--	--	--	--	--	57.8
2850000	--	--	--	--	--	--	--	51.9

Table B-6. Peak deflections measured on the approach side.

Peak Deflections (mils)

Slab No. / Cycles	1	2	3	4	5	6	7	8	9
1	-40.00	-32.00	-35.50	-45.50	-50.10	-51.00	-60.50	-54.50	-38.50
1000	-40.50	-42.00	-40.00	-47.50	-46.50	-47.50	-62.50	-58.00	-40.00
2000	-41.50	-43.20	-42.00	-45.50	-53.50	--	-63.30	-58.50	-39.00
5000	-42.20	-42.00	-44.00	-44.00	-56.50	-54.00	-65.00	-59.00	-40.50
10000	--	-43.00	-42.80	-50.20	-57.50	-55.00	-65.00	-62.00	-40.10
20000	-56.70	-43.00	-49.90	-52.00	-60.00	-61.50	-66.00	-63.00	-40.10
50000	-55.50	-43.00	-48.00	-50.20	-59.50	-65.00	-66.00	-63.50	-40.50
100000	-54.00	-42.50	-53.30	-57.00	-59.00	-64.50	-65.00	-63.50	-41.50
250000	--	--	-66.00	--	--	--	--	--	--
300000	-55.00	-46.70	--	-56.00	-94.50	-63.30	-65.00	-63.50	-42.50
350000	--	--	--	--	--	-84.50	--	--	--
600000	-55.50	-43.50	--	-57.50	--	--	-69.00	-62.50	-42.50
825000	--	--	--	-92.00	--	--	--	--	--
900000	70.00	-44.00	--	--	--	--	--	--	--
1200000	--	--	--	--	--	--	-69.00	-65.00	-42.50
1500000	--	-75.20	--	--	--	--	--	--	--
1800000	--	--	--	--	--	--	-67.50	-64.00	-43.30
2400000	--	--	--	--	--	--	-69.00	-65.50	-44.50
2700000	--	--	--	--	--	--	-82.00	--	--
3000000	--	--	--	--	--	--	--	-60.10	-45.00
3300000	--	--	--	--	--	--	--	-54.00	--
3600000	--	--	--	--	--	--	--	--	-45.50
4200000	--	--	--	--	--	--	--	--	-47.00
4800000	--	--	--	--	--	--	--	--	-47.00
5400000	--	--	--	--	--	--	--	--	-47.50
6000000	--	--	--	--	--	--	--	--	-47.50
6600000	--	--	--	--	--	--	--	--	-52.00

Table B-6 (Continued). Peak deflections measured on the approach side.

Peak Deflections (mils)

Slab No. / Cycles	10	11	12	13	14	15	16	17	18
1	-49.00	-63.39	-37.87	--	-54.33	-52.81	-55.31	-72.45	-42.17
1000	-51.00	-55.93	-39.41	-60.69	-56.64	-57.68	-60.27	--	-43.69
2000	-52.00	-55.14	-41.67	-60.79	-57.25	-58.90	-58.44	-76.76	-43.29
5000	-52.50	-55.56	-42.01	-61.05	-37.51	-58.34	-59.86	-77.59	-44.12
10000	-52.50	-56.77	-34.87	-54.83	-37.16	-55.66	-57.01	-76.64	-45.53
20000	-53.00	-58.63	-35.35	-55.25	-36.02	-54.34	-56.96	-68.37	-46.44
50000	-54.00	-59.12	-34.42	-54.38	54.83	-56.05	-52.25	-62.21	-44.33
100000	-55.50	-53.69	-34.24	-55.39	-36.34	-56.18	-51.06	-62.32	-40.16
250000	-63.50	--	--	--	--	--	--	--	--
281000	--	--	--	--	--	-71.80	--	--	--
293000	--	--	--	--	--	--	--	--	-59.55
300000	--	-55.44	-36.65	-56.89	-45.84	--	-49.22	-69.02	--
558000	--	--	--	-72.00	--	--	--	--	--
600000	--	--	-35.65	--	-69.93	--	-53.51	-71.36	--
722500	--	--	--	--	--	--	-69.00	--	--
900000	--	--	-40.04	--	-52.23	--	--	-77.25	--
1200000	--	--	--	--	-56.83	--	--	-81.94	--
1500000	--	--	-34.76	--	--	--	--	-83.37	--
1599000	--	--	--	--	--	--	--	--	--
1622000	--	--	--	--	-72.14	--	--	--	--
1800000	--	--	--	--	--	--	--	-90.04	--
1900000	--	--	--	--	--	--	--	-90.62	--
2400000	--	--	-33.17	--	--	--	--	--	--
3000000	--	--	-35.83	--	--	--	--	--	--
3600000	--	--	-38.04	--	--	--	--	--	--
4200000	--	--	-42.64	--	--	--	--	--	--
5102000	--	--	-44.45	--	--	--	--	--	--

Table B-6 (Continued). Peak deflections measured on the approach side.

Peak Deflections (mils)

Slab No. / Cycles	19	20	21	22	23	24	25	26	27
1	-62.82	--	-55.57	-56.78	-58.11	-65.57	-63.38	-64.85	-60.08
1000	-66.70	-56.82	-58.44	-64.27	-61.44	-71.17	-70.79	-69.23	-68.53
2000	-67.26	--	-58.13	-64.80	-62.99	-66.20	-71.98	-71.53	-67.97
5000	-66.85	-59.45	-57.74	-65.77	-62.44	-70.04	-67.78	-74.16	-67.31
10000	-67.32	-61.60	-58.18	-54.56	-62.20	-72.51	-66.32	-72.06	-71.49
20000	-68.30	-57.94	-56.00	-55.90	-61.05	-74.34	-66.53	-67.06	-64.00
50000	-69.41	-56.52	-58.99	-56.92	-62.09	-75.44	-66.47	-70.27	-64.09
100000	-72.85	-56.98	-62.94	-55.80	-62.08	-73.44	-68.01	-68.07	-64.01
125000	--	--	--	--	--	-85.12	--	--	--
290000	--	--	--	--	--	--	-77.20	-85.38	--
300000	-64.45	-58.64	-57.01	-59.43	-59.18	--	--	--	-64.23
533000	--	--	--	-80.54	--	--	--	--	--
565000	--	--	--	--	-71.51	--	--	--	--
600000	-64.90	-56.90	-58.23	--	--	--	--	--	-64.68
900000	-95.96	-62.66	-58.94	--	--	--	--	--	-63.57
1200000	--	-60.95	-58.50	--	--	--	--	--	-63.81
1800000	--	--	-60.84	--	--	--	--	--	-64.33
2400000	--	--	-59.30	--	--	--	--	--	-64.56
2441000	--	-63.64	--	--	--	--	--	--	--
3000000	--	--	-63.52	--	--	--	--	--	--
3600000	--	--	-57.60	--	--	--	--	--	--
4200000	--	--	-61.54	--	--	--	--	--	--
4800000	--	--	-59.91	--	--	--	--	--	--
5400000	--	--	-61.62	--	--	--	--	--	--
6000000	--	--	-60.08	--	--	--	--	--	-66.05
6600000	--	--	-60.46	--	--	--	--	--	-65.29
7200000	--	--	--	--	--	--	--	--	-66.25
7800000	--	--	--	--	--	--	--	--	-64.89

Table B-6 (Continued). Peak deflections measured on the approach side.

Peak Deflections (mils)

Slab No.	28	29	30	31	32	33	34	35
Cycles								
1	-53.99	-85.07	-68.72	-70.02	-64.58	-67.28	-63.96	-55.82
1000	-56.80	-88.20	-75.05	-71.32	-69.77	-72.46	-65.29	-55.72
2000	-57.46	-89.09	-71.98	-72.74	-72.24	-70.28	-65.93	-58.92
5000	-56.28	-87.62	-71.80	-72.40	-70.22	-71.21	-58.89	-56.09
10000	-59.11	-69.87	-65.93	-66.15	-69.40	-68.79	-58.69	-52.68
20000	-57.84	-69.24	-66.98	-66.62	-64.61	-66.71	-58.21	-56.33
50000	-59.28	-74.30	-67.29	-66.76	-68.46	-69.44	-59.09	-58.86
100000	-59.93	-73.05	-67.65	-67.53	-70.73	-74.88	-59.12	-52.88
200000	--	--	--	-82.75	--	--	--	--
246000	--	--	-82.89	--	--	--	--	--
300000	-60.67	-78.40	--	--	-91.16	-76.13	-59.89	-54.43
350000	--	--	--	--	-83.89	--	--	--
470000	--	--	--	--	--	-73.50	--	--
600000	-60.57	-74.88	--	--	--	--	-64.20	-54.36
805000	--	--	--	--	--	--	-71.16	--
859000	-73.67	--	--	--	--	--	--	--
900000	--	-90.34	--	--	--	--	--	-54.82
965000	--	-79.76	--	--	--	--	--	--
1200000	--	--	--	--	--	--	--	-54.89
1500000	--	--	--	--	--	--	--	-54.60
1800000	--	--	--	--	--	--	--	-54.54
2100000	--	--	--	--	--	--	--	-54.58
2400000	--	--	--	--	--	--	--	-57.11
2825000	--	--	--	--	--	--	--	-64.61
2850000	--	--	--	--	--	--	--	-66.59

Table B-7. Peak differential deflections measured on the approach side.

Peak Differential Deflections (mils)

Slab No. / Cycles	1	2	3	4	5	6	7	8	9
1	2.92	1.94	7.40	3.68	6.20	6.13	5.30	2.00	1.62
1000	3.93	3.28	7.40	4.56	6.20	6.13	5.98	3.00	1.62
2000	4.34	3.54	8.83	5.28	7.94	--	6.13	4.50	1.64
5000	4.74	5.03	9.82	5.28	8.50	6.83	6.50	5.00	1.64
10000	--	5.38	10.85	6.84	8.98	7.04	6.60	5.00	1.64
20000	7.52	5.64	12.30	7.20	10.58	8.38	6.60	5.50	1.75
50000	7.80	5.64	13.53	9.33	10.98	15.60	6.36	5.00	1.98
100000	7.80	6.32	14.84	12.15	12.55	17.90	6.92	5.50	2.05
250000	--	--	65.40	--	--	--	--	--	--
300000	9.35	6.62	--	15.60	72.08	22.73	6.93	5.50	2.05
350000	--	--	--	--	--	63.40	--	--	--
600000	11.66	7.58	--	18.63	--	--	7.72	6.50	2.05
825000	--	--	--	83.64	--	--	--	--	--
900000	45.85	8.33	--	--	--	--	--	--	--
1200000	--	--	--	--	--	--	7.12	6.00	2.82
1500000	--	64.73	--	--	--	--	--	--	--
1800000	--	--	--	--	--	--	7.13	6.50	3.30
2400000	--	--	--	--	--	--	8.78	5.50	4.76
2700000	--	--	--	--	--	--	43.48	--	--
3000000	--	--	--	--	--	--	--	9.10	5.60
3300000	--	--	--	--	--	--	--	17.40	--
3600000	--	--	--	--	--	--	--	--	5.85
4200000	--	--	--	--	--	--	--	--	5.98
4800000	--	--	--	--	--	--	--	--	6.78
5400000	--	--	--	--	--	--	--	--	7.98
6000000	--	--	--	--	--	--	--	--	8.28
6600000	--	--	--	--	--	--	--	--	31.63

Table B-7 (Continued). Peak differential deflections measured on the approach side.

Peak Differential Deflections (mils)

Slab No. / Cycles	10	11	12	13	14	15	16	17	18
1	6.50	1.95	1.26	--	1.05	1.97	3.64	5.64	2.95
1000	7.00	0.35	1.55	4.88	2.52	2.22	4.31	--	2.42
2000	7.00	0.38	2.13	4.99	2.61	3.90	4.09	7.06	2.27
5000	7.50	0.88	2.47	5.76	1.07	3.84	5.21	7.97	2.19
10000	7.50	1.08	2.68	4.33	0.70	3.32	4.09	8.62	2.12
20000	9.00	1.05	2.48	5.79	1.38	7.57	5.09	9.30	2.43
50000	9.00	1.46	2.88	6.32	1.01	9.44	3.43	6.58	4.11
100000	8.90	1.30	2.51	5.05	4.92	12.86	3.25	6.69	5.05
250000	40.00	--	--	--	--	--	--	--	--
281000	--	--	--	--	--	50.22	--	--	--
293000	--	--	--	--	--	--	--	--	28.09
300000	--	2.86	5.53	6.99	4.26	--	5.20	19.53	--
558000	--	--	--	57.21	--	--	--	--	--
600000	--	--	5.15	--	7.41	--	6.00	25.69	--
722500	--	--	--	--	--	--	38.84	--	--
900000	--	9.40	5.34	--	9.14	--	--	36.48	--
1200000	--	2.10	--	--	9.83	--	--	41.07	--
1500000	--	--	5.19	--	--	--	--	47.17	--
1599000	--	3.63	--	--	--	--	--	--	--
1622000	--	--	--	--	59.81	--	--	--	--
1800000	--	--	--	--	--	--	--	57.06	--
1900000	--	--	--	--	--	--	--	59.23	--
2400000	--	--	5.74	--	--	--	--	--	--
3000000	--	--	6.90	--	--	--	--	--	--
3600000	--	--	9.23	--	--	--	--	--	--
4200000	--	--	14.39	--	--	--	--	--	--
5102000	--	--	23.38	--	--	--	--	--	--

Table B-7 (Continued). Peak differential deflections measured on the approach side.

Peak Differential Deflections (mils)

Slab No. / Cycles	19	20	21	22	23	24	25	26	27
1	0.73	--	0.26	2.18	2.04	5.71	5.04	6.25	1.91
1000	2.14	6.60	1.05	3.33	1.93	8.32	6.55	8.87	2.99
2000	2.00	--	0.76	3.43	1.93	10.50	6.88	9.00	2.79
5000	1.44	6.92	0.51	4.49	2.17	11.94	6.51	9.11	2.95
10000	1.27	7.21	0.65	4.81	3.47	14.45	7.13	13.45	2.61
20000	1.19	4.15	0.63	5.14	3.77	16.14	7.08	13.49	3.89
50000	1.19	4.52	1.06	5.46	3.00	17.54	8.62	14.69	4.18
100000	1.78	5.22	0.78	6.66	4.13	23.43	9.36	14.82	4.28
125000	--	--	--	--	--	42.28	--	--	--
290000	--	--	--	--	--	--	45.95	39.69	--
300000	2.20	5.26	2.25	7.76	4.49	--	--	--	4.24
533000	--	--	--	38.60	--	--	--	--	--
565000	--	--	--	--	27.93	--	--	--	--
600000	3.28	3.00	2.83	--	--	--	--	--	4.87
900000	59.19	5.56	2.99	--	--	--	--	--	4.21
1200000	--	3.97	3.47	--	--	--	--	--	3.81
1800000	--	--	3.15	--	--	--	--	--	5.13
2400000	--	--	3.64	--	--	--	--	--	5.70
2441000	--	9.10	--	--	--	--	--	--	--
3000000	--	--	3.39	--	--	--	--	--	--
3600000	--	--	2.94	--	--	--	--	--	--
4200000	--	--	4.48	--	--	--	--	--	--
4800000	--	--	3.54	--	--	--	--	--	--
5400000	--	--	3.40	--	--	--	--	--	--
6000000	--	--	4.77	--	--	--	--	--	5.73
6600000	--	--	5.18	--	--	--	--	--	5.89
7200000	--	--	--	--	--	--	--	--	6.79
7800000	--	--	--	--	--	--	--	--	6.02

Table B-7 (Continued). Peak differential deflections measured on the approach side.

Peak Differential Deflections (mils)

Slab No.	28	29	30	31	32	33	34	35
Cycles								
1	4.80	1.78	3.75	4.37	4.96	4.38	3.85	3.68
1000	7.69	5.77	7.08	5.78	7.04	6.38	5.38	4.45
2000	8.12	5.72	7.86	6.06	7.28	7.00	5.47	4.50
5000	9.07	6.01	8.32	6.45	8.46	8.74	5.90	5.79
10000	10.68	8.00	5.70	7.63	8.14	9.10	6.15	5.42
20000	10.70	7.81	5.95	8.55	8.02	10.83	6.36	6.16
50000	11.28	9.57	7.23	9.29	8.44	11.59	6.74	5.98
100000	12.34	9.58	8.83	11.29	10.04	12.80	7.12	6.19
200000	--	--	--	35.52	--	--	--	--
246000	--	--	39.93	--	--	--	--	--
300000	13.28	7.69	--	--	32.37	17.88	7.27	6.31
350000	--	--	--	--	36.53	--	--	--
470000	--	--	--	--	--	31.56	--	--
600000	13.33	8.56	--	--	--	--	10.91	6.65
805000	--	--	--	--	--	--	37.13	--
859000	33.77	--	--	--	--	--	--	--
900000	--	26.88	--	--	--	--	--	6.96
965000	--	35.92	--	--	--	--	--	--
1200000	--	--	--	--	--	--	--	7.03
1500000	--	--	--	--	--	--	--	7.27
1800000	--	--	--	--	--	--	--	7.48
2100000	--	--	--	--	--	--	--	9.46
2400000	--	--	--	--	--	--	--	10.85
2825000	--	--	--	--	--	--	--	27.28
2850000	--	--	--	--	--	--	--	32.01

Table B-8. Crack width data summary*.

Crack width (inches)

Slab No. / Cycles	11	12	13	14	15	16	17	18	19
Crack initiation	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.010	0.007	-0.155	0.012	0.064	0.002	0.017	0.022	0.027
5000	--	0.007	-0.150	0.012	0.064	0.002	0.009	--	0.027
10000	--	--	-0.150	0.012	0.064	0.002	0.004	--	0.027
20000	--	0.010	-0.150	0.012	0.064	0.003	0.002	0.022	0.027
50000	0.015	0.017	-0.150	0.012	0.065	0.003	0.010	--	0.029
100000	0.015	0.017	-0.150	0.012	0.069	0.005	0.010	--	0.032
281000	--	--	--	--	0.355	--	--	--	--
300000	0.015	0.017	-0.150	0.015	--	0.006	0.010	--	0.032
558000	--	--	0.118	--	--	--	--	--	--
600000	0.016	0.017	--	0.009	--	0.018	0.003	--	0.032
722500	--	--	--	--	--	0.191	--	--	--
900000	0.016	0.152	--	0.010	--	--	0.003	--	0.206
1200000	0.019	--	--	0.023	--	--	0.010	--	--
1457000	--	--	--	0.044	--	--	--	--	--
1500000	--	0.157	--	--	--	--	0.010	--	--
1599000	0.038	--	--	--	--	--	--	--	--
1622000	--	--	--	0.455	--	--	--	--	--
1800000	--	--	--	--	--	--	0.010	--	--
1900000	--	--	--	--	--	--	0.010	--	--
2400000	--	0.162	--	--	--	--	--	--	--
3000000	--	0.162	--	--	--	--	--	--	--
3600000	--	0.168	--	--	--	--	--	--	--
4200000	--	0.275	--	--	--	--	--	--	--
5102000	--	0.946	--	--	--	--	--	--	--

Table B-8 (continued). Crack width data summary.

Slab No.	20	21	22	23	24	25	26	27	28
Cycles									
Crack initiation	0.000	0.000	0.000	0.000	0.000	0.000	-0.041	0.000	0.000
1	0.006	0.000	-0.003	0.016	0.027	0.026	0.011	-0.022	0.009
5000	--	0.000	-0.003	0.019	0.030	--	0.019	-0.022	0.013
10000	--	0.000	-0.003	0.019	0.030	0.031	0.019	-0.022	0.017
20000	0.008	0.000	-0.004	0.019	0.031	0.031	0.019	-0.022	0.018
50000	0.008	0.001	-0.004	0.021	0.033	0.031	0.022	-0.022	0.018
100000	0.009	--	-0.002	0.021	0.046	0.031	0.219	-0.022	--
125000	--	--	--	--	0.140	0.032	--	--	--
300000	0.009	0.004	-0.002	0.021	--	--	--	-0.020	0.019
532000	--	--	0.143	--	--	--	--	--	--
533000	--	--	0.439	--	--	--	--	--	--
542000	--	--	--	0.066	--	--	--	--	--
565000	--	--	--	0.220	--	--	--	--	--
600000	0.010	0.004	--	--	--	--	--	-0.020	0.021
859000	--	--	--	--	--	--	--	--	0.229
900000	0.011	0.004	--	--	--	--	--	-0.020	--
1200000	0.013	0.004	--	--	--	--	--	-0.020	--
1800000	0.021	0.003	--	--	--	--	--	-0.020	--
2400000	0.096	0.002	--	--	--	--	--	-0.019	--
2441000	0.205	--	--	--	--	--	--	--	--
3000000	--	0.004	--	--	--	--	--	--	--
4200000	--	0.006	--	--	--	--	--	--	--
5400000	--	0.007	--	--	--	--	--	--	--
6000000	--	0.007	--	--	--	--	--	-0.016	--
6600000	--	0.012	--	--	--	--	--	--	--
7800000	--	--	--	--	--	--	--	-0.016	--

Table B-8 (continued). Crack width data summary.

Slab No. / Cycles	29	30	31	32	33	34	35
Crack initiation	0.000	0.000	0.000	0.033	0.007	0.000	0.000
1	0.001	0.004	0.019	0.033	0.007	0.018	-0.010
5000	--	--	--	--	--	0.018	-0.009
10000	0.002	0.008	0.020	0.033	0.013	0.018	-0.009
20000	0.002	0.011	0.023	0.033	0.015	0.019	-0.007
50000	0.005	0.012	0.024	0.034	0.019	0.019	-0.007
100000	0.006	0.012	0.025	0.034	0.020	0.020	-0.007
180000	--	--	0.113	--	--	--	--
200000	--	0.044	0.158	--	--	--	--
246000	--	0.249	--	--	--	--	--
300000	0.006	--	--	0.069	0.020	0.021	-0.007
350000	--	--	--	0.156	--	--	--
458000	--	--	--	--	0.142	--	--
465000	--	--	--	--	0.237	--	--
470000	--	--	--	--	0.263	--	--
600000	0.006	--	--	--	--	0.036	-0.007
765000	--	--	--	--	--	0.058	--
780000	--	--	--	--	--	0.031	--
805000	--	--	--	--	--	0.215	--
900000	0.010	--	--	--	--	--	-0.007
960000	0.068	--	--	--	--	--	--
965000	0.077	--	--	--	--	--	--
1200000	--	--	--	--	--	--	-0.007
1800000	--	--	--	--	--	--	-0.005
2400000	--	--	--	--	--	--	0.009
2850000	--	--	--	--	--	--	0.358

NOTE: * Crack width data was not taken if cracking was evident around the measuring studs.

APPENDIX C

LOAD TRANSFER HISTORIES OF TEST SPECIMENS

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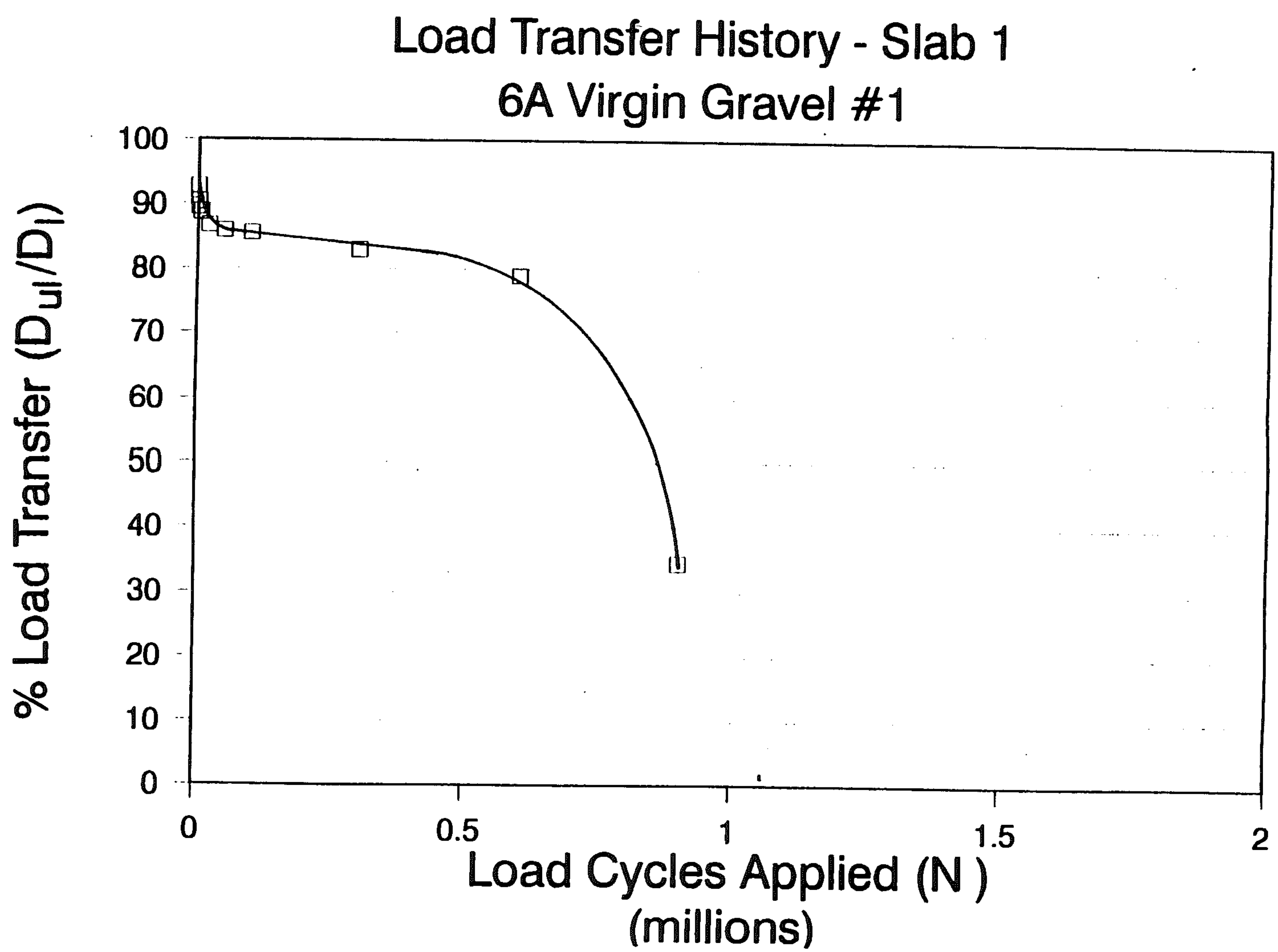

Figure C-1. Load transfer history for slab 1.

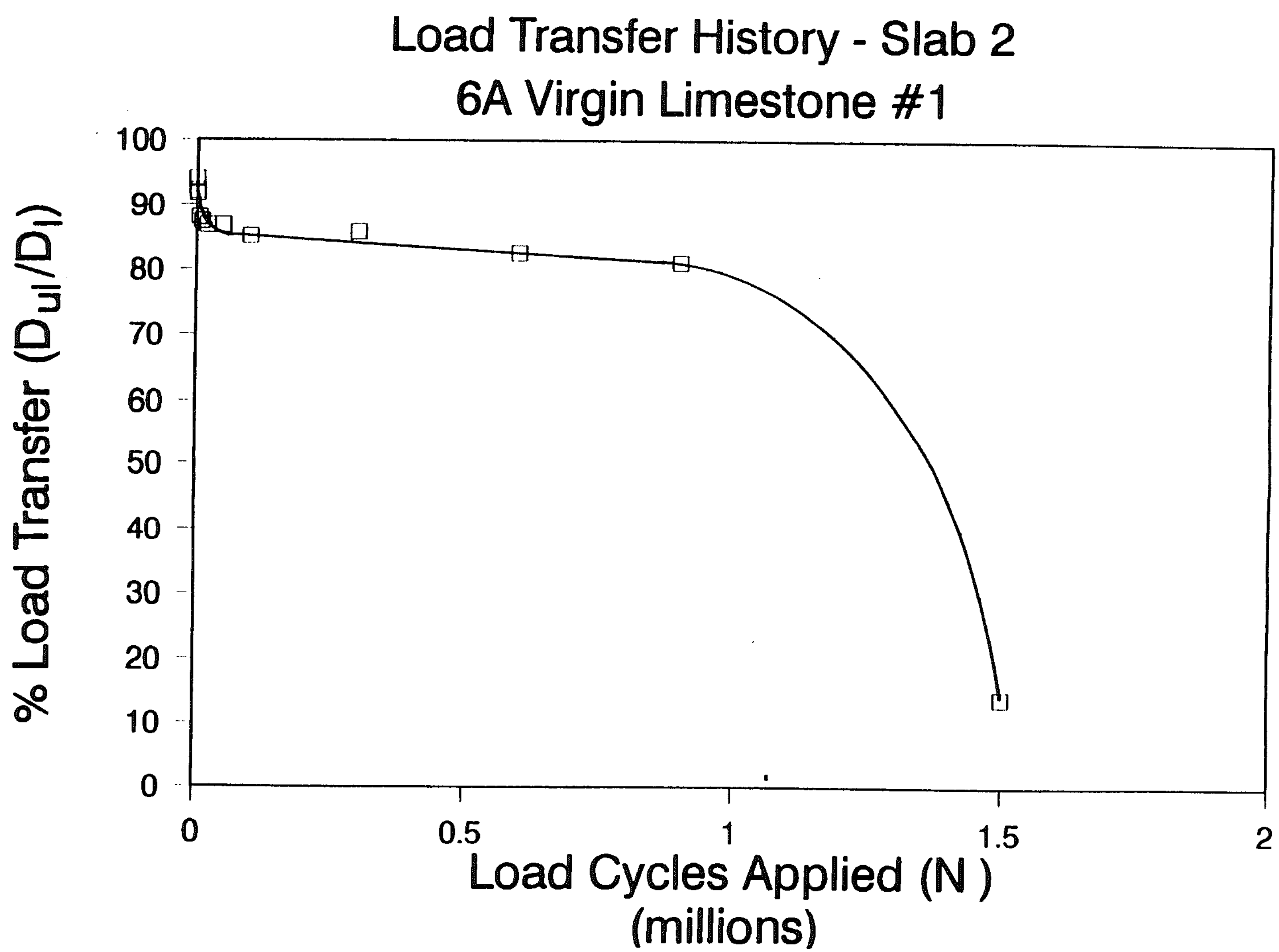

Figure C-2. Load transfer history for slab 2.

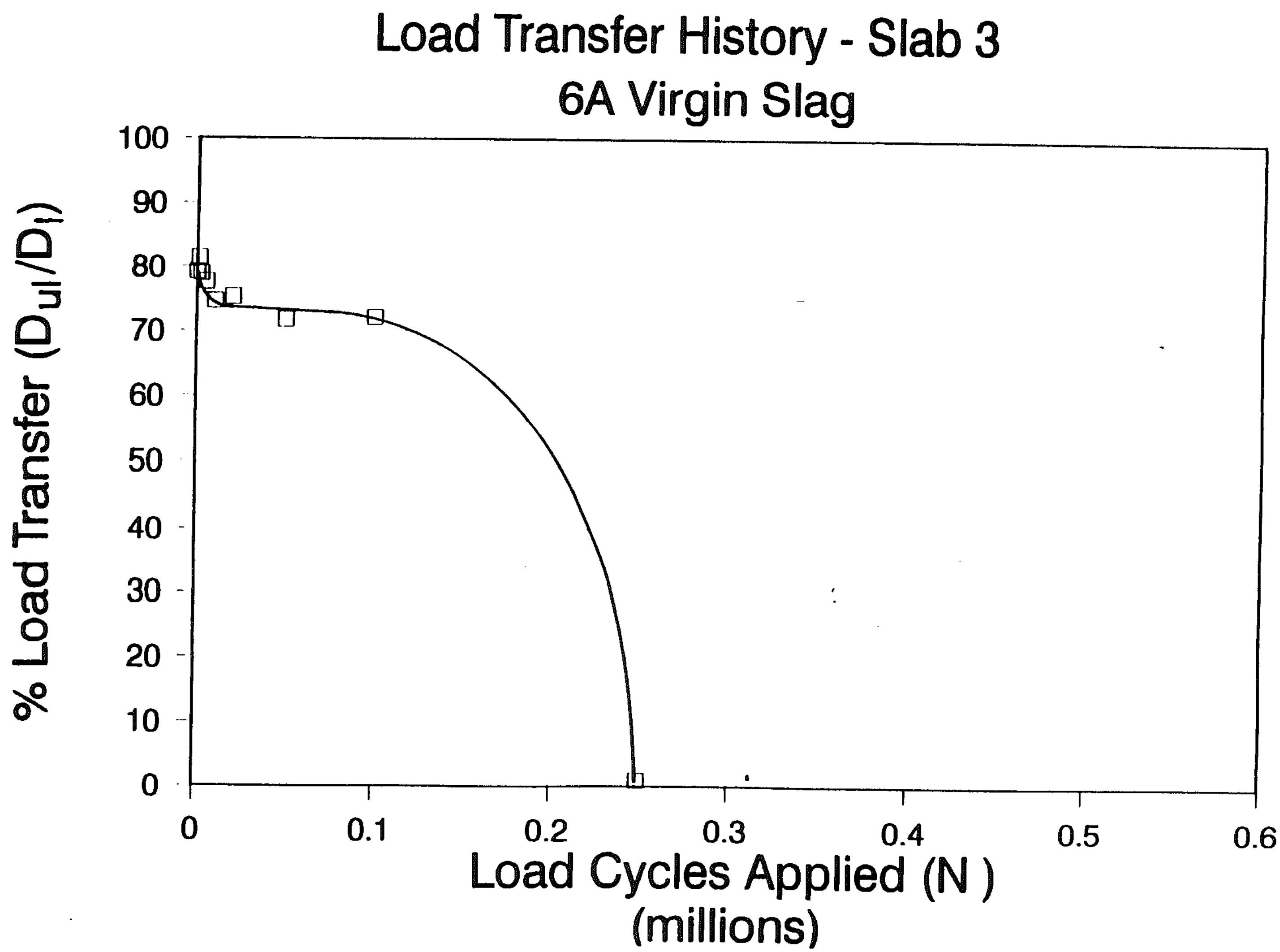

Figure C-3. . Load transfer history for slab 3.

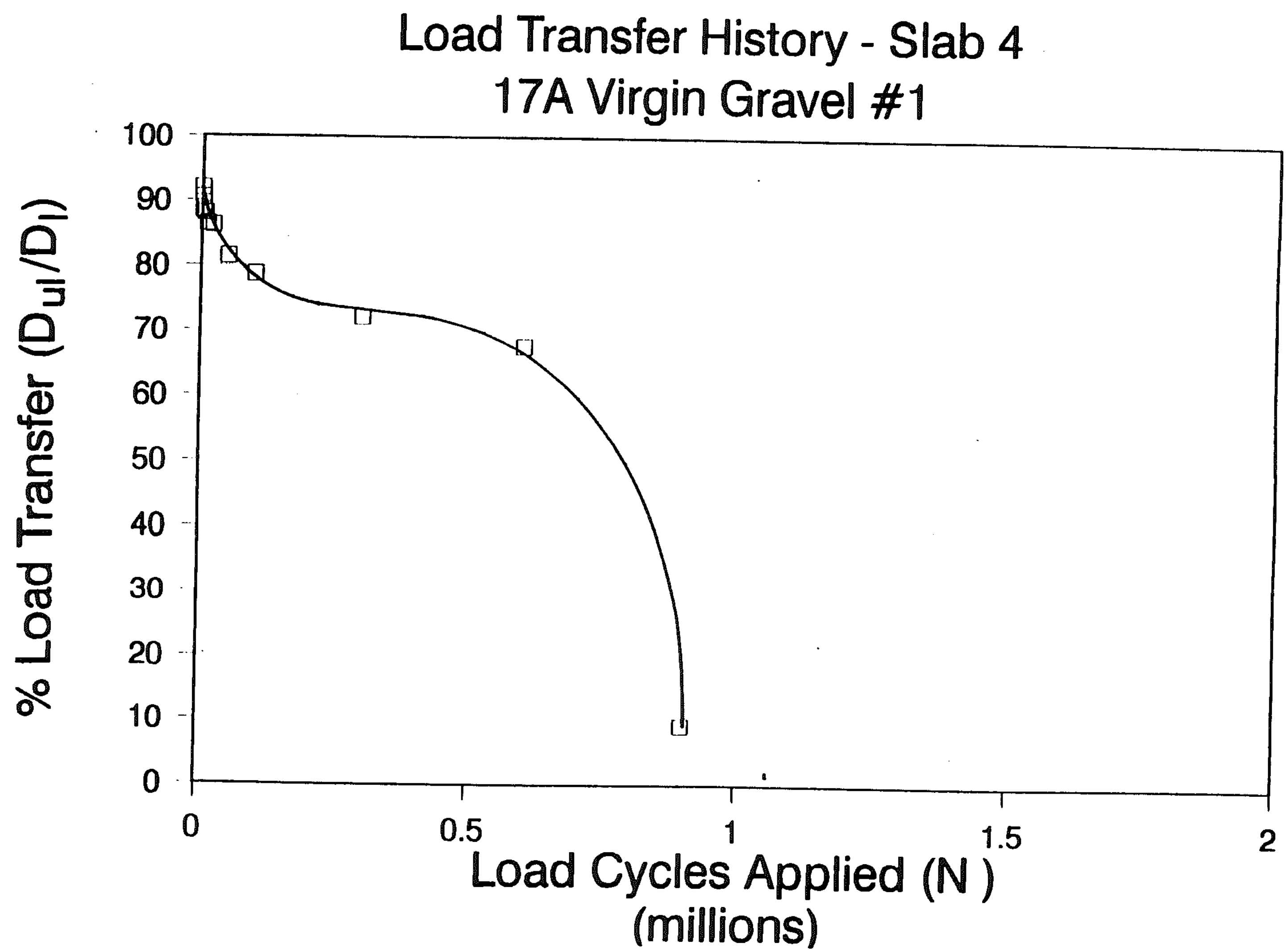

Figure C-4. Load transfer history for slab 4.

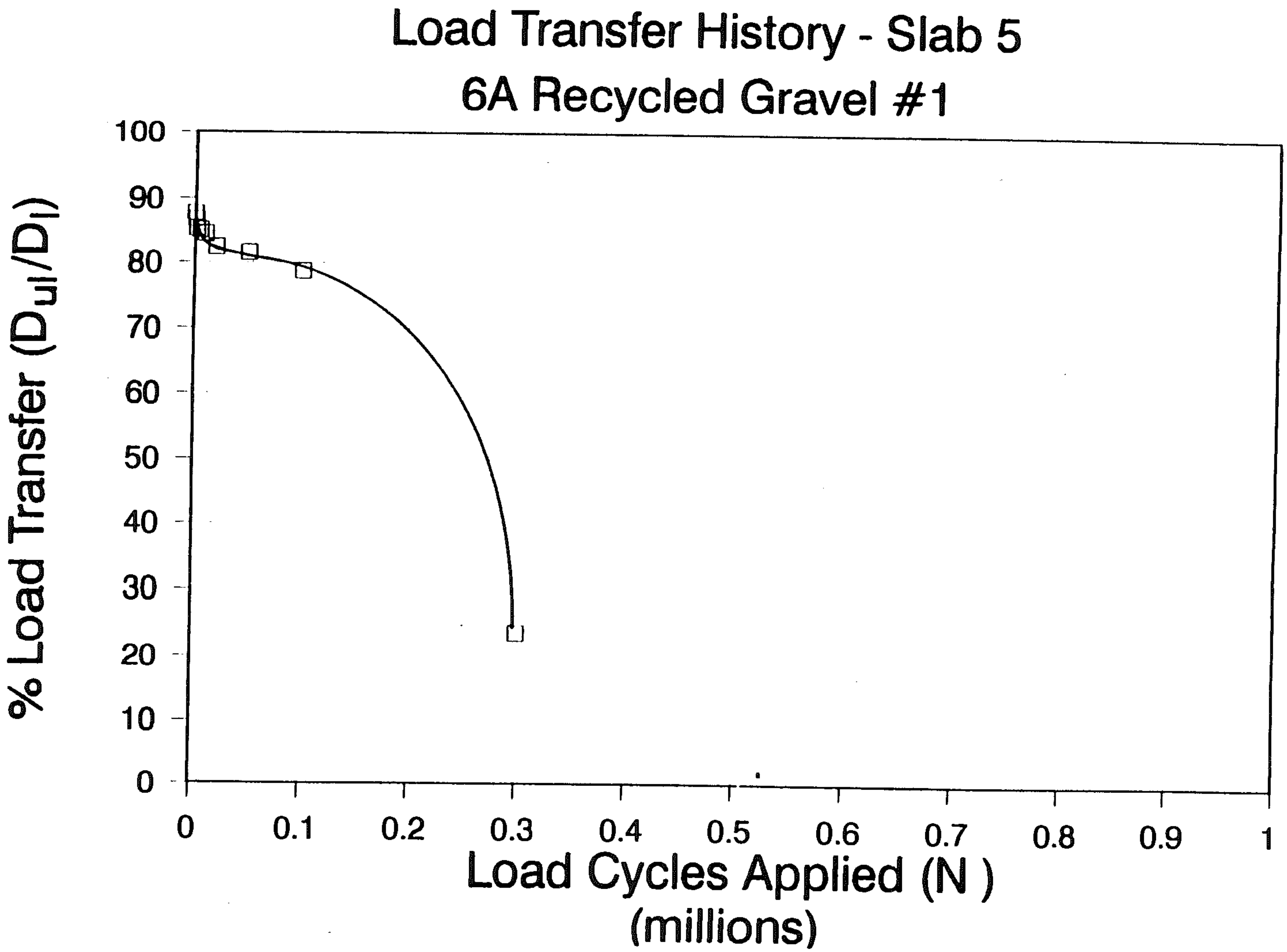

Figure C-5. Load transfer history for slab 5.

