EFFECTS OF TRAFFIC INDUCED VIBRATIONS

ON BRIDGE DECK REPAIRS

by

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16. Abstract (Limit: 200 words)

The effects of traffic induced vibrations on concrete-steel bond strength and concrete compressive strength was studied for full depth repairs of reinforced concrete bridge decks. The specimens had blockouts to represent full depth patch repair areas. Two bar sizes, #5 and #8, two top covers, 3 in. and 1-1/2 in., and four slumps, ranging from 1-1/2 in. to 7-1/2 in., were used. The bond tests used modified cantilever specimens. Standard 6 in. x 12 in. cylinders were used for the study of the effect of traffic induced vibrations on compressive strength.

Based on the results, traffic induced vibrations are not detrimental to the quality of repair concrete when low slump concrete is used and the reinforcing bars are securely fastened to the structure before the concrete placement. They may be detrimental when medium slump (4 to 5 in.) concrete is used. They are, however, detrimental to the quality of repair concrete when high slump concrete is used. Bond strength of #5 bars is affected more detrimentally than that of #8 bars when subjected to the traffic induced vibrations.

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Introduction

1.1 General

Reinforced concrete bridge decks are repaired to insure structural safety and to restore riding comfort. For a number of years there has been concern about maintaining traffic on bridges during the repair process. The concern has centered on the detrimental effects that traffic induced vibrations may have on the quality of the concrete used in the repair. It is important, therefore, to know if the vibrations weaken the repaired deck.

Effects of vibrations on both plastic and hardened concrete are well known. Vibrations help to consolidate plastic concrete, while vibrations of considerable intensity may be sustained without damage by hardened concrete (6,9). However, not much is known about the effects of vibrations, especially traffic induced vibrations, on concrete which is still setting up.

Relatively few studies (17,19,23,24,26) have considered the effects of maintaining traffic on bridge decks with newly placed repair concrete. Most of the evidence suggests that maintaining traffic during placement, setting, and curing of repair concrete does not lower its quality. Core analysis, compression tests, observation of bridge deck cracking and bond tests have been used in the past (17,18,19,24) to study the effects of traffic induced vibrations on newly placed concrete in widened decks and overlays. Some of the work is not applicable because the vibrated and control specimens did not have the same geometry and strength (24), or fabrication procedures (8). Doubts still remain.

In light of the importance of the problem, this study was undertaken to determine whether vibrations caused by traffic are detrimental to newly placed repairs in reinforced concrete bridge decks.

1.2 Background

When placed in the forms, concrete is honeycombed with irregularly distributed air pockets (2). When vibrated in this plastic state, it readily assumes the shape of the form. Vibrations consolidate the plastic concrete by momentarily liquefying the material. A vibrator sets the particles in motion and reduces the internal friction to about five percent of the value at rest (3). The mixture becomes fluid and attains a denser configuration due to gravity. As the hydration process progresses, the concrete becomes stiffer and finally becomes hard within a few hours. Hardened concrete a few days old can sustain vibrations, such as those produced by traffic, without causing any structural damage. Little is known about the effect vibrations have on newly placed concrete which is in the process of setting up. Can vibrations be applied to such concrete without damaging its quality? If so, what is the intensity of such "safe vibrations"? Unfortunately there is insufficient data available to indicate a reasonably precise value. A few studies (10,14,28) have been done to determine safe intensities of vibrations which can be applied to brick , stone masonry, and concrete buildings.

Crandell (10) expresses the intensity of "safe vibrations" for building structures, such as schools and churches, in terms of an energy ratio. The energy ratio is the ratio of the square of the acceleration to the square of the frequency for a given type of vibration. After studying the effects of blast vibrations on various residential buildings, Crandell concluded that vibrations having energy ratios up to 3.0 can be safely sustained by this type of structure.

Other researchers (6,9,28) express the intensity of vibration in terms of the peak particle velocity. Northwood et. al (28) studied the effect of successively increasing the intensity of blast vibrations on 30 to 70 year old houses made of brick masonry. The newer houses had concrete basements. They found that the extent of damage correlates better with peak particle velocity than with acceleration. Hence, they recommend that the peak particle velocity be used to express the intensity of vibration; 3 in./sec. appears to be the threshold intensity of vibration which may cause minor damage, such as plaster cracking.

Akins and Dixon (6) found that peak particle velocities developed by construction equipment, such as dozers and pavement breakers, are typically less than 0.2 in./sec. Since experience shows that vibrations from such equipment are harmless to concrete during its placement and curing, they feel that a peak particle velocity of 0.2 in./sec. is "safe" for concrete from the time of placement to an age of one day. They feel that data is limited on the effect of vibrations on concrete from one to seven days old, hence, their recommendation for a "safe" level of vibrations for such concrete (2 in./sec.) is based on work done on mature concrete (14,28).

Schmidt et. al (9), based on a survey of German literature, as well as limited testing by the German cement industry, found that vibrations can have different consequences depending on intensity, duration, and the time elapsed after concrete placement. The critical period for the detrimental effects of vibration on young concrete extends from three to fourteen hours after preparation. During this period, the stresses caused by high intensity vibrations may exceed the low strength of young concrete, and a reduction in strength may occur. They feel that particle velocities up to 0.79 in./sec., at amplitudes less than 0.028 in., have no adverse effect on properly compacted young concrete of "normal use". They also specify a peak particle velocity of 2.5 in./sec. as safe for hardened concrete with a compressive strength of 800 psi.

In 1980, Montero (26) followed the construction process of a bridge widening and carried out vibrational analysis of that bridge. He calculated a maximum particle velocity of 4.87 in./sec. using service loads. This value is much higher than the 0.2 in./sec. recommended by Akins and Dixon (6) for one day old concrete. It even exceeds the value of 2.0 in./sec. recommended by them for concrete that is one to seven days old, and 2.5 in./sec. recommended by Schmidt et. al (9) for concrete having 800 psi strength. Montero pointed out that since the peak particle velocity of 4.87 in./sec. far exceeds that recommended by Akins and Dixon, there is a possible detrimental effect on the concrete in the widened part of the deck. He also suggests the possibility of a loss of bond along the longitudinal joint between the old and new parts of a deck.

In 1980, Manning (23) analyzed the available information on the effects of traffic induced vibrations on bridge deck repairs. He reviewed current deck repair practices followed by state highway and transportation agencies and summarized the work done by other researchers on the effects, measurement, and analysis of traffic vibrations. He came to the conclusion that, provided the concrete is well-proportioned, traffic induced vibrations are not detrimental to the quality of deck repairs.

Deteriorated decks are frequently repaired to maintain their durability. Sometimes, field engineers are concerned that traffic induced vibrations will cause defects in freshly poured concrete (23). The possibility of damage makes it difficult to decide whether to allow traffic during the deck repair or to reroute it. Rerouting involves cost and inconvenience to the public. According to Manning (23), such costs are estimated to be in the range of 10 to 20 percent of the total repair bill. Clearly if traffic vibrations are harmless to the quality of the repair concrete, the cost of rerouting traffic can be eliminated. On the other hand, if these vibrations are harmful to the quality of the repair concrete, they must be avoided, or reduced to "safe" levels by measures such as lane closure or speed limits during the repair.

1.3 Previous Work

A number of studies have been done to determine the effects of delayed vibration, revibration, or continuous vibration on newly placed concrete. Both the source and the time of application of vibrations have differed.

In delayed vibration studies, concrete is vibrated a few hours after the initial placement with or without initial compaction by hand tamping (23).

In revibration studies, a vibrator is reapplied to concrete within a few hours of initial consolidation at the time of placement (31). These vibrations are applied while a running vibrator can still sink into the concrete under its own weight and reliquefy it (2,31).

In the studies involving continuous vibration, concrete specimens have been subjected to vibrations from some source for several hours, beginning at the time of placement. Pile driving, laboratory vibrators, and traffic have been used to induce the vibrations (8,17,24).

In 1938, Davis, Brown and Kelly (12) studied the effects of delayed vibration and sustained jigging. The delayed vibration study used 7/8 in. diameter, 18 in. long deformed test bars placed vertically in 6 in. X 6 in. cylindrical specimens. 4 in. slump concrete was consolidated in the molds by hand tamping. The delayed vibrations were applied 0 to 9 hours after placement by vibrating clamped specimens on a 3600 rpm vibrating table for 15 seconds, connecting the top end of vertical test bars to the shaft of a 7500 rpm internal vibrator for 30 seconds, or operating a 3000 rpm air hammer axially against the top end of a test bar for 30 seconds. Increases in ultimate bond strength up to 62% were recorded, and the effect of delayed vibration up to 9 hours of placement was found to be positive in all cases.

In the sustained jigging tests, 1 in. plain round bars, surrounded by a 4 in. helix of 3/16 in. wire, were embedded vertically in 6 in. X 6 in. specimens which were rigidly connected to a jigging table vibrated with an amplitude of 1/4 in. at 300 rpm. The concrete had water-cement ratios of 0.58 and 0.53. Slump values for the jigged specimens were not reported, however. Jigging was begun after the concrete was placed by hand tamping. In all cases, improved ultimate bond strengths at 28 days were found. Bond strength increased as the period of jigging was increased from 1/2 hour to 2 hours, after which there was no significant change for jigging up to 6 hours. No specimen was subjected to jigging after 6 hours of placement.

In a field study, Davis et. al (29) observed detrimental effects of vibrations on test specimens due to transportation. Specimens were moved from a job site to a field laboratory by train shortly after they were made. The concrete slump is not provided, but it should have been in the range of 5 to 6 in. based on the method of construction used. They observed that most of the 3/4 in. aggregate had segregated to the bottom half of the 12 in. cylinders. The top third of the cylinders had such a high water-cement ratio that failure in compression occurred principally in this portion of the specimens. The time taken to transport the cylinders from the site to the laboratory is not provided, but the study shows the effects of short-term continuous vibration on the compressive strength of high slump concrete.

In 1942, two separate studies (22,25) showed the negative effects of revibration on both bond and concrete compressive strength. Larnach (22) used 4 in. X 4 in. X 6 in. specimens with a horizontally cast 1/4 in. black mild steel round bar. Water-cement ratios between 0.44 and 0.53 were used. Slump values were not reported. Relative movement between test bar and steel mold was prevented during placement and revibration. The test bars were positioned at the center of the specimen cross section and had a 6 in. embedment length. Initial consolidation was obtained using both external and surface vibration. The specimens were revibrated for 2 minutes at varied intervals by applying a 6000 rpm vibrator to the top flanges of the mold. 4 in. cubes were also made in the same manner for studying the effect of revibration on compressive strength. The revibration caused a reduction in bond strength. The reduction was less than 9 % for a interval of one hour or less between the time of placement and revibration. But when the interval was increased from one to three hours, the reduction in bond increased to 33 %. Similarly for compressive strength, revibration had no significant effect until the period between placement and revibration was 30 minutes or more. The detrimental effects of revibration on bond and compressive strength were most pronounced when revibration was applied 3 to 6 hours after placement. The 33% reduction for revibration 3 hours after placement was the maximum reduction obtained in bond strength. A maximum reduction in compressive strength of 19% occurred when revibration was done 6 hours after placement. No specimens were revibrated more than 6 hours after placement.

In addition to the revibration, Larnach (22) studied the effects of relative movement of the test bars during placement and consolidation. 1/4 in. diameter mild steel bars were placed horizontally in 4 in. X 4 in. X 6 in. specimens. The bars were free to move within the molds. Water-cement ratios of 0.44, 0.48 and 0.53 were used, representing progressively more fluid concrete (slumps were not reported). During placement, the concrete was the consolidated up to a level above the test bar by placing a 6000 rpm vibrator on the bar. The bond strength was improved by 2% for the water-cement ratio of 0.44. There were reductions of 3% and 5% in bond strength when the water-cement ratio was increased to 0.48 and 0.53, respectively.

Menzel (25) used 4 7/8 in. X 18 in. X 15 in. specimens with 3/4 in. deformed bars, possessing 2 in. concrete cover, to study the effect of revibration on bond strength. Each specimen contained two horizontally cast test bars, one 2 in. below the top face and the other 2 in. above the bottom face. An internal vibrator was used for both initial consolidation at the time of placement and revibration one hour after placement. The test bars were held in place during vibration. Initial vibration was applied for a period which "was judged to be right amount of internal vibration." The concrete used had a slump of 2 to 3 in. The bottom-cast bars were largely unaffected due to revibration, while the top-cast bars had more than a 60% drop in ultimate steel stress at splitting during the pullout tests.

In 1957, Vollick (30) obtained results which were quite the opposite of those obtained by Larnach (22). Revibration applied up to 4 hours after placement resulted in an improved 28 day compressive strength using 6 X 12 cylinders. 28 day impact hammer readings gave similar results. An internal vibrator was used for both initial placement and revibration. The concrete used had a 3 in. slump. The impact hammer indicated a maximum increase in compressive strength of 22.5%, with an average of 14 %. Based on test cylinders, revibration improved the average concrete compressive strength by 19%.

The contradiction between Vollick's and Larnach's results is difficult to explain. One possible difference could have been the period for which concrete was revibrated. Larnach revibrated for 2 minutes, while Vollick revibrated for 20 seconds. Since Larnach did not provide the plastic concrete properties, his results must be viewed with caution.

In 1970, Bastian (8) seemed to show that vibrations caused by pile driving operations do not have a detrimental effect on the compressive strength of concrete. To establish this, he removed 4 in. diameter cores from concrete piles which were constructed at a pile driving site. The cast-in-place piles were subjected to vibrations from nearby pile driving for a period of seven hours beginning at the time of placement. The cores were tested for compressive strength after four days. 4 in. diameter control cylinders were also made and tested at four days. He found that the test cores had about 4 % higher strength than the control cylinders. The results, however, cannot be considered valid because the test specimens (i.e., the cores taken from the piles and the control cylinders) were not the same.

Furr and Ingram (18) studied bonded concrete overlays on bridge decks. In one of the test series, they attached freshly poured concrete test cylinders to the bridge decks. The decks were subjected to continuous vibrations at a frequency of 400 rpm and an amplitude of 0.044 in. at the time of placement, which was gradually reduced to 0.008 in. at the end of 48 hours. The slump of the concrete was not reported. However, virtually all overlays require low slump concrete (less than 1 in.). According to Furr and Ingram the cylinders subjected to the vibrations showed "considerably" higher strengths than non-vibrated control cylinders.

In 1974, the Massachusetts Department of Public Works (24) conducted a study of the effects of traffic induced vibrations on concrete-steel bond strength. Two types of specimens were made and tested simultaneously. While the field specimens, made and tested at a bridge widening site, were subjected to traffic induced vibrations, the laboratory specimens were made under controlled laboratory conditions and were not subjected to any traffic vibrations. During the pullout tests, all the field test bars failed in tension at the ultimate force of the bar without splitting the concrete. The laboratory specimens, however, did exhibit concrete splitting and test bar pullout. Comparing the results from the two types of specimens, it was concluded that bond strength is not affected by the traffic induced vibrations. Unfortunately, the field and control specimens differed in both geometry and compressive strength. The field specimens were an integral part of an 8 1/2 in. thick deck slab that had additional bars placed in both directions, while the laboratory specimens were 6 in. X 6 in. X 21 in. long. The #6 test bar was the only steel in the laboratory specimens. The field specimens had an average strength of 5750 psi, while the laboratory specimens had an average strength of 4400 psi. Hence, the results are of little use in determining the effects of traffic induced vibrations on bond strength.