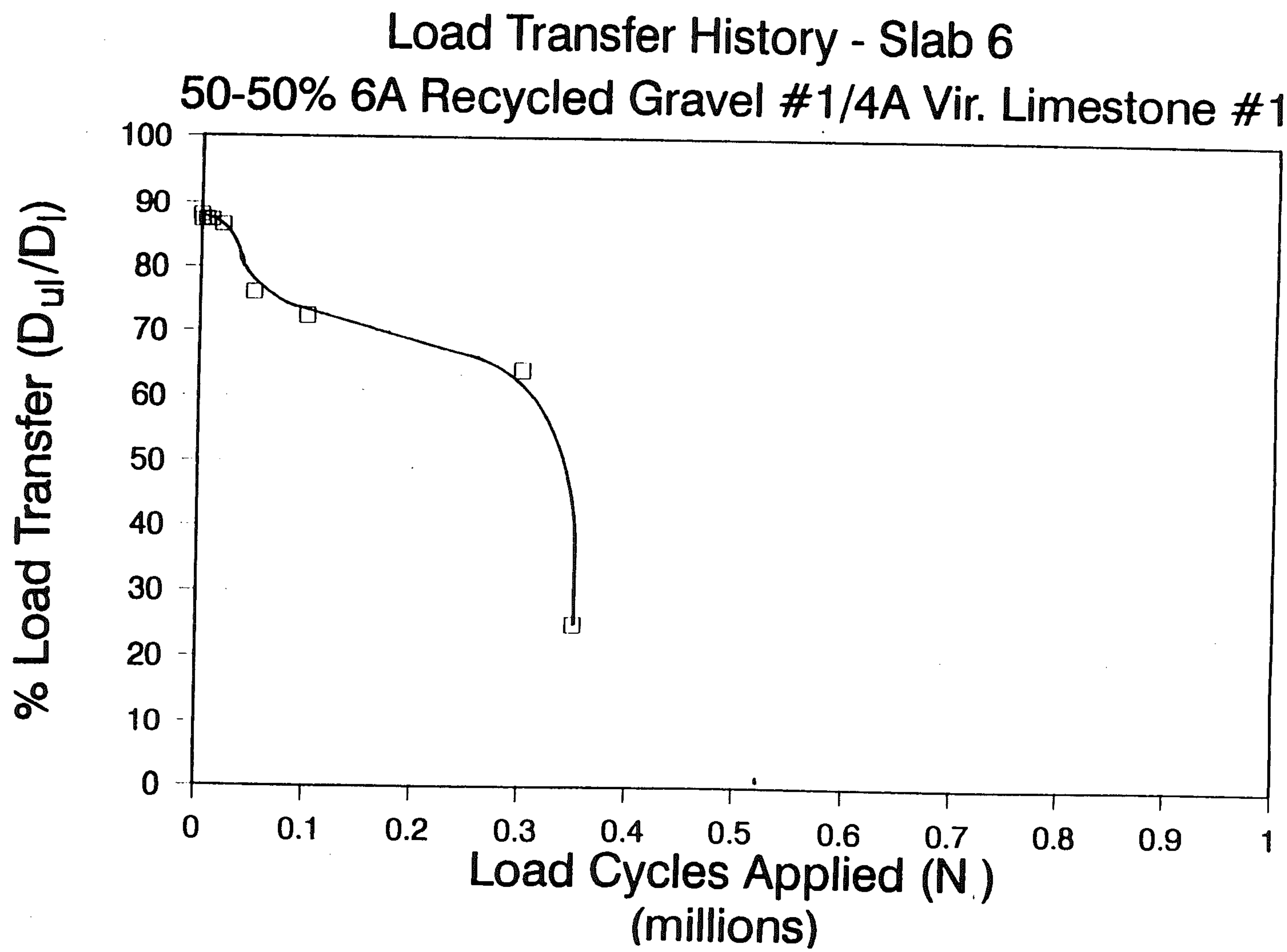

Figure C-6. Load transfer history for slab 6.

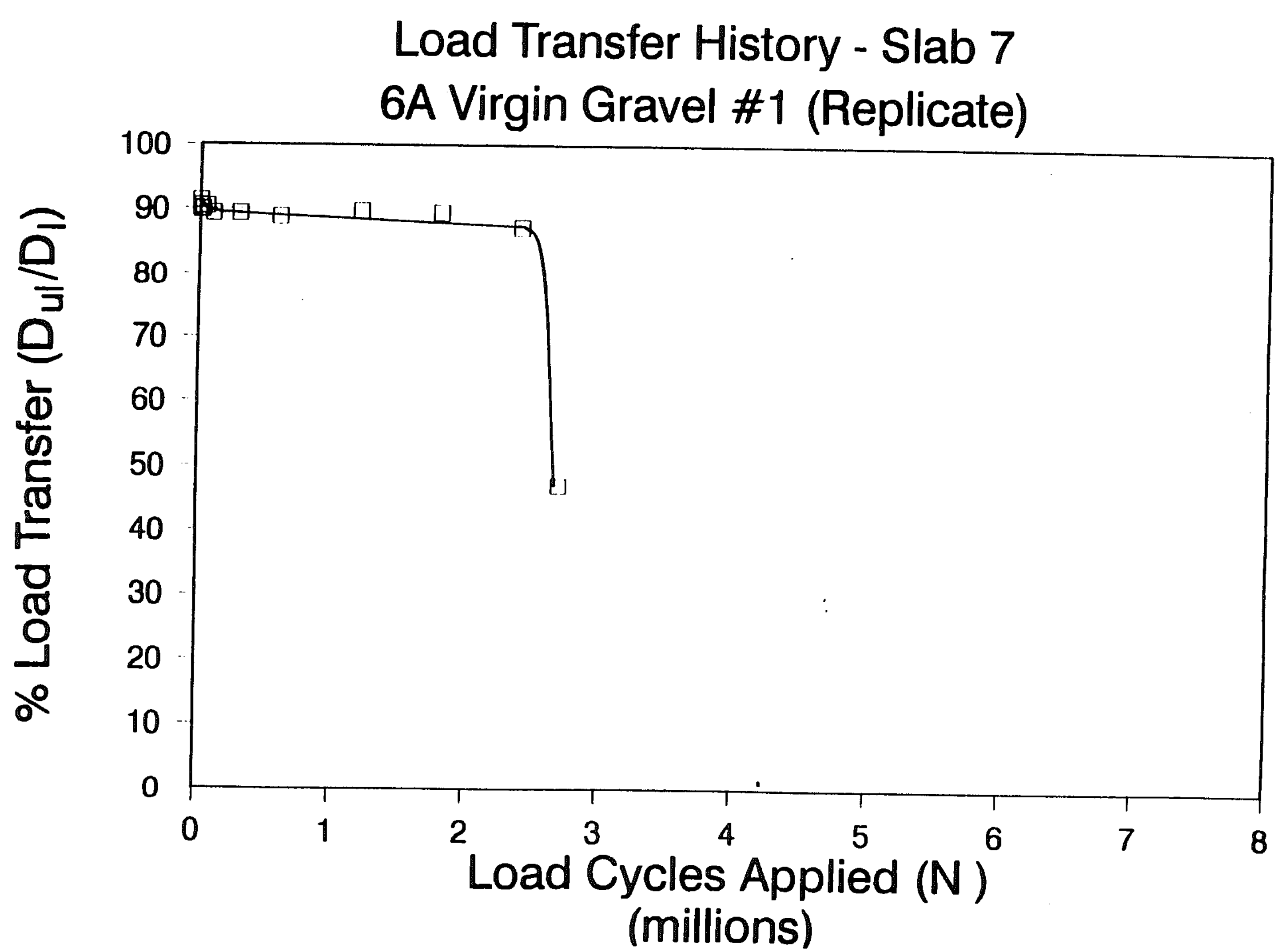

Figure C-7. Load transfer history for slab 7.

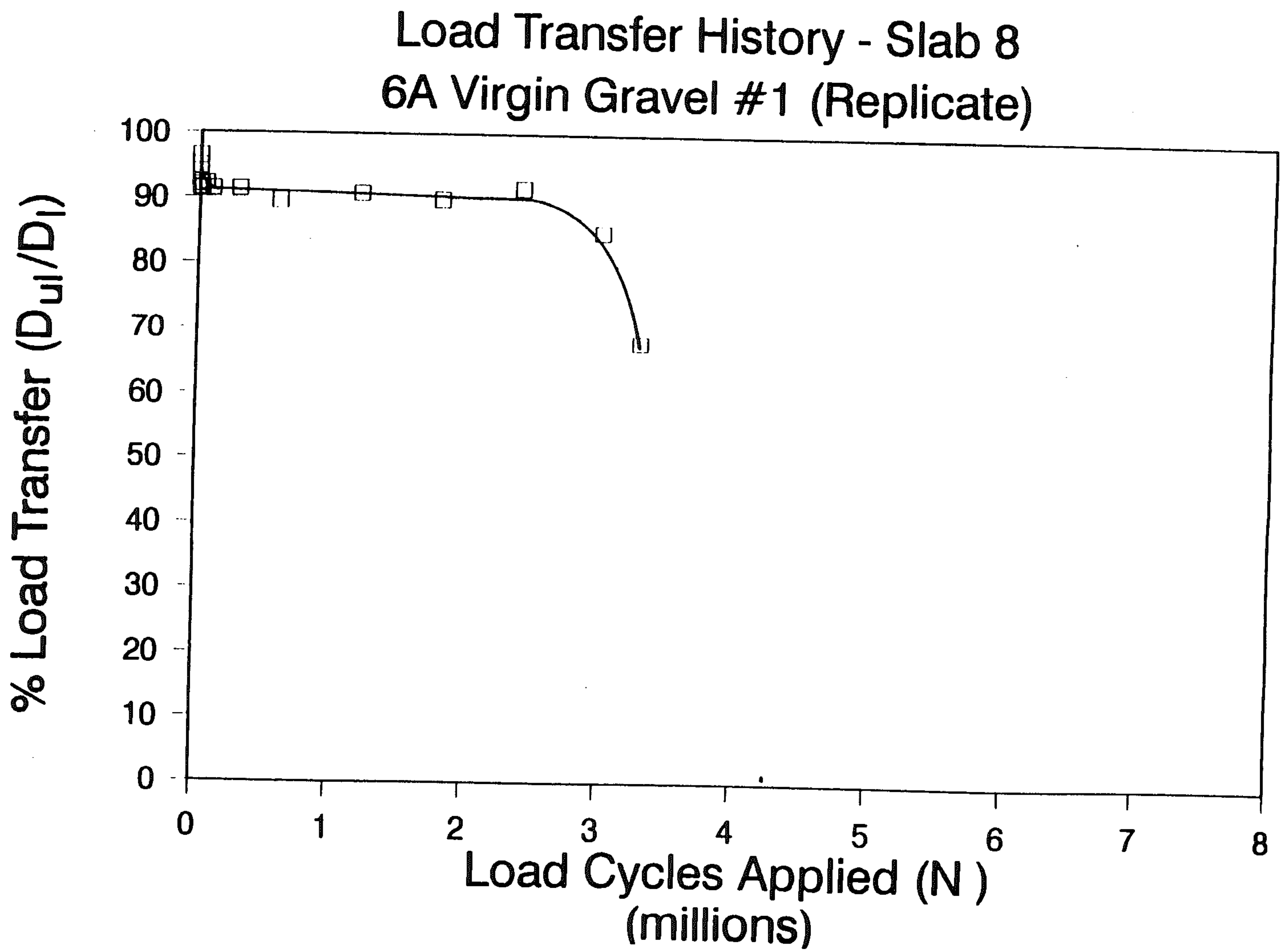

Figure C-8. Load transfer history for slab 8.

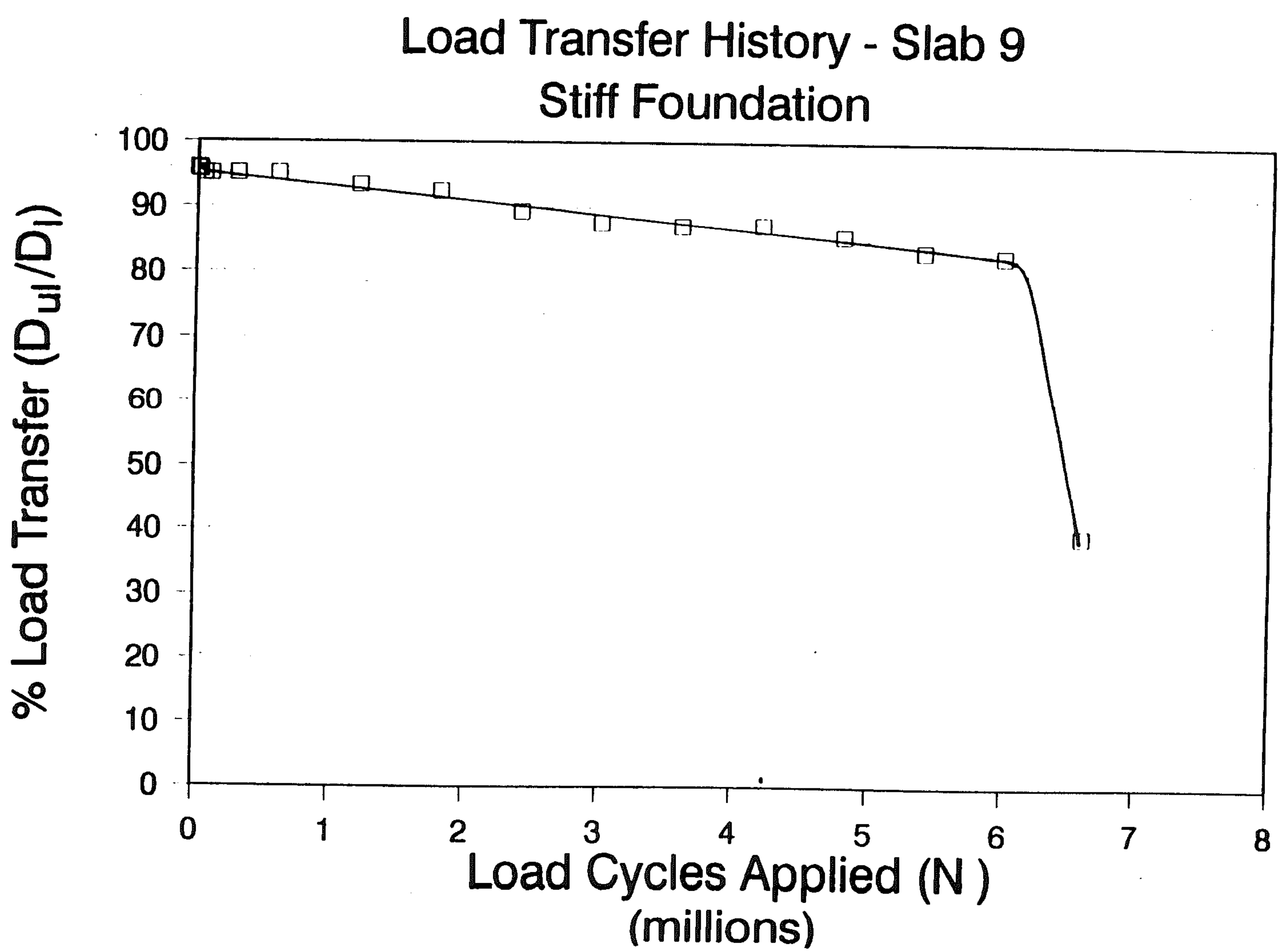

Figure C-9. Load transfer history for slab 9.

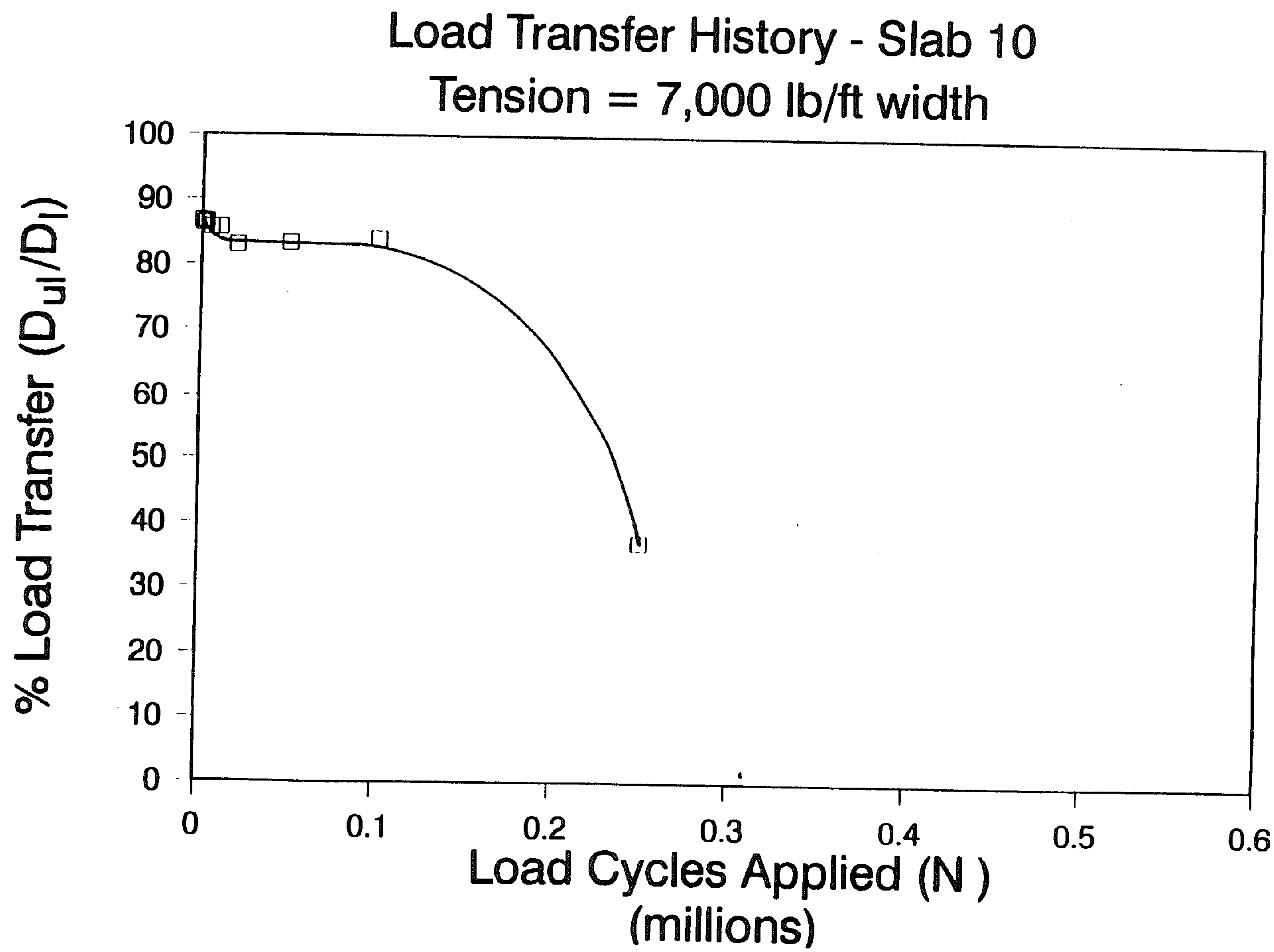

Figure C-10. Load transfer history for slab 10.

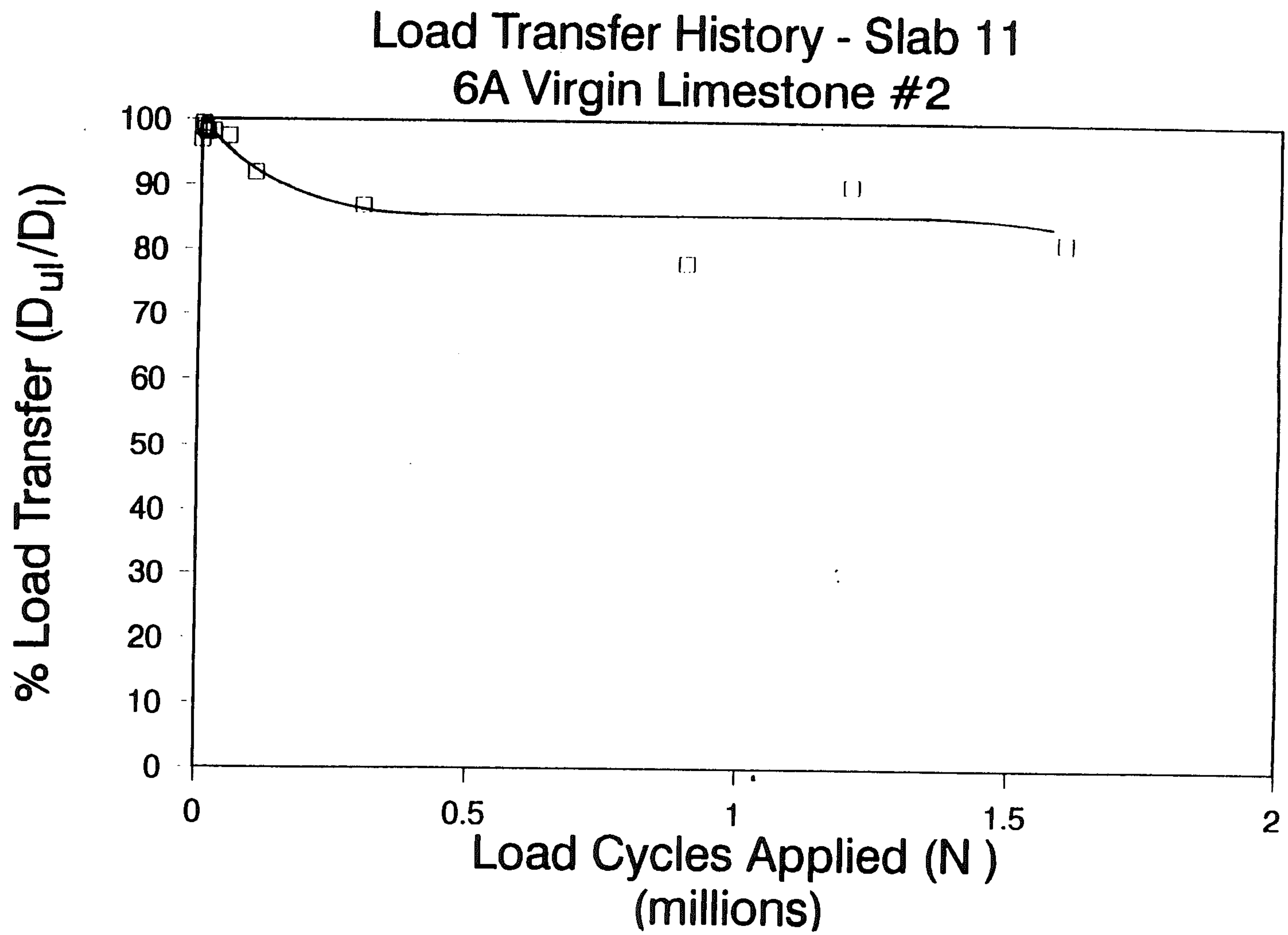

Figure C-11. Load transfer history for slab 11.

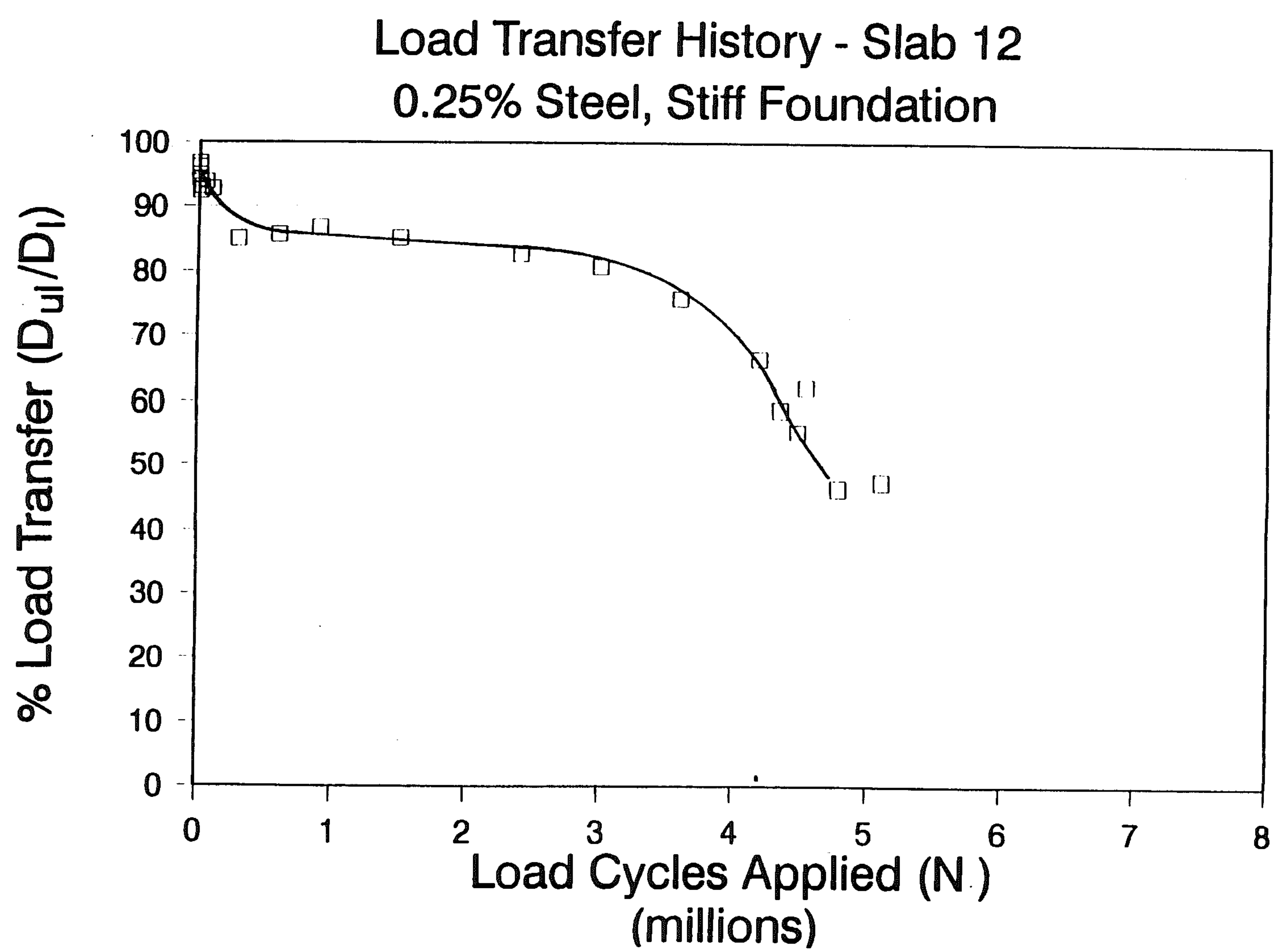

Figure C-12. Load transfer history for slab 12.

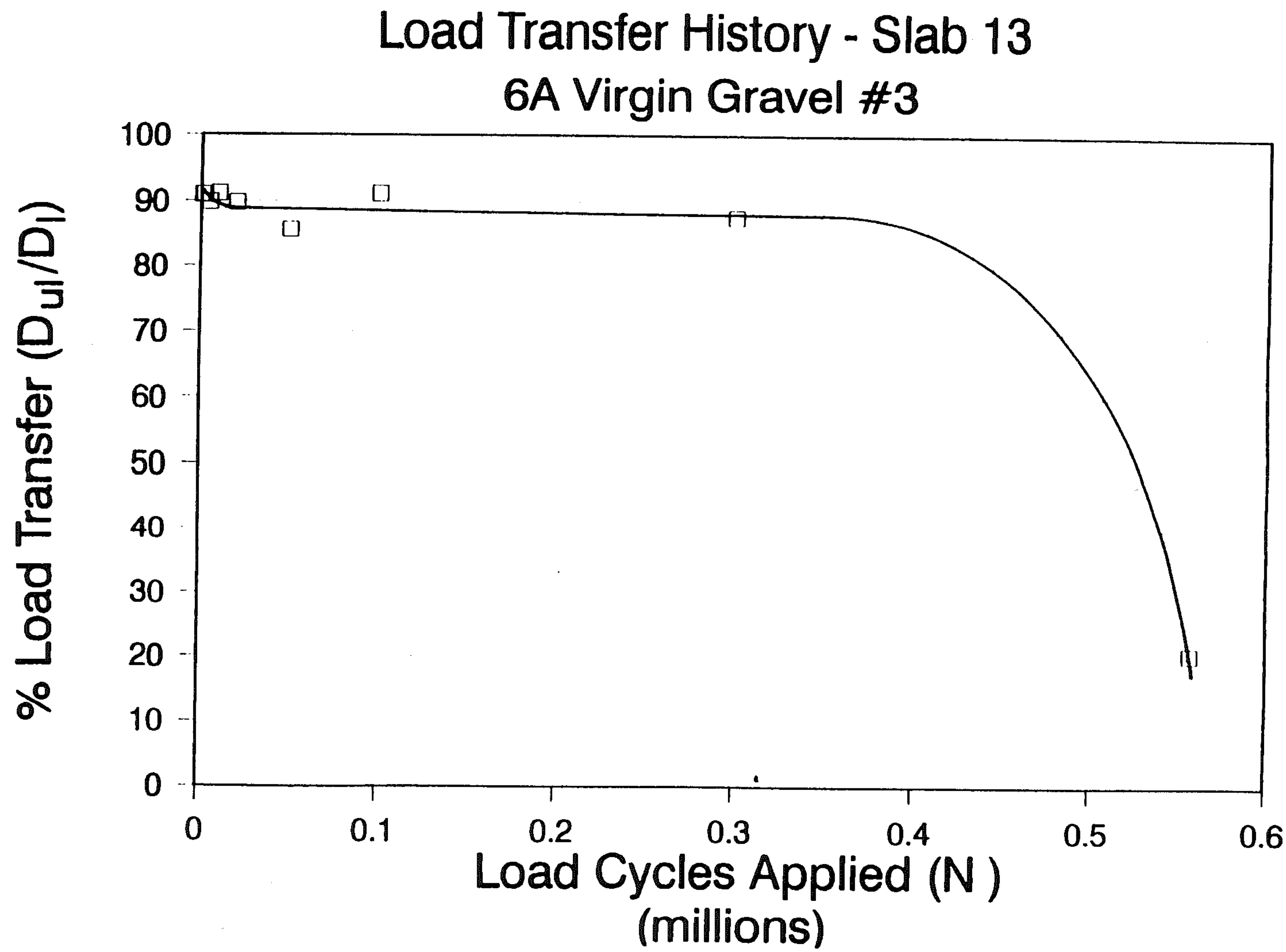

Figure C-13. Load transfer history for slab 13.

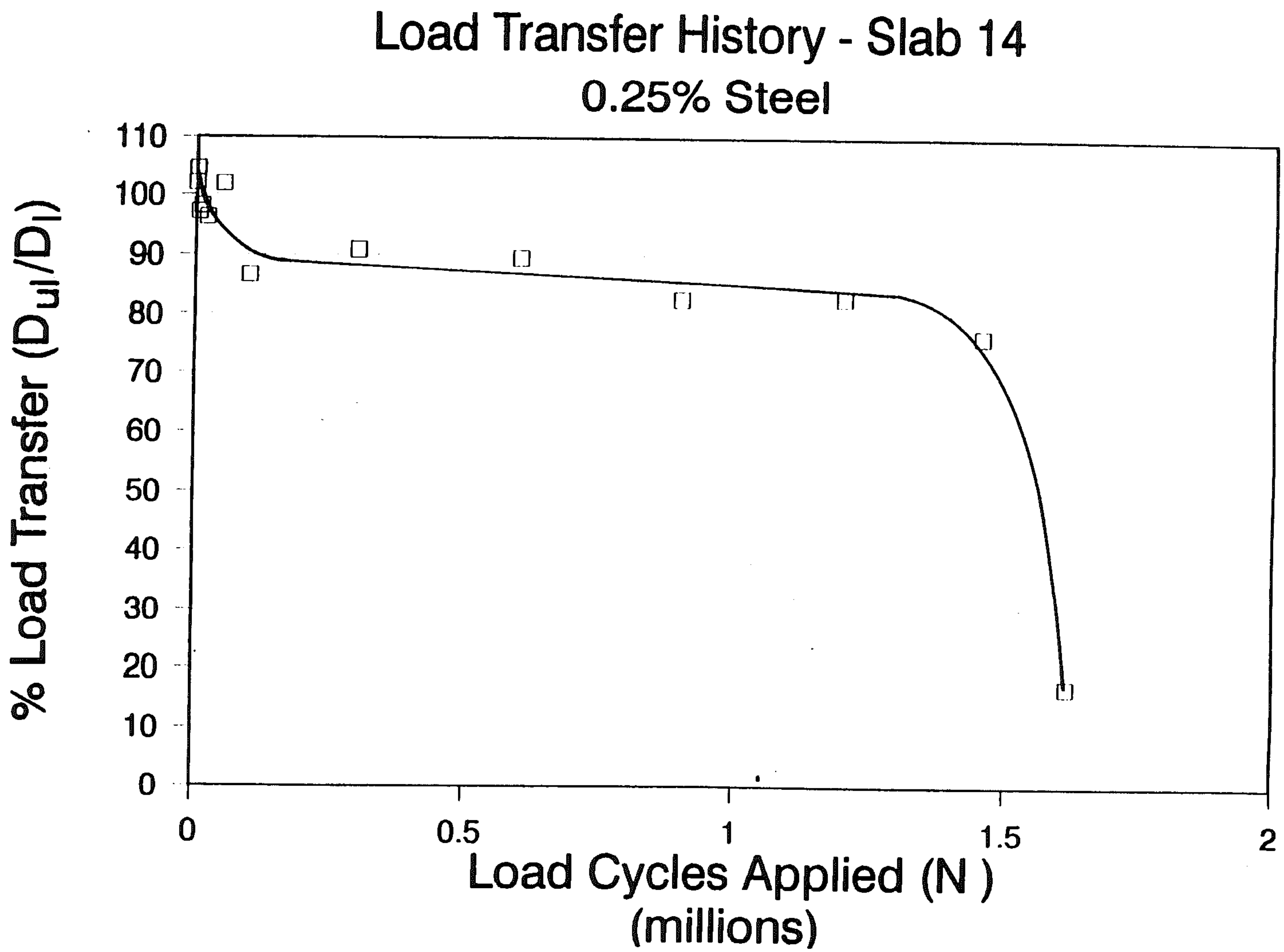

Figure C-14. Load transfer history for slab 14.

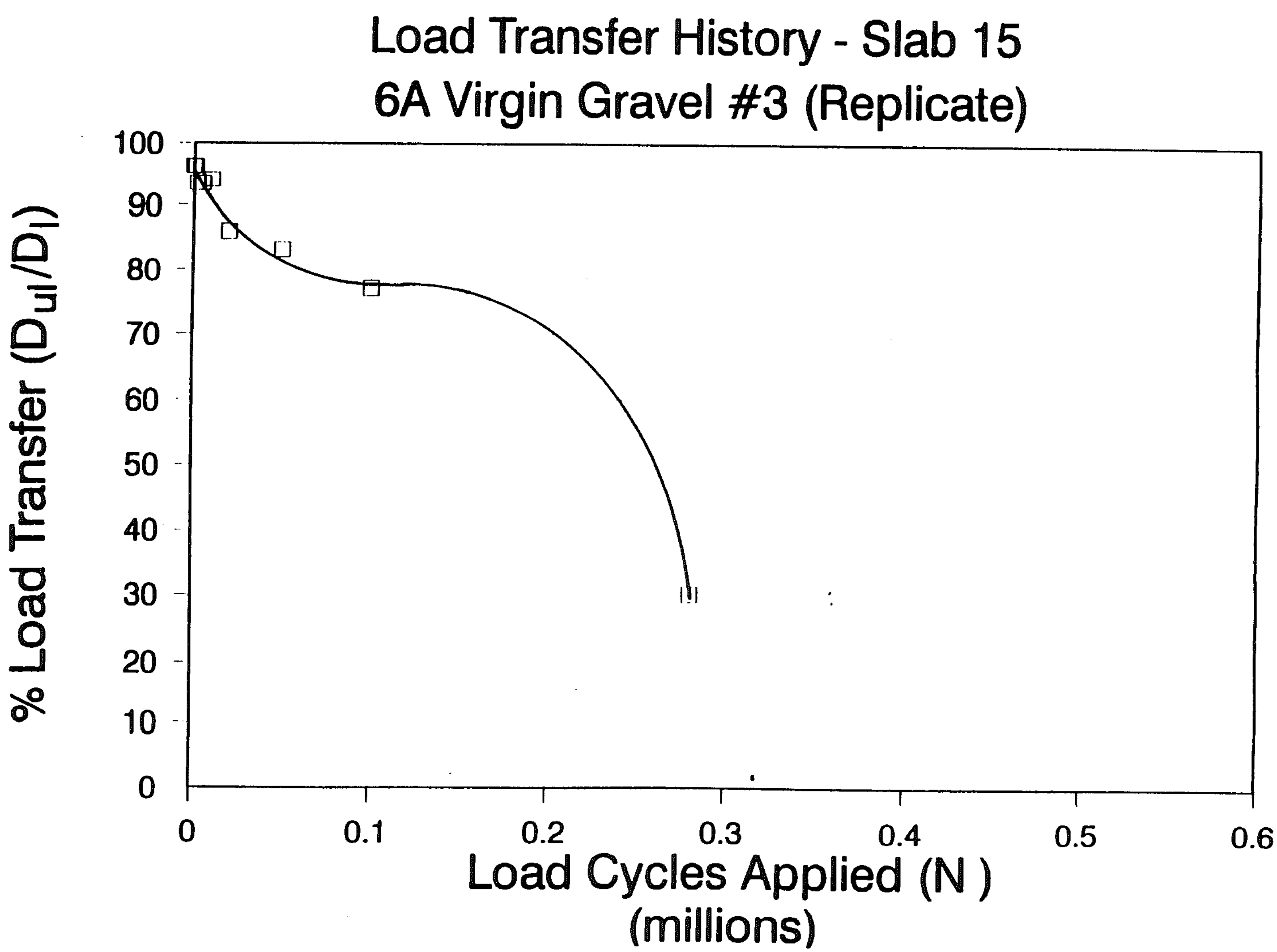

Figure C-15. Load transfer history for slab 15.

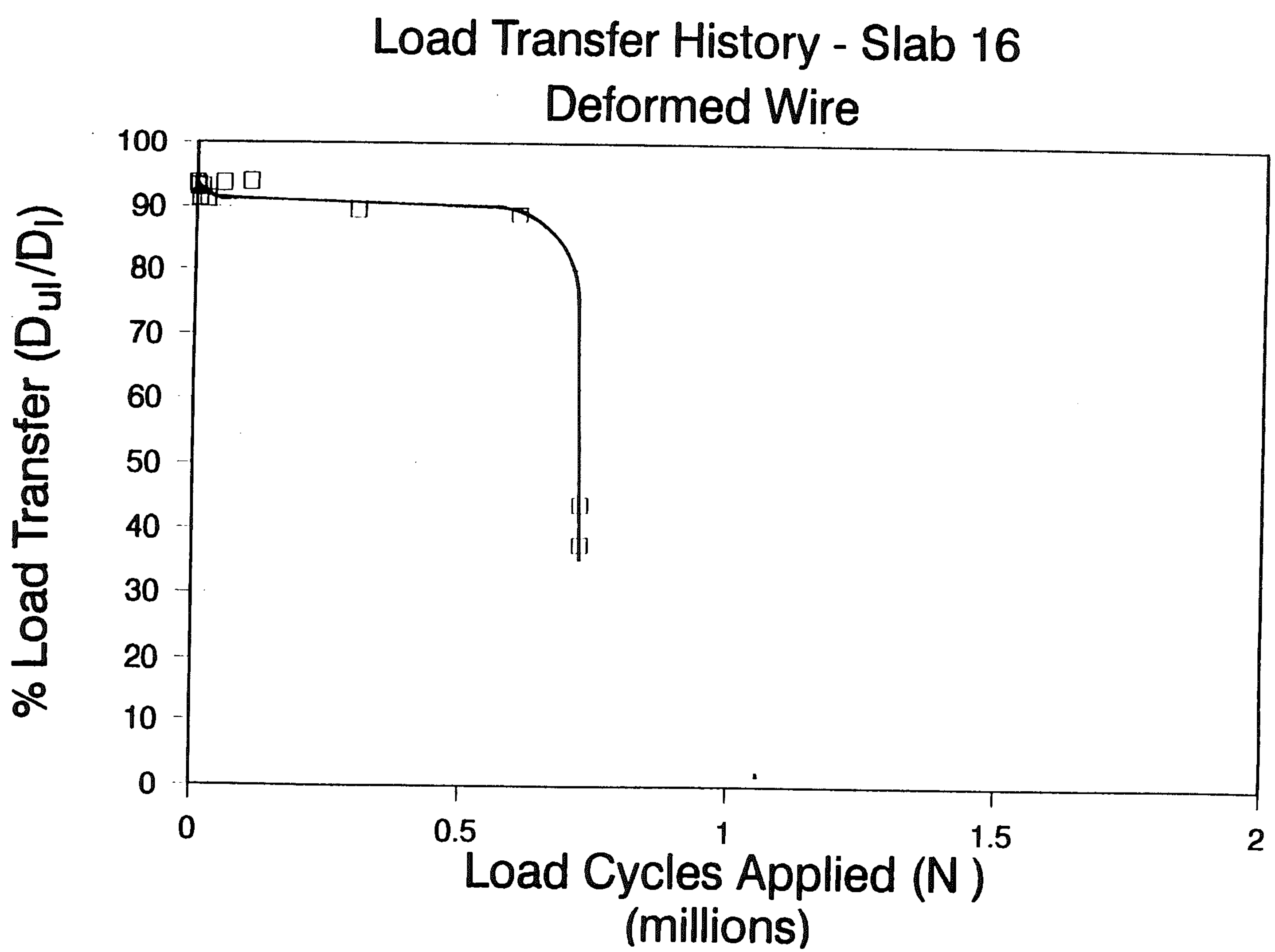

Figure C-16. Load transfer history for slab 16.

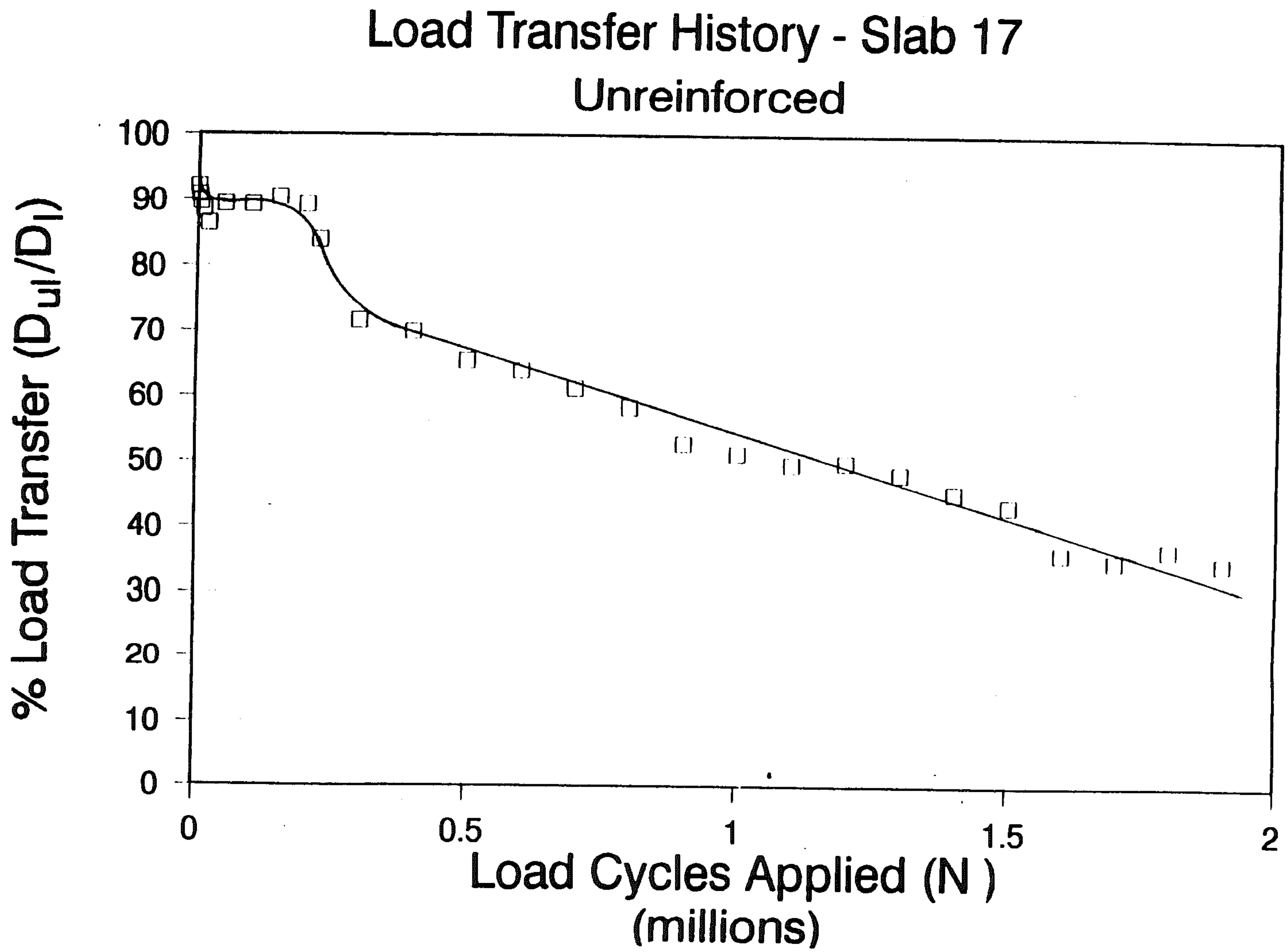

Figure C-17. Load transfer history for slab 17.