In 1981, Furr and Fouad (17) studied the effects of maintaining traffic on existing lanes of a bridge while it was being widened. The study consisted of both field and laboratory investigations. The field investigation included the visual inspection of 30 bridges which had been widened under traffic, a comparison of cores from areas disturbed by traffic induced vibration with those from undisturbed areas, and field measurements of traffic induced vibrations. The laboratory investigations included a study of the effects of traffic induced vibrations on beams prepared in the laboratory, and a core study of the laboratory beams. The laboratory investigations were done to determine the age and the curvature at which the traffic vibrated concrete may develop cracks, and to compare the results of laboratory core study with those from the field core study. The laboratory beams represented a transverse portion of the widened part of a deck. Five 12 in. X 7 in. X 10 ft 8 1/2 in. laboratory beams were made using concrete with a reported slump range of 3 in. to 6 in. They were mounted on flexible supports and were loaded at one end to apply the simulated traffic induced vibrations. When loaded, the beams assumed a curvature equal to the transverse curvature of the new part of the deck when traffic was allowed on the old part. Four of the beams were subjected to a single large intermittent displacement of 0.5 in. or 0.3 in. amplitude applied at a frequency of 0.5 cycles per second at 5 minute intervals for 24 hours, beginning at the time of placement. The fifth beam was subjected to the large intermittent displacements superimposed on small continuous vibrations of 0.02 in. amplitude at 6 cycles per second.

During the field investigations, the visual inspection did not show any damage caused by maintaining traffic during construction. However, a dye test for bond within cores taken from both laboratory beams and the bridge decks showed that loss of bond between the concrete and the steel was possible in some cases. Five of the 109 cores taken from the bridge decks and four of the eight cores from laboratory beams showed clear evidence of relative movement and, hence, loss of bond between the concrete and the reinforcing steel. This relative movement was limited, however, to areas very close to the longitudinal joint, and the core study did not show any movement a few feet away from the joint. By comparing the core strengths taken from the disturbed and undisturbed areas, it was found that traffic induced vibrations did not have an adverse effect on compressive strength. The laboratory beams showed that, for a 7 in. depth, a curvature of about 0.000036 is required to develop cracking in fresh concrete "approximately at the time of set." The field measurements of vibrations showed, however, that the largest curvature encountered was 0.0000114 for a deck having about the same thickness as the laboratory beams. Hence, Furr and Fouad concluded that normal traffic can be allowed during bridge deck widening.

1.4 Object and Scope

The effects of simulated traffic induced vibrations on concrete-steel bond strength and concrete compressive strength are studied for full depth repairs of reinforced concrete bridge decks.

The study consisted of five groups of test slabs. Each group included two traffic vibrated and one control test slab, each 4 ft X 8 ft X 1 ft. The test slabs had 23 in. X 18 in. X 12 in. blockouts to represent full depth patch repair areas. Two bar sizes, #5 and #8, two top covers, 3 in. and 1-1/2 in., and four slumps, ranging from 1-1/2 in. to 7-1/2 in., were used. A total of 40 bars were tested. The bond tests used modified cantilever beam specimens (13). Standard 6 in. X 12 in. cylinders were attached to the test slabs to study the effects of traffic induced vibrations on compressive strength.

The results are plotted and analyzed. Bond values are compared with values obtained from other tests in the series and with those predicted by AASHTO (1) and ACI (4). Compressive strengths of the traffic vibrated and the control cylinders are compared. Recommendations are made for bridge deck repair.

Chapter 2

Experimental Investigation

2.1 General

To study the effects of traffic induced vibrations on bond and compressive strength, 4 ft X 8 ft X 1 ft slabs and 6 in. X 12 in. cylinders were subjected to simulated traffic induced vibrations. The strengths obtained were compared to control specimens which were not subjected to the traffic vibrations. The slabs and cylinders were attached to a 24 ft bridge frame, which in turn was subjected to the vibrations generated by an MTS actuator (Fig. 2.1). Each specimen had blockouts representing full depth repair areas in a reinforced concrete bridge deck. Traffic induced vibrations were represented by continuous small amplitude vibrations on which intermittent large amplitude vibrations were superimposed.

2.2 Test Specimens

The study used five groups of test slabs. Each group consisted of two traffic vibrated and one control slab. The specimens were 4 ft X 8 ft X 1 ft, matching typical 4 to 6 ft transverse spans in reinforced concrete bridge decks. 1-1/2 in. and 3 in. covers and #5 and #8 bars were used. The slabs contained 23 in. X 18 in. X 12 in. blockouts for repair (Fig. 2.2). A #5 or # 8 test bar was used in each blockout, with one bar size used in each vibrated slab. The blockouts were located so that the pullout of one bar would not interfere with another. Dummy bars (not tested) were placed 6 in. on either side of the test bars. The control slabs contained four test bars, while the vibrated slabs contained two.

Test specimens were constructed using timber forms. Form sheathing, made of 3/4 in. A-B plywood, was protected with 3 coats of polyurethane clear gloss finish. Form sides were butted against the form bases and were held in position using double 2 X 4 wales. The wales were connected at corners and third points along the length of the form, with 1/4 in. diameter all-thread rods. The slabs were cast with 23 in. X 18 in. X 12 in. box forms to make the blockouts representing the repair areas (Fig. 2.2). The box forms, made of 3/4 in. A-B plywood, were braced inside with 2 X 2 and 2 X 4 horizontal braces in both directions and had holes to accommodate the bars and pipes for the test slab.

Each traffic vibrated slab had thirty two 5/8 in. diameter mild steel bolts, with a 6 in. embedment, to provide shear connection with the bridge frame. The bolts were spaced at 6 in. and were placed in pairs in double rows 4 ft. 9 in. apart to coincide with the girders on the bridge frame. The test slabs were reinforced with a bottom layer of #5 bars, spaced at 1 ft longitudinally and laterally, supported by 1 1/2 in. chairs. Two #5 bars were placed at the top longitudinally, 4-1/2 in. on either side of the slab center line to carry the cantilever moment due to self weight (Fig. 2.3). A 36 in. #4 bar with 2 1/2 in. top cover was placed diagonally at each internal corner of the blockouts to arrest cracks. Four lifting hooks were provided, two on each short face of the slab.

The test bars extended 18 in. from the face of the slabs to facilitate pullout. The test bars in the traffic vibrated slabs were clamped to the formwork during the simulated traffic vibrations to restrict relative movement between the test bar and the formwork (Fig 2.2 and 2.4). The bars in the control slabs were supported at openings in the forms (Fig. 2.2).

The bonded length of a test bar was limited by using a bond breaking collar of polyvinyl chloride (PVC) pipe. The bond breaking pipe had an inside diameter equal to the test bar diameter. A steel pipe was butted against the unloaded face of the test bar and was coupled to the bar using another piece of PVC pipe (Fig. 2.4). The pipe and bar diameters were equal. The steel pipe was used to provide access to the unloaded face of the test bar for measuring slip values during the pullout test. The joints at the two ends of the bonded length were sealed with G. E. silicone rubber caulk to prevent leakage of mortar. The caulk was allowed to cure before concrete placement.

The embedment lengths of the test bars were initially selected based on earlier studies (13). A 5 in. length was used for the #5 test bars of Group 1. This was reduced to 4 in. for the other four groups because the control test bars of Group 1 yielded before pullout. The #8 bars had a 9 in. embedment length in all five groups.

To study the effect of traffic induced vibrations on the compressive strength of concrete, 6 in. X 12 in. test cylinders were clamped to the traffic vibrated slabs. The steel molds were clamped to the top of the slabs, using three 1/4 in. lead inserts. Control cylinders were filled simultaneously and were placed on the floor. Both ready-mixed concrete and laboratory mixed concrete were used for the cylinders. Eight cylinders, four traffic vibrated and four control, were made using ready-mixed repair concrete for Slab Groups 3-5. A group of 18 cylinders, nine traffic vibrated and nine control, were made using laboratory mixed concrete.

2.3 Materials

2.3.1 Concrete

The concrete for the slabs and the repairs was supplied by a local ready-mix plant. Concrete for the last group of 18 cylinders was laboratory mixed and used the same materials as the ready-mixed concrete. Type I cement and 3/4 in. nominal maximum size coarse aggregate were used. The coarse aggregate was crushed limestone supplied by a local quarry, and the fine aggregate was Kansas River sand. Mix designs for the slabs, repairs, and laboratory mixed concrete are given in Table 2.1.

2.3.2 Steel

All reinforcing bars were ASTM A 615 (7) Grade 60 steel. Stress-strain curves for the #5 and #8 bars used are shown in Fig. 2.5. The bar deformation patterns are shown in Fig. 2.6. The geometric properties were obtained following ASTM A 615 (7) and are given in Table 2.2.