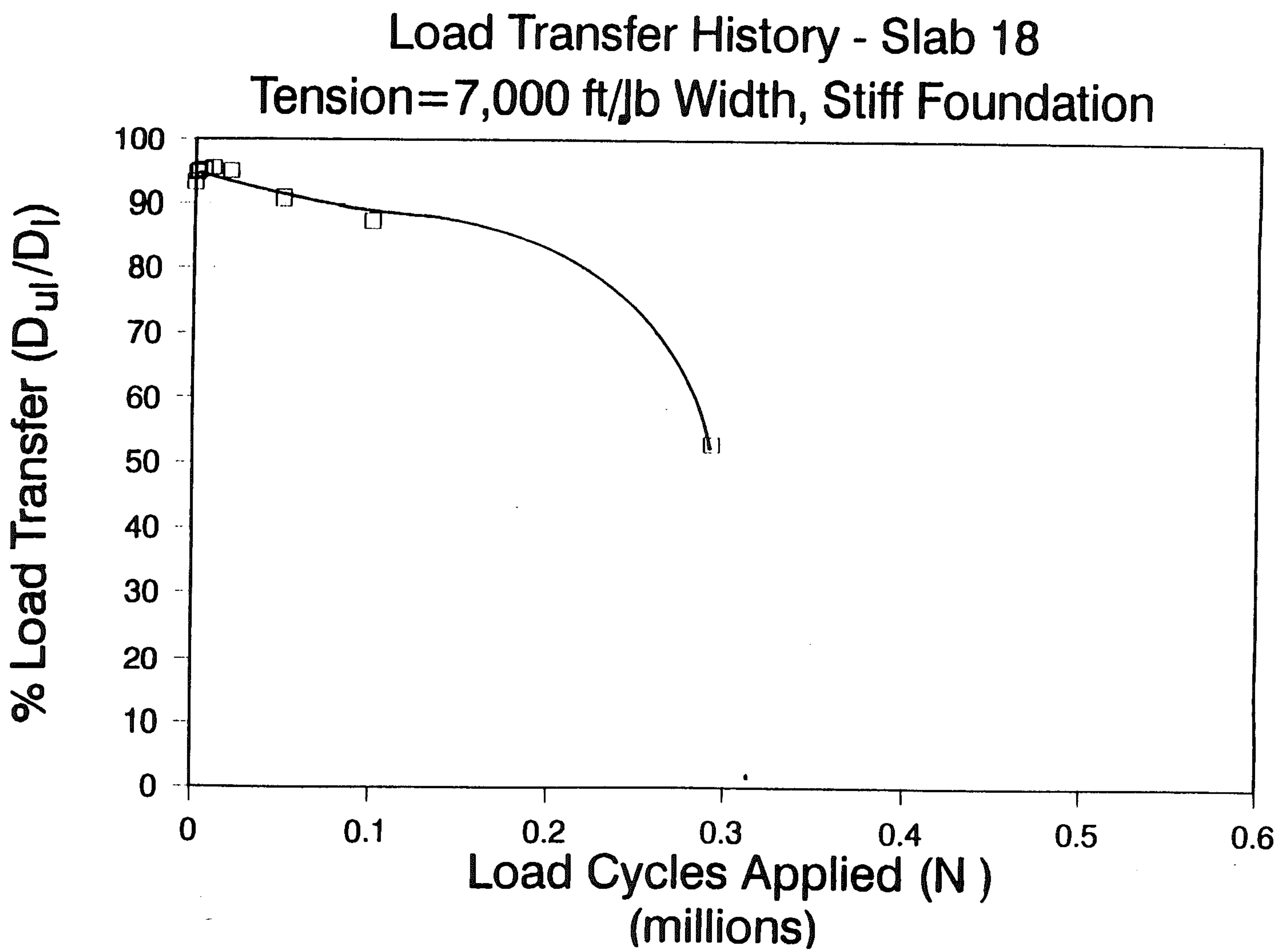

Figure C-18. Load transfer history for slab 18.

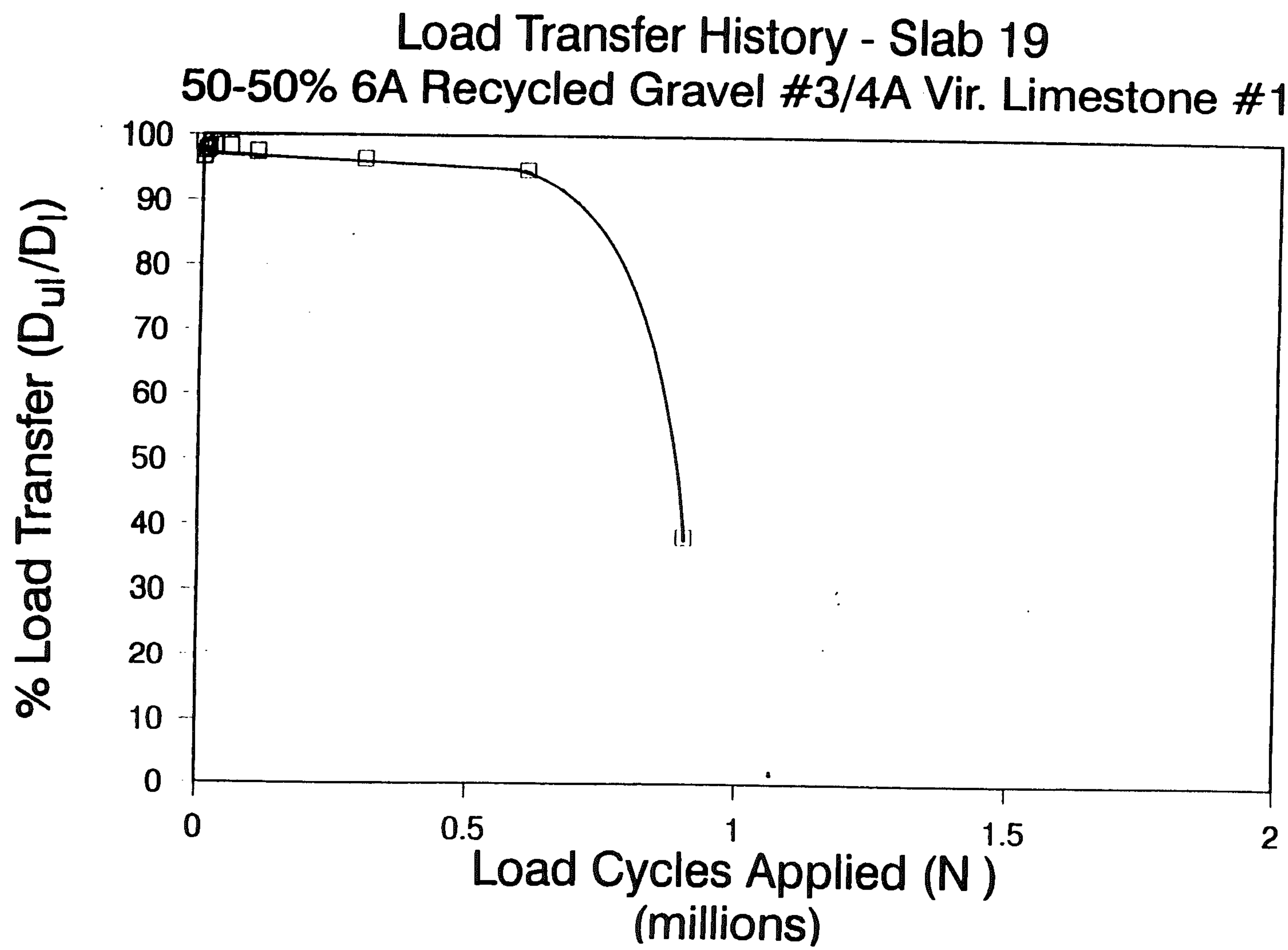

Figure C-19. Load transfer history for slab 19.

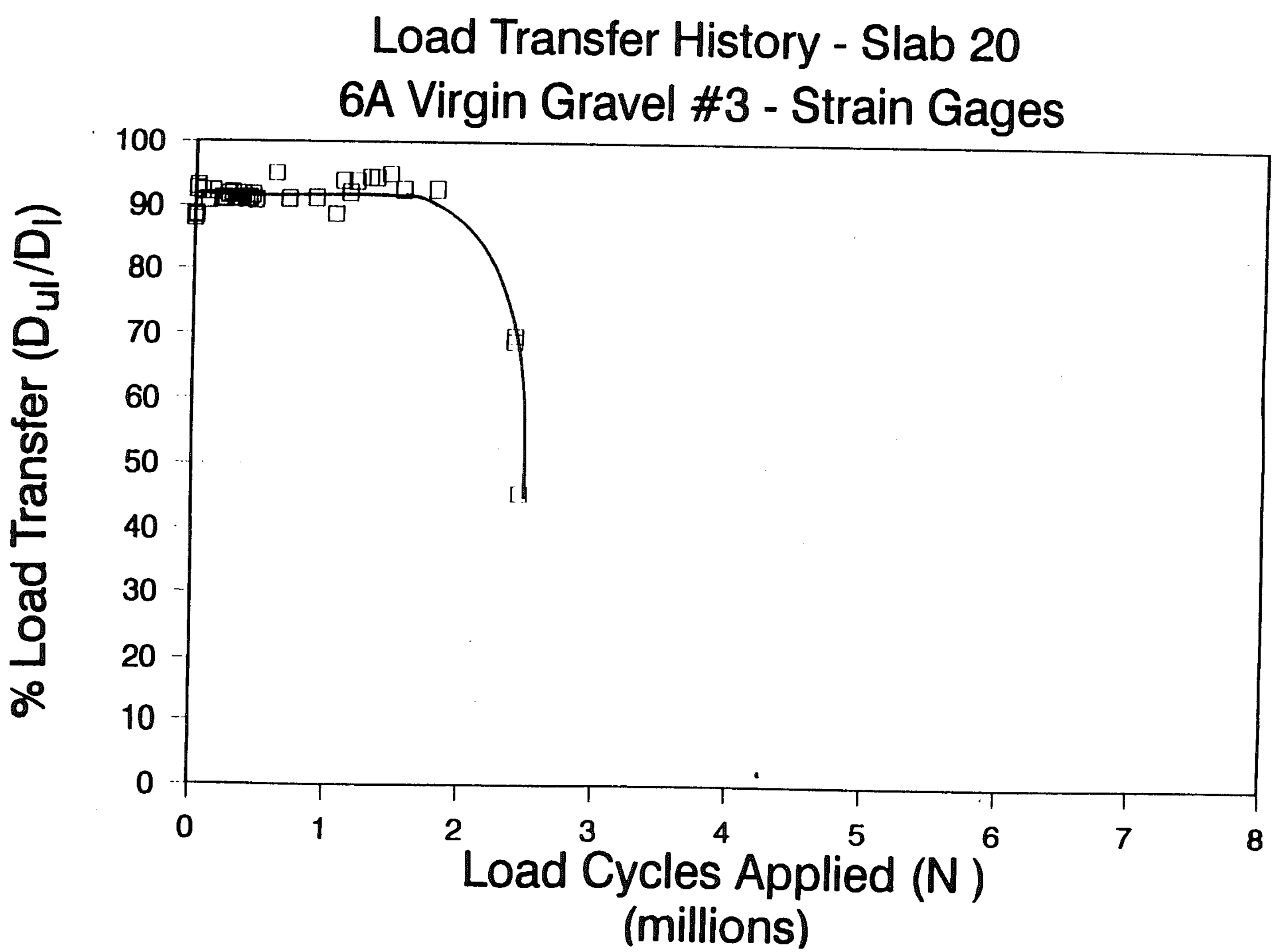

Figure C-20. Load transfer history for slab 20.

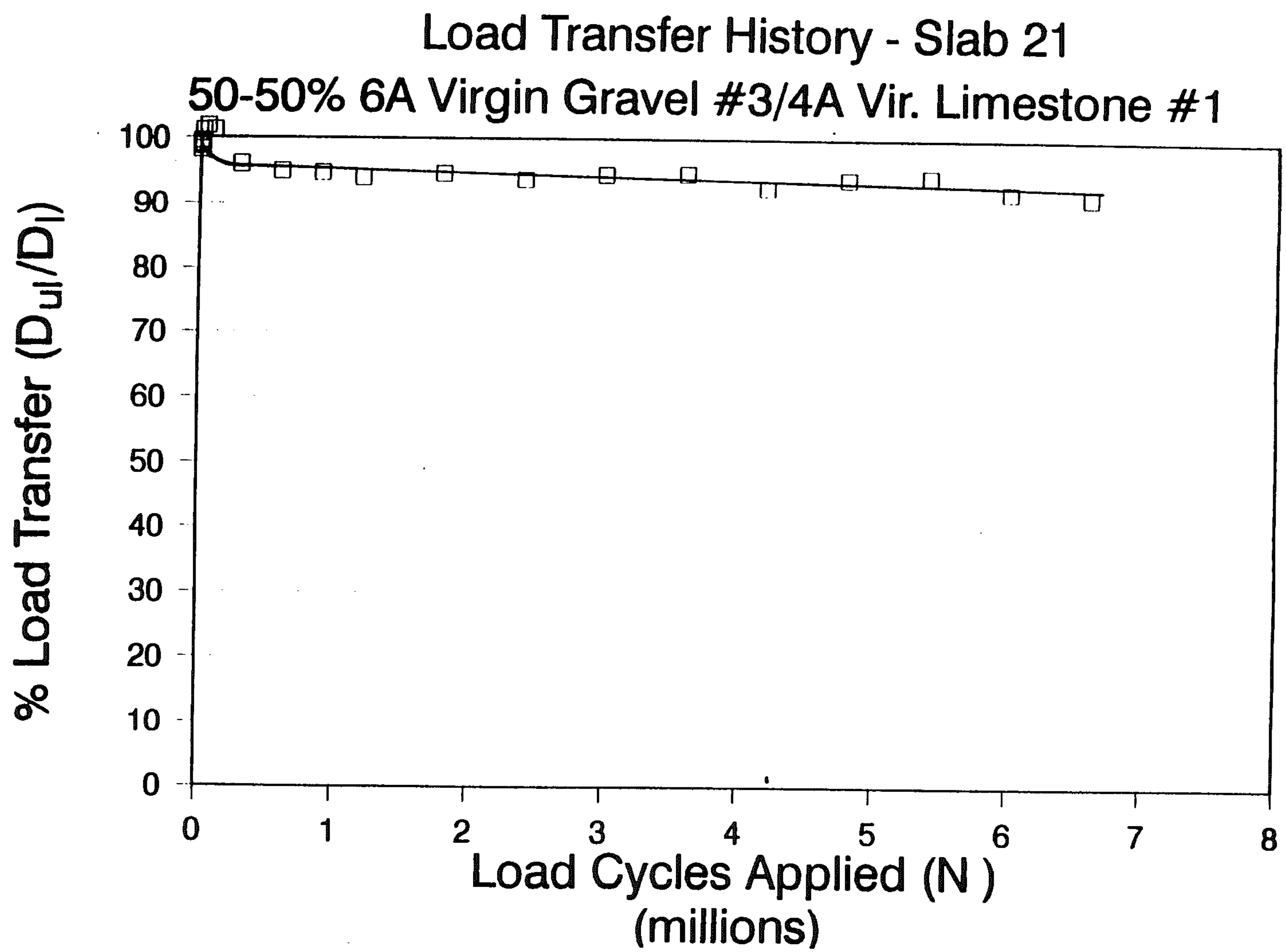

Figure C-21. Load transfer history for slab 21.

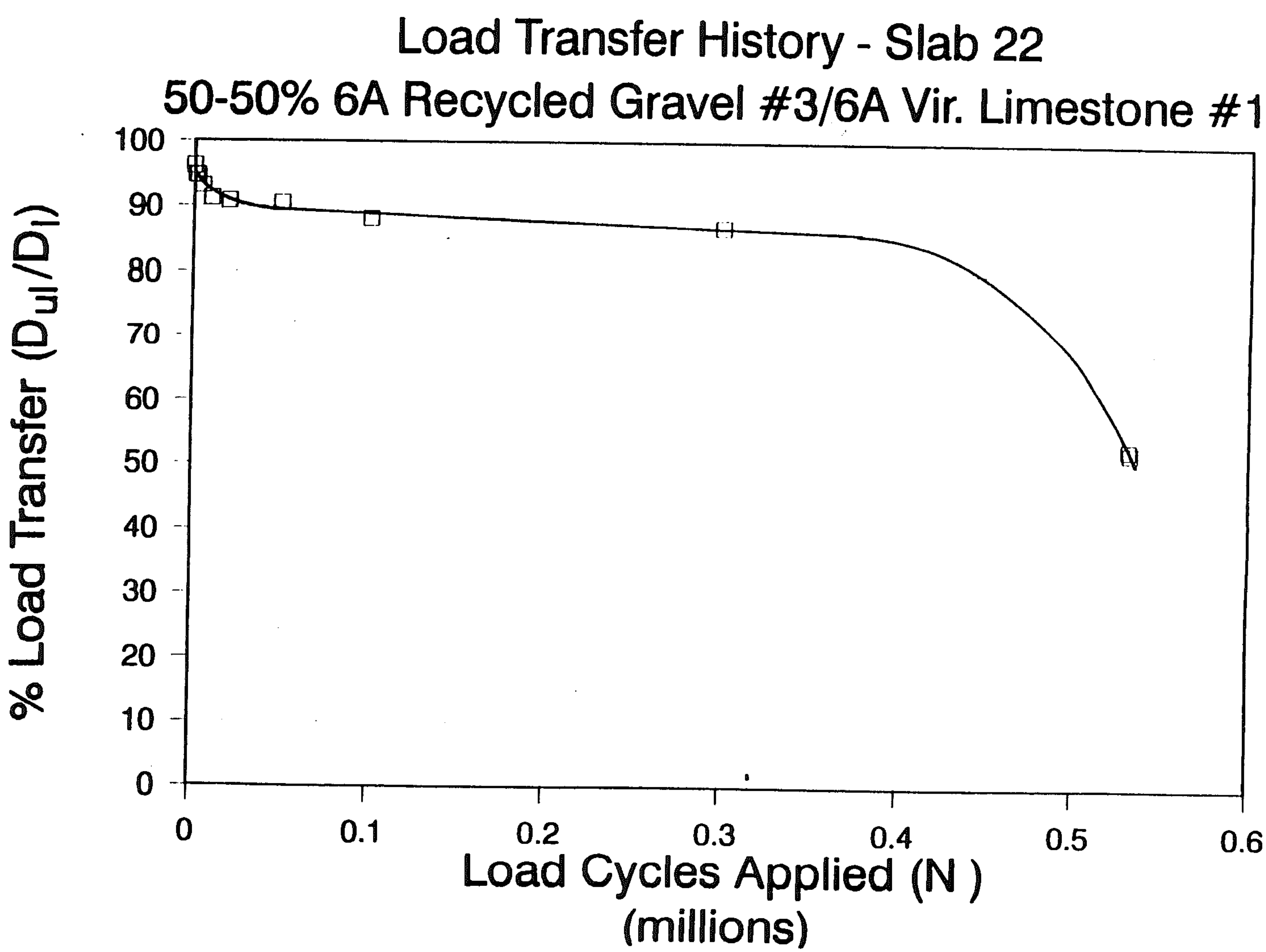

Figure C-22. Load transfer history for slab 22.

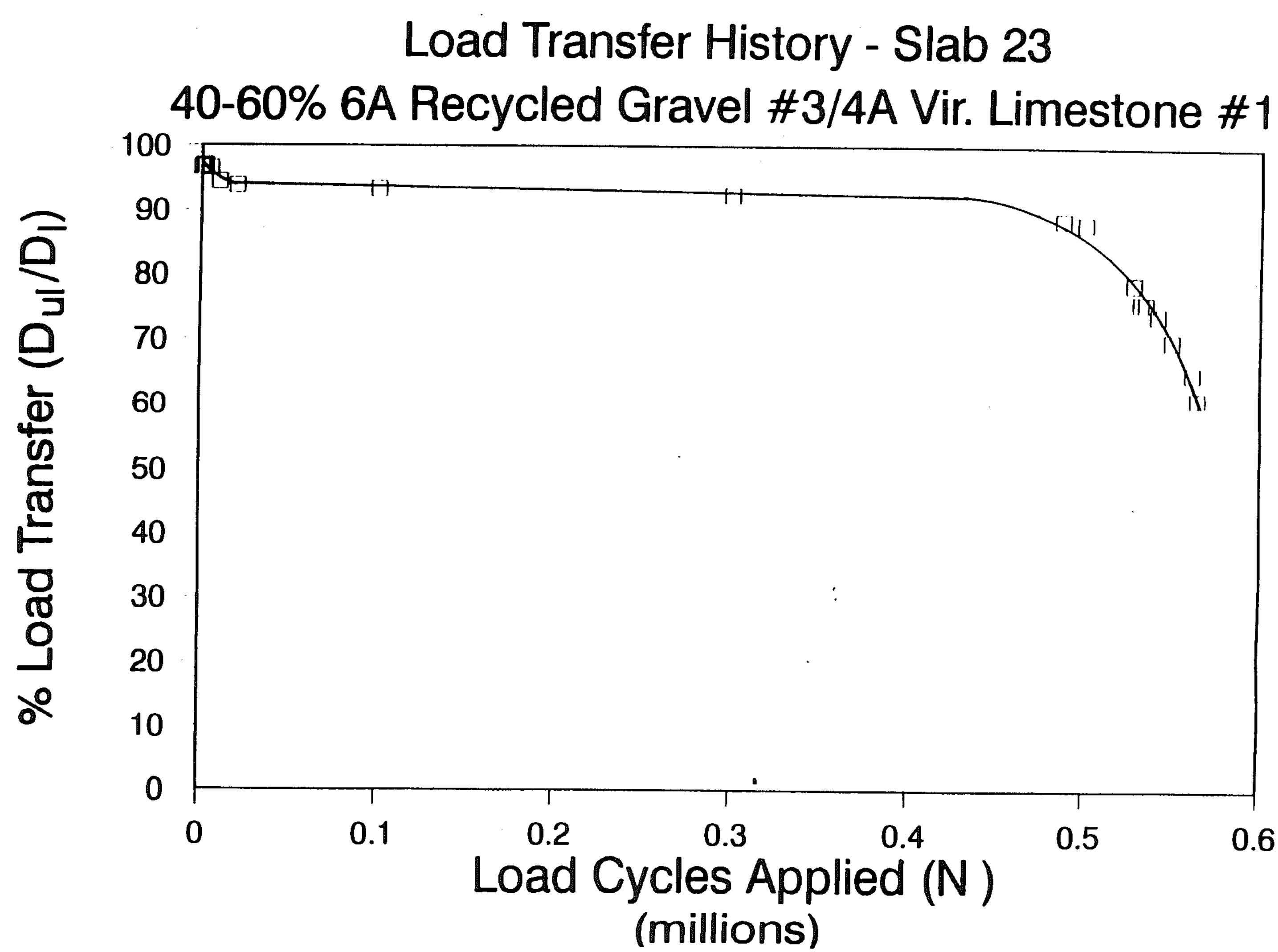

Figure C-23. Load transfer history for slab 23.

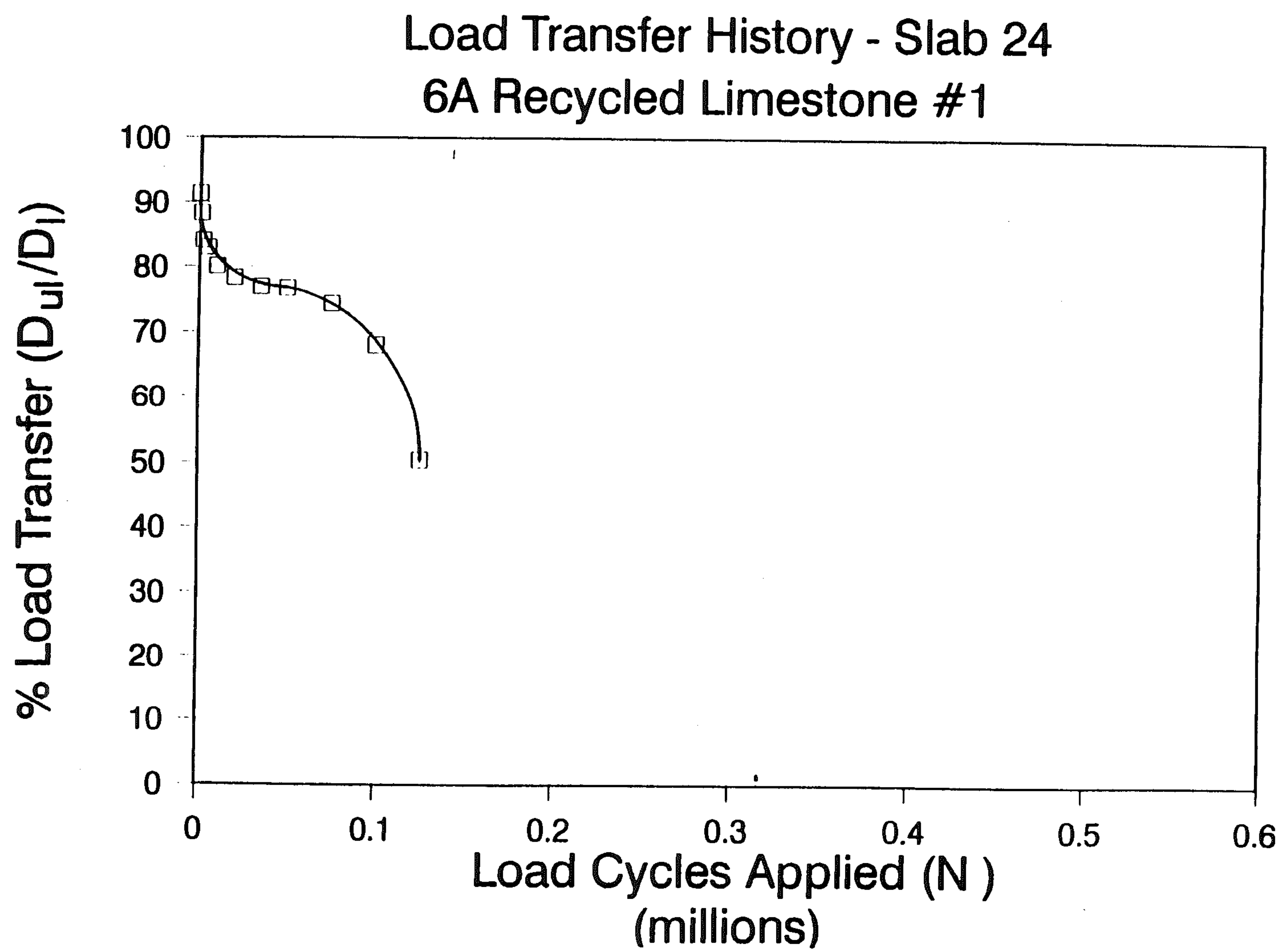

Figure C-24. Load transfer history for slab 24.

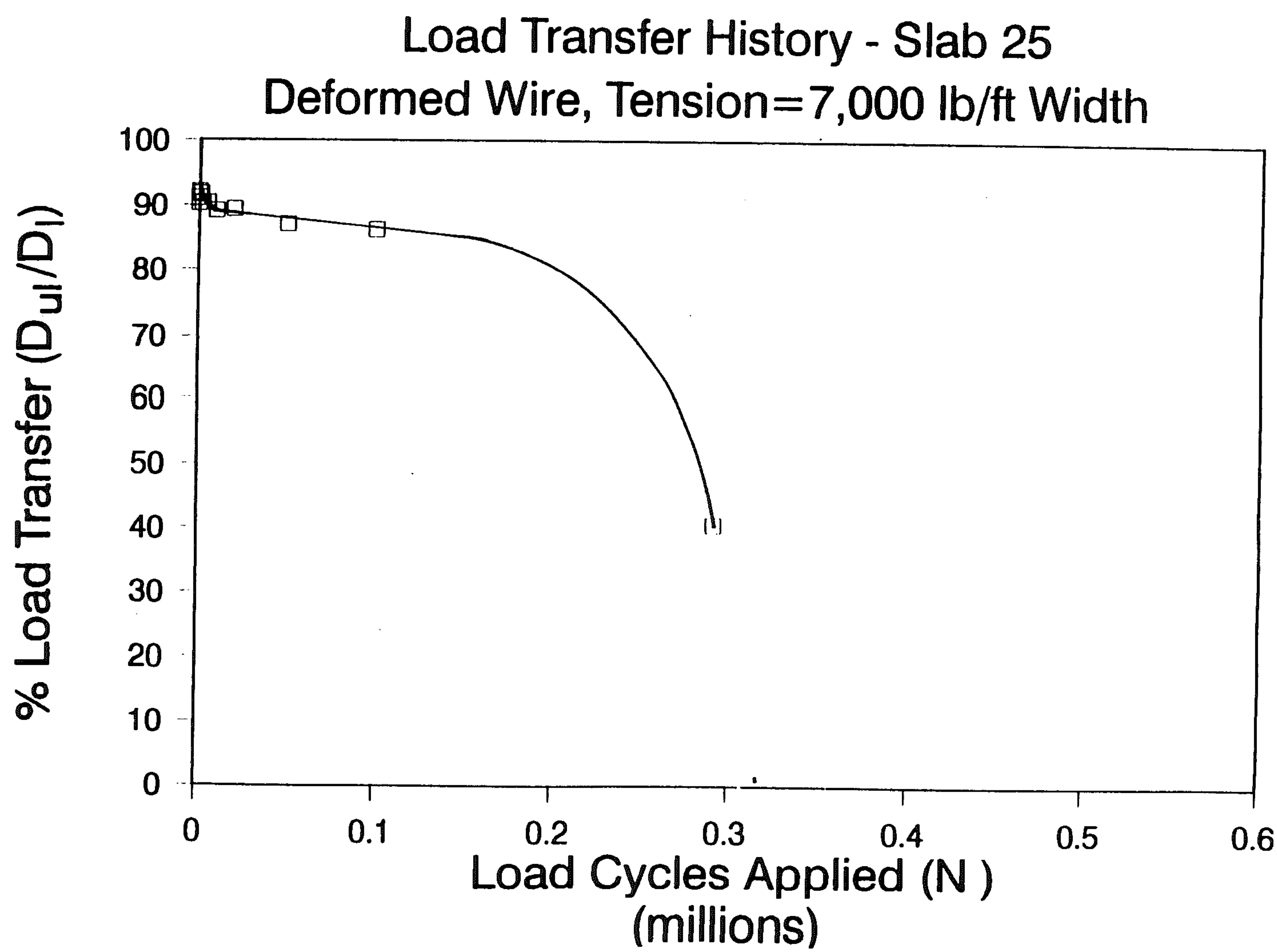

Figure C-25. Load transfer history for slab 25.

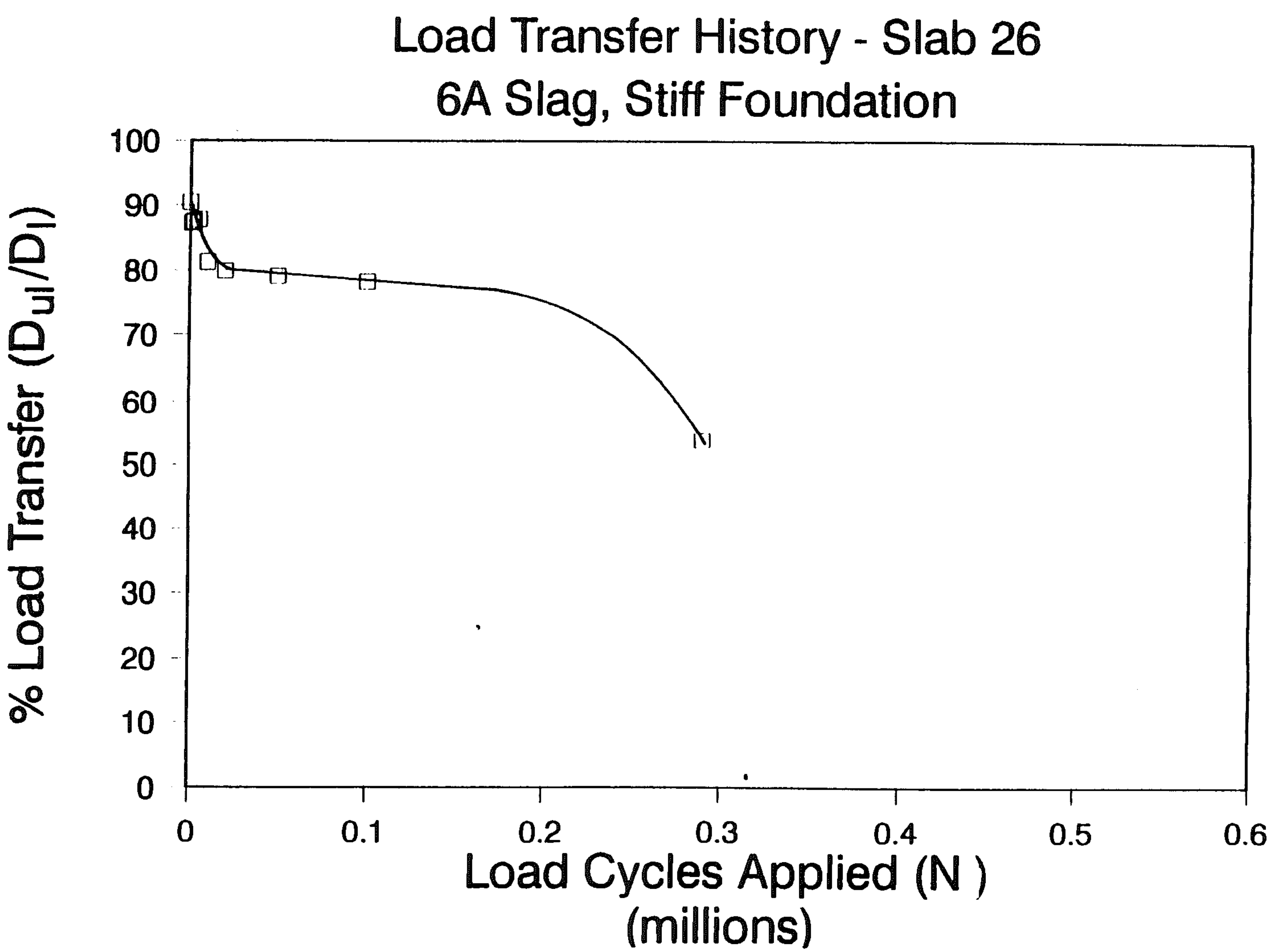

Figure C-26. Load transfer history for slab 26.

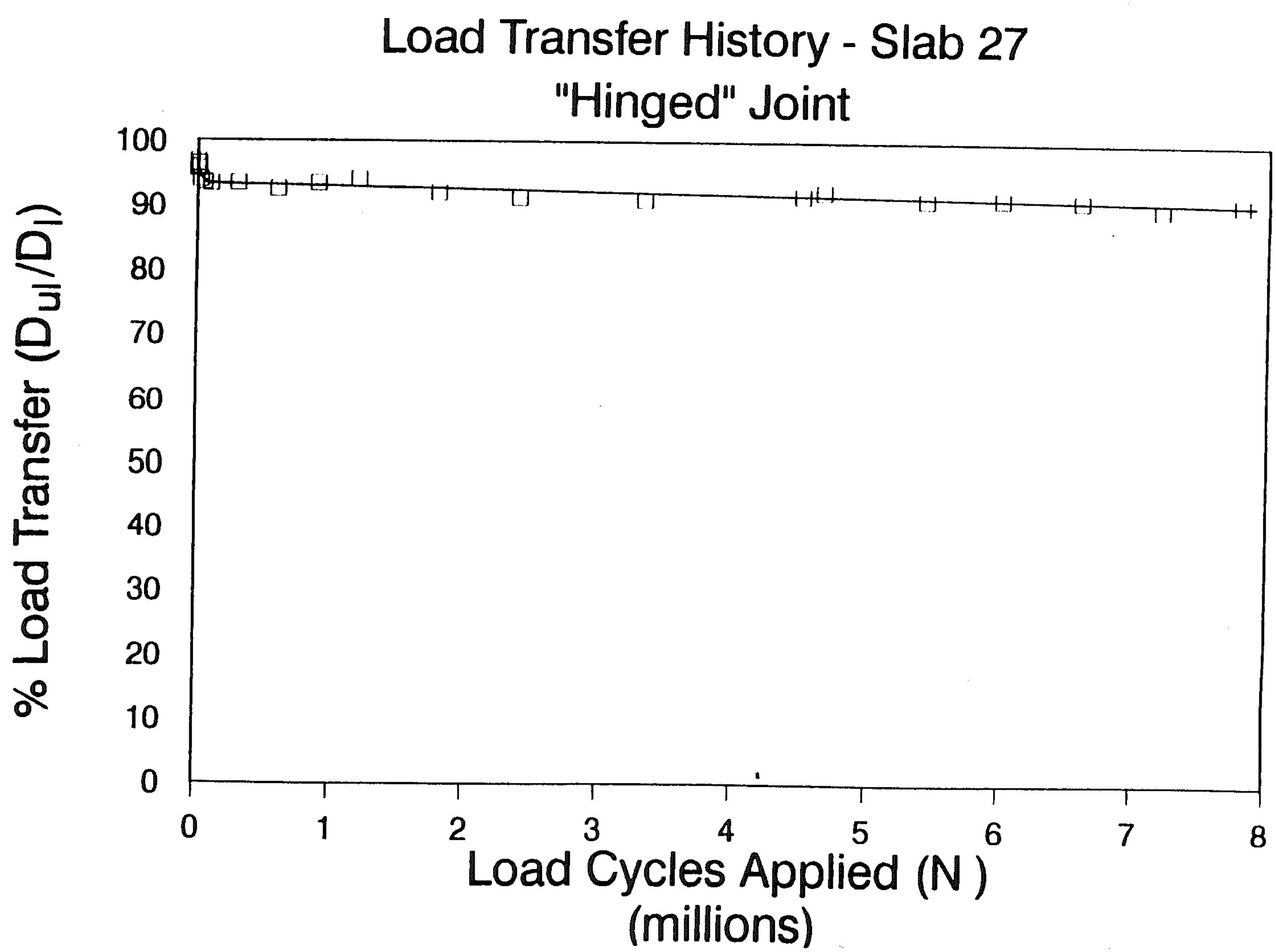

Figure C-27. Load transfer history for slab 27.

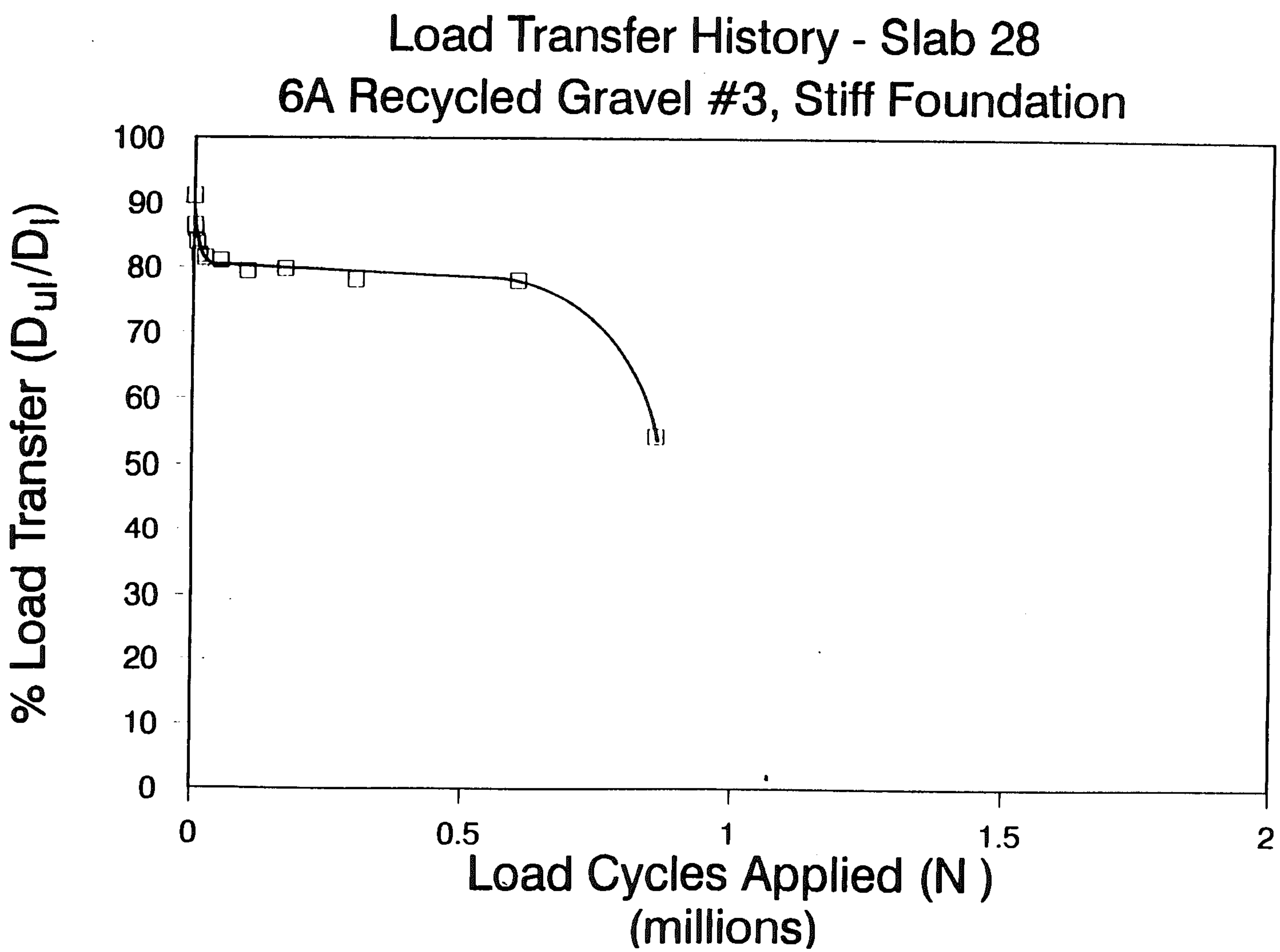

Figure C-28. Load transfer history for slab 28.

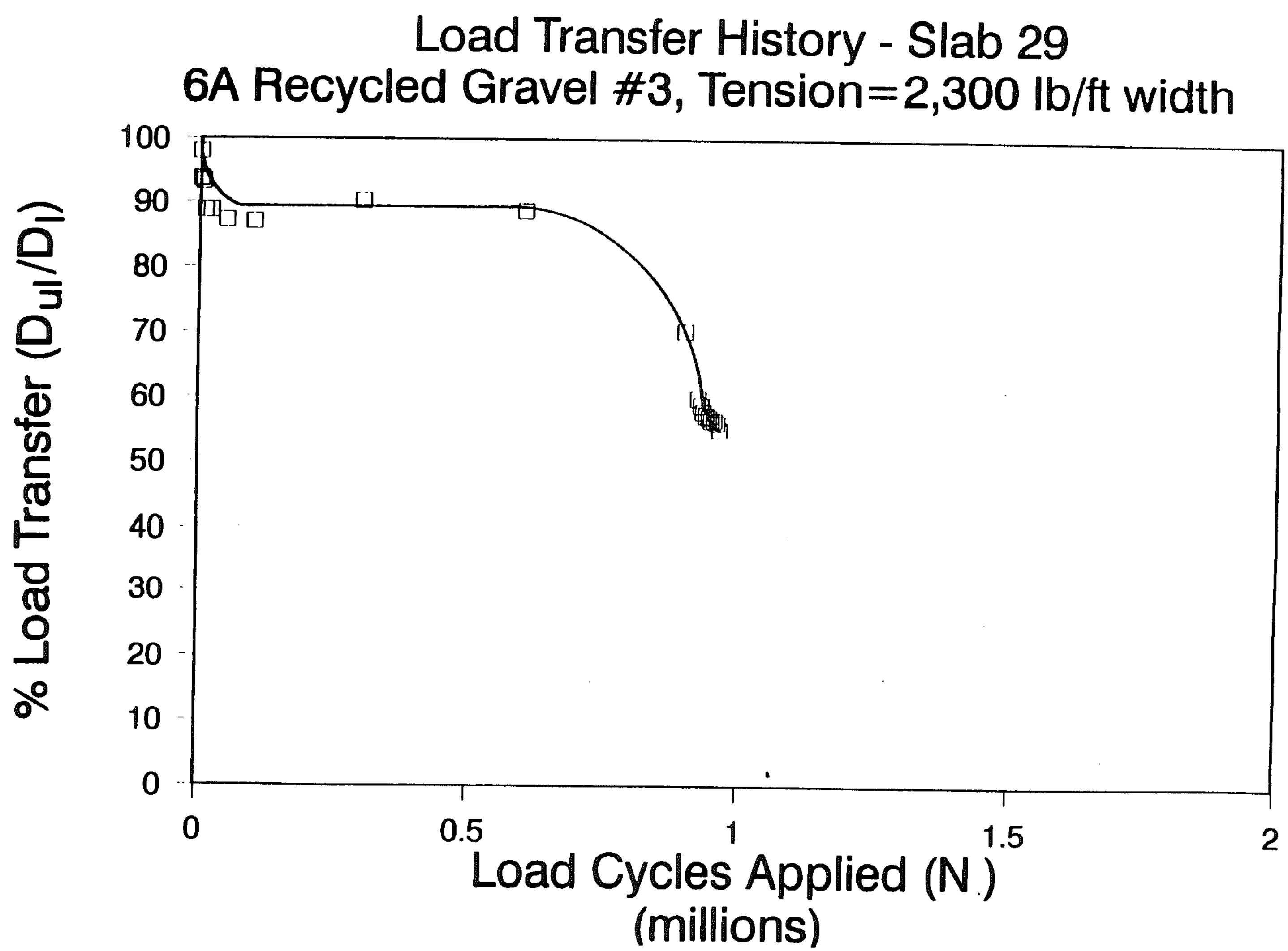

Figure C-29. Load transfer history for slab 29.

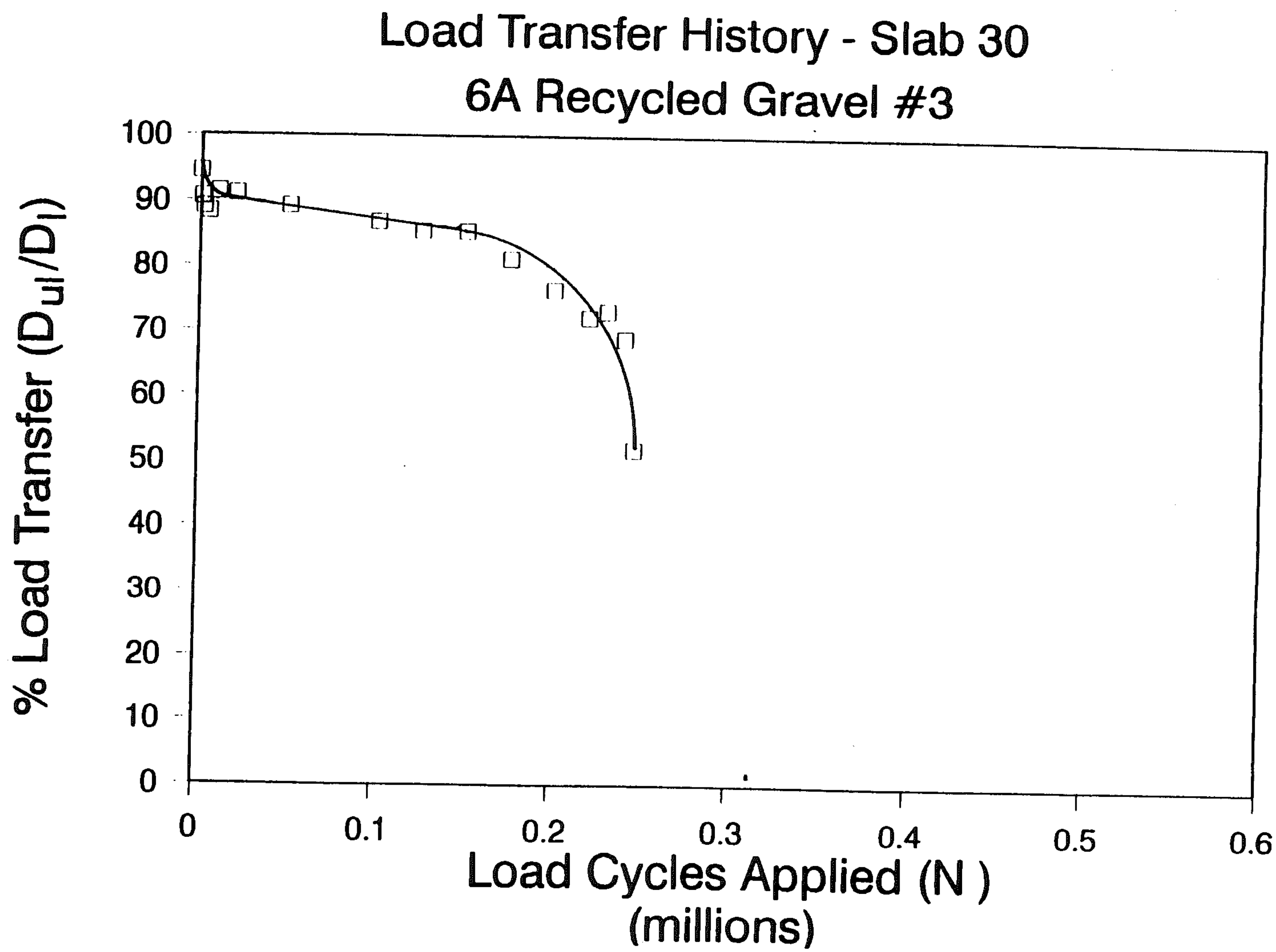

Figure C-30. Load transfer history for slab 30.

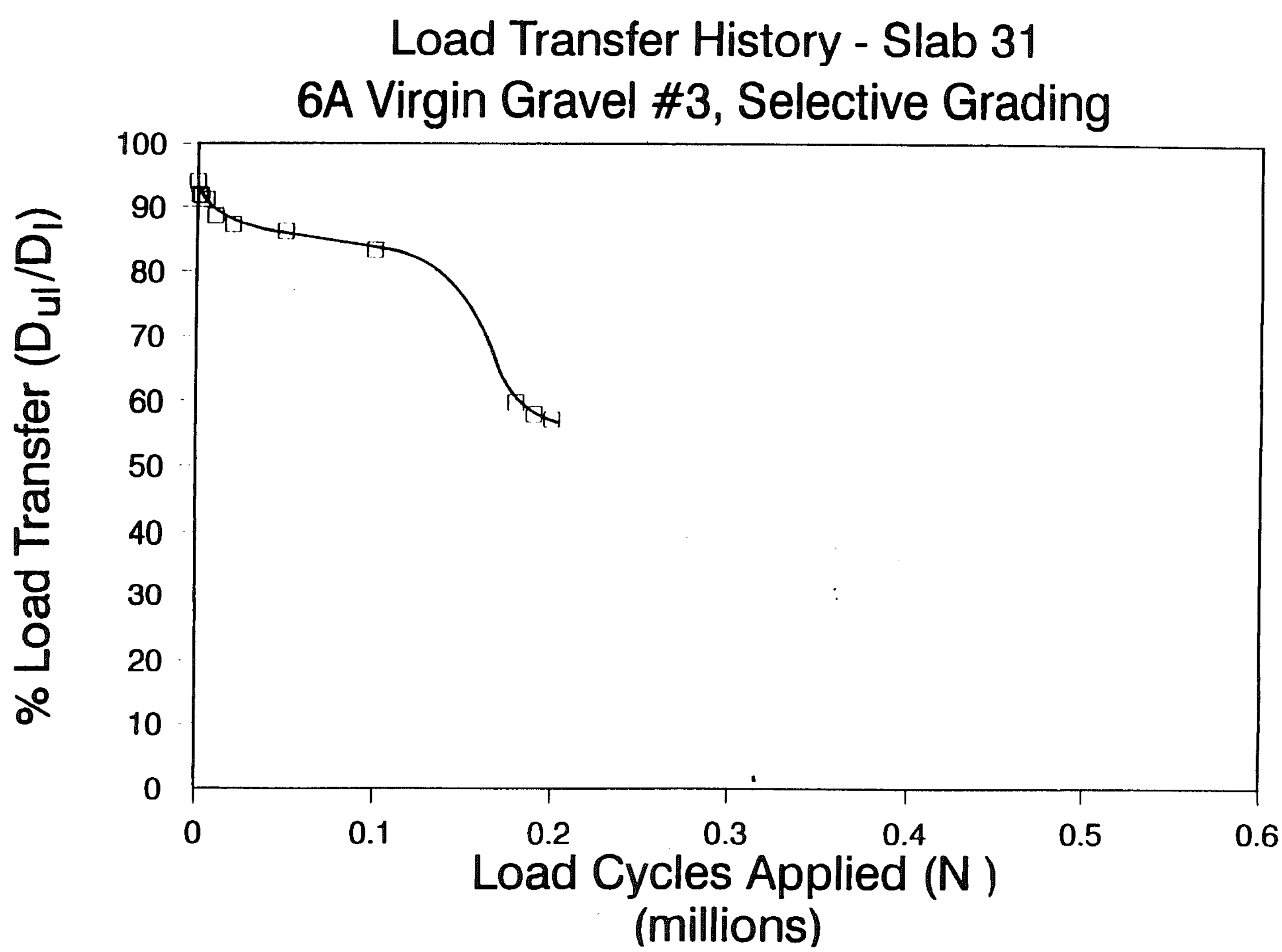

Figure C-31. Load transfer history for slab 31.

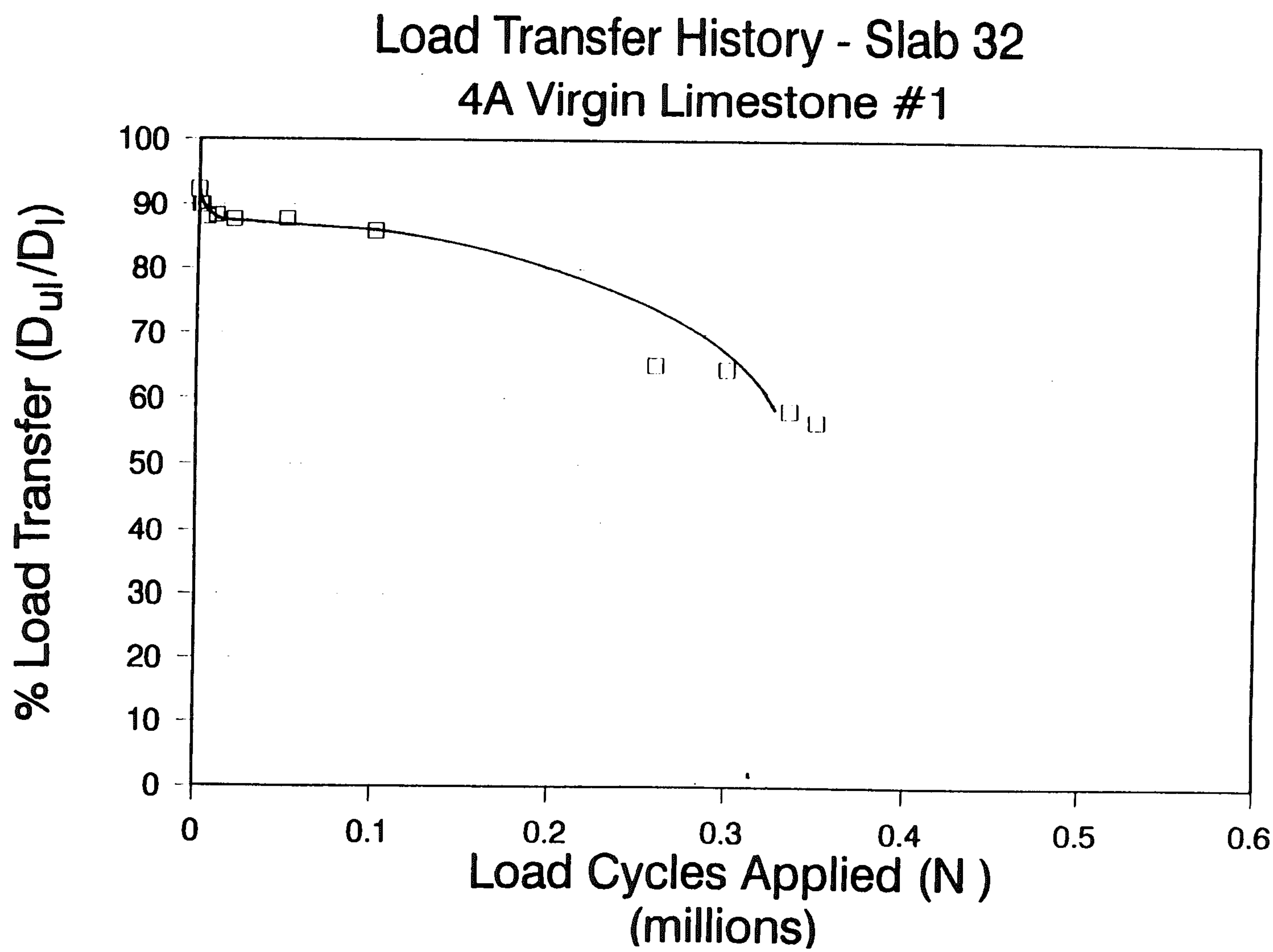

Figure C-32. Load transfer history for slab 32.

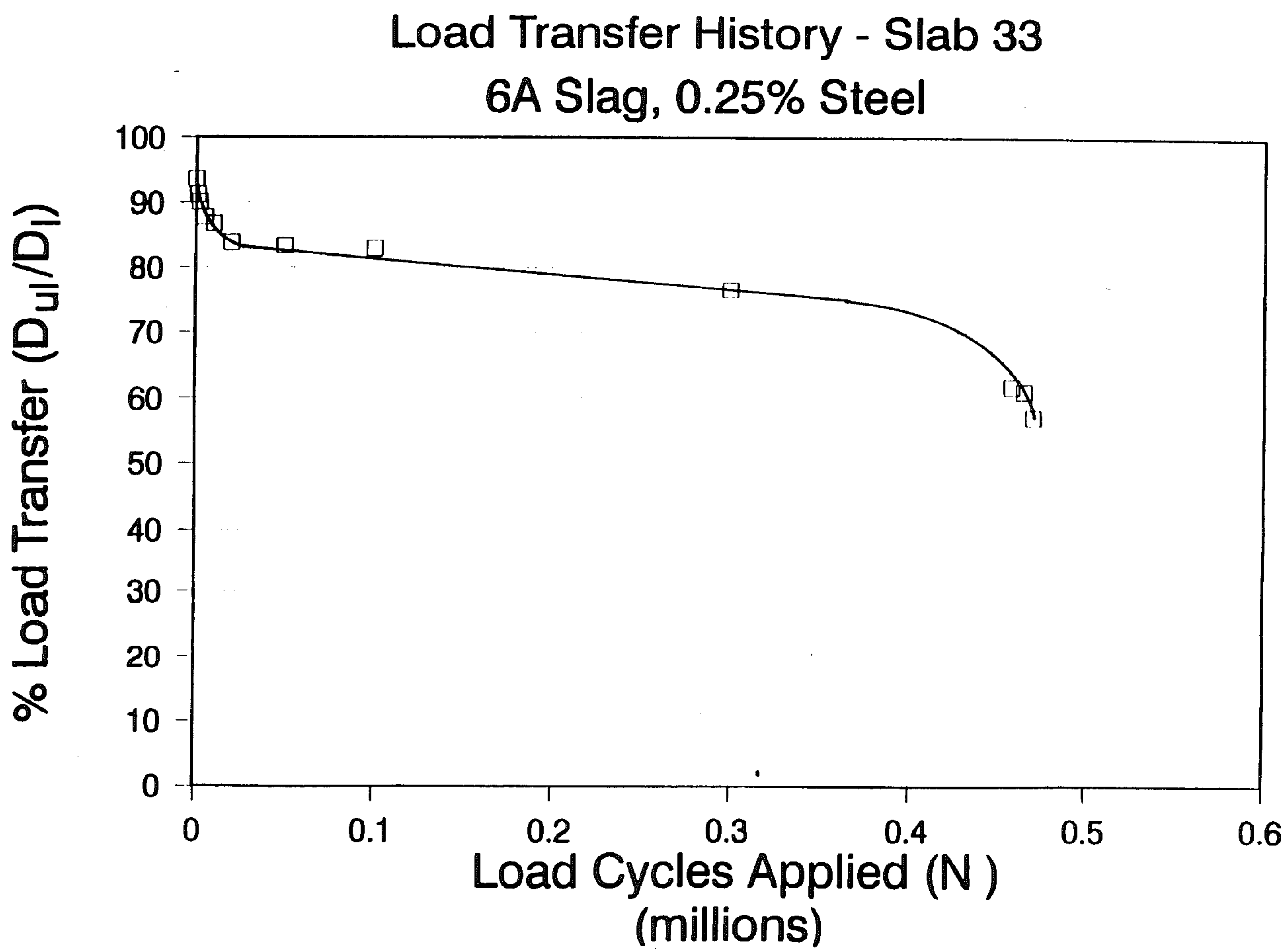

Figure C-33. Load transfer history for slab 33.

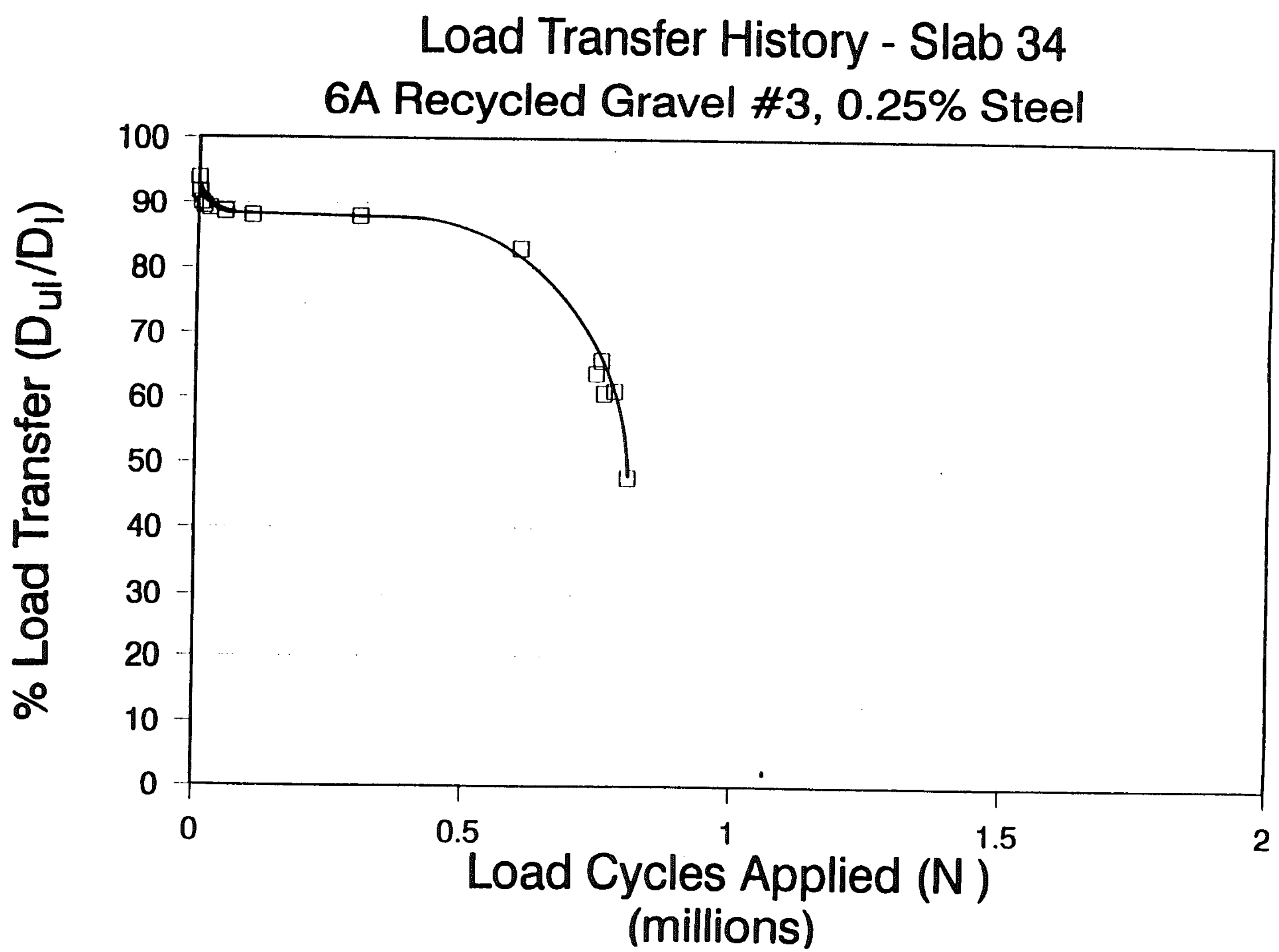

Figure C-34. Load transfer history for slab 34.

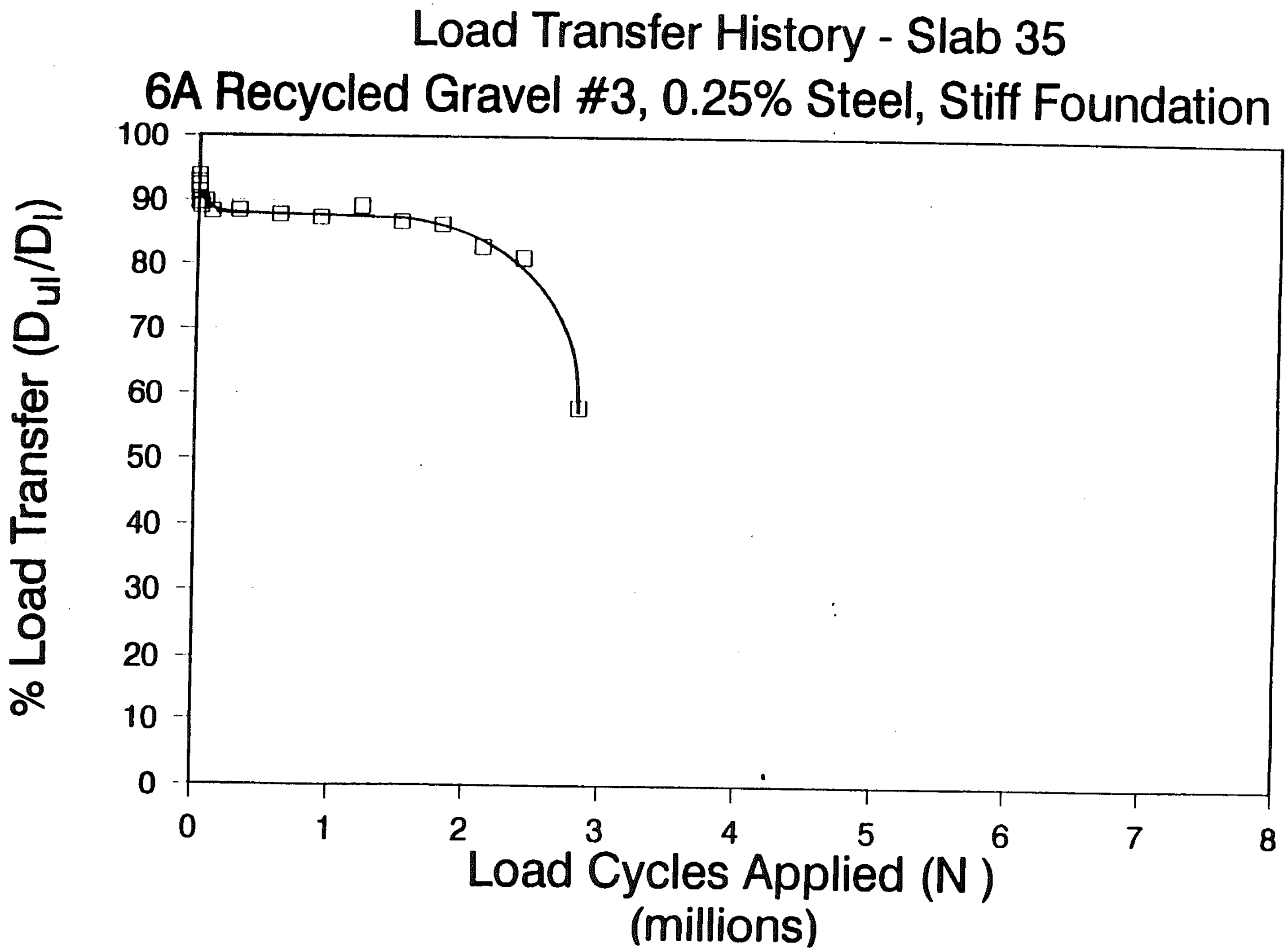

Figure C-35. Load transfer history for slab 35.

APPENDIX D

CRACK WIDTH HISTORIES OF TEST SPECIMENS AND LOAD TRANSFER VERSUS CRACK WIDTH OF TEST SPECIMENS

List of conversions for Appendix D

1 in = 25.4 mm

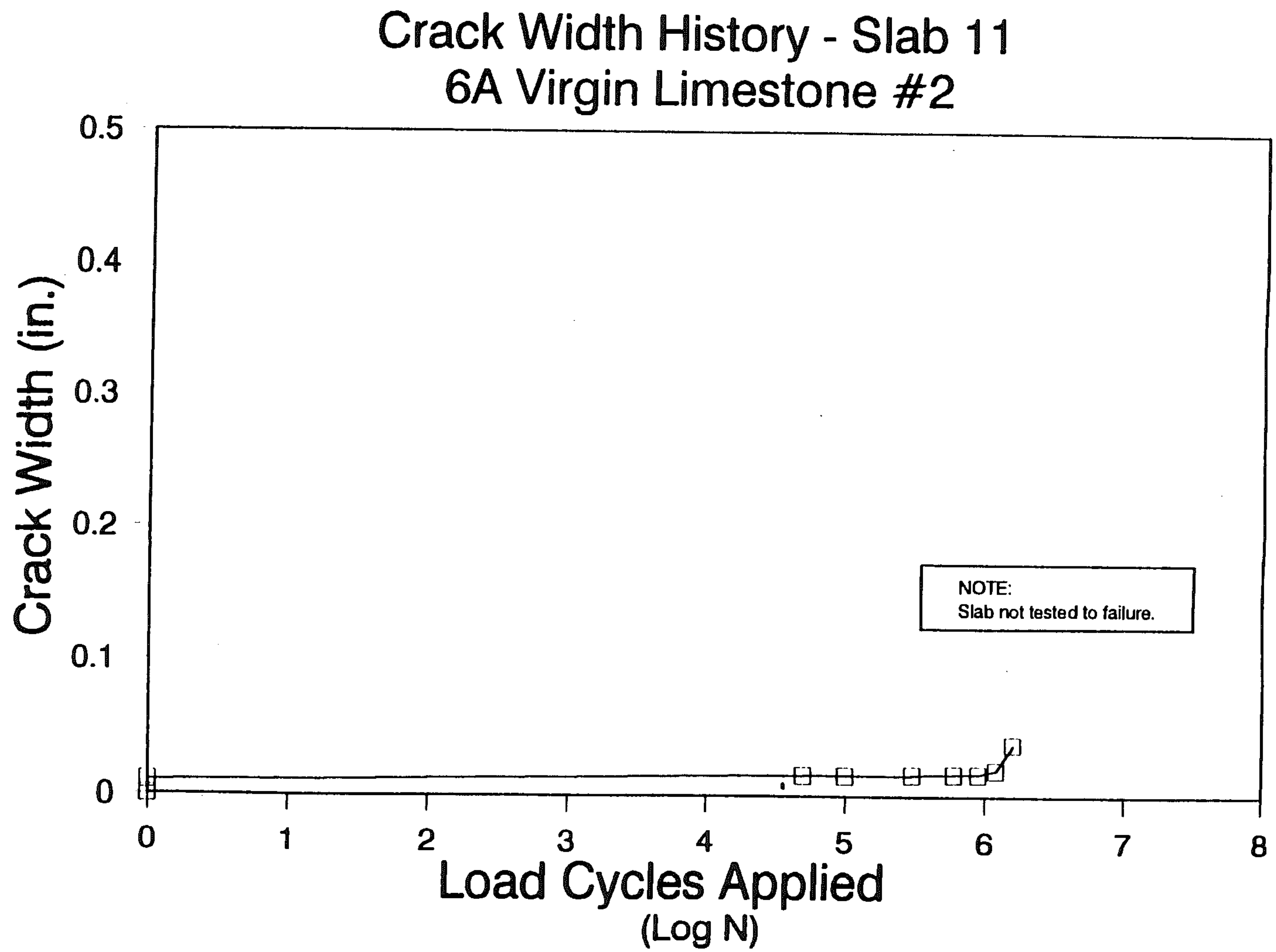

Figure D-1. Crack width history for slab 11.

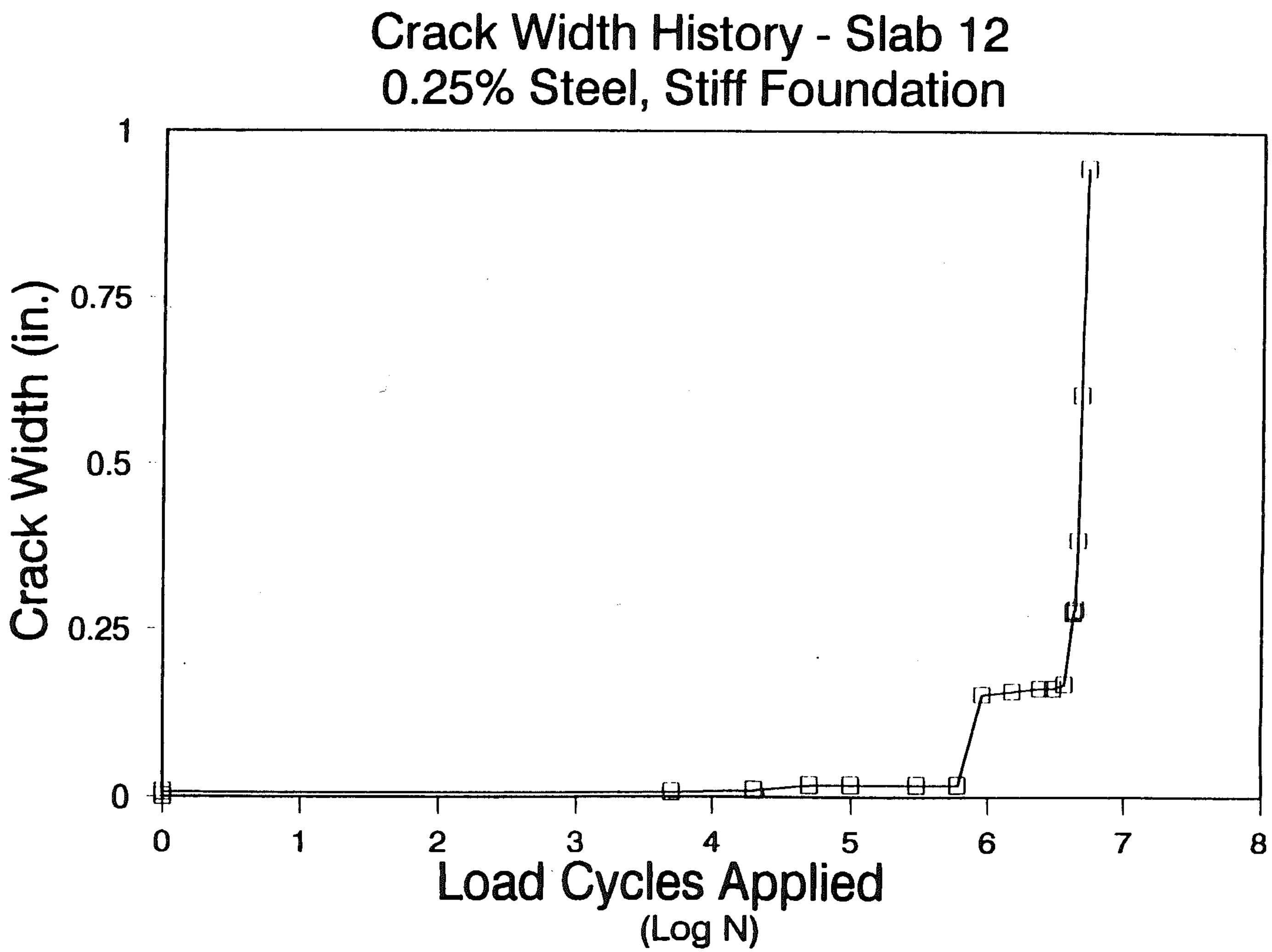

Figure D-2. Crack width history for slab 12.

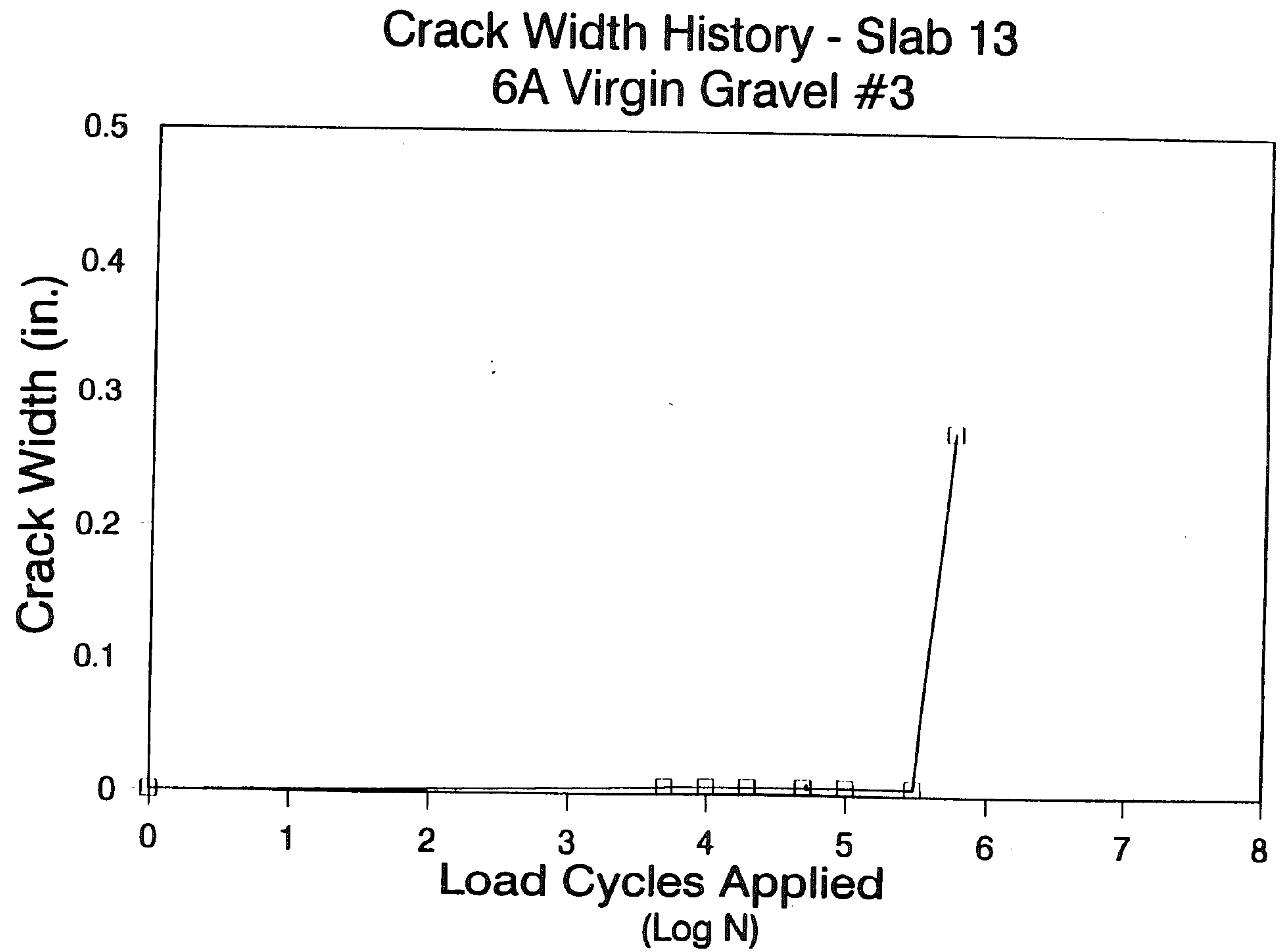

Figure D-3. Crack width history for slab 13.

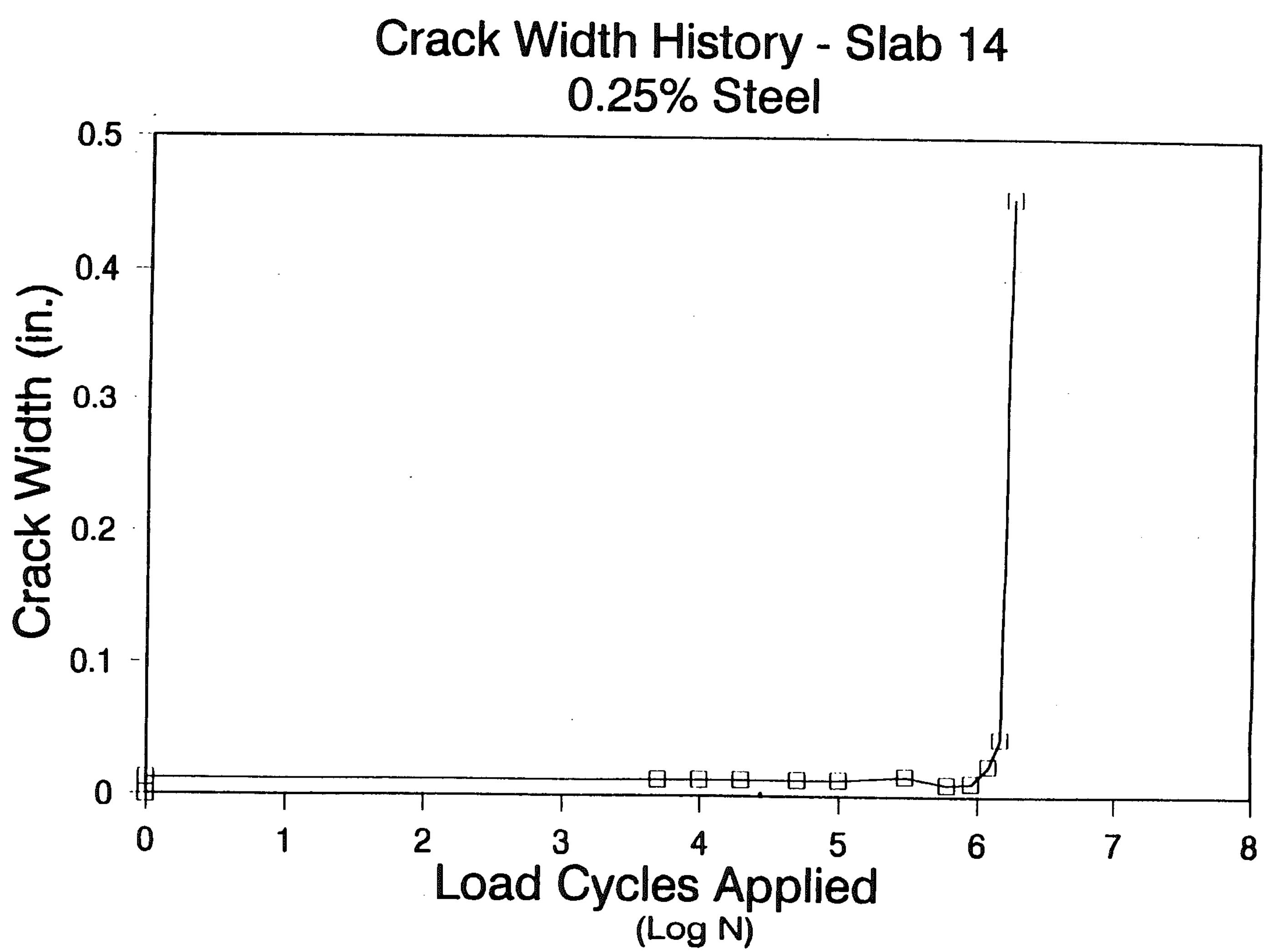

Figure D-4. Crack width history for slab 14.