2.4 Construction of Test Specimens

The forms for the three slabs in a group were filled one after another. Concrete was vibrated in place by inserting an internal vibrator at one ft spacings, 10 minutes after placing concrete in the forms. The vibrator was a 1 7/8 in. internal pneumatic vibrator, rated at 11500 cycles per minute, with a 0.04 in. amplitude. The vibrator was inserted in the concrete for about 10 seconds until the surface was creamy. The slab was then hand screeded using a metal edged screed. Two passes of the screed were made in the longitudinal direction of the slab. After screeding, the concrete was floated using a magnesium bull float. It was covered with a polyethylene sheet to prevent loss of moisture until the concrete had gained a strength of at least 3000 psi. The polyethylene sheet was then removed, and the forms were stripped.

The blockouts in the three slabs were cleaned using a water blaster rated at 2000 psi until all laitence and carbonation were removed. After the water blasting, the two traffic vibrated slabs were bolted to the bridge frame, and the formwork for the repair areas was secured. Side forms were secured to the face of the slab using twelve 1/4 in. diameter all-thread rods embedded in the slab (Fig. 2.4). The base forms were fitted snug between the webs of the two beams of the bridge frame and were supported by 2 X 4s resting on the bottom flanges of the two beams. Fig. 2.4 shows the arrangement provided to restrict movements between the test bar and the side form during the vibrations. The control slab was moved back to its base form on the floor and tied with 1/4 in. diameter all-thread rods.

Concrete was placed in the repair areas within 24 hours of water blasting. Repair areas with same size test bars in traffic vibrated and control slabs were cast simultaneously. The repair concrete was consolidated 10 minutes after placing it in the forms, using an 1-1/2 in. internal electric vibrator. The vibrator was inserted 5 in. from each corner of the repair areas for about 10 seconds. The concrete was hand screeded using a metal edged screed. Two passes of the screed were made perpendicular to the test bars. Following screeding, the repair was floated with a magnesium hand float.

While concrete was being placed in the blockouts, eight 6 in. X 12 in. concrete cylinders were prepared. Two cylinders were clamped to each of the traffic vibrated slabs at the longitudinal center line (Fig. 2.1). Four control cylinders were placed on the floor next to the bridge frame. The traffic vibrated and control cylinders were handled identically prior to the application of the traffic induced vibrations.

2.5 Application of Traffic Induced Vibrations

2.5.1 Load Frame

To apply simulated traffic induced vibrations to the bridge frame, a vertical load frame was built to support an MTS actuator (Fig. 2.1). It consisted of two W12X50 A 36 columns and a pair of MC18X58 sections connecting the two columns at 11 ft 3 1/2 in. height. The columns were prestressed to the laboratory structural floor through 27 in. X 30 in. X 1-1/2 in. plates attached to each. 1-1/4 in. diameter Howlett prestressing bars and a 60 ton hydraulic jack were used to apply 90 kips prestressing force (after all losses). The connections for both the load frame and bridge were friction type and used high strength bolts. Lock nuts were provided for all connections for protection against the vibrations.

A 55 ton capacity MTS actuator was attached to the load frame by four 1-1/2 in. mild steel rods. The rods were prestressed to 25 kips each, connecting the swivel head to the cross beam of the load frame. Four 1 in. diameter mild steel load rods, with a prestressing force of 10 kips each, connected the swivel base to the bridge frame (Fig. 2.1).

2.5.2 Bridge Frame

The bridge frame consisted of two 25 ft W12X50 sections, braced at three locations along the 24 ft simple span (Fig. 2.1). Two braces, W6X9 sections, were provided 4 ft from each support, and the third, a W8X24 cross beam, was clamped over the two beams at midspan with a prestressing force of 40 kips. The cross beam transferred the traffic induced vibrations to the bridge frame. To prevent the bridge frame from lifting off its supports during the application of the traffic vibration, the weight of the frame was increased by attaching two dummy slabs to the frame 3 ft from each support.

The traffic vibrated slabs were bolted to the bridge frame 2 ft 11 in. on either side of midspan (Fig. 2.1). A linear variable differential transformer (LVDT) was connected to each slab. The core rods of the LVDTs were connected at the center of the slabs, using four 1/4 in.

diameter bolts that had been cast into the slab. The LVDTs were mounted on vertical stands supported on the floor. One LVDT was used as the feedback transducer for the closed-loop control of the actuator. Both LVDTs served to monitor the amplitude and frequency of the traffic vibrations. Tracings of the traffic vibrations were obtained at regular intervals on a paper recorder.

2.5.3 Traffic Vibrations

The simulated traffic induced vibrations were produced by the MTS closed-loop electro-hydraulic testing system. The slab centerline vibrations consisted of continuous small vibrations of 0.04 in amplitude (peak to peak) at 4 Hz frequency, on which intermittent large vibrations of 0.5 in. amplitude (peak to peak) at 0.5 Hz frequency were superimposed at 4 minute intervals. The amplitudes and frequencies of the small and the large vibrations were selected based on the field measurements of vibrations done by Furr and Fouad (17) and Csagoly et. al (11), and load history studies done by Kennedy (21). The intermittent large vibrations correspond to passage of a heavy truck, at 55 miles per hour, on a 240 ft three span bridge which is vibrating at its natural frequency in response to the flow of traffic (represented by the continuous small vibrations). Simulated traffic induced vibrations were started 10 minutes after the repair concrete was floated and continued for 30 hours. A typical time trace of the vibrations, as recorded on an oscilloscope connected to the feedback LVDT, is shown in Fig. 2.7.

The repair area forms remained in place until the concrete had reached a strength of 3000 psi as measured by the control cylinders. The three test slabs were then moved to the pullout test area using a 3 ton capacity overhead crane.

2.6 Pullout Tests

The specimens were designed as modified cantilever beam specimens (13) providing a more realistic bond test than can be obtained with a direct pullout test; the former ensures that both the steel and the concrete surrounding the bar are in tension during pullout, while the direct pullout places the concrete in compression.

The pullout tests were performed using the frame shown in Fig. 2.8 (13). The two 60 ton capacity hydraülic jacks were powered by an Amsler pump. The reactive force of the jacks during the pullout test was transferred to bottom half of the test slab face by the frame (Fig. 2.8). The frame and the test slab were tied down to the structural floor to prevent either from lifting during the test. Each jack had a 1 in. diameter load rod made of cold rolled steel and instrumented with strain gages. The two load rods transferred the load to a pair of 2 in. X 5 in. cold rolled bars placed at right angles to the rods, which were attached to the test bar through a rocker and wedge grip assembly.

To read the bar slip, spring loaded LVDTs were used. Two LVDTs were attached to the loaded end of the test bar with LVDT core rods bearing against the face of the slab. A third LVDT was attached to the steel pipe and was spring loaded against the unloaded face of the test bar. The "loaded end" LVDTs were attached to the test bar 1-1/4 in. from the face of the slab. Both load rods and the three LVDTs were monitored using the data acquisition system.

During the test, the load was increased at approximately 2 kips per minute for the #5 bars, and 3 kips per minute for the #8 bars. Pullout force versus loaded end slip was plotted during the test. For the #8 bars, load and slip readings were recorded first at every 10 second interval, then at 5 second intervals as the ultimate bond force was approached. These readings were recorded every 5 seconds for #5 bars. A test was stopped when the unloaded end slip was about twice the value corresponding to the ultimate bond force, and the load had dropped 5 kips and 1 kip for a #8 and a #5 bar, respectively.

Pullout tests on each group of three slabs, at ages ranging from 4 to 10 days, were performed during a 10 hour period. The control and traffic vibrated test cylinders and the cylinders for the slab concrete were tested immediately following the pullout tests.

2.7 Test Results

2.7.1 Pretest Observations

Repair areas were inspected with a magnifying glass before the pullout tests. The repairs in Group 4 had settlement cracks. Each crack had about the same length and tended to follow the test bars and dummy bars in both the control and the traffic vibrated slabs. This group had 1-1/2 in. cover and had been "repaired" with 4 in. slump concrete.