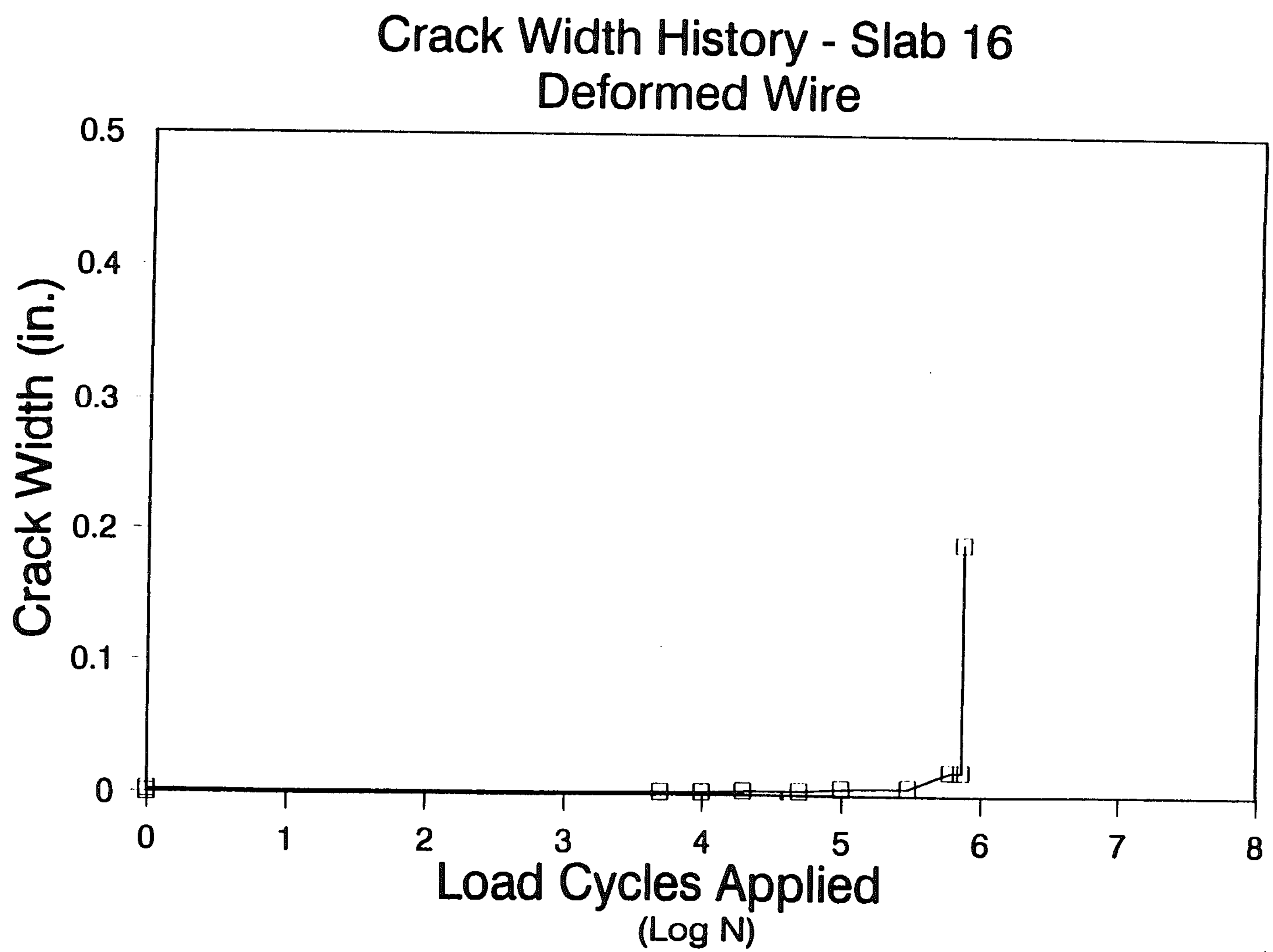

Figure D-6. Crack width history for slab 16.

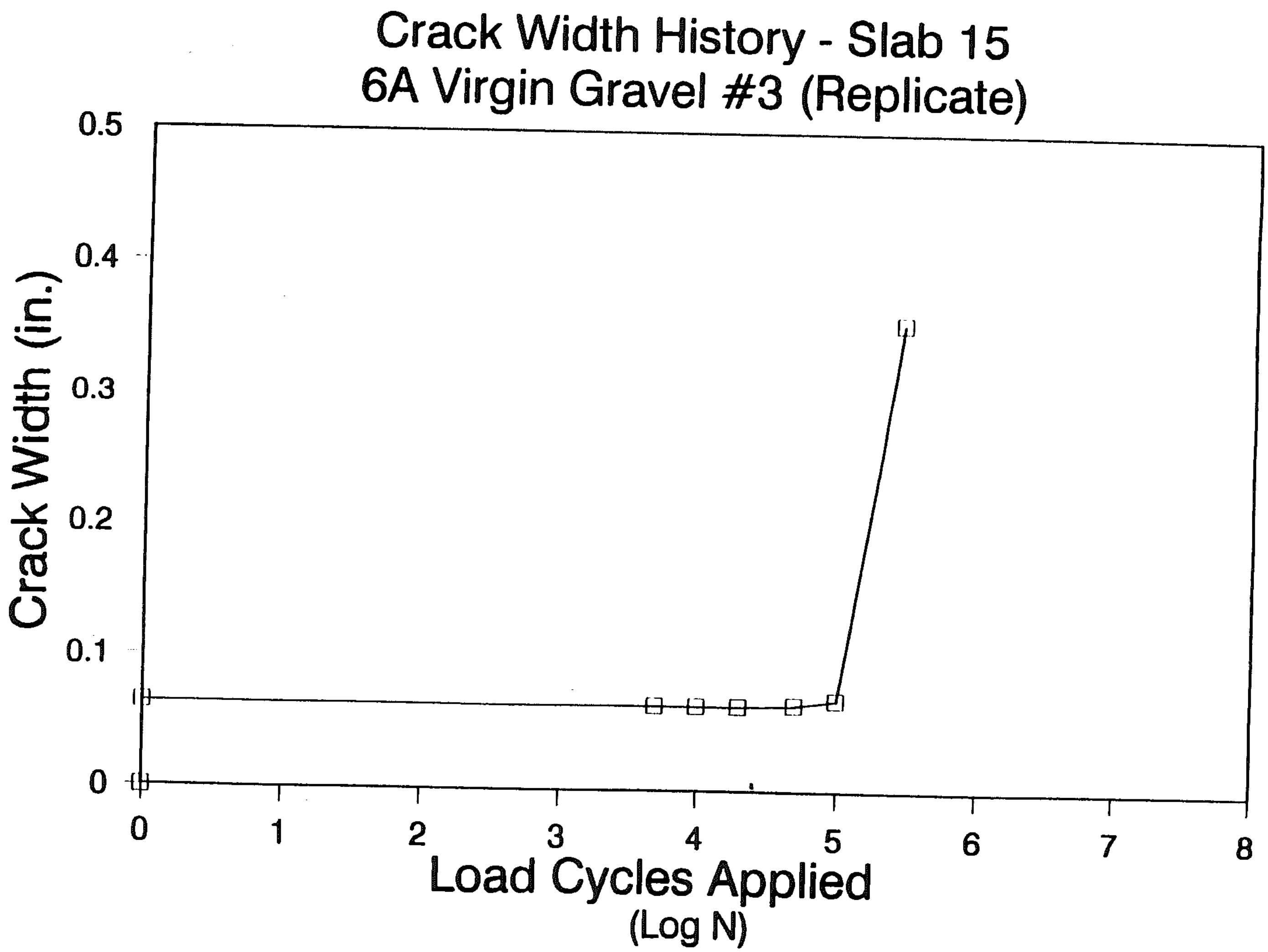

Figure D-5. Crack width history for slab 15.

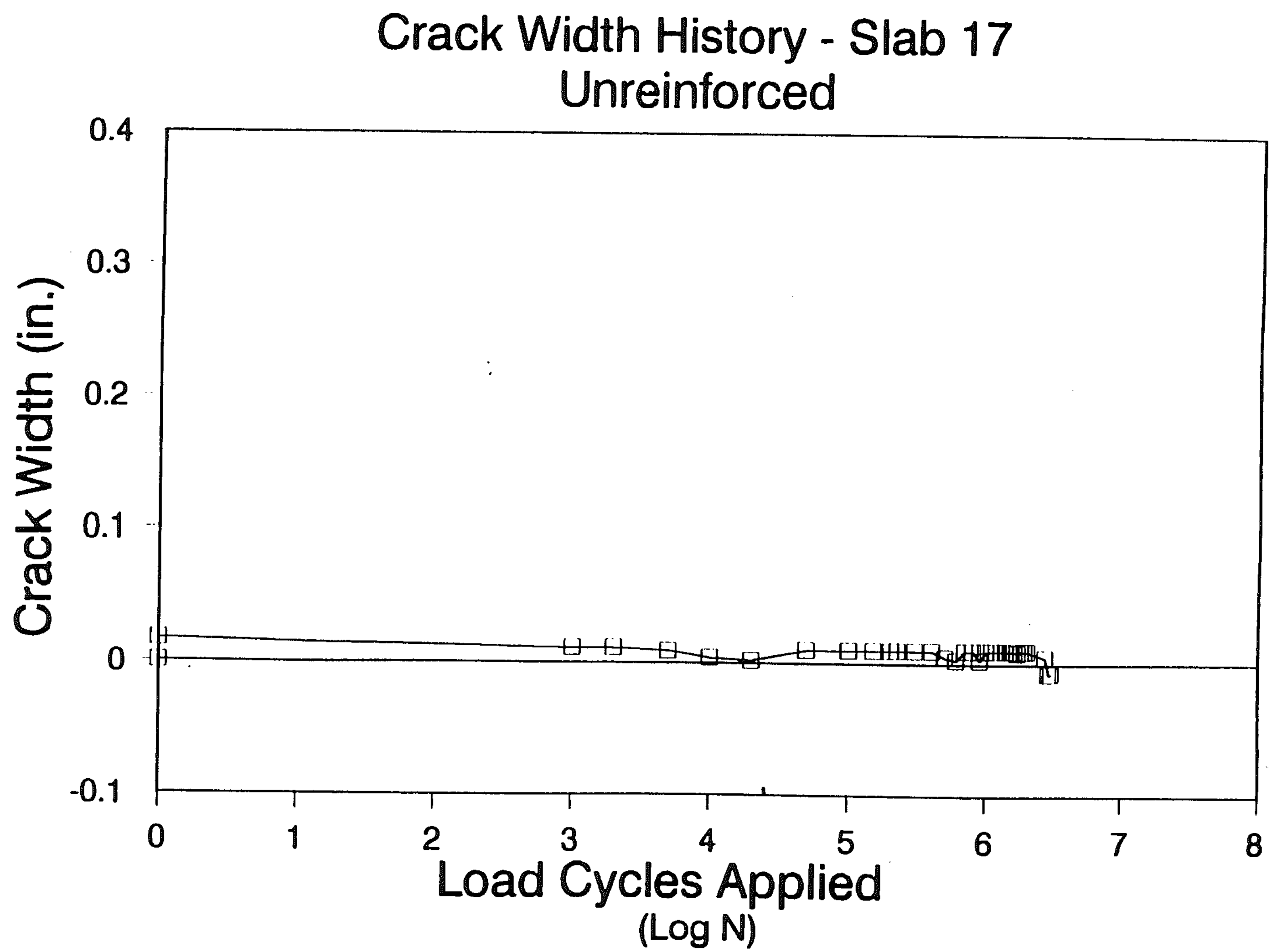

Figure D-7. Crack width history for slab 17.

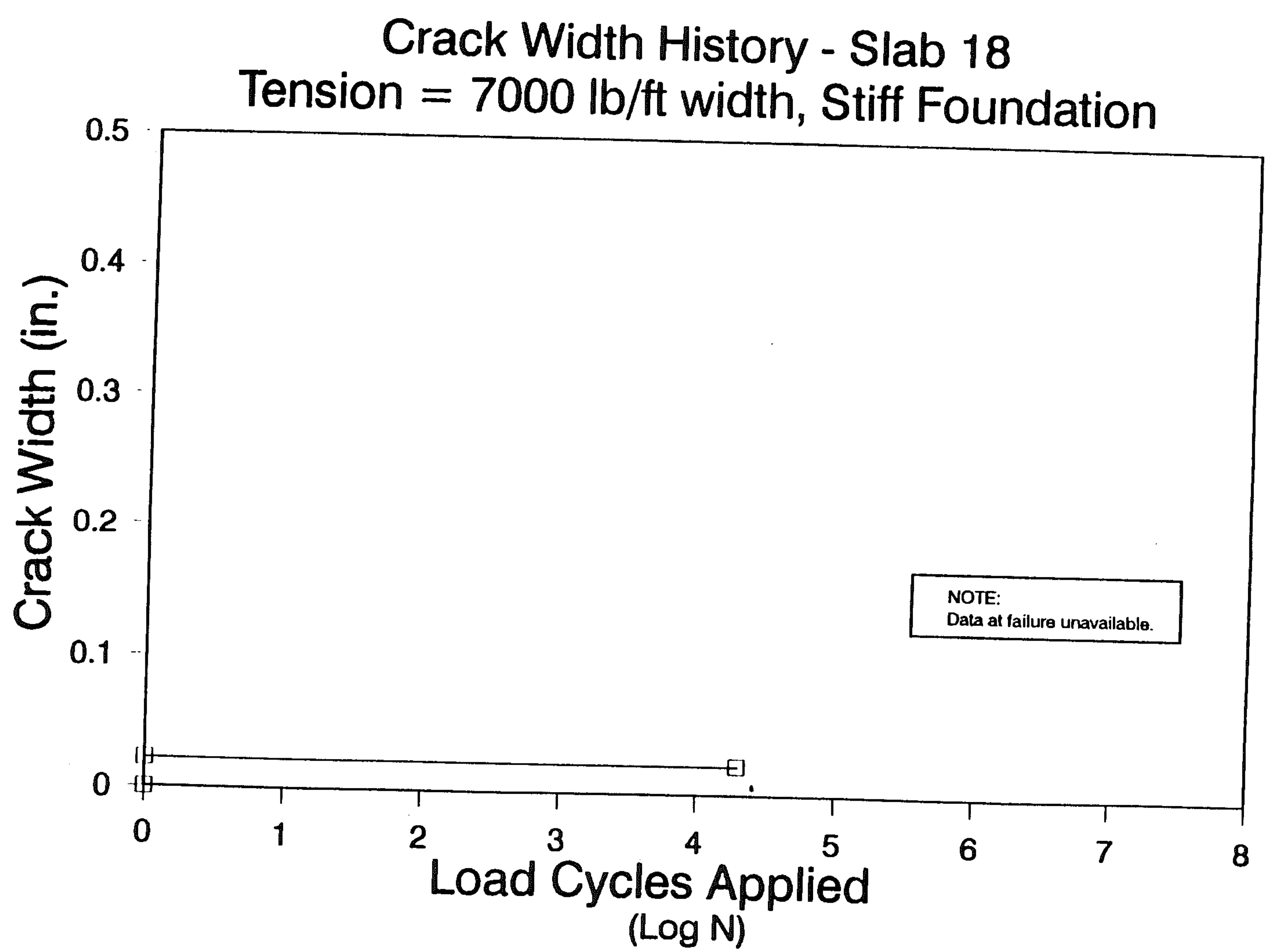

Figure D-8. Crack width history for slab 18.

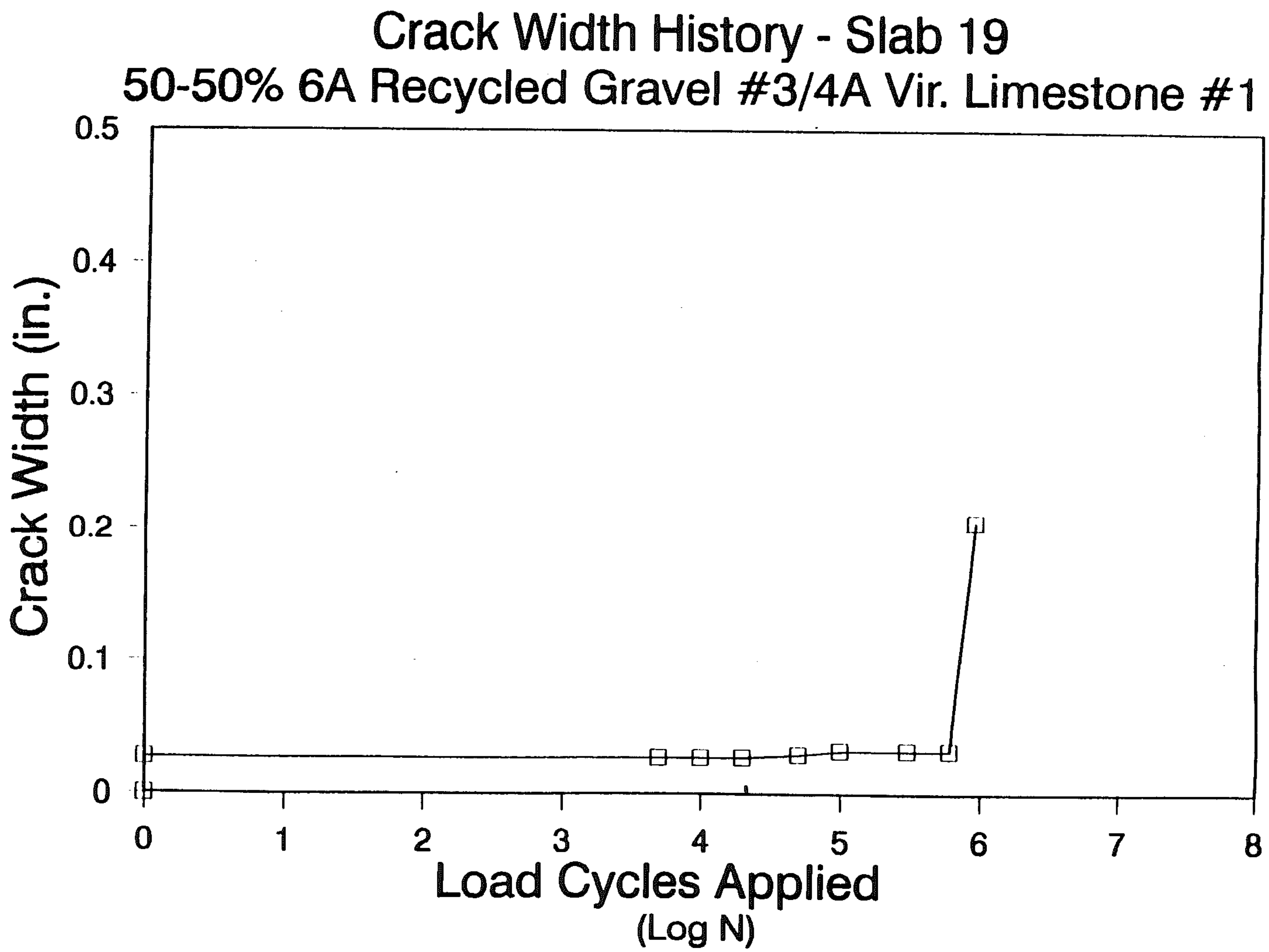

Figure D-9. Crack width history for slab 19.

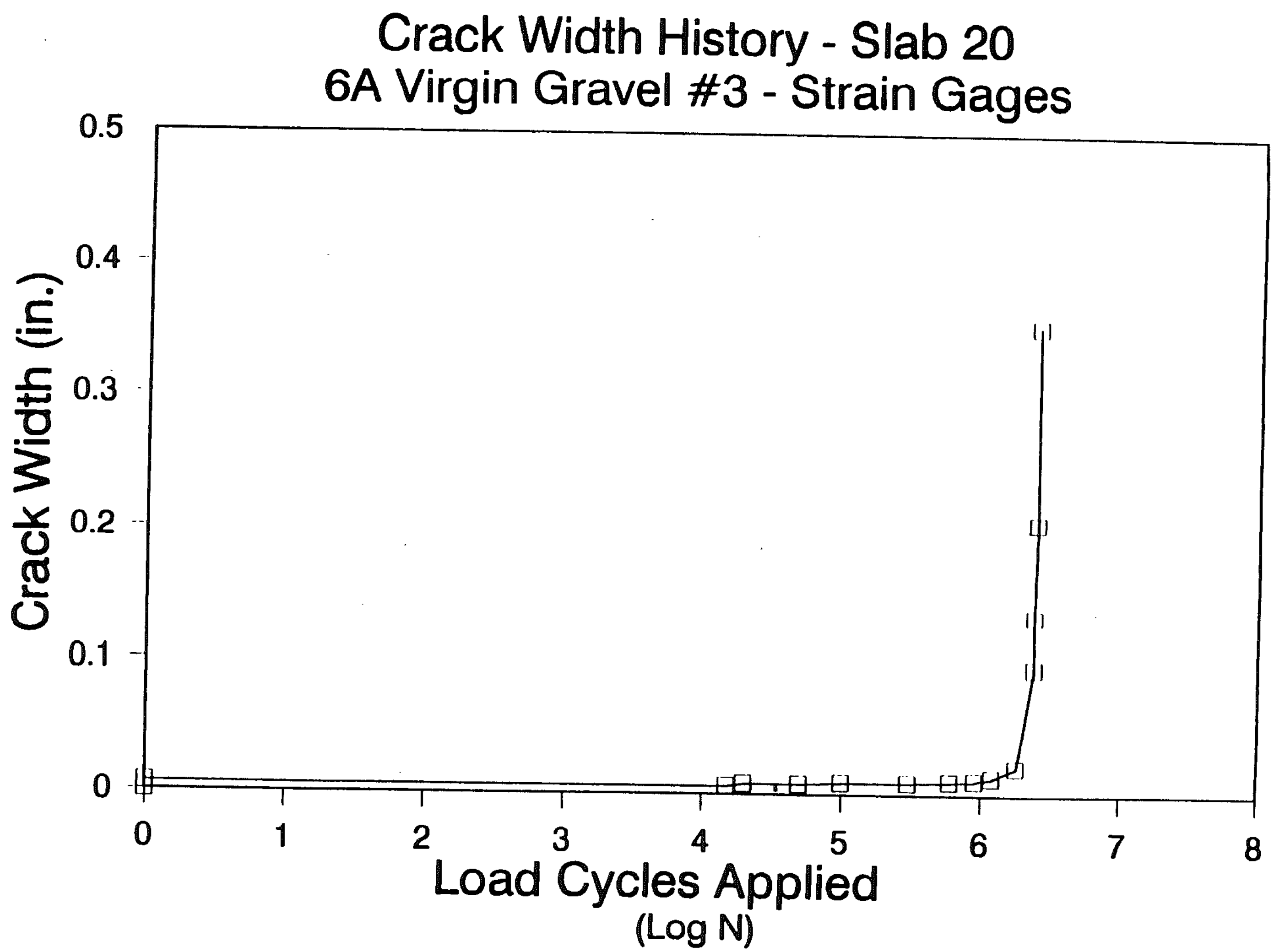

Figure D-10. Crack width history for slab 20.

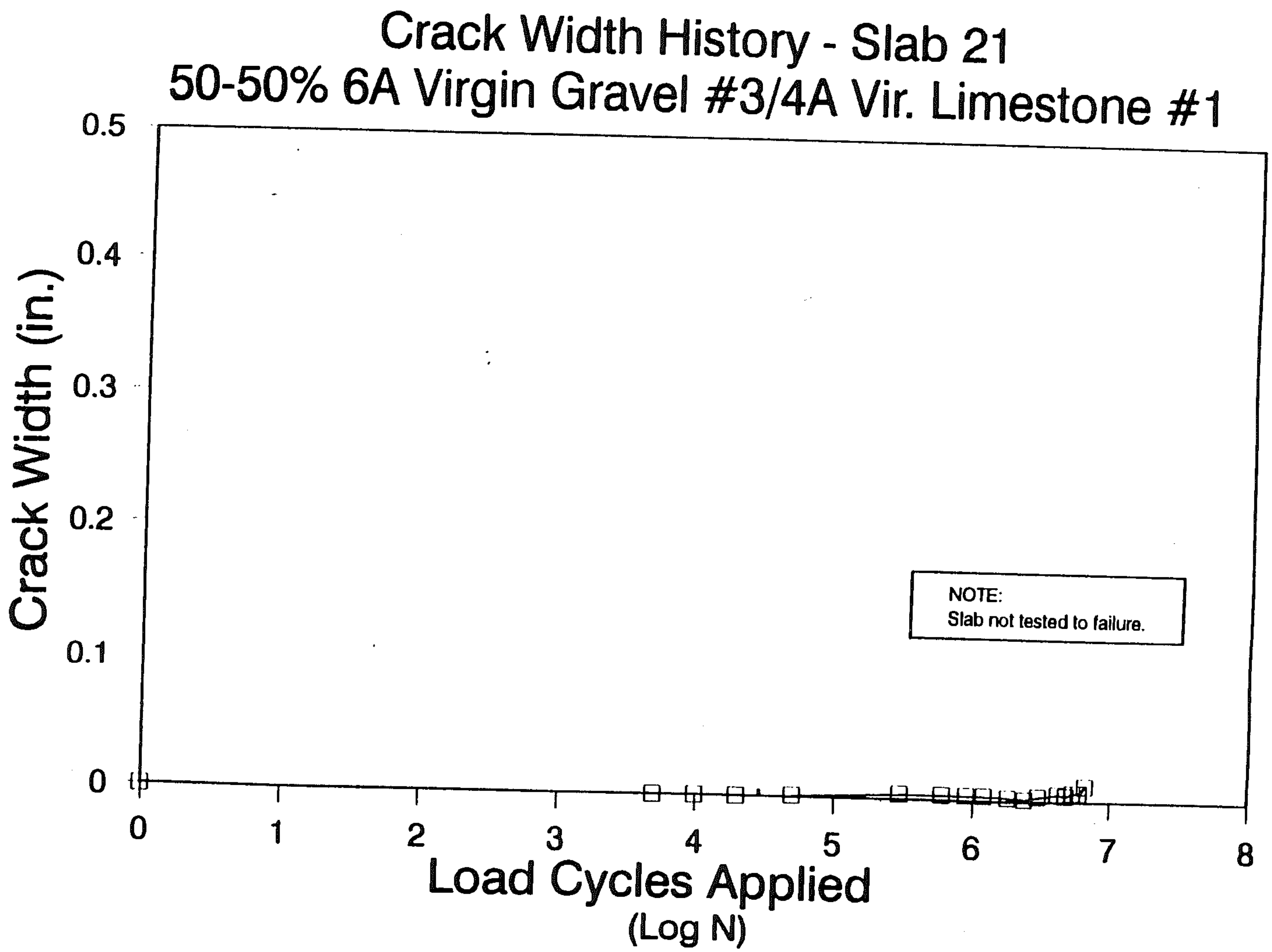

Figure D-11. Crack width history for slab 21.

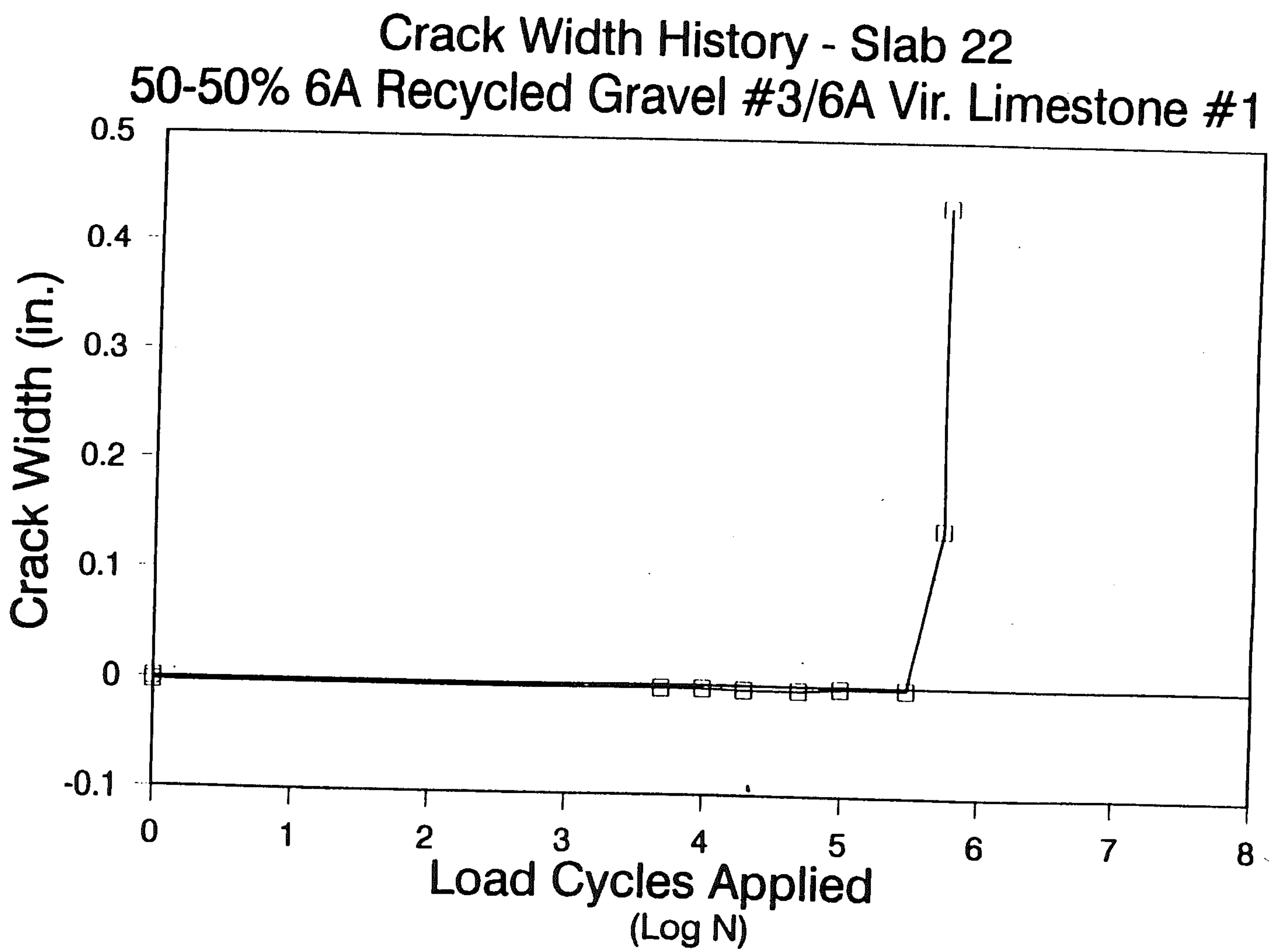

Figure D-12. Crack width history for slab 22.

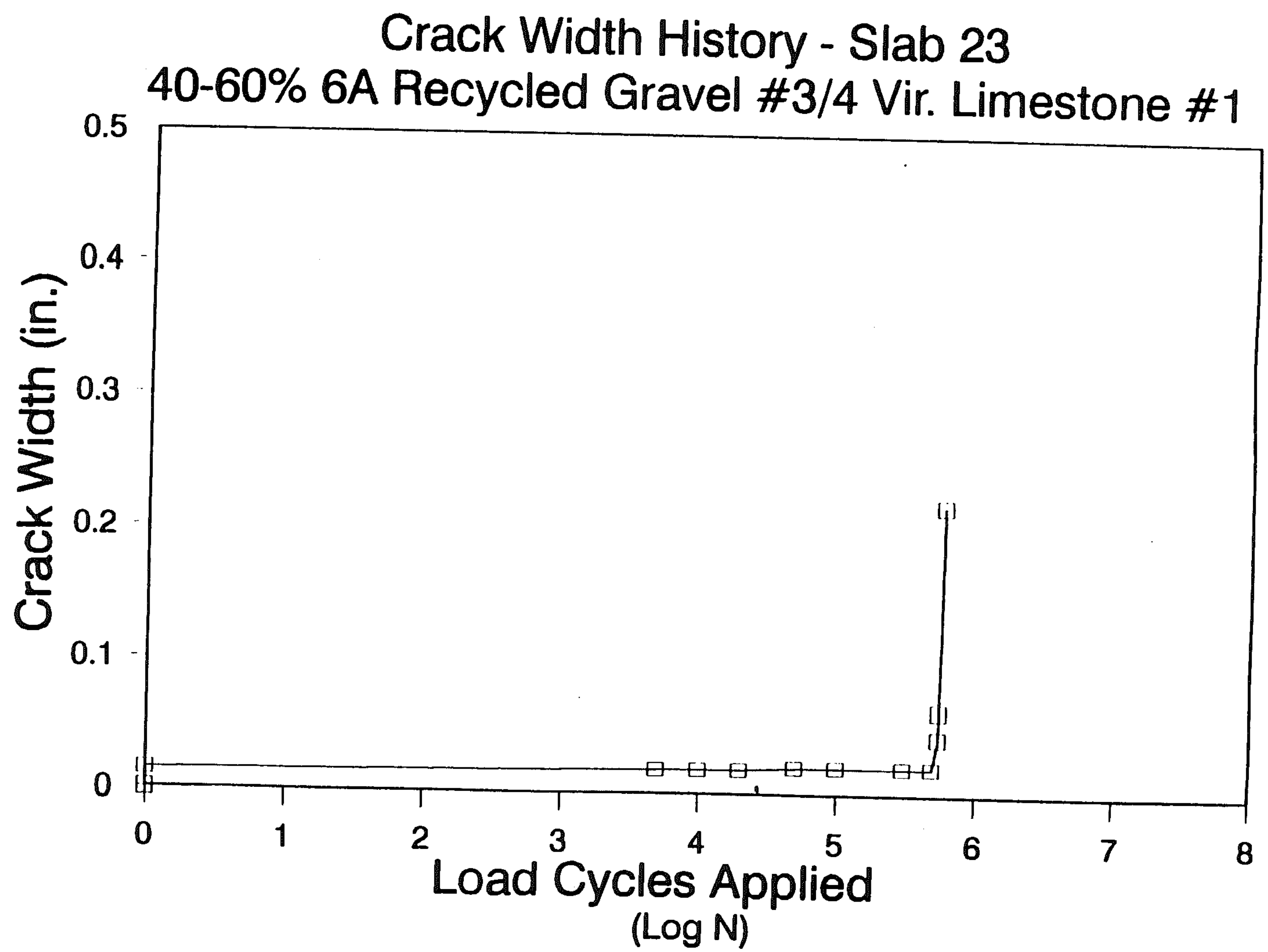

Figure D-13. Crack width history for slab 23.

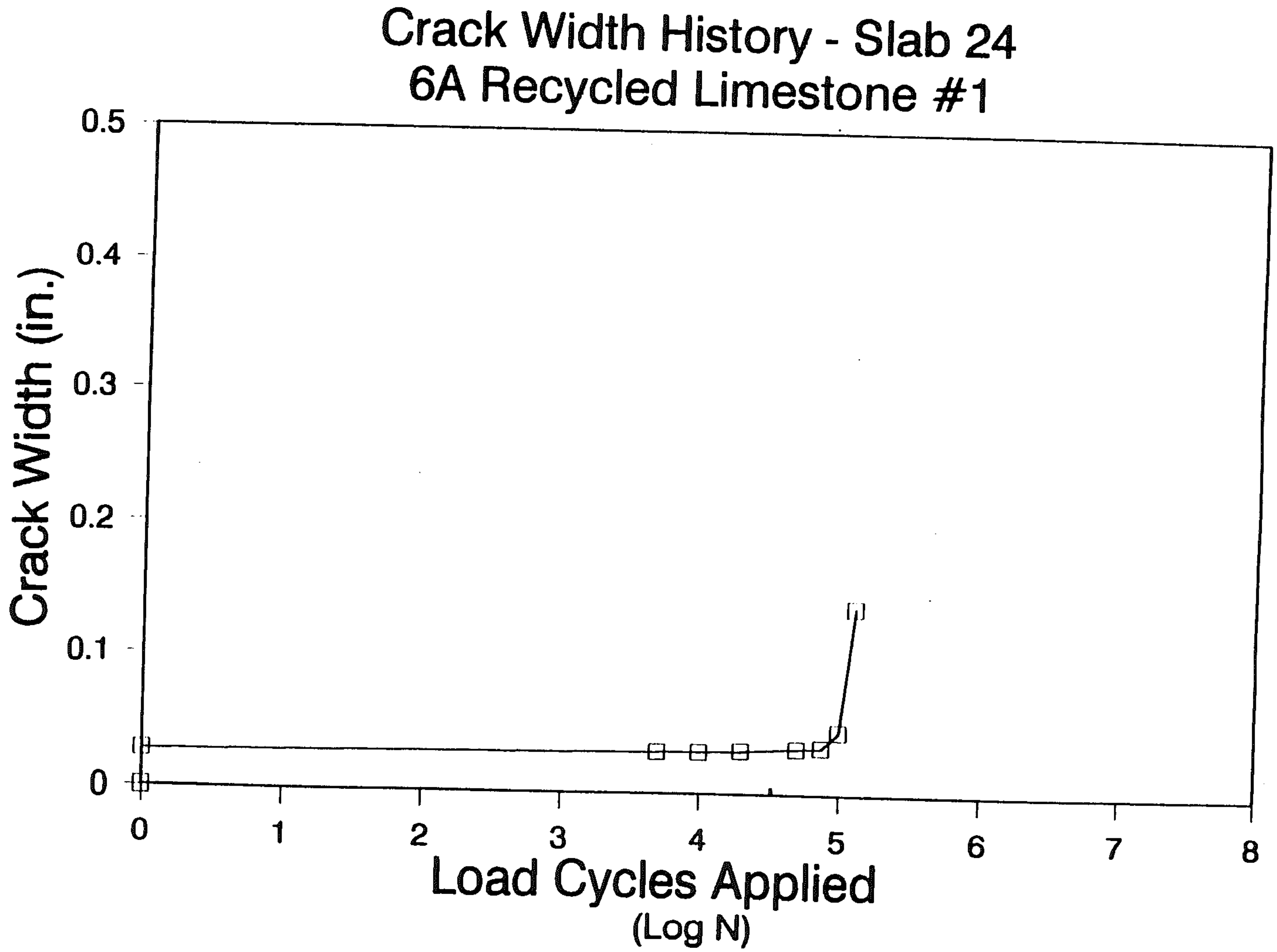

Figure D-14. Crack width history for slab 24.

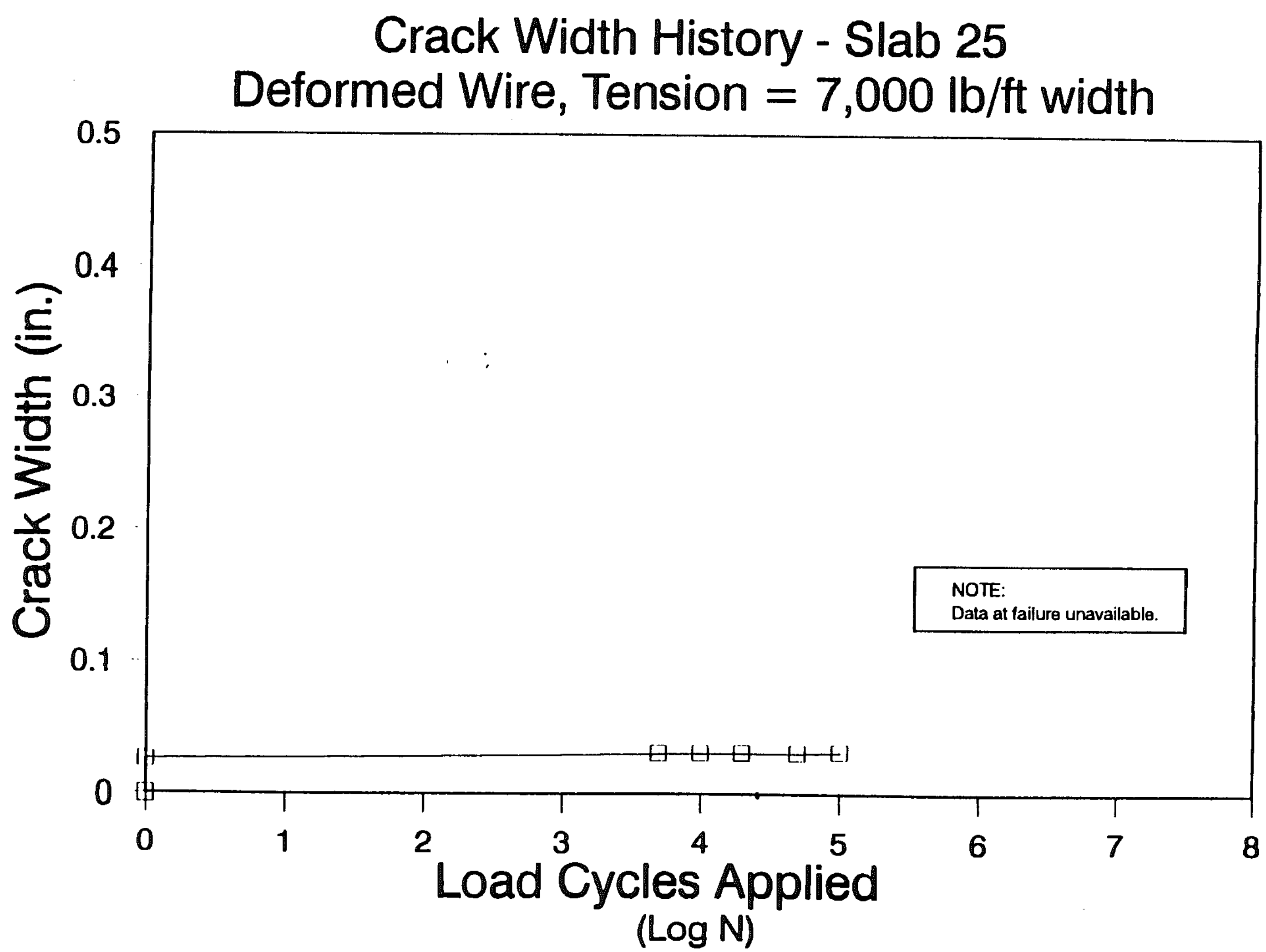

Figure D-15. Crack width history for slab 25.

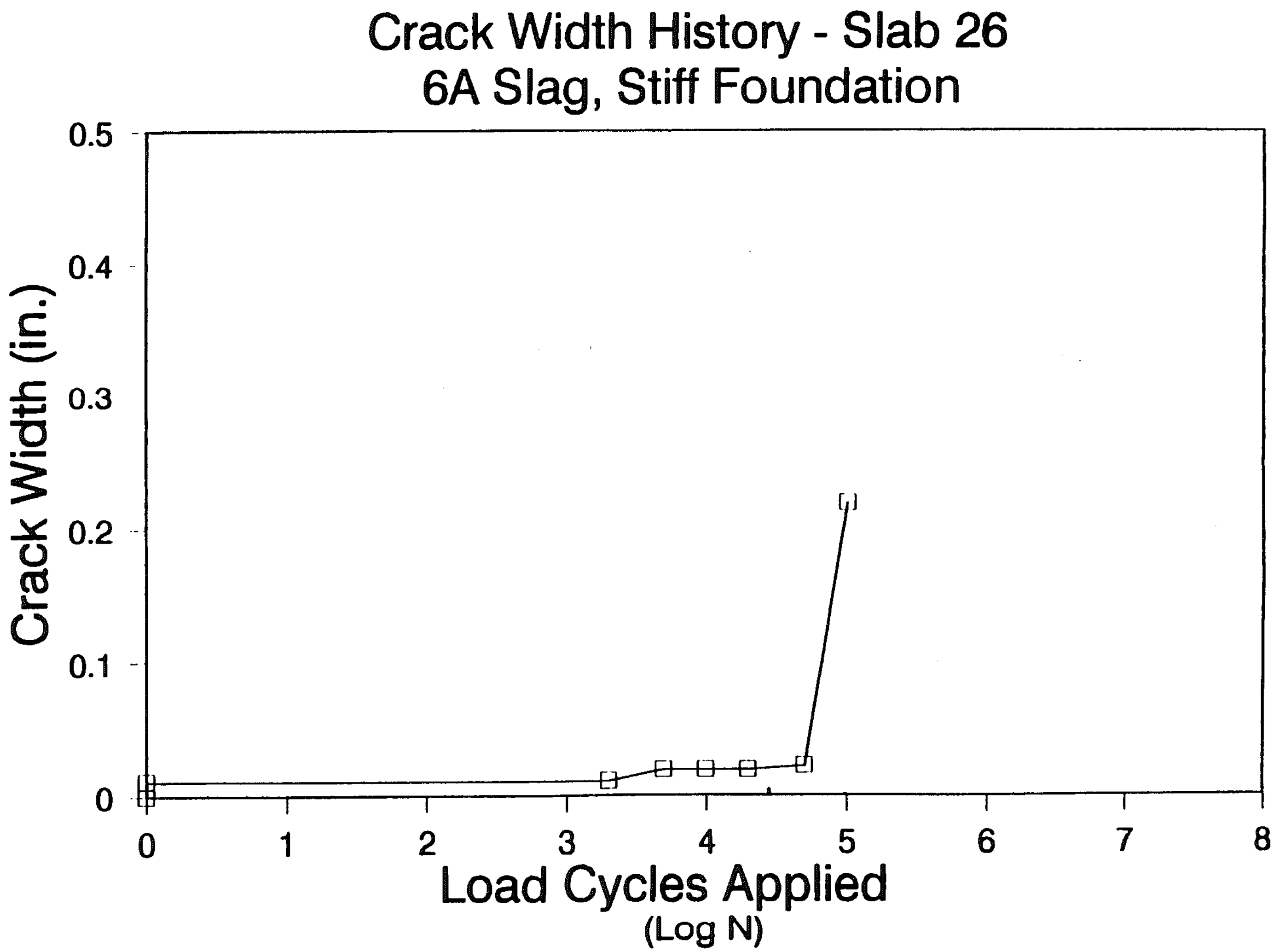

Figure D-16. Crack width history for slab 26.

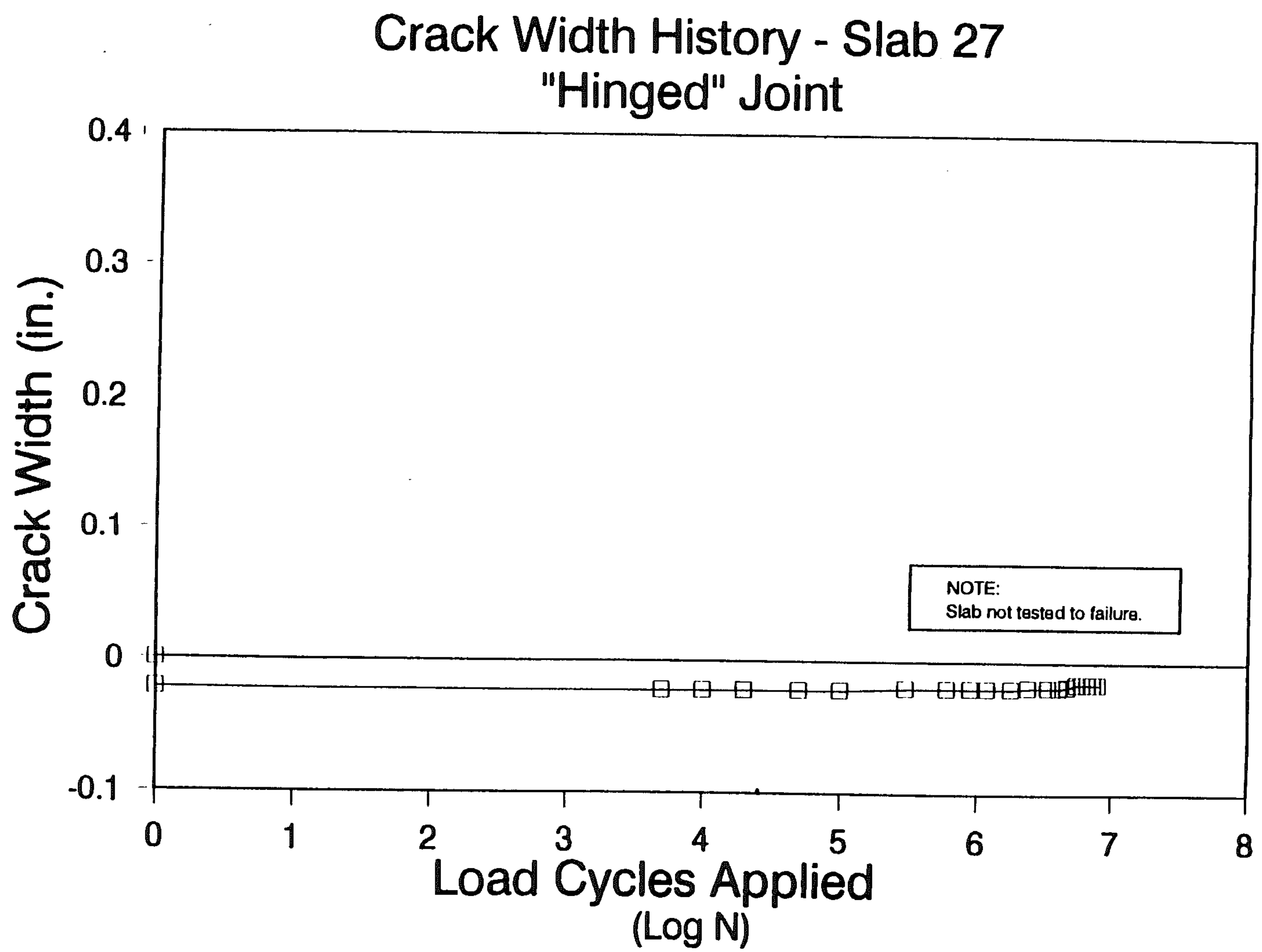

Figure D-17. Crack width history for slab 27.

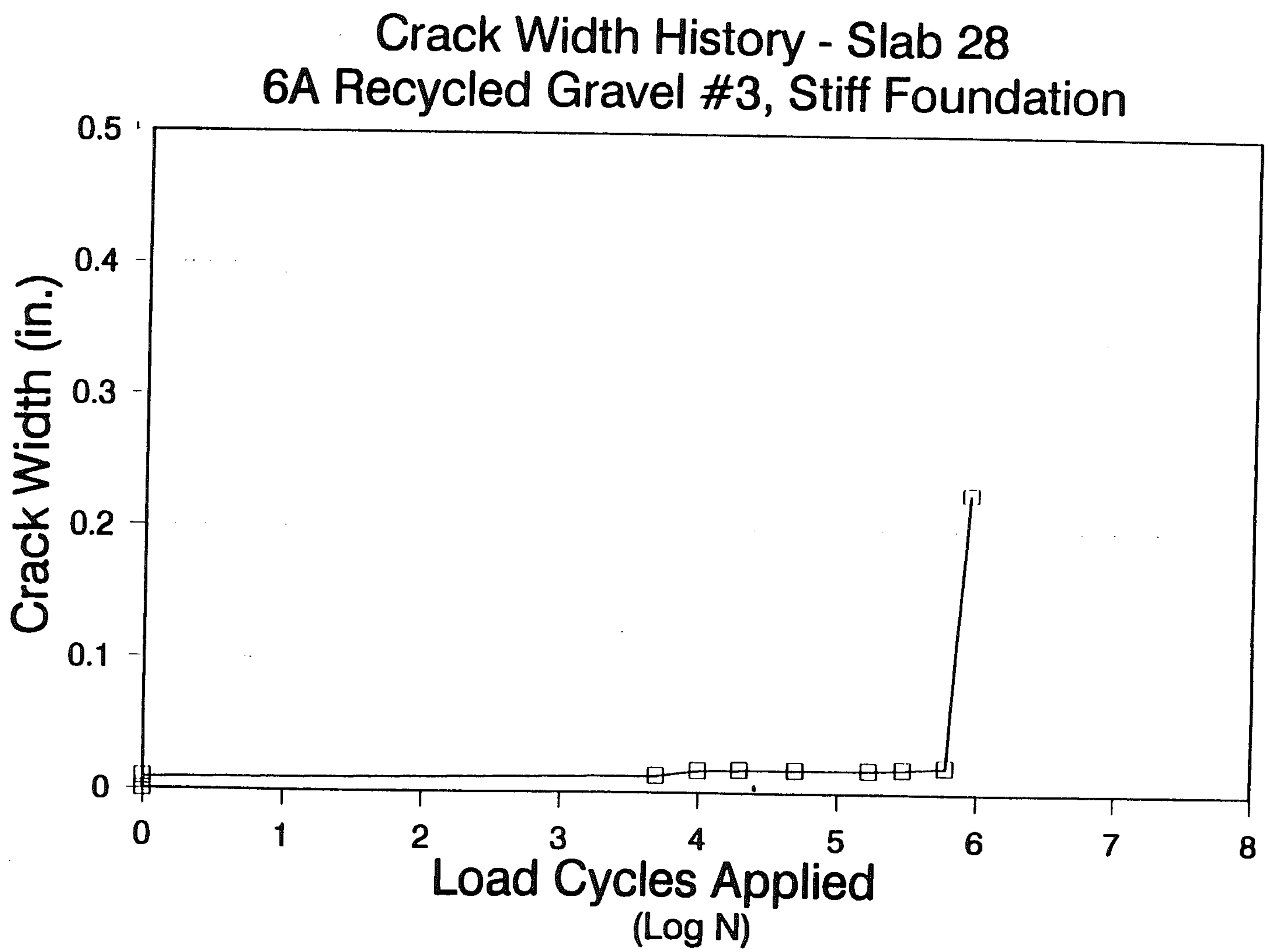

Figure D-18. Crack width history for slab 28.

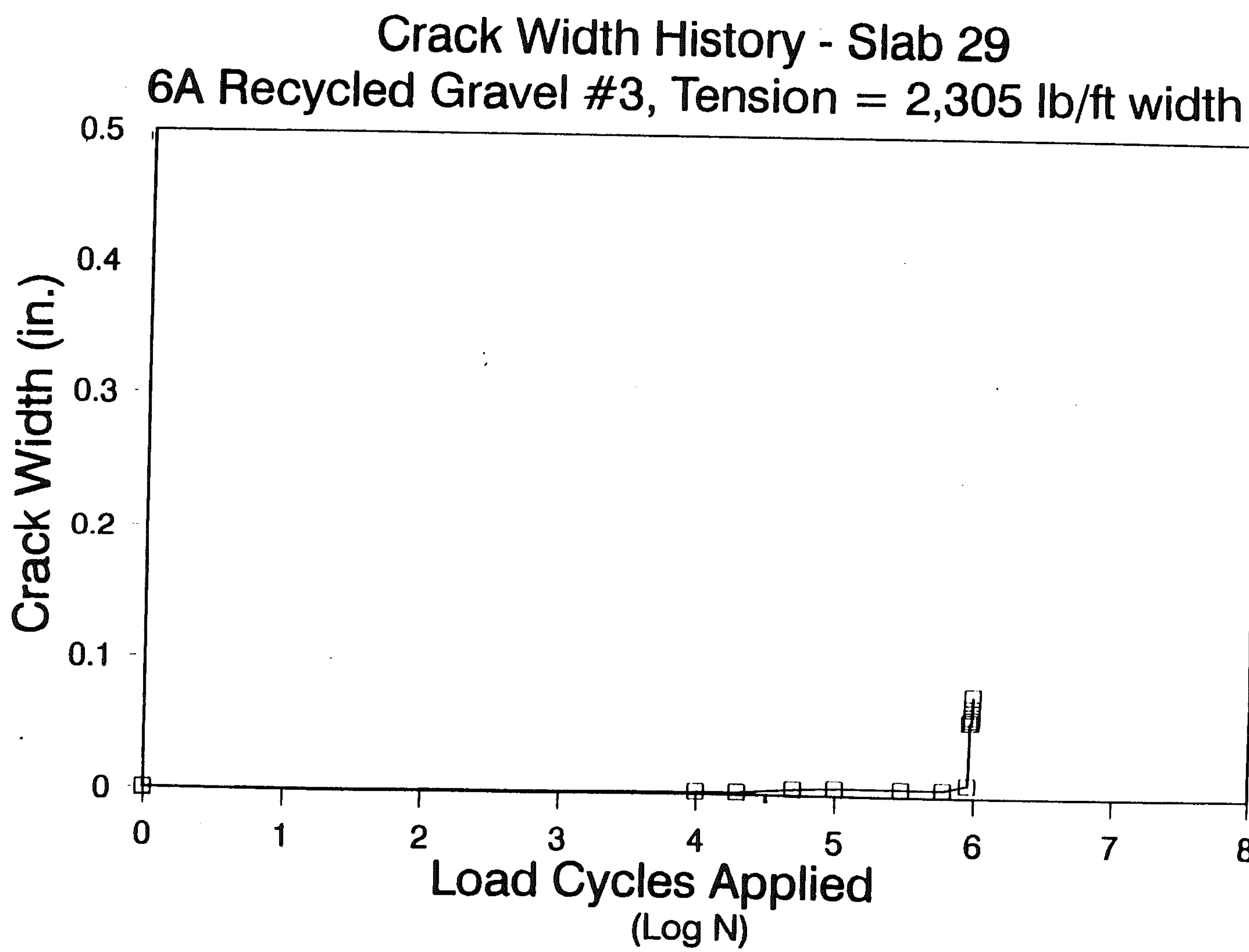

Figure D-19. Crack width history for slab 29.

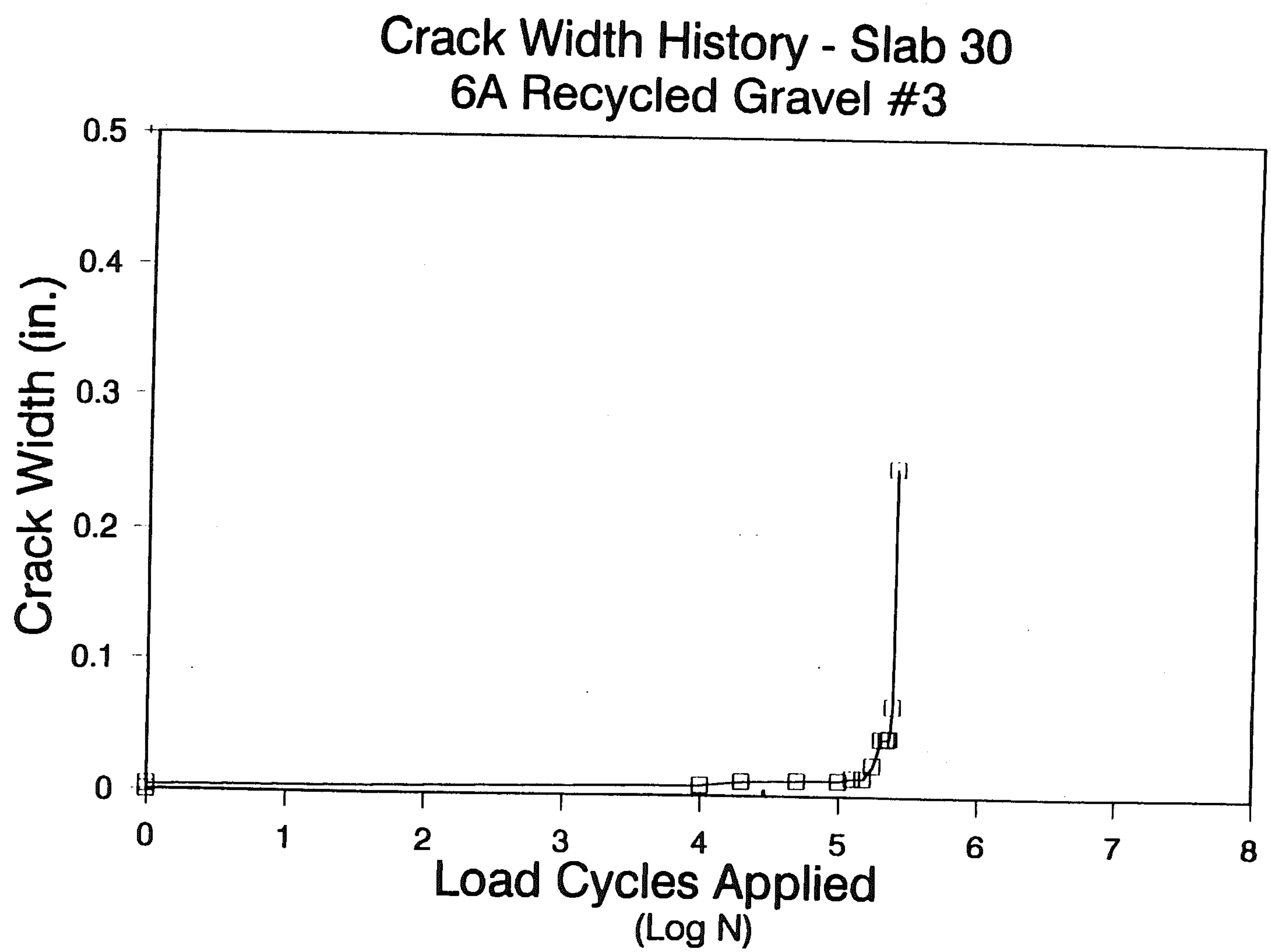

Figure D-20. Crack width history for slab 30.

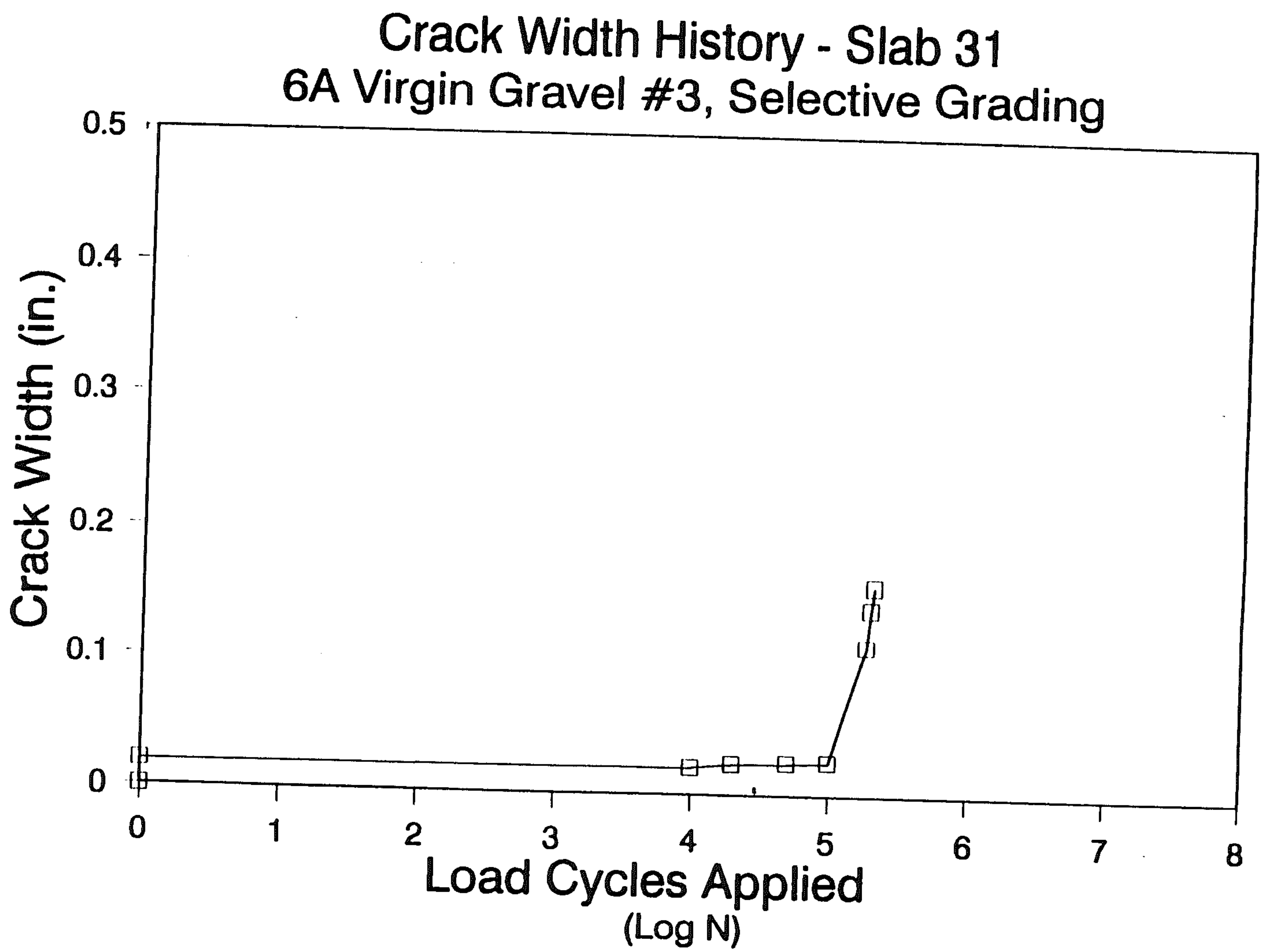

Figure D-21. Crack width history for slab 31.

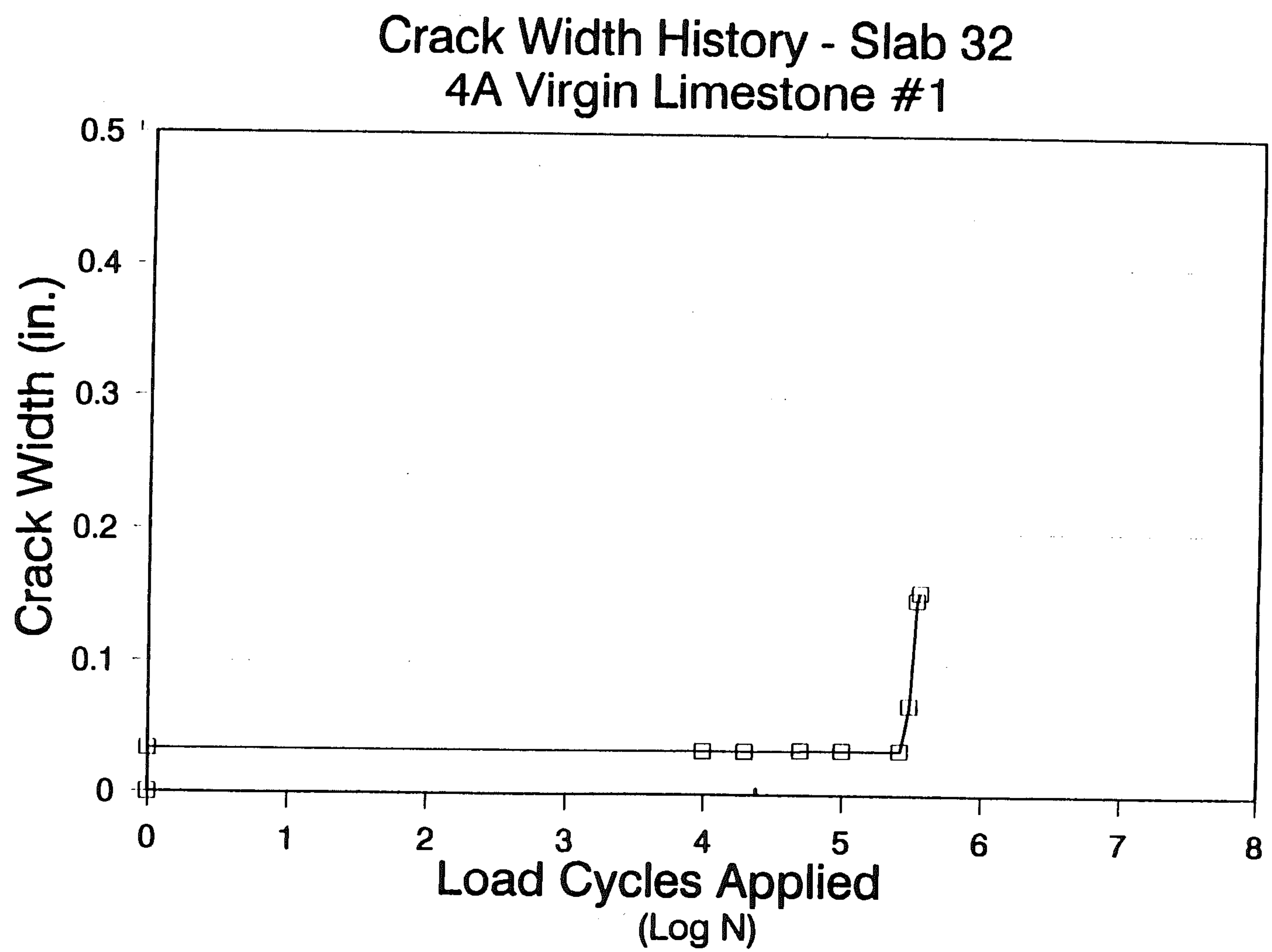

Figure D-22. Crack width history for slab 32.

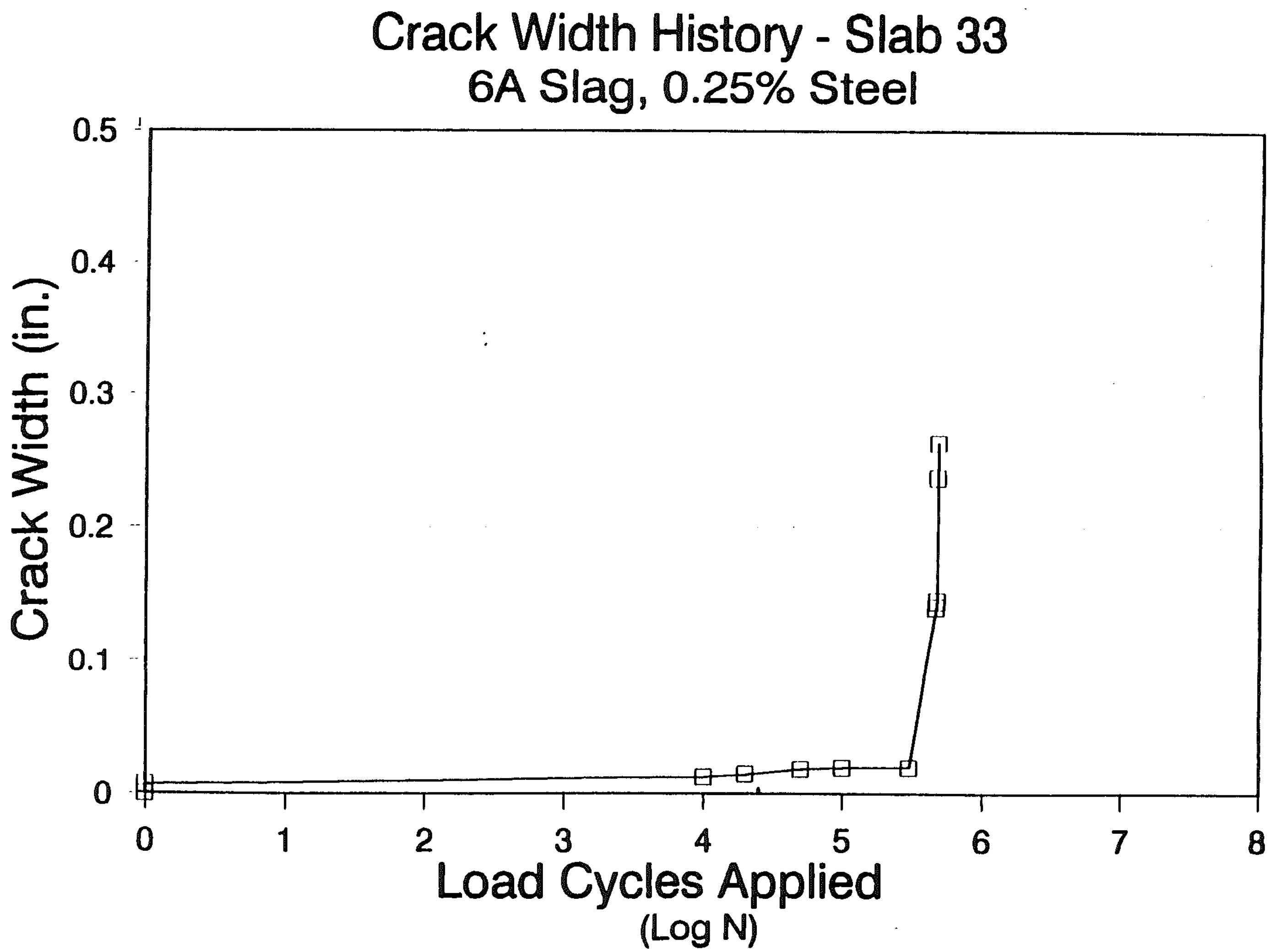

Figure D-23. Crack width history for slab 33.

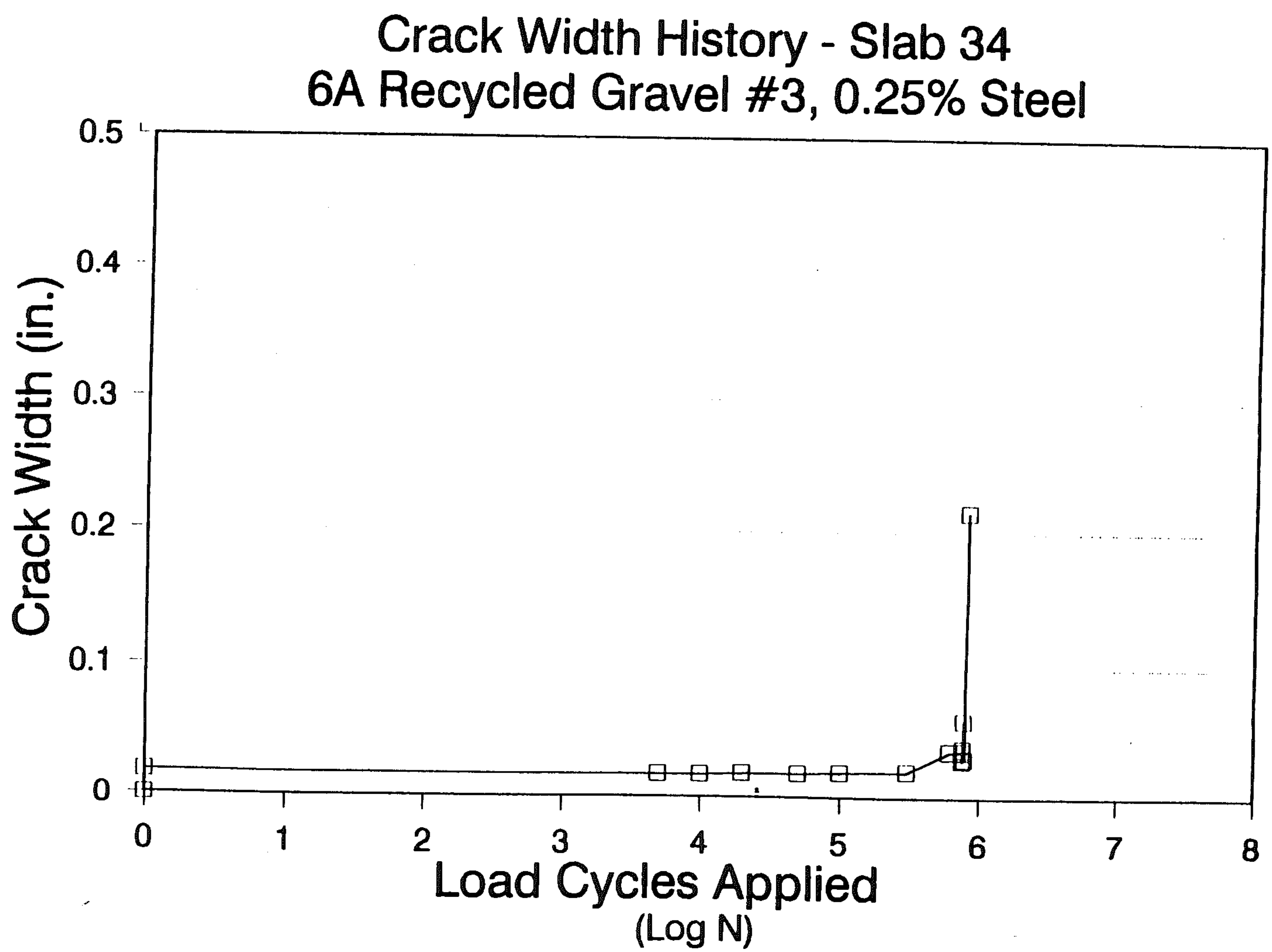

Figure D-24. Crack width history for slab 34.

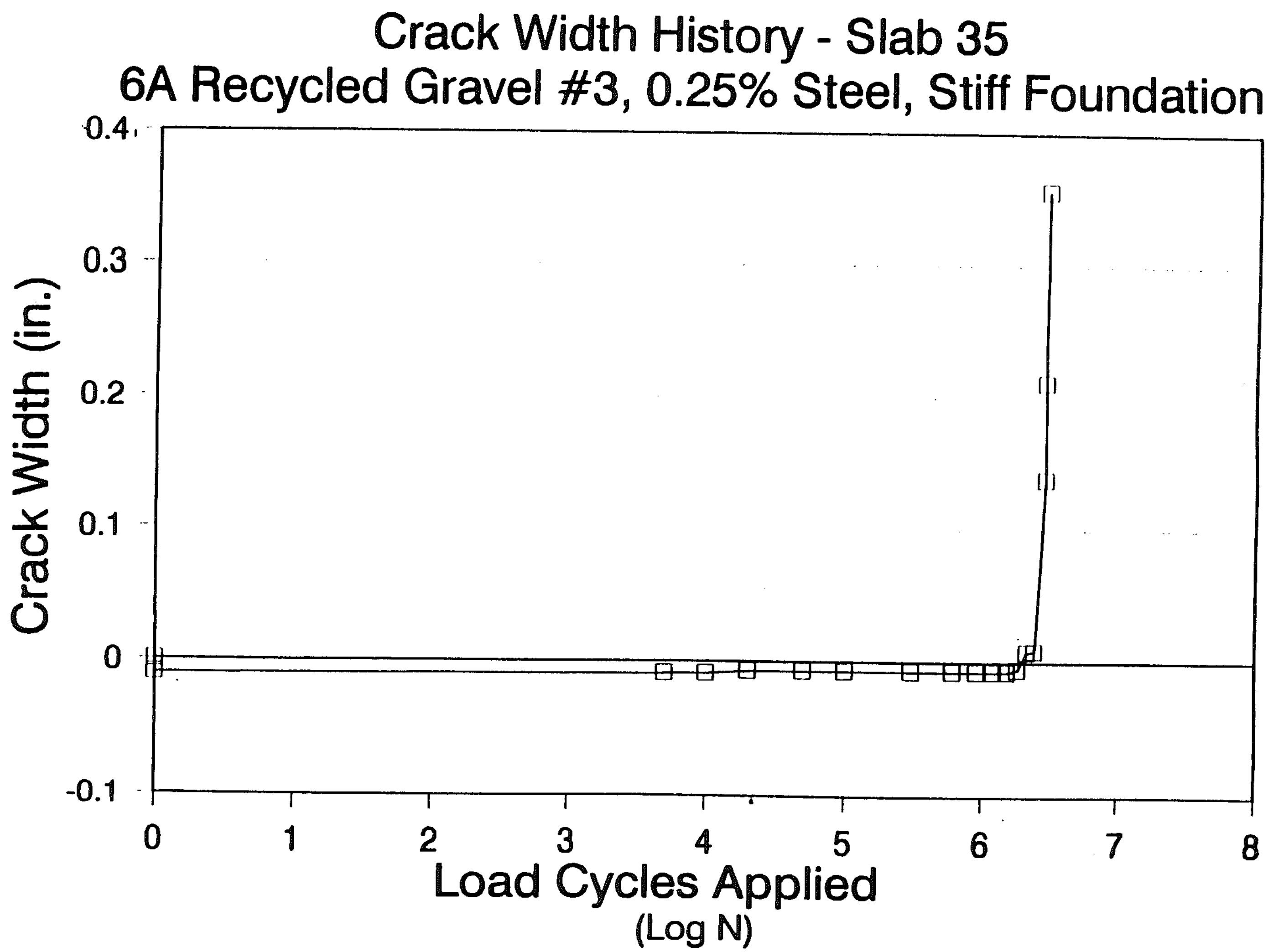

Figure D-25. Crack width history for slab 35.

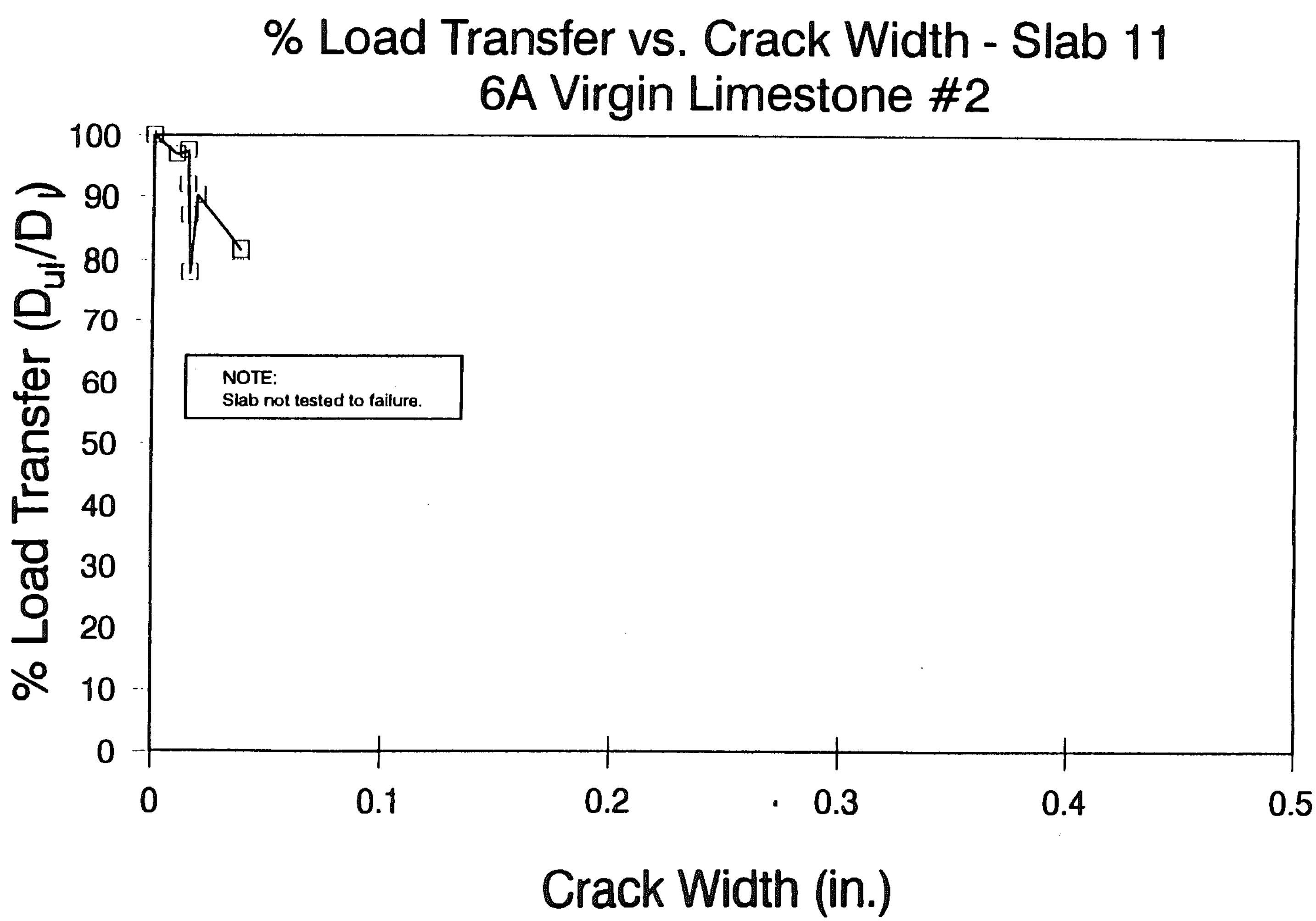

Figure D-26. Percent load transfer versus crack width for slab 11.

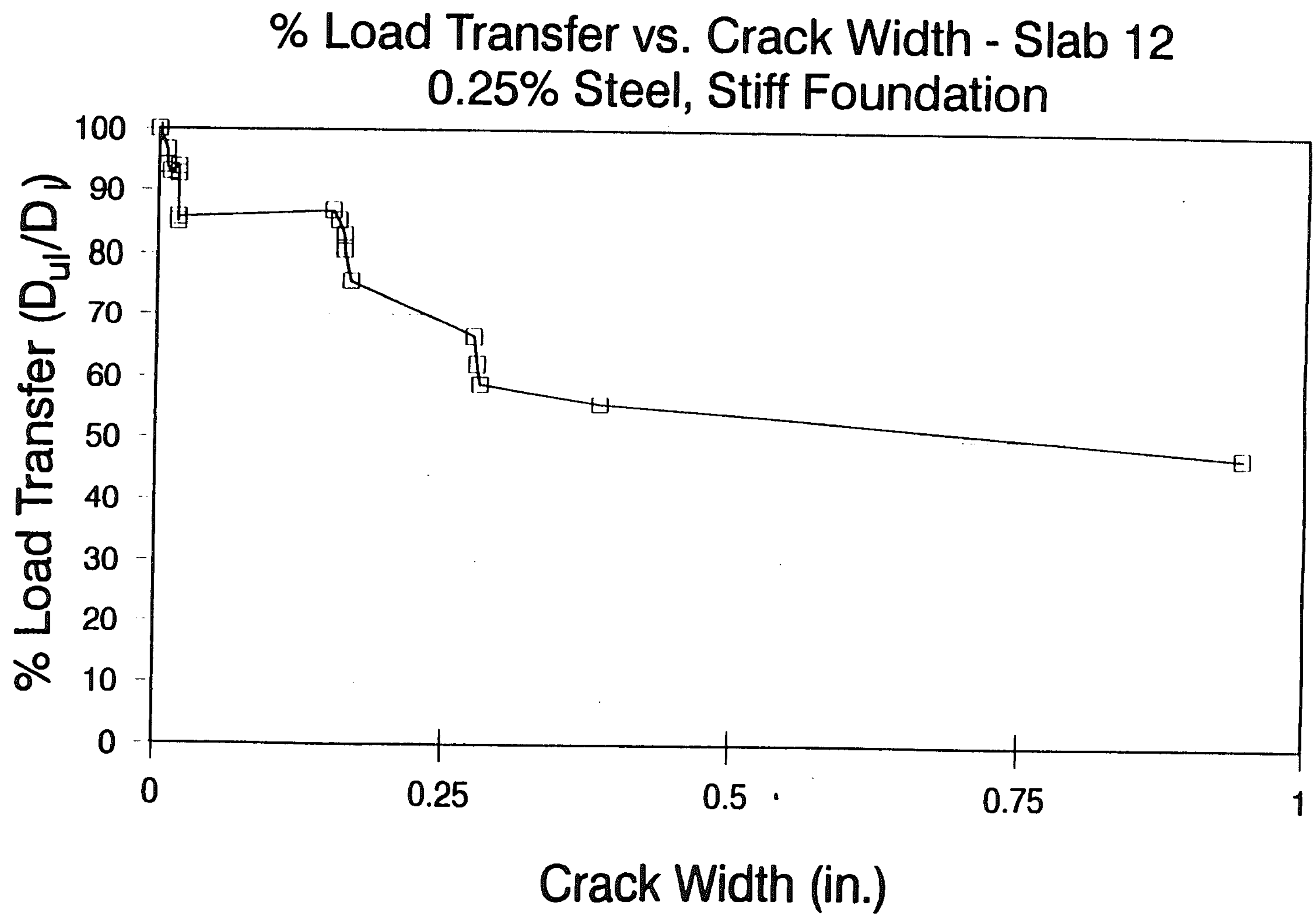

Figure D-27. Percent load transfer versus crack width for slab 12.