The repair areas in Group 5 (1-1/2 in. cover and 7-1/2 in. slump) had both shrinkage and settlement cracks. The intensity of the shrinkage cracks and the length of settlement cracks were about the same for control and traffic vibrated slabs. The shrinkage cracks were uniformly distributed over the repair area, while the settlement cracks tended to follow the test and dummy bars, as in the Group 4 slabs.

Group 1,2 and 3 (3 in. cover, and 1-1/2 in., 4-1/2 in., and 1-1/2 in. slump, respectively) exhibited no cracking.

2.7.2 Pullout Tests

Typical load versus loaded end slip, and load versus unloaded end slip curves are presented in Fig. 2.9 and 2.10. A summary of test data is presented in Table 2.3.

The failure modes upon pullout were dependent on cover and bar size. All failures could be described as splitting failures, except those for #5 bars with 3 in. cover, which did not exhibit any splitting.

For the #8 bars with 3 in. cover, longitudinal cracking was observed when the pullout force reached about half of its ultimate value. It started above the PVC bond breaker and advanced toward the unloaded end of the test bar. As the ultimate load was approached, the crack grew in length and width, while transverse cracks appeared at 60 to 70 degrees from the longitudinal crack. Once the ultimate bond force was attained, the load dropped slowly at first and then at a faster rate. The bar pulled out by splitting the concrete. Generally, both a horizontal crack between the two dummy bars and a vertical crack below the test bar through the depth of the specimen were observed at pullout. A typical crack pattern is shown in Fig. 2.11.

For the #8 bars with 1-1/2 in. cover, a longitudinal crack started and grew in the same way as it did for the #8 bars with 3 in. cover. The crack, however, appeared at a lower load, and overall, the splitting cracks during pullout were more numerous, longer and sometimes wider, than for the bars with the 3 in. cover. The load dropped sharply after reaching ultimate, and the bars pulled out at a faster rate than the bars with 3 in. cover, sometimes splitting the top cover concrete. A horizontal crack between the two dummy bars and a crack below the test bar generally appeared as the pullout force reached ultimate.

The #5 bars with 3 in. cover never cracked or split the concrete, even after the bar pulled out. Unlike the #8 bars, the #5 bars pulled out of the concrete by shearing the local surrounding concrete. Once the peak bond force value was attained, the #5 bars pulled out at a relatively slower rate, and the force dropped very slowly.

The #5 bars with 1-1/2 in. cover showed very little cracking. As the test progressed, fine cracks of about 2 to 4 in. in length appeared above, and in the direction of, the test bar. Transverse cracks rarely appeared, even after the force reached its maximum value. The maximum load was generally lower, and the force dropped at a faster rate after reaching ultimate than for the #5 bars with 3 in. cover.

In general, there was no appreciable difference in the failure modes of the traffic vibrated and control test bars. For low to medium slump concrete, the average ultimate load of the traffic vibrated test bars was equal to or slightly higher than the load for the control test bars. For high slump concrete, the average ultimate load of the traffic vibrated test bars was lower than for the control test bars.

2.7.3 Compressive Strength

Compressive strengths of the traffic vibrated and control cylinders are summarized in Table 2.4. There was no visible difference in the failure mode of the traffic vibrated and control test cylinders, except in one case; one of the four traffic vibrated test cylinders of Group 5, (7-1/2 in. slump) crushed locally at its top end.

In general with low slump (1-1/2 in.) repair concrete, the traffic vibrated cylinders were stronger than the control cylinders. For medium slump, the traffic vibrated and control cylinders had similar strengths, while at higher slumps the control cylinders were invariably stronger.

Chapter 3

Analysis of Data

3.1 General

Test results described in Chapter 2 are analyzed in this chapter to determine the effects of traffic induced vibrations on concrete-steel bond strength and concrete compressive strength in bridge deck repairs. Bond strengths are compared with those predicted by AASHTO (1) and ACI (4).

The analysis reveals that for low slump (1-1/2 in.) concrete, bond and compressive strength are improved by traffic induced vibrations. For medium slump (4 to 5 in.), there is no appreciable effect. For high slump (7 to 8 in.), bond and compressive strength are reduced due to the traffic vibrations.

The traffic induced vibrations are more detrimental to the bond strength of #5 bars than #8 bars.

3.2 Bond Strength

The effects of traffic induced vibrations are studied as functions of slump, cover, and bar size. These effects are analyzed using the ultimate bond force. All of the test bars, except two in the first group, pulled out during the tests.

Two #5 control bars in the first group yielded before pullout. As the yield force for these two bars was attained, the unloaded end slip value became stationary, while the loaded end slip value kept increasing without any significant increase in the force. These tests were stopped to avoid a tensile failure of the test bar. Thus, for these two bars the maximum recorded load is used in place of the ultimate bond force for the analysis. The maximum recorded pullout forces are given in Table 2.3.

3.2.1 Effect of Slump

The effects of traffic induced vibration on bond strength are presented in Fig. 3.1 and 3.2. The concrete slump ranged from 1-1/2 in. to 7-1/2 in. for bars with 3 in. and 1-1/2 in. cover. In these figures, each point represents the ratio of the bond strength of a traffic vibrated test bar to the average of the bond strengths of the two control test bars of the same group.

3.2.1.1 Bars with 3 in. Cover

The bars with 3 in. cover were cast with 1-1/2 in. and 4-1/2 in. slump concrete. For the bars with 3 in. cover and 1-1/2 in. slump, the traffic induced vibrations improved the average ultimate bond strength by 2.7% and 6.9% for #5 and #8 bars, respectively. When the slump was increased to 4-1/2 in., the traffic vibrated #5 and #8 bars had 0.5% lower and 5.6% higher average bond strengths, respectively, compared to the control bars of the same group (Table 3.1). Thus, the traffic induced vibrations did not appear to have a detrimental effect, even with an increase in slump to 4-1/2 in. The two #5 control bars from Group 1 yielded before pullout. Had their yield strengths been somewhat higher, they would have provided higher bond strengths, and that would have given a flatter line for the #5 bars in Fig. 3.1.

3.2.1.2 Bars with 1-1/2 in. Cover

4 in. and 7-1/2 in. slump concrete was used with bars having 1-1/2 in. cover. A comparison of the values for traffic vibrated and control bars (Table 3.1) shows that when 4 in. slump concrete was used, the traffic induced vibrations decreased the average bond strength of the #5 bars by 7.8% but increased the bond strength of the #8 bars by 7.0%. When the slump was increased to 7-1/2 in., the average bond strengths of the #5 and #8 bars dropped by 5.9% and 3.1%, respectively.

3.2.2 Effect of Cover

Since the data for both the 3 in. and 1-1/2 in. cover is limited, no conclusions can be made about the influence of cover on the effect of traffic induced vibrations on bond strength. The influence of cover on bond strength, independent of construction history, is discussed in Section 3.2.4.

3.2.3 Effect of Bar Size

Traffic induced vibrations appear to be more detrimental to the bond strength of #5 bars than to #8 bars.

Fig. 3.1 and 3.2 and Table 3.1 show that at all slumps, the ratio of bond strengths of traffic vibrated bars to control bars is lower for #5 bars than for #8 bars. A comparison of the ratios of the ultimate bond strength indicates that for low slump concrete, the vibrations improved the average bond strength of #5 bars by only 2.7%, compared to a 6.9% improvement for #8 bars. For medium slump concrete (4 in. and 4-1/2 in.) , the vibrations decreased the average bond strength by 4.2% for #5 bars but improved it by 6.3% for #8 bars. For high slump concrete, the #5 bars had a 5.9% drop in the average bond strength compared to a 3.1% drop for the #8 bars.

Both bar sizes had the same total thickness of slab (12 in.) and had the same cover, either 1-1/2 in. or 3 in.; thus, the depth of concrete below the steel differed by 3/8 in. due the difference in bar size. This difference in the depth of concrete below the steel is small and had a minor effect at most. The difference in relative strengths is likely tied to the difference in failure modes.