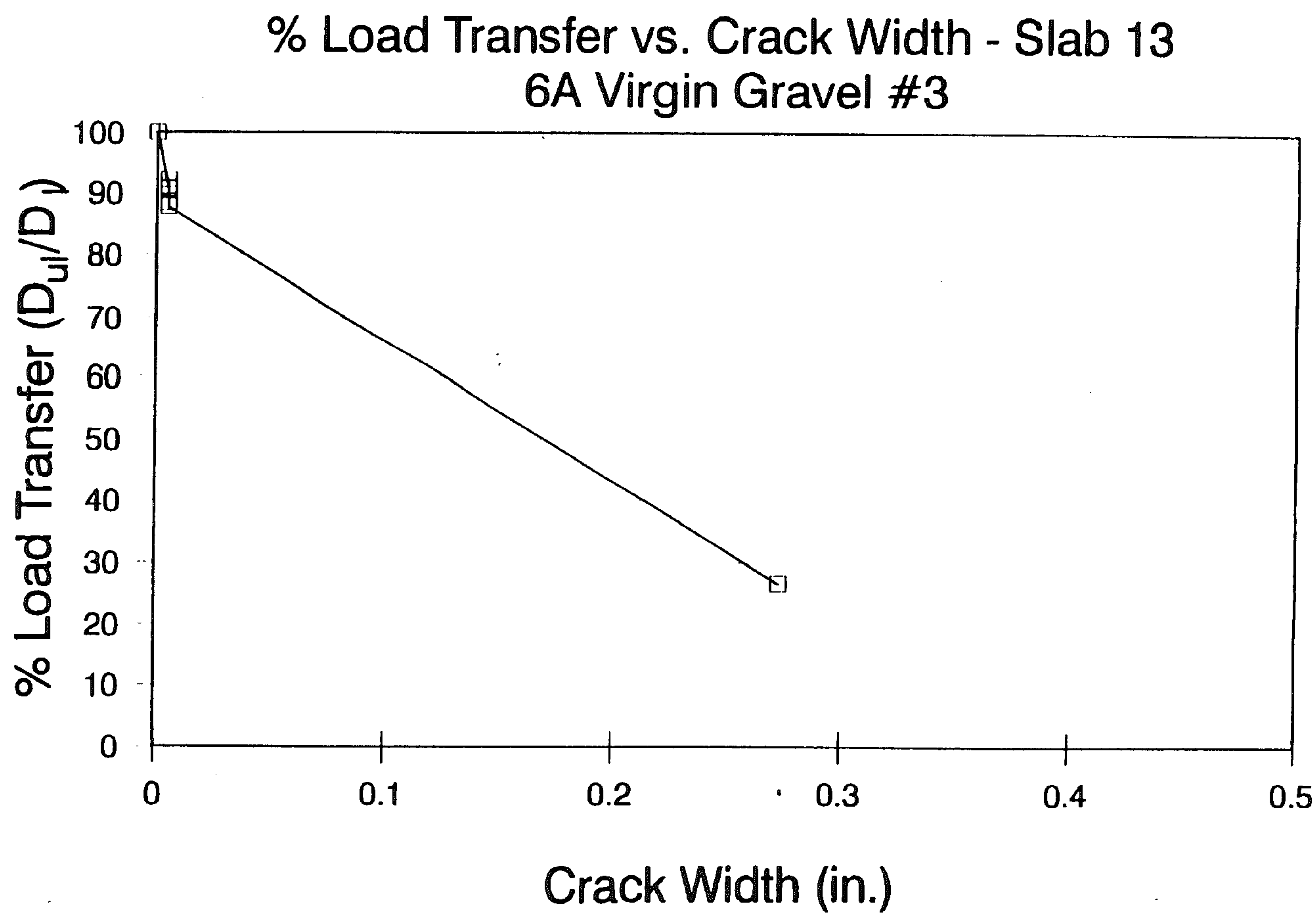

Figure D-28. Percent load transfer versus crack width for slab 13.

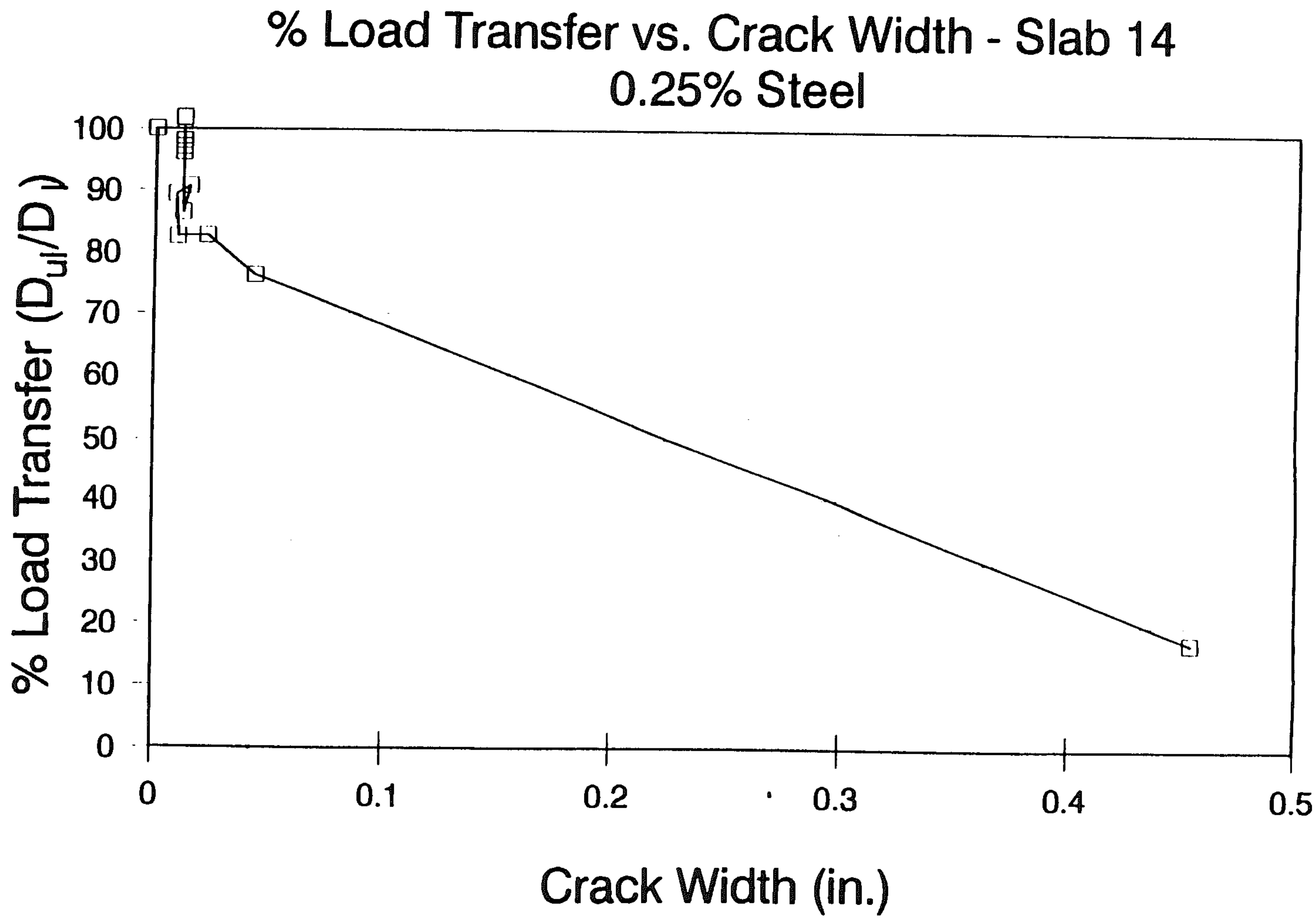

Figure D-29. Percent load transfer versus crack width for slab 14.

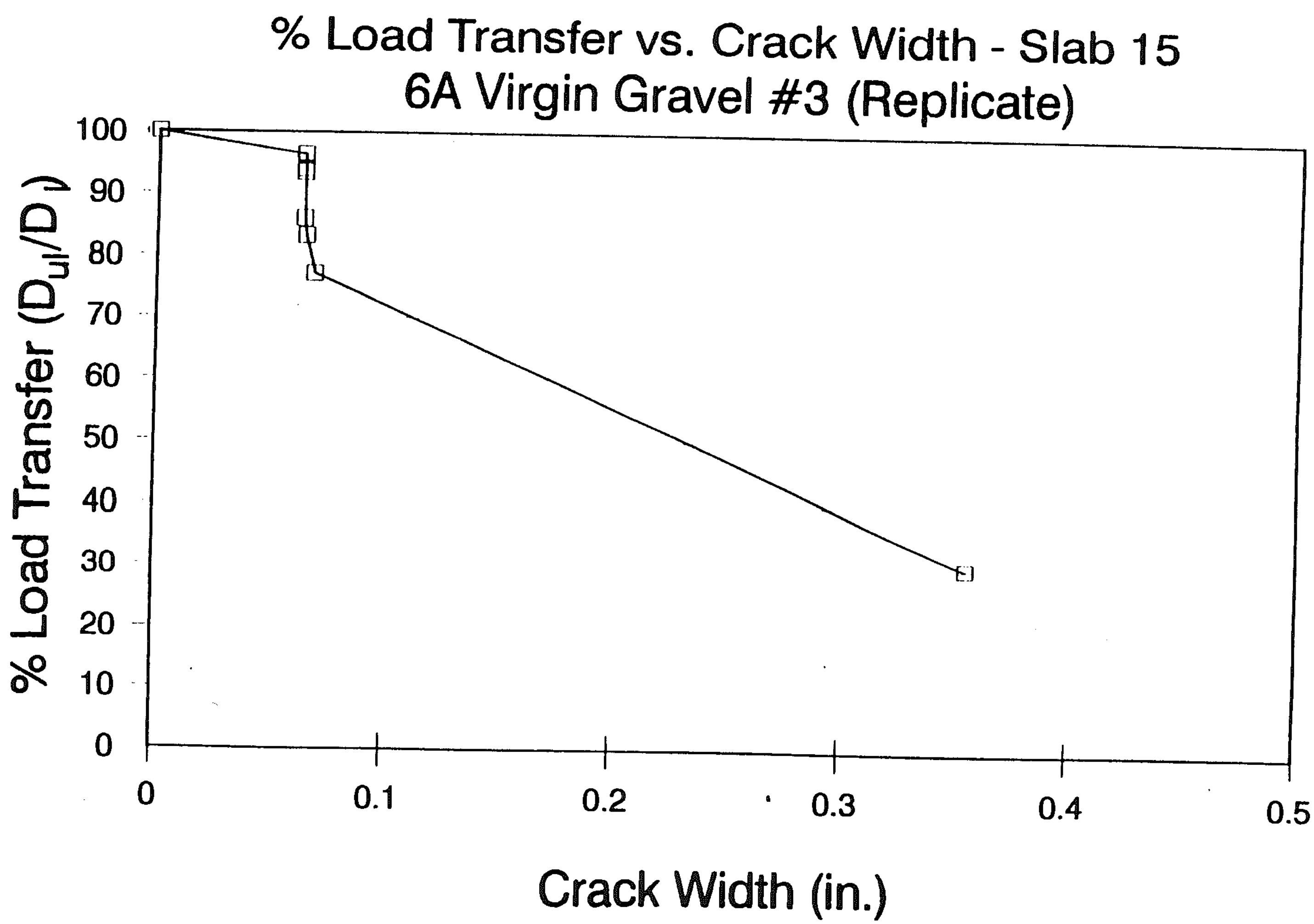

Figure D-30. Percent load transfer versus crack width for slab 15.

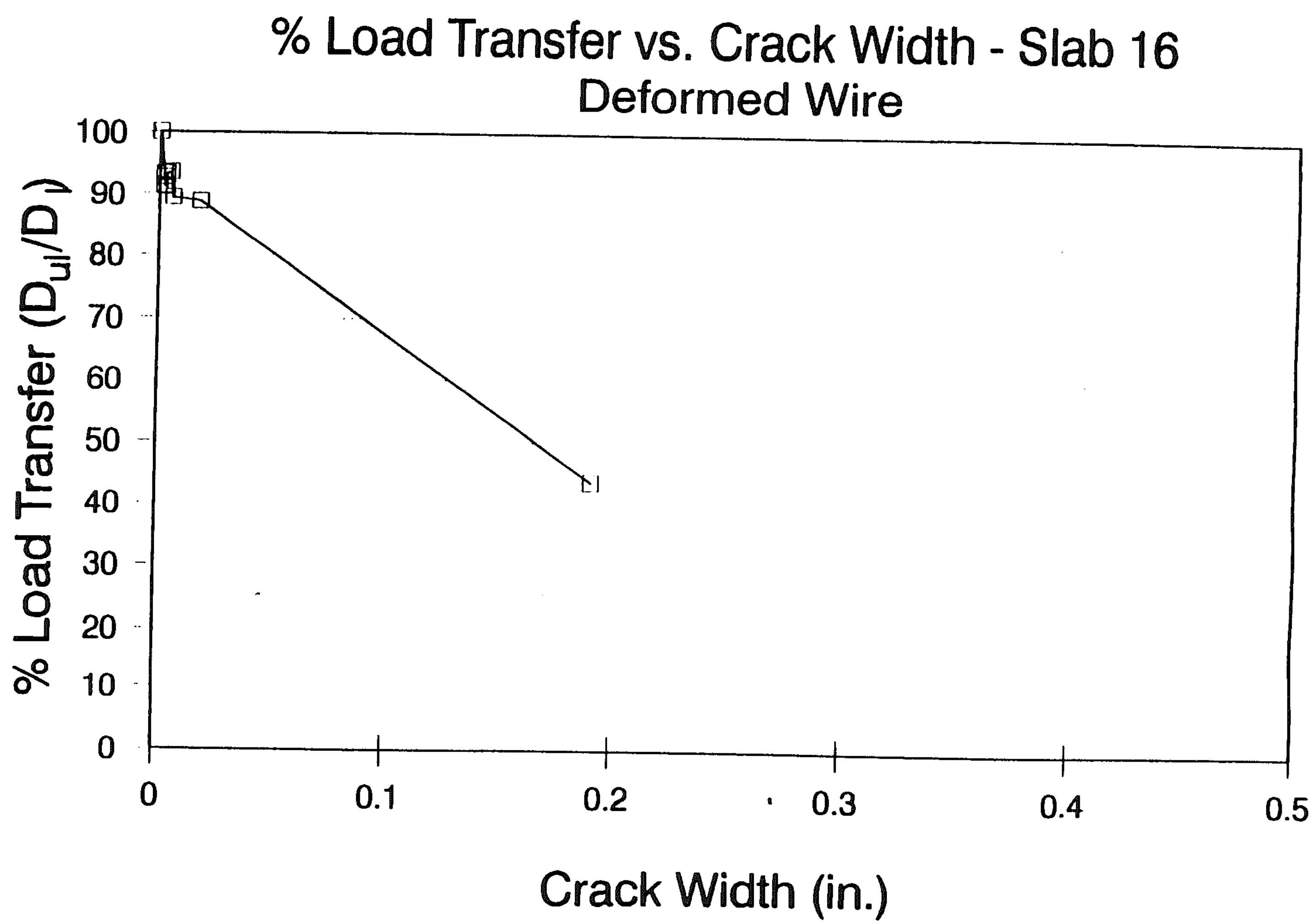

Figure D-31. Percent load transfer versus crack width for slab 16.

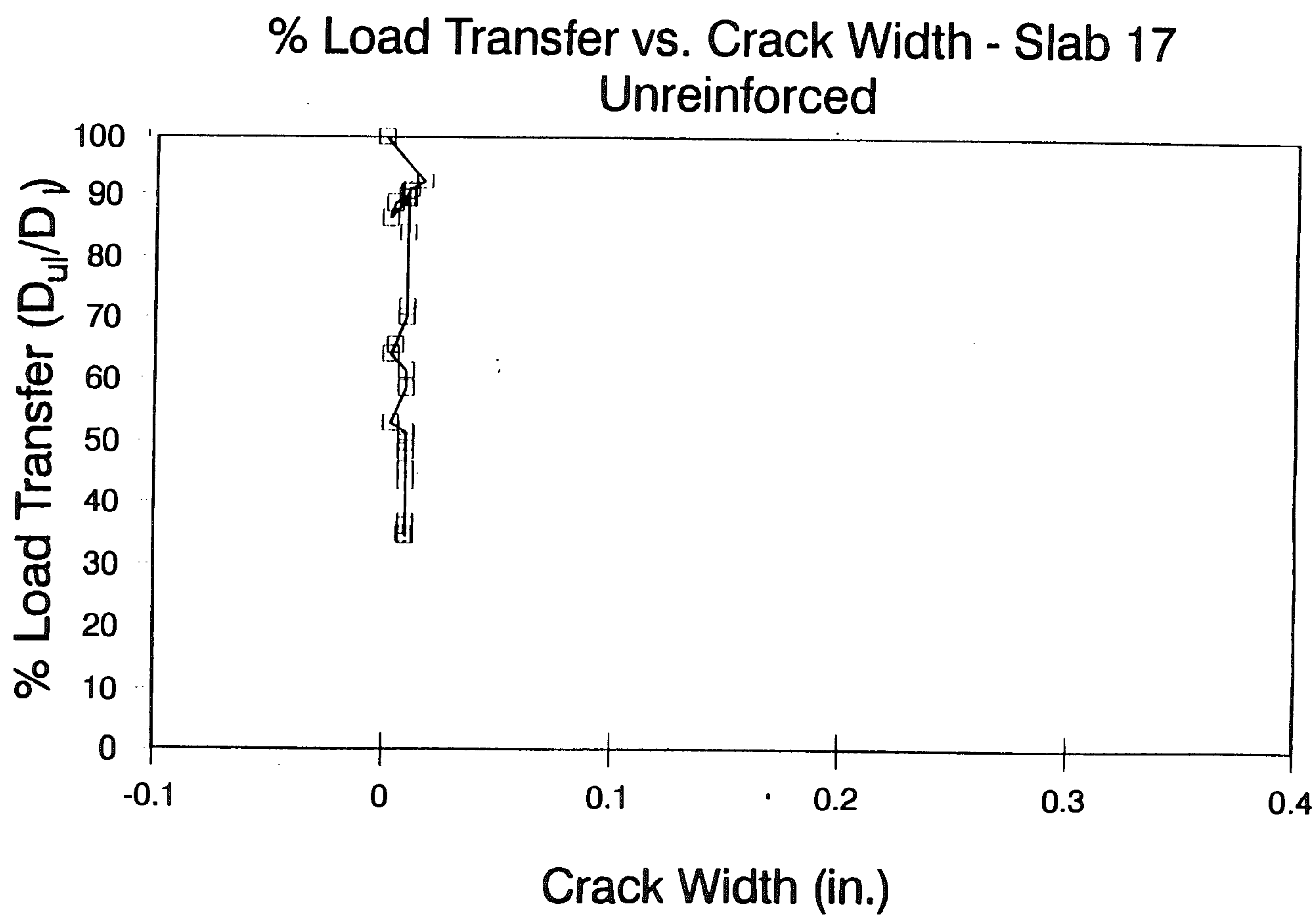

Figure D-32. Percent load transfer versus crack width for slab 17.

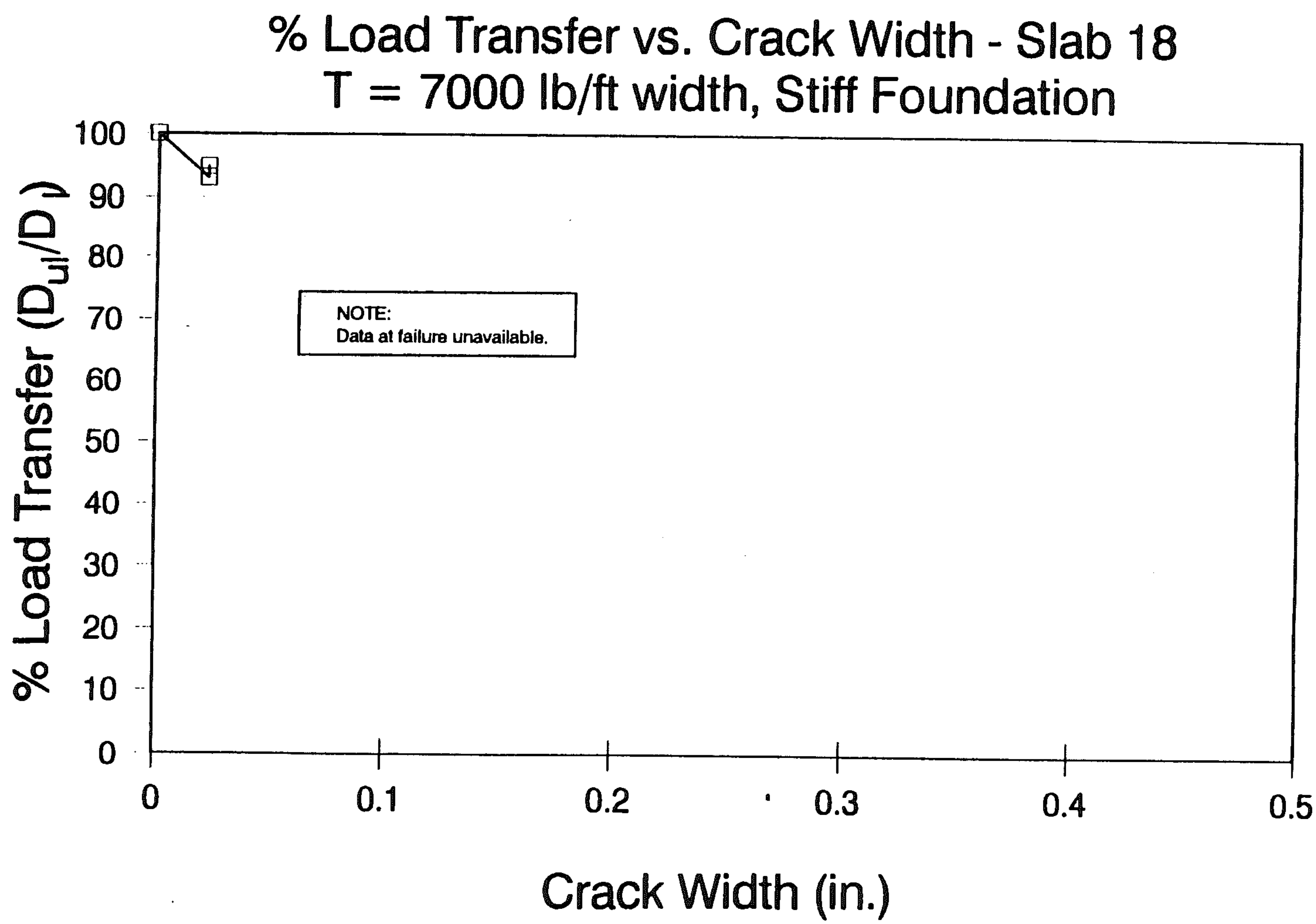

Figure D-33. Percent load transfer versus crack width for slab 18.

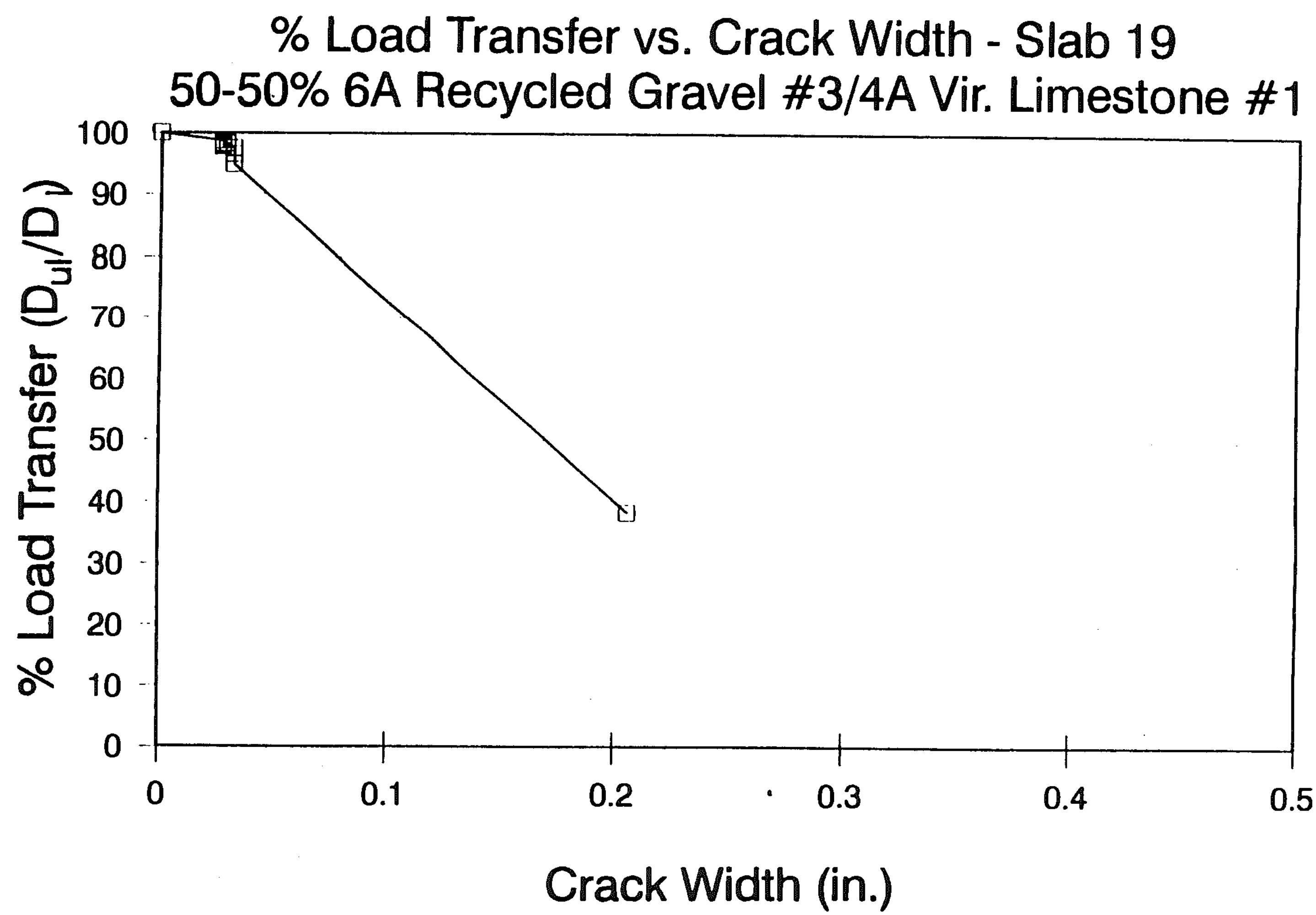

Figure D-34. Percent load transfer versus crack width for slab 19.

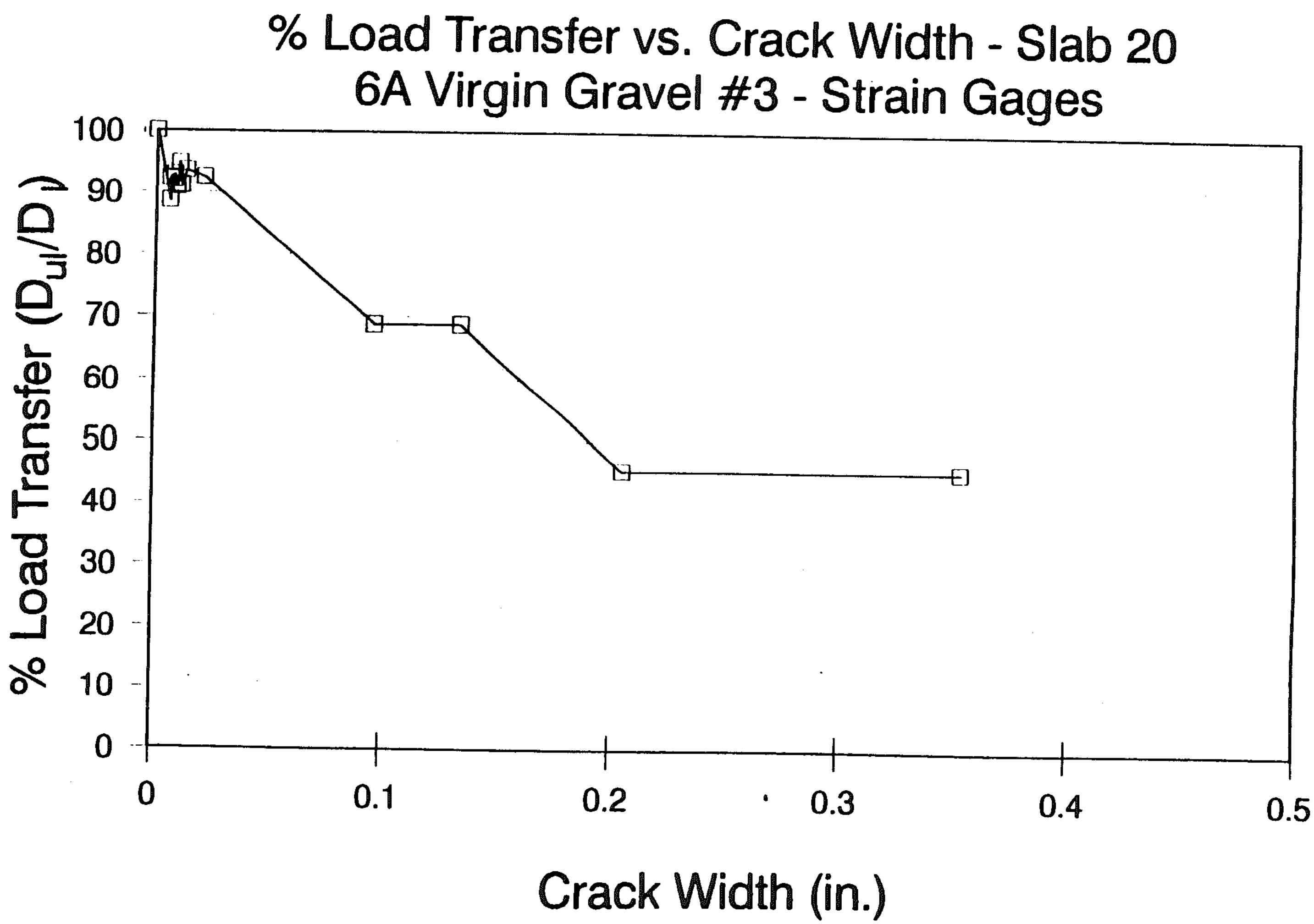

Figure D-35. Percent load transfer versus crack width for slab 20.

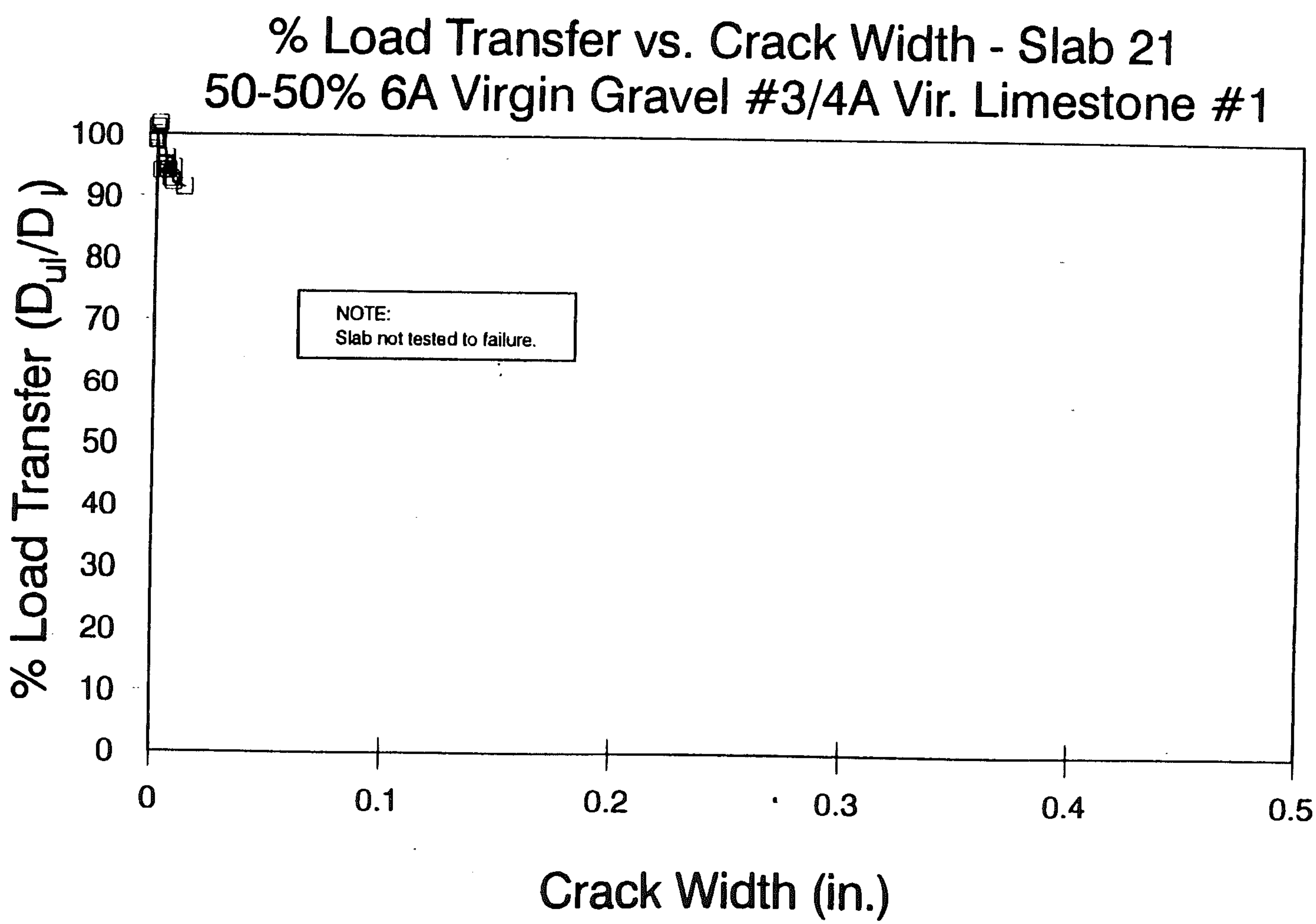

Figure D-36. Percent load transfer versus crack width for slab 21.

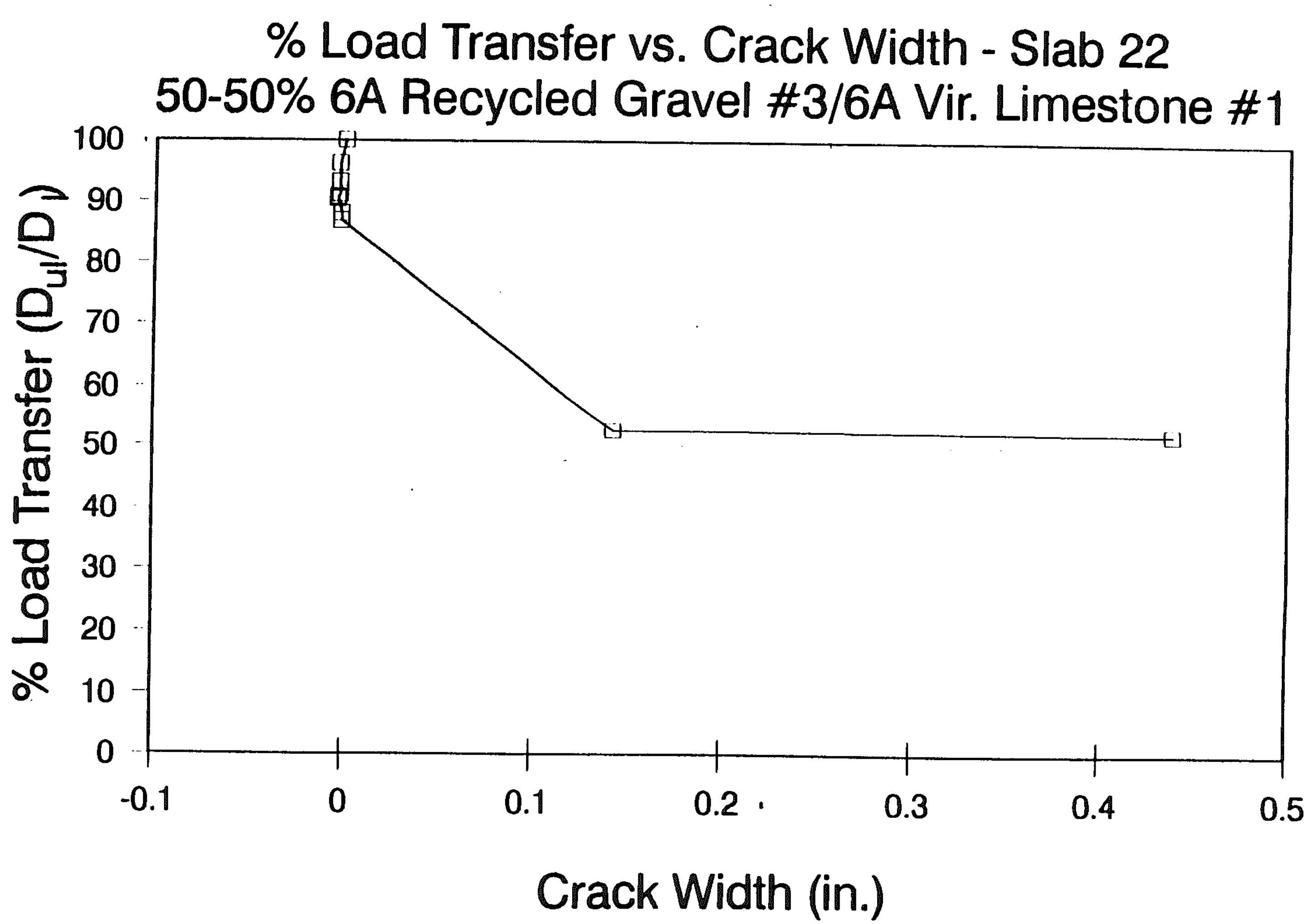

Figure D-37. Percent load transfer versus crack width for slab 22.

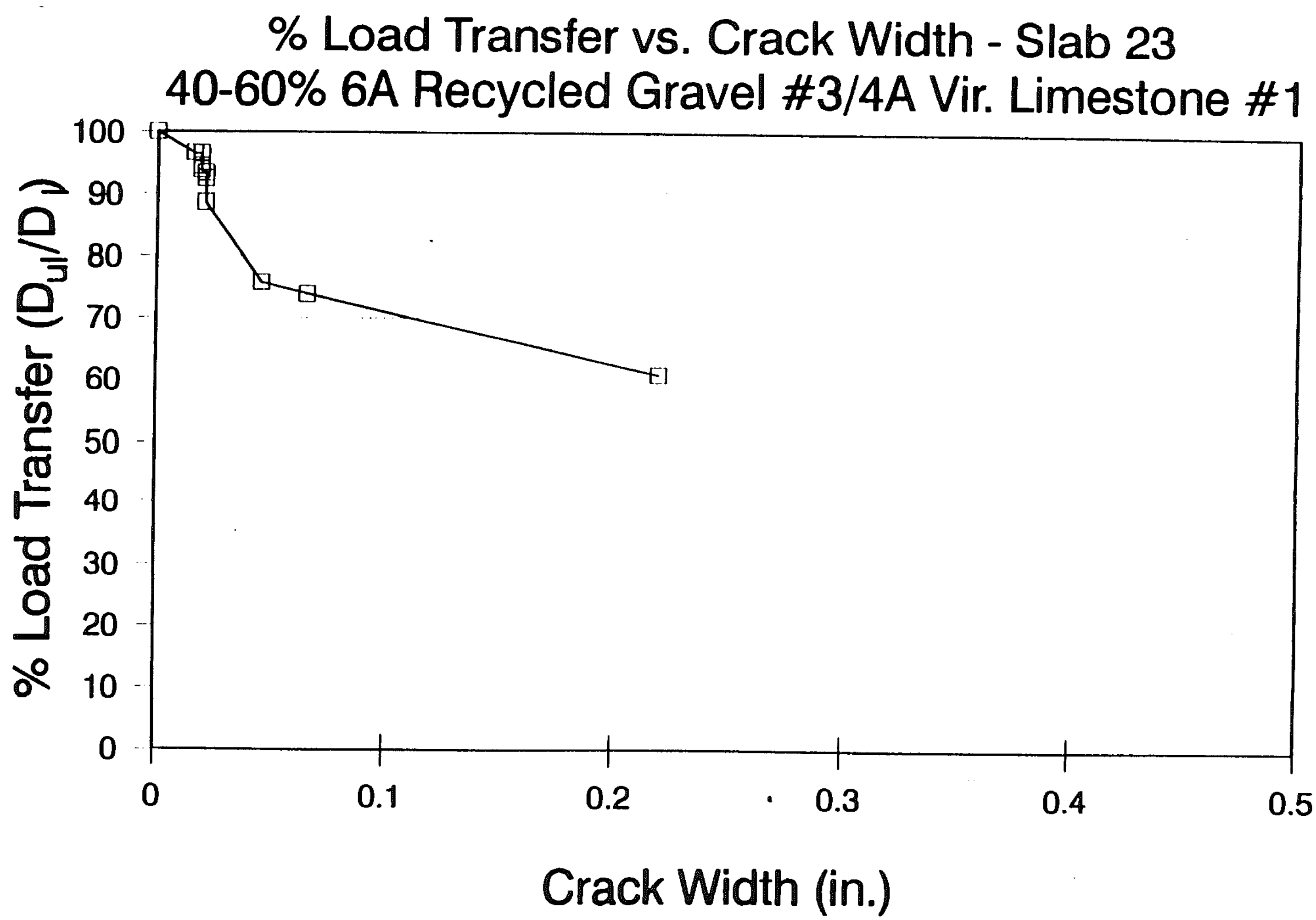

Figure D-38. Percent load transfer versus crack width for slab 23.

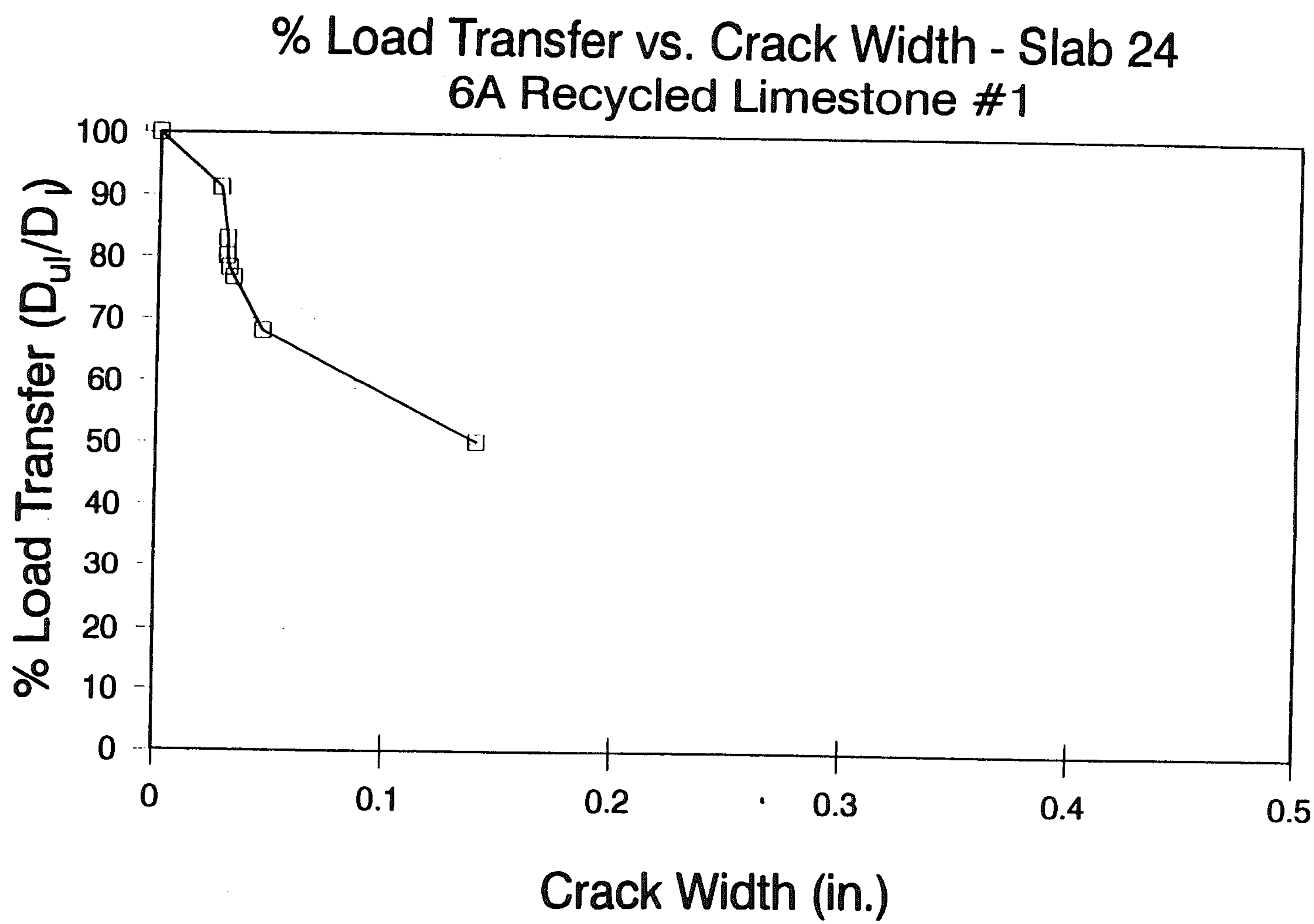

Figure D-39. Percent load transfer versus crack width for slab 24.

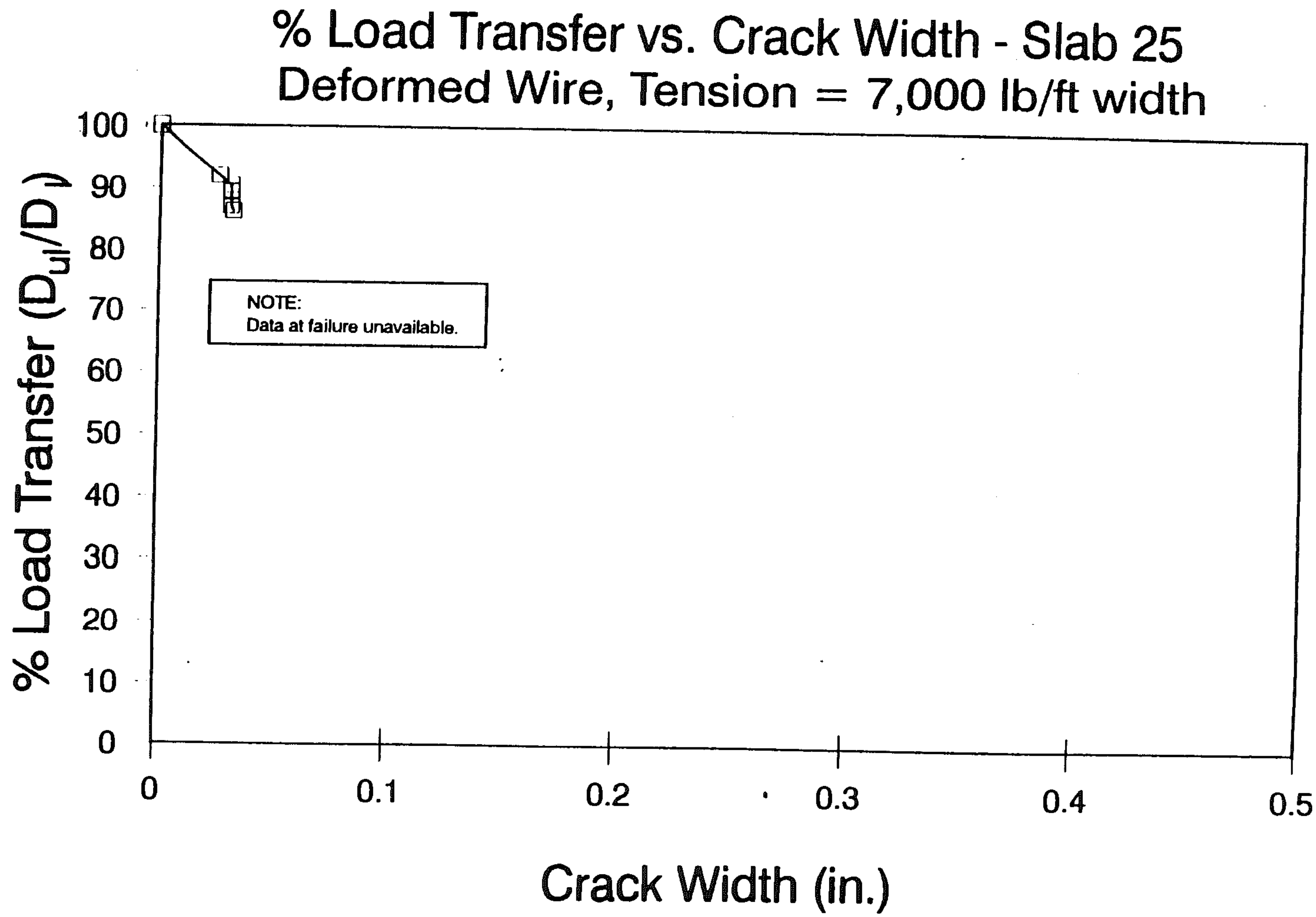

Figure D-40. Percent load transfer versus crack width for slab 25.

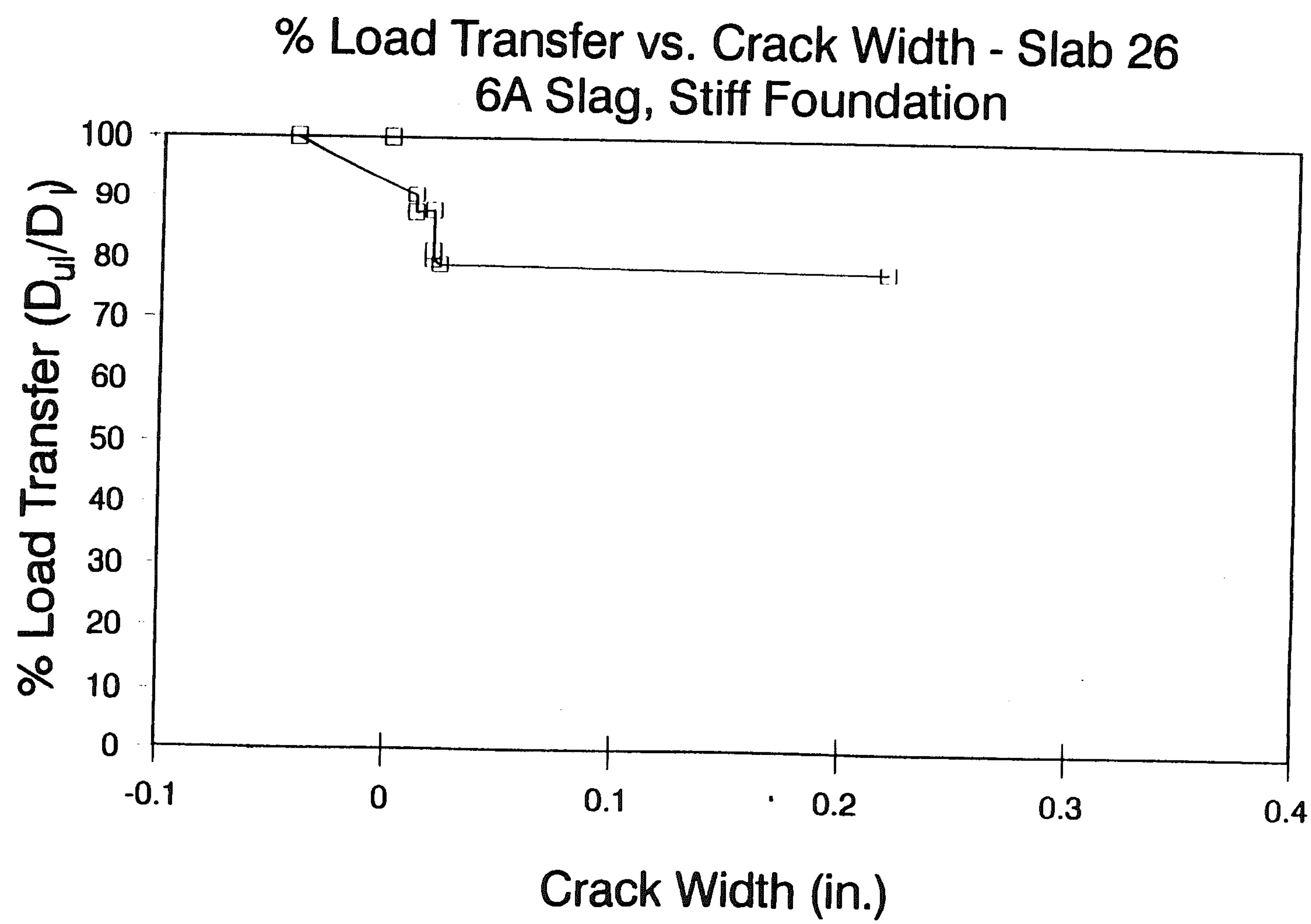

Figure D-41. Percent load transfer versus crack width for slab 26.

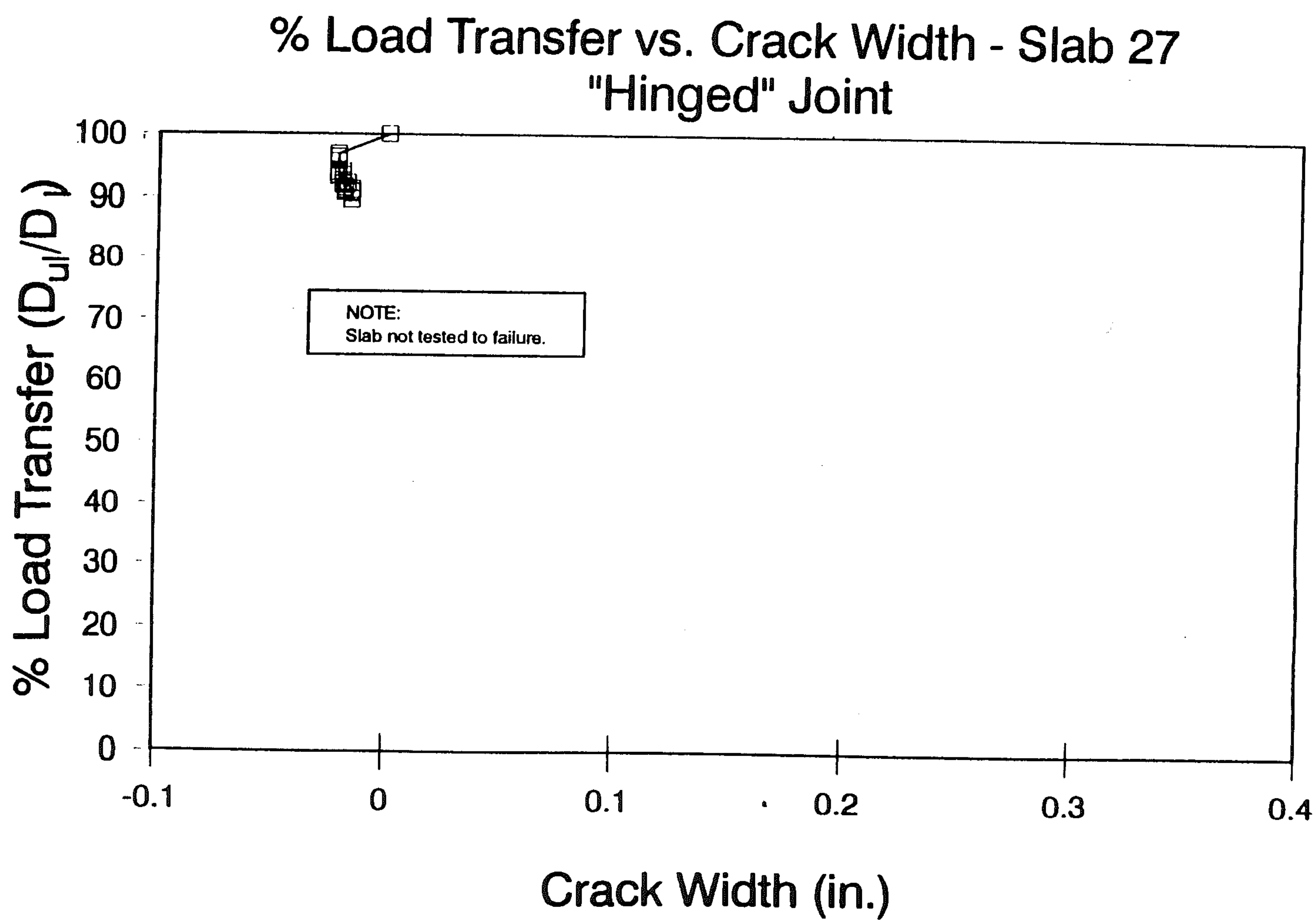

Figure D-42. Percent load transfer versus crack width for slab 27.

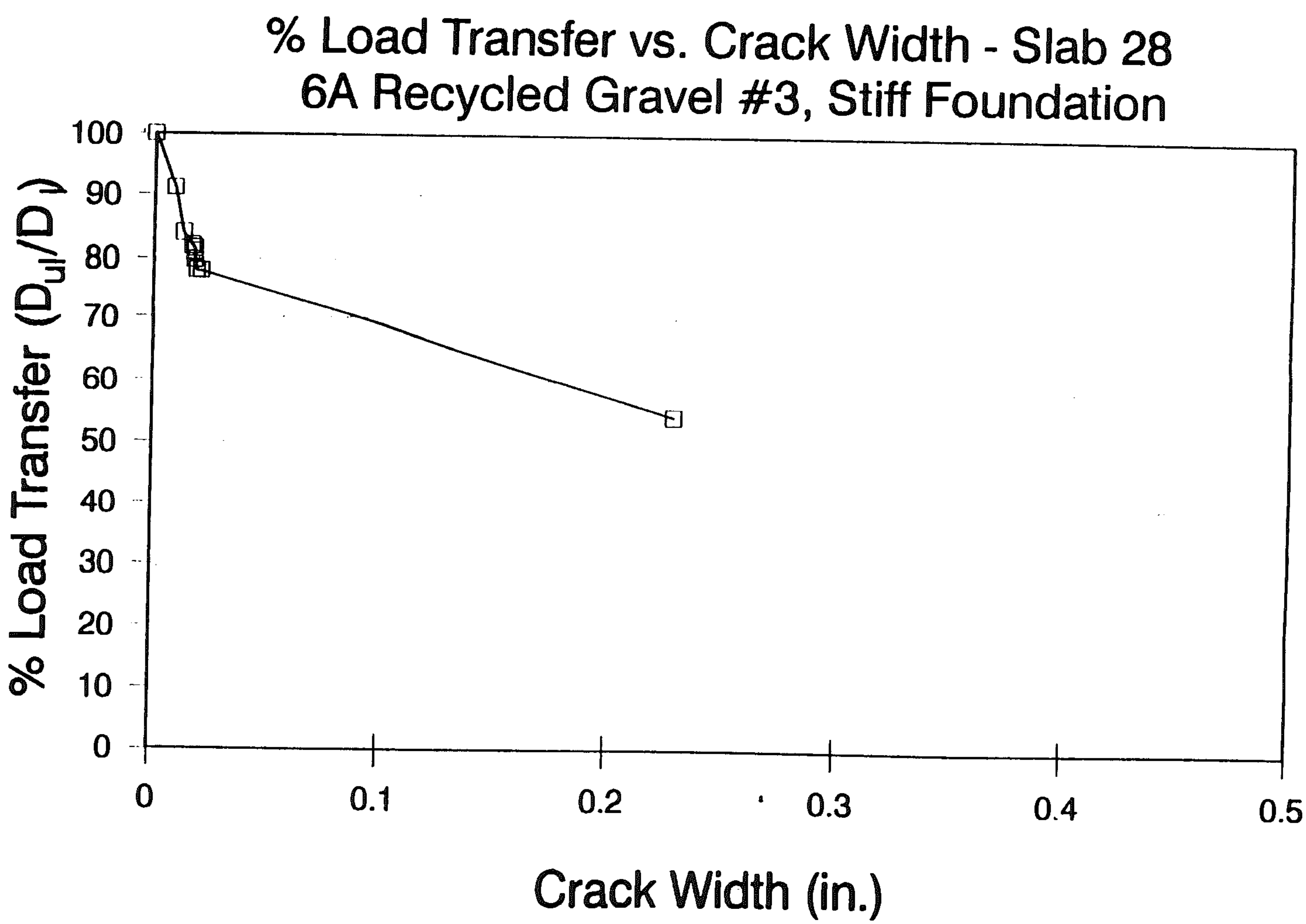

Figure D-43. Percent load transfer versus crack width for slab 28.

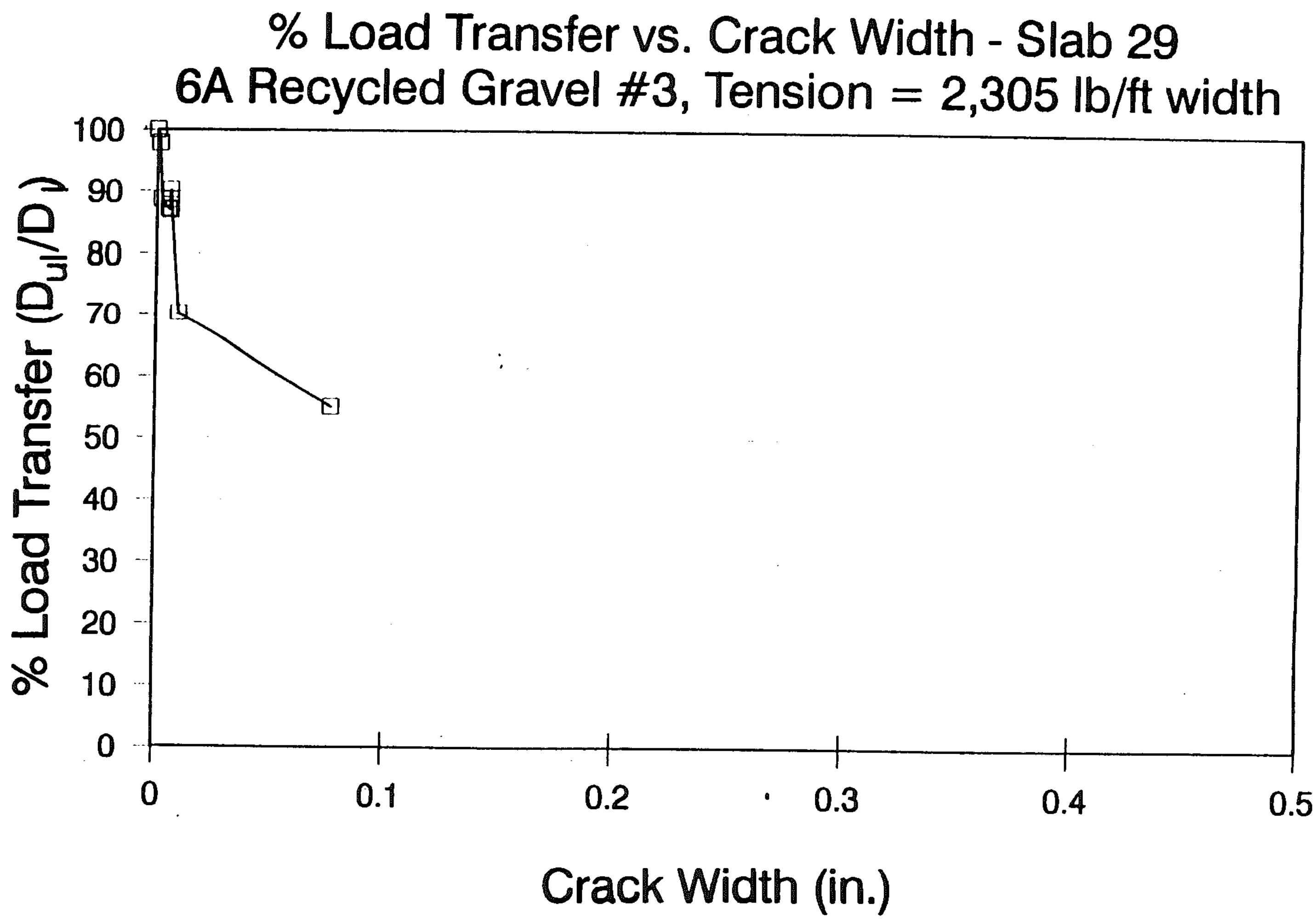

Figure D-44. Percent load transfer versus crack width for slab 29.

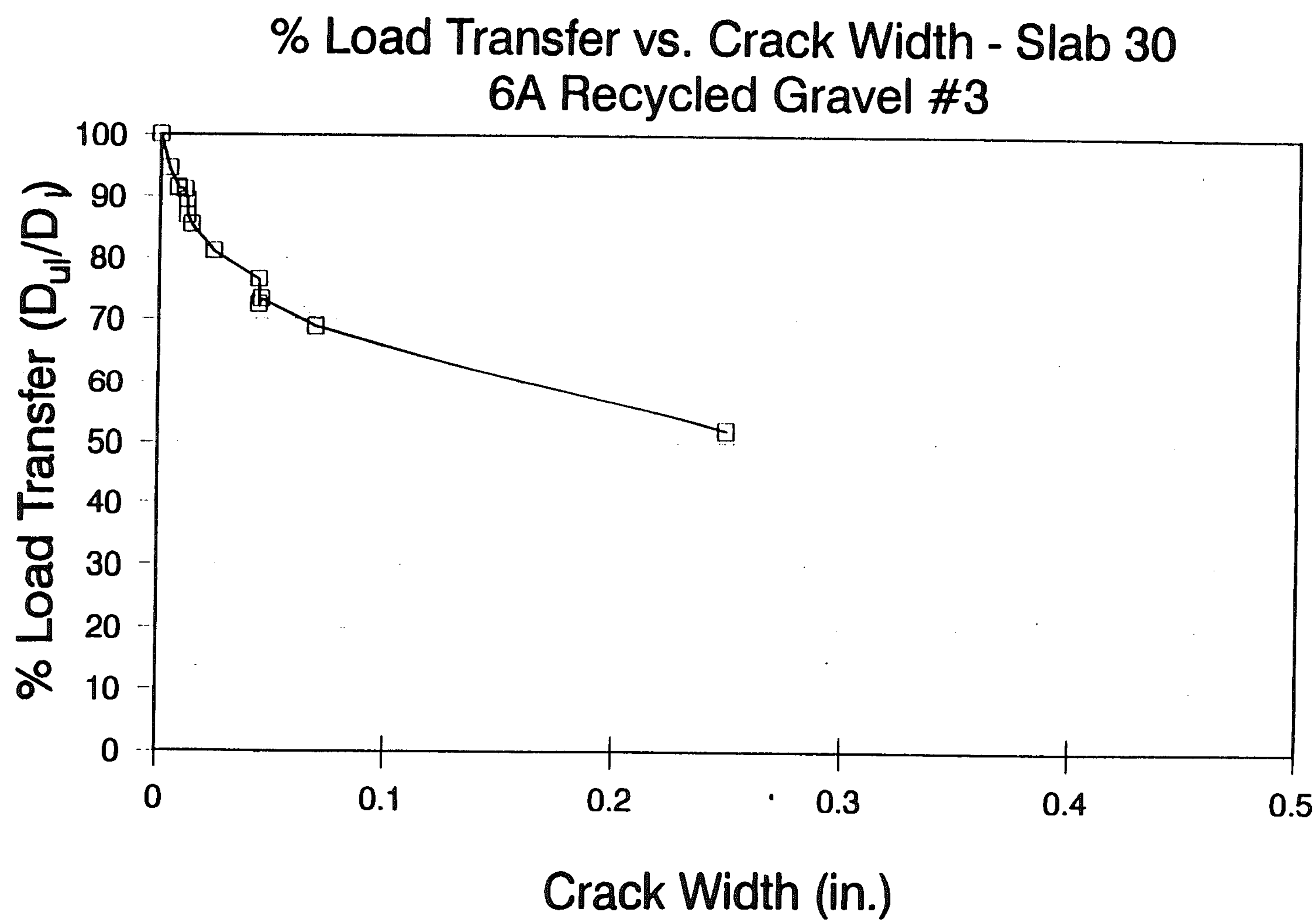

Figure D-45. Percent load transfer versus crack width for slab 30.

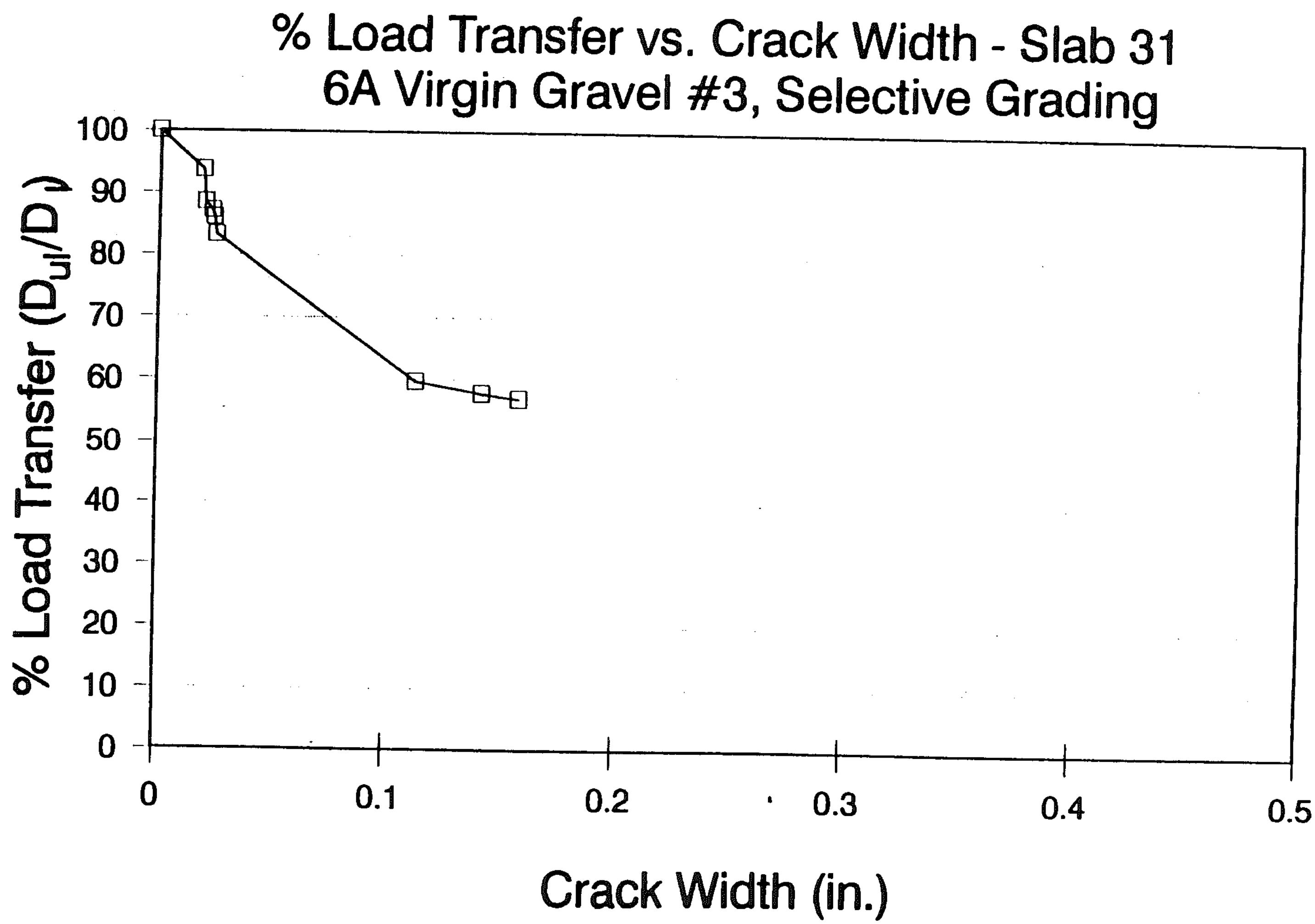

Figure D-46. Percent load transfer versus crack width for slab 31.

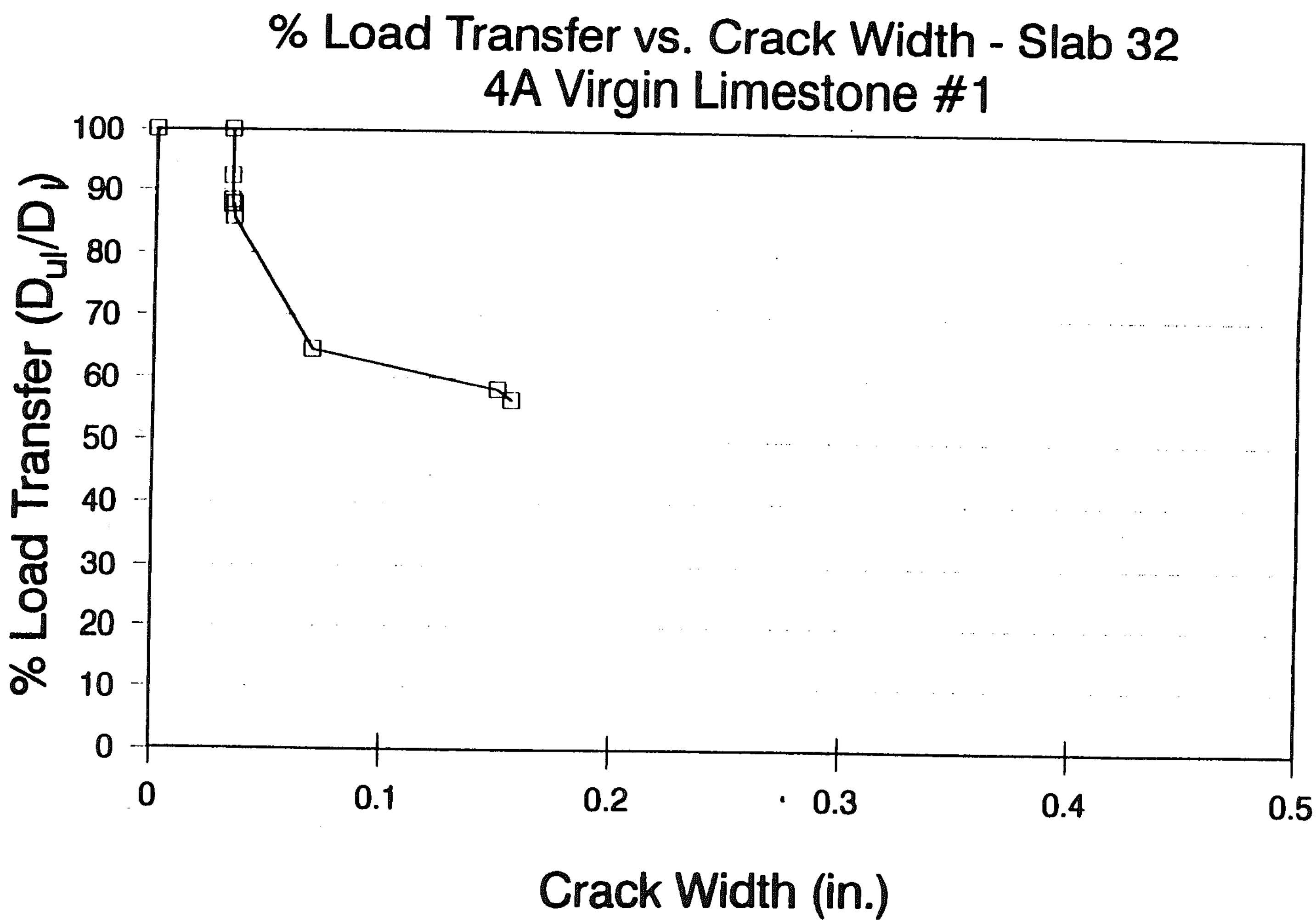

Figure D-47. Percent load transfer versus crack width for slab 32.

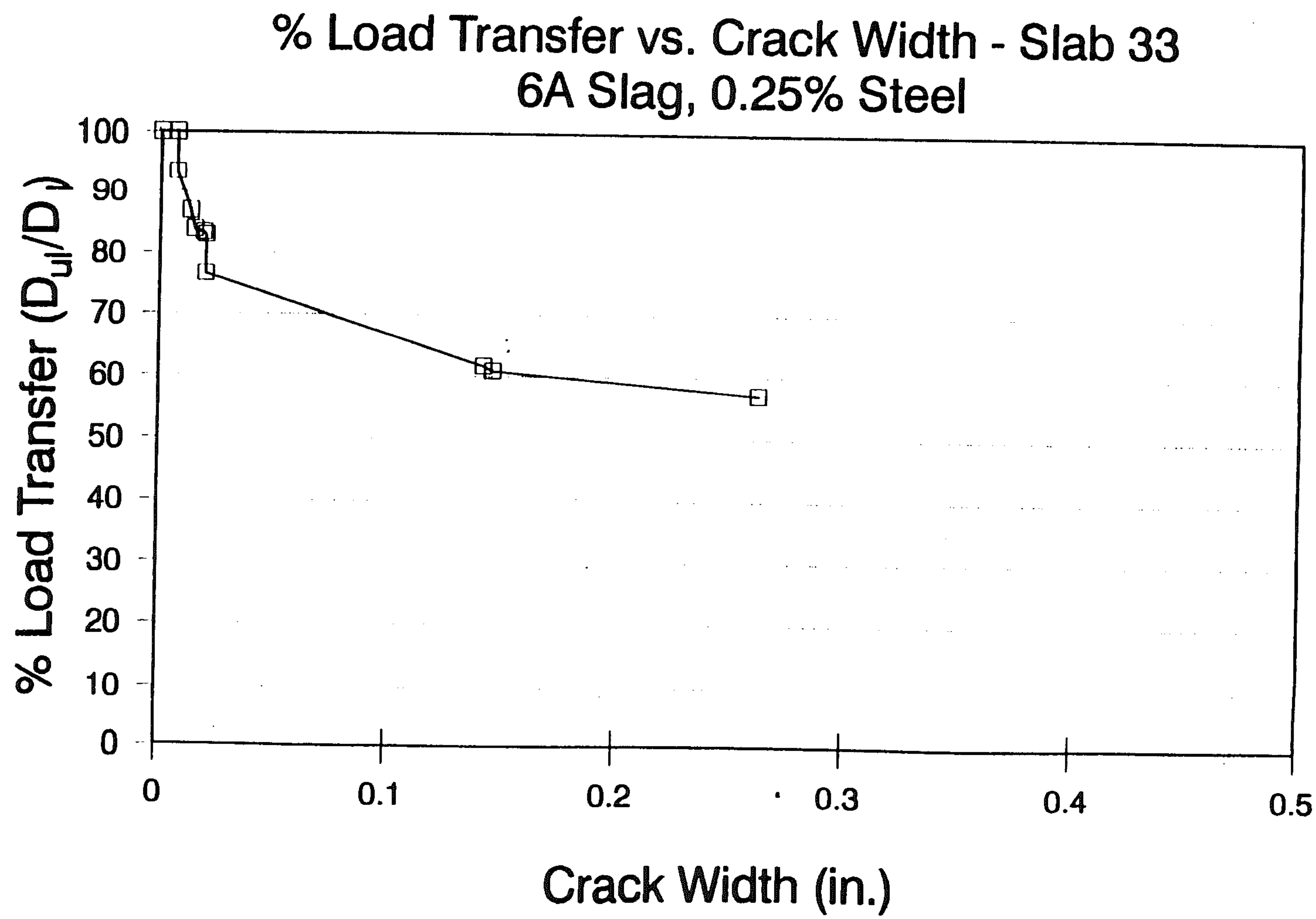

Figure D-48. Percent load transfer versus crack width for slab 33.

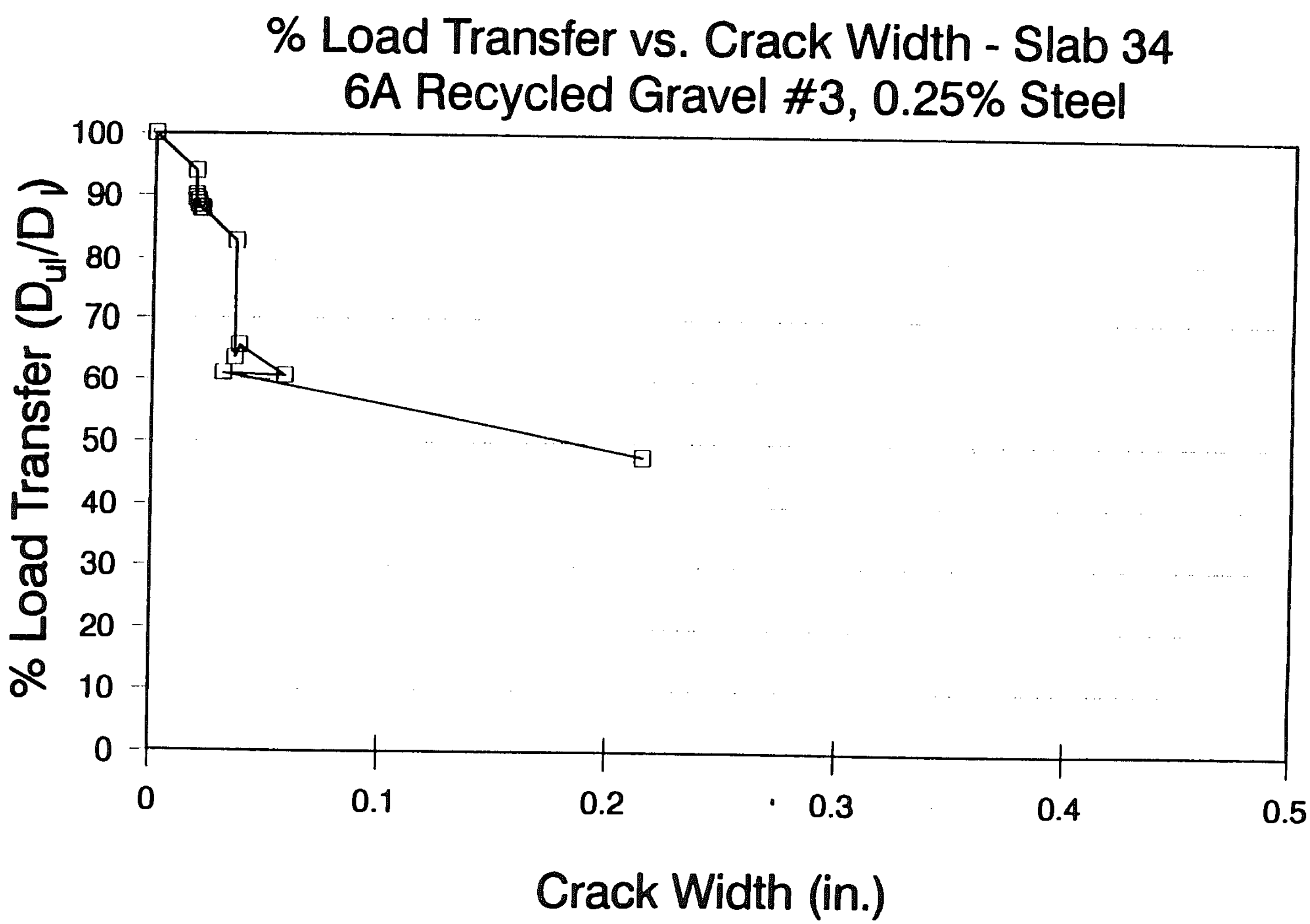

Figure D-49. Percent load transfer versus crack width for slab 34.

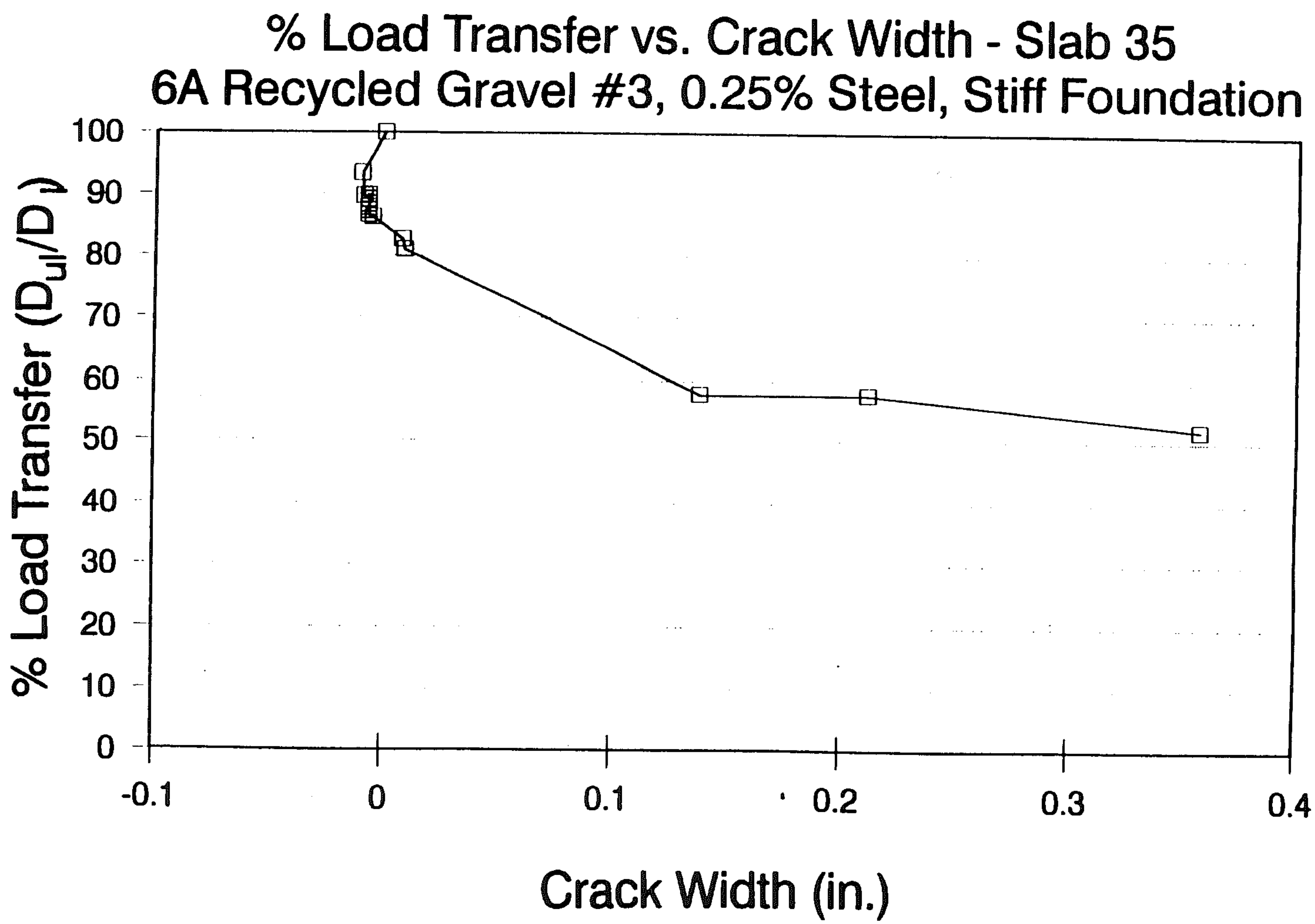

Figure D-50. Percent load transfer versus crack width for slab 35.