While #8 bars pulled out by splitting the concrete both above and below the bars, the #5 bars pulled out primarily by shearing the local surrounding concrete. Although some splitting of the top concrete was observed for #5 bars with 1-1/2 in. cover, the concrete in the lower part of the repair area was not affected when the #5 bars were pulled out. The concrete in the top part of the repair area had a high water-cement ratio, lower strength, and local settlement cracks. The concrete in the bottom of the repair had a lower water-cement ratio, higher strength and improved consolidation due to the traffic induced vibrations. Thus, the #8 bars split higher quality concrete during the pullout, while the #5 bars split or sheared lower quality concrete.

3.2.4 Comparison with Design Values

The bond strengths obtained for both the traffic vibrated and the control bars are compared with values derived from the ACI (4) and AASHTO (1) expressions for development length in Table 3.1. The ratios of bond strengths observed in this study versus the values derived from the ACI and AASHTO expressions are plotted as a function of compressive strength and cover in Fig. 3.3 and 3.4. The following expressions for bond strengths are derived from the ACI and AASHTO expressions for the development length of reinforcing bars.

$$T = 1.25 \bullet 625\pi L d_b \tag{1}$$

$$T = 1.25 \bullet 25L\sqrt{f'_c} \tag{2}$$

in which L = the embedment length (in.); d_b = nominal bar diameter (in.); and f'_c = concrete compressive strength (psi). The 1.25 factors are included to account for the 20% reduction in development length (equivalent to 25% increase in bond strength) allowed when the lateral spacing of bars is 6 in. or more.

As shown in Fig. 3.3 and 3.4, all #5 and #8 test bars have higher bond strengths than indicated by Eq. (1) and (2). For 3 in. cover, the average ratio of test versus predicted bond strengths is 254%, while that for the 1-1/2 in. cover is 176%. The test bar bond strengths range from 46% to 214% higher than those given by these two expressions. This high strength may be attributed, at least in part, to the short development lengths used in this study; it is well known that bars with short bonded lengths develop a higher pullout force per unit length than do longer bars (13,15,16). The points for #8 bars (Fig. 3.4) are more closely grouped than the points for #5 bars (Fig. 3.3) because Eq. (2), which is used for #8 bars, takes into account concrete strength, while Eq. (1),which is used for #5 bars, does not.

Fig. 3.3 and 3.4 also indicate that increased cover improves the bond strength. This is true for both traffic vibrated as well as control bars, as has been found by other studies (13,20,27).

3.3 Compressive Strength

The effect of traffic induced vibrations on concrete compressive strength also appears to be a function of slump. Fig. 3.5 illustrates the effect of the traffic induced vibrations on concrete compressive strength when standard 6 in. X 12 in. cylinders are used. The compressive strengths of the traffic vibrated and control cylinders are given in Table 2.4.

The compressive strengths of the laboratory mixed and the ready mixed concrete are affected in a similar manner by traffic vibrations. For low slump concrete, the compressive strength is improved slightly by the traffic vibrations, 0.5% for the laboratory mixed concrete and 4.1% for the ready mixed concrete. For medium slump concrete, neither mix is affected significantly; the ready mixed concrete (4 in. slump) shows a 2.5% improvement, while the laboratory mixed concrete (5 in. slump) loses 1.3% in compressive strength because of the traffic vibrations. For high slump concrete, traffic induced vibrations have a greater effect, with the compressive strengths of laboratory mixed concrete (7-3/4 in. slump) and ready mixed concrete (7-1/2 in. slump) decreasing 4.8% and 7.7%, respectively.

3.4 Discussion

3.4.1 Peak Particle Velocity

The simulated traffic induced vibrations used in this study produced a peak particle velocity of about 1.4 in./sec. Akins and Dixon (6), however, feel that a peak particle velocity of 0.2 in/sec. is "safe" for concrete up to one day old. Similarly, Schmidt et. al (9) feel that a peak particle velocity of 0.79 in./sec. is safe for young concrete. The peak particle velocity produced in this study exceeded the "safe" values recommended by both of these studies. Since there was no adverse effect on low slump concrete from traffic induced vibrations with a peak particle velocity as high as 1.4 in./sec., previous "safe" limits are clearly conservative when low slump concrete is used. However, the lower limits may be valid for high slump concrete, since compressive strengths dropped up to 7.7% and bond strengths dropped up to 5.9% when high slump concrete was used.

3.4.2 Bond Strength

The influence of slump on the traffic vibrated concrete appears to be caused by concrete segregation. The higher the slump, the greater the amount of bleed water that will rise in the repair areas subjected to traffic vibration. This was observed in the traffic vibrated specimens using high slump (7-1/2 in.) concrete. The bleeding results in increased settlement cracking and a high water-cement ratio which produces a lower concrete strength in the top part of the repair. The low strength affects the bond strengths of top-cast bars adversely. Lower slump concrete, on the other hand, is expected to attain better consolidation due to the traffic vibrations, producing a denser material, which results in slightly better bond strength.

Davis, Brown and Kelly (12) showed the positive effect of sustained jigging after up to 6 hours of placement on bond strength. They used plain round bars and did not report the slump value; still, their results indicate the possibility of improvement in bond strength due to vibrations at some value of slump.

Larnach (22) found that when bars were brought in contact with a vibrator during placement, there was a tendency towards reduction (5%) in bond strength for wetter mixes (water-cement ratio of 0.53). The drier mixes (water-cement ratio of 0.44), on the other hand, had a slight increase (2%) in bond strength.

The studies done by Larnach (22) and Menzel (25) showed the negative effects of revibration on bond strength. Larnach, who used concrete with a water-cement ratio between 0.44 and 0.53 and did not provide slump values, reported a reduction of 33% in bond strength; Menzel, using 2 to 3 in. slump concrete, reported more than a 60% drop in bond strength. Since revibration and traffic induced vibrations do not necessarily affect the bond strength in a similar way, these results do not necessarily contradict the results of this study for low and medium slump concrete.

Furr and Fouad (17) used concrete with 3 to 6 in. slump in the study of bridge widening. Four of the eight traffic vibrated cores containing test bars showed clear evidence of relative movement and, hence, loss of bond. Their results are consistent with the results obtained in this study for the medium and high slump specimens.

3.4.3 Compressive Strength

The effect of slump on the compressive strength of traffic vibrated concrete also appears to depend on segregation and bleeding. Traffic vibrations will accentuate bleeding in high slump concrete and result in weaker concrete near the top surface. On the other hand, traffic vibrations help consolidate the low slump concrete, producing slightly higher compressive strengths.

The effects of traffic induced vibrations on low, medium, and high slump concrete obtained in this study agree with the findings of other researchers (18,29,30).

Furr and Ingram (18) observed that traffic induced vibrations "substantially" improve the compressive strength of concrete used for overlays; most overlays have very low slump. Vollick (30) noted a 19% improvement in compressive strength of 3 in. slump concrete due to revibration, although revibration does not necessarily affect the compressive strength in the same way as do traffic vibrations.

The observation that traffic induced vibrations produce higher quantities of bleed water and reduce the compressive strength of high slump concrete agrees with the study by Davis et. al (29), which showed that vibrations produced by transportation segregated high slump concrete and increased the water-cement ratio in the upper part of the specimens.

3.5 Recommendations

Traffic induced vibrations provide a slight improvement in bond strength and concrete compressive strength when low slump concrete is used. They can be slightly detrimental or may have no effect when medium slump (4 to 5 in.) concrete is used. But they reduce both bond and compressive strength when high slump concrete is used.

It is significant that the test bars were tightly secured to the slabs and forms. This restricted the relative movement between the bars and the slab. Therefore, these results do not apply when bars are free to move relative to the bridge deck.

Although bond strengths ranged from 46% to 214% higher than those predicted by the ACI and the AASHTO governing expressions for development length, it is the effects of the vibrations that are of prime interest, i. e. the test specimens were prepared under carefully controlled conditions and represent very high quality construction. The observed variation in strength should be superimposed upon strengths obtained in practice.

Based on this work, it is recommended that traffic can be maintained during bridge deck repair provided:

1. Low slump (less than 3 in.) repair concrete is used.
2. The reinforcing bars in the repair area are securely fastened to the structure prior to concrete placement.

Chapter 4

Summary and Conclusions

4.1 Summary

The purpose of this study was to determine the effects of simulated traffic induced vibrations on concrete-steel bond strength and concrete compressive strength for full depth repairs in reinforced concrete bridge decks. Fifteen test slabs, five control and ten traffic vibrated, with 23 in. X 18 in. repair areas were used to study the effect of traffic induced vibrations on bond. Forty pullout tests were performed using #5 and #8 bars embedded in 8 ft X 4 ft X 1 ft slabs. To study the effect of traffic vibrations on compressive strength, standard 6 in. X 12 in. cylinders were attached to the slabs. The traffic induced vibrations consisted of small continuous vibrations of 0.02 in. amplitude at 4 cycles per second over which large intermittent vibrations of 0.5 in. amplitude (peak to peak) at 0.5 cycle per second were superimposed at 4 minute intervals. Test results were plotted and analyzed to study the effects of traffic induced vibrations as functions of slump, cover, and bar size. The bond strengths of traffic vibrated and control bars were compared with bond values derived from ACI and AASHTO expressions.

4.2 Conclusions

The following conclusions are based on the test results and analysis described in this report.

1. Traffic induced vibrations are not detrimental to reinforced concrete bridge deck repairs provided that:
   a. Low slump (less than 3 in.) concrete is used in the repairs; and
   b. Reinforcing bars are securely fastened to the structure before the concrete placement.
2. Traffic induced vibrations may be detrimental if medium slump (4 to 5 in.) concrete is used for bridge deck repair.
3. Traffic induced vibrations are detrimental to the quality of high slump (over 5 in.) repair concrete.
4. Traffic induced vibrations appear to be more detrimental to the bond strength of #5 bars than #8 bars.
5. Increased cover increases the bond strength of the both #5 and #8 bars.
6. ACI and AASHTO expressions for development length provided conservative bond strength values for all test bars. However, a portion of the high bond strength is due to the high quality construction and short embedment lengths used.

4.3 Recommendations for Future Study

In this study, traffic induced vibrations reduced both bond and compressive strengths when high slump concrete was used. Therefore, there is some concern that traffic vibrations could have a similar effect when special concretes, such as latex modified concrete which is placed at high slump, are used for overlays and full depth repairs. It would be of interest to study the effects of traffic induced vibrations using these special concretes.

No conclusions could be drawn about the influence of cover on the effect of traffic vibrations on bond strength. It would be useful to determine whether traffic vibrations are more detrimental to the bond strength of higher cover bars or of lower cover bars.

All of the test bars in this study were well secured to the formwork prior to the application of traffic vibrations. In practice, however, some relative movement may be found at lap splices in repair areas. This suggests the need for studying the effect of relative movement on bond strength for repairs subjected to traffic vibrations.

The traffic vibrations were started 10 min. after concrete placement and were continued for 30 hours. A minimum delay period after placement might be found after which traffic induced vibrations have no effect, even on high slump repair concrete. If so, traffic vibrations would need to be avoided only for such a delay period when high slump concrete was used.

References

1. AASHTO Subcommittee on Bridges and Structures, Standard Specifications for Highway Bridges, 1977, American Association of State Highway and Transportation Officials, Washington, D.C., 1977, 914 pp.

2. ACI Committee 309, "Recommended Practice for Consolidation of Concrete (ACI 309-72) (Reaffirmed 1978)," American Concrete Institute, Detroit, Michigan, 1972, 40 pp.

3. ACI Committee 309, "Behavior of Fresh Concrete during Vibration (ACI 309.1R-81)," ACI Journal, Proceedings, Vol. 78, January-February 1981, pp. 36-53.

4. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, Michigan, 1983, 111 pp.

5. ACI Committee 546, "Guide for Repair of Concrete Bridge Superstructures (ACI 546.1R-80)," Concrete International, Vol. 2, No. 9 September 1980, pp. 69-88.

6. Akins K.P., and Dixon, D.E., "Concrete Structures and Construction Vibrations," SP-60 American Construction Institute, Detroit, Michigan, 1979, pp. 213-247.

7. ASTM A615, "Standard Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement," Annual Book of ASTM Standards, Part 4, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1980, pp. 588-593.

8. Bastian, C.E., "The Effect of Vibrations on Freshly Poured Concrete," Foundation Facts, Vol. 6, No. 1, 1970, pp. 14-17.

9. Bonzel, Justin and Schmidt, Michael, "Effects of Vibrations on Fresh and Young Concrete (Einfluss Von Erschutterungen auf frischen und auf jungen Beton)," Beton (Dusseldrof), Vol. 30, No. 9, Sept. 1980, pp. 333-337, and No. 10, Oct. 1980, pp. 372-378.

10. Crandell, F.J., "Ground Vibration due to Blasting and its Effect on Structures," Journal of the Boston Society of Civil Engineers, Vol. 36, No. 2, April 1949, pp. 222-246.

11. Csagoly, P.F.; Campbell, T.I.; and Agarwal, A.C., "Bridge Vibration Study," Report No. RR181, Sept. 1972, Ministry of Transportations and Communications, Ontario, 26 pp.

12. Davis, R.E., Brown, E.H. and Kelly, J.W., "Some Factors Influencing the Bond Between Concrete and Reinforcing Steel," American Society for Testing Materials, Proceedings of the Forty-first Annual Meeting, Vol. 38, Part II, Philadelphia, 1938, pp. 394-406.

13. Donahey, Rex C. and Darwin, David, "Effects of Construction Procedures on Bond in Bridge Decks," Structural Engineering and Engineering Materials SM Report, No. 7, University of Kansas Center of Research, Inc., Lawrence, Kansas, January 1983, 125 pp.

14. Edwards, A.T., and Northwood, T.D., "Experimental Studies of the Effects of Blasting on Structures," The Engineer, Vol. 210, No. 5462 September 30, 1960, pp. 538-546. (Research Paper No. 105 of the Division of Building Research, NRC, Ottawa.)

15. Ferguson, Phil M. and Thompson, J. Neils, "Development Length for Large High Strength Reinforcing Bars," ACI Journal, Proceedings, Vol. 62, No. 1, Jan. 1965, pp. 71-91.

16. Ferguson, Phil M. and Thompson, J. Neils, "Development Length of High Strength Reinforcing Bars in Bond," ACI Journal, Proceedings, Vol. 59, No. 7, July 1962, pp. 887-922.

17. Furr, H.L. and Fouad, F.H., "Bridge Slab Concrete Placed Adjacent to Moving Live Load," Research Report No. 226-1F, Texas Dept. of Highways and Public Transportation, Jan. 1981, pp. 131.

18. Furr, H. and Ingram, L., "Concrete Overlays for Bridge Deck Repair," Highway Resesearch Record, No. 400, Highway Research Board, Washington D.C. 1972, pp. 93-104.

19. Goodpasture, D.W., "Effect of Traffic on Bridge Deck Cracking," ACI Journal, Proceedings, Vol. 70, No. 12, Dec. 1973, pp. 793-794.

20. Jimenez, Rafael, Gergely, Peter: and White, Richard N., "Shear Transfer Across Cracks in Reinforced Concrete," Report 78-4, Department of Structural Engineering, Cornell University, Ithaca, N.Y., Aug. 1978, 357 pp.

21. Kennedy, John H., "Portable Instruments for Load History Studies," Report No. FHWA TS-80-232, Aug. 1980, 60 pp.

22. Larnach, W.J., "Changes in Bond Strength Caused by Re-vibration of Concrete and Vibration of Reinforcement," Magazine of Concrete Research, London, Vol. 4, No. 10 July 1952, pp. 17-21.

23. Manning, David G., "Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs," NCHRP Synthesis of Highway Practice 86, Dec. 1981, 40 pp.

24. Massachusetts Department of Public Works, "Pull-out Test on Steel Reinforcing Bars Subjected to Vibrations," 1974, 9 pp.

25. Menzel, C.A., "Effects of Settlement on Concrete on Results of Pull-out Bond Tests," Research Department Bulletin 41, Research and Development Laboratories of the Portland Cement Association, Nov. 1952, 149 pp.

26. Montero, A.C., "Effects of Maintaining Traffic During Widening of Bridge Decks (A Case Study)," Thesis, The Ohio State University, Columbus, 1980, 78 pp.

27. Morita, S. and Fujii, S., "Bond Capacity of Deformed Bars Due to Splitting of Surrounding Concrete," Bond in Concrete, edited by P. Bartos, Applied Science Publishers, London, 1982, pp. 331-341.

28. Northwood, T.D., Crawford, R., and Edwards, A.T., "Blasting Vibrations and Building Damage" The Engineer, Vol. 215, No. 5601, May 31, 1963, pp. 973-978. (Research Paper No. 186 of the Division of Building Research, NRC, Ottawa.)

29. Tuthill, L.H., and Davis, H.E., "Overvibration and Revibration of Concrete," ACI Journal, Proceedings, Vol. 35, No. 4, Sept. 1938, pp. 41-47.

30. Vollick, C.A., "Effects of Revibrating Concrete," ACI Journal, Proceedings, Vol. 54, No.39, March 1958, pp. 721-732.

31. Tuthill, L.H., "Revibrations Reexamined," Concrete Construction, Vol. 22, No. 10, Oct. 1977, pp. 537-539.

32. Whiffen, A.C. and Leonard, D.R., "A Survey of Traffic Induced Vibrations," Transport and Research Laboratory, Report LR 418, England, 1971, 26 pp.

Table 2.1(a) Concrete Mix Design and Properties
(Ready-mixed Concrete)

Test Group	Slab Concrete W/C Ratio	Cement #	Water #	Aggregate Fine # *	Aggregate Coarse # +	Slump in.	Air %	Strength psi	Repair Concrete W/C ratio	Cement #	Water #	Aggregate Fine # *	Aggregate Coarse # +	Slump in.	Air %	Strength psi
1	0.44	591	235	1470	1482	4	na++	5160	0.46	579	267	1448	1449	1 1/2	5	3480
2	0.46	579	265	1453	1441	1 1/4	4 1/2	na++	0.49	614	300	1413	1425	4 1/2	2	3410
3	0.44	555	244	1455	1545	4 1/4	10 1/2	2960	0.44	555	244	1455	1536	1 1/2	7	2960
4	0.44	555	244	1455	1536	3 1/4	5 1/2	5160	0.44	564	248	1491	1455	4	7	3230
5	0.44	555	244	1455	1536	5 1/2	7 1/2	3760	0.44	680	300	1300	1440	7 1/2	7 1/2	3000

+Crushed Limestone - Hamm's quarry, Perry, KS
Bulk specific gravity = 2.52, absorption = 3.5%
Maximum size = 3/4 inch
*Kansas river sand - Lawrence Sand Co., Lawrence KS
Bulk specific gravity = 2.62, absorption = 0.5%
Fineness modulus = 3.0
Air entraining agent - Vinsol resin
Design air entrainment = 6%
Slump and air values are as measured
++These values were not recorded

Table 2.1(b) Concrete Mix Design and Properties
(Laboratory mixed Concrete)

Cylinder Group	W/C ratio	Cement #	Water #	Aggregate Fine #	Aggregate Coarse #	Slump in.	Air %	Strength psi
1	0.44	680	300	1300	1440	7 3/4	5 1/2	3770
2	0.44	645	284	1375	1438	5	4 1/2	3870
3	0.44	555	244	1536	1435	1 1/2	5	3930

Materials used are same as test slabs
Design air entrainment = 6%
Slump and air values are as measured

Table 2.2 Test Bar Data

Bar Size	#5	#8
Deformation Spacing, in.	0.336	0.545
Deformation Height, in.	0.041	0.057
Deformation Angle, Deg.	54	50
Deformation Gap, in.	0.118	0.313
Nominal Weight, lb/ft	1.012	2.650
Deformation Bearing Area, sq. in./in. length	0.196	0.239
Yield Strength, ksi	59.50	63.47
Tensile Strength, ksi	102.9	104.6

Table 2.3 Test Specimen Variables and Bond Strength

Slab	Bar No.	Bar Size	Concrete Strength Vib. psi	Concrete Strength Con. psi	Slump in.	Embed. Length in.	Cover in.	Specimen Type V-Vib C-Con	Bond Strength kips
1b	1	#5	na	3480	1 1/2	5	3	V	17.8
1b	2								18.3
1a	3							C	18.9Y
1a	4								18.4Y
1a	5	#8				9			41.8
1a	6								44.2
1c	7							V	42.4
1c	8								43.6
2a	9	#5	na	3410	4 1/2	4	3	C	15.6
2a	10								19.3
2b	11							V	16.1
2b	12								18.7
2a	13	#8				9		C	42.8
2a	14								43.1
2c	15							V	43.5
2c	16								47.3
3a	17	#5	3080	2960	1 1/2	4	3	C	11.7
3a	18								13.3
3b	19							V	13.1
3b	20								14.1
3a	21	#8				9		C	35.2
3a	22								36.8
3c	23							V	39.4
3c	24								42.5
4a	25	#5	3310	3230	4	4	1 1/2	C	9.92
4a	26								10.6
4b	27							V	8.99
4b	28								10.0
4a	29	#8				9		C	25.2
4a	30								25.2
4c	31							V	26.4
4c	32								27.5
5a	33	#5	2770	3000	7 1/2	4	1 1/2	C	12.4
5a	34								12.8
5b	35							V	11.5
5b	36								12.2
5a	37	#8				9		C	24.0
5a	38								30.5
5c	39							V	25.8
5c	40								27.1

Y after load indicates pullout force exceeded yield strength

Table 2.4 Comparison of Vibrated and Control Strength

Test No.	Concrete Source	Slump in.	# of Cylinders (V+C)	Strength Vibrated psi	Strength Control psi	Vib/Con Ratio
3	RM *	1 1/2	(4+4)	3330	3200	1.041
4	RM	4	(4+4)	3310	3230	1.025
5	RM	7 1/2	(4+4)	2770	3000	0.923
	LM +	1 1/2	(3+3)	3950	3930	1.005
	LM	5	(3+3)	3820	3870	0.987
	LM	7 3/4	(3+3)	3590	3770	0.952

* Ready-Mixed Concrete
+Laboratory Mixed Concrete

Table 3.1 Bond Strength Comparisons

Bar No.	Bar Size	Slump in.	Cover in.	Embedment Length in.	Specimen Type V-Vib C-Con	Bond Strength kips	Ave. C Bond Strength kips	V/C Ratio %	Ave. V/C Ratio %	ACI, AASHTO Load kips	Test/ ACI, AASHTO Load %
1	#5	1 1/2	3	5	V	17.8		95.2		7.67	232
2						18.3		97.9	96.6		238
3					C	18.9Y					246
4						18.4Y	18.7				240
5	#8			9		41.8				16.6	252
6						44.2	43.0				266
7					V	42.4		98.6			255
8						43.6		101.4	100.0		263
9	#5	4 1/2	3	4	C	15.6				6.14	254
10						19.3	17.5				314
11					V	16.1		92.0			262
12						18.7		106.9	99.5		305
13	#8			9	C	42.8				16.4	261
14						43.1	43.0				262
15					V	43.5		101.2			265
16						47.3		110.0	105.6		288
17	#5	1 1/2	3	4	C	11.7				6.14	191
18						13.3	12.5				217
19					V	13.1		104.8			213
20						14.1		112.8	108.8		230
21	#8			9	C	35.2				15.3	230
22						36.8	36.0				240
23					V	39.4		109.4			257
24						42.5		118.1	113.8		278
25	#5	4	1 1/2	4	C	9.92				6.14	162
26						10.6	10.3				173
27					V	8.99		87.3			146
28						10.0		97.1	92.2		166
29	#8			9	C	25.2				16.0	158
30						25.2	25.2				158
31					V	26.4		104.8			165
32						27.5		109.1	107.0		172
33	#5	7 1/2	1 1/2	4	C	12.4				6.14	202
34						12.8	12.6				209
35					V	11.5		91.3			187
36						12.2		96.8	94.1		199
37	#8			9	C	24.0				14.8	162
38						30.5	27.3				206
39					V	25.8		94.5			174
40						27.1		99.3	96.9		183

Y after load indicates pullout force exceeded yield strength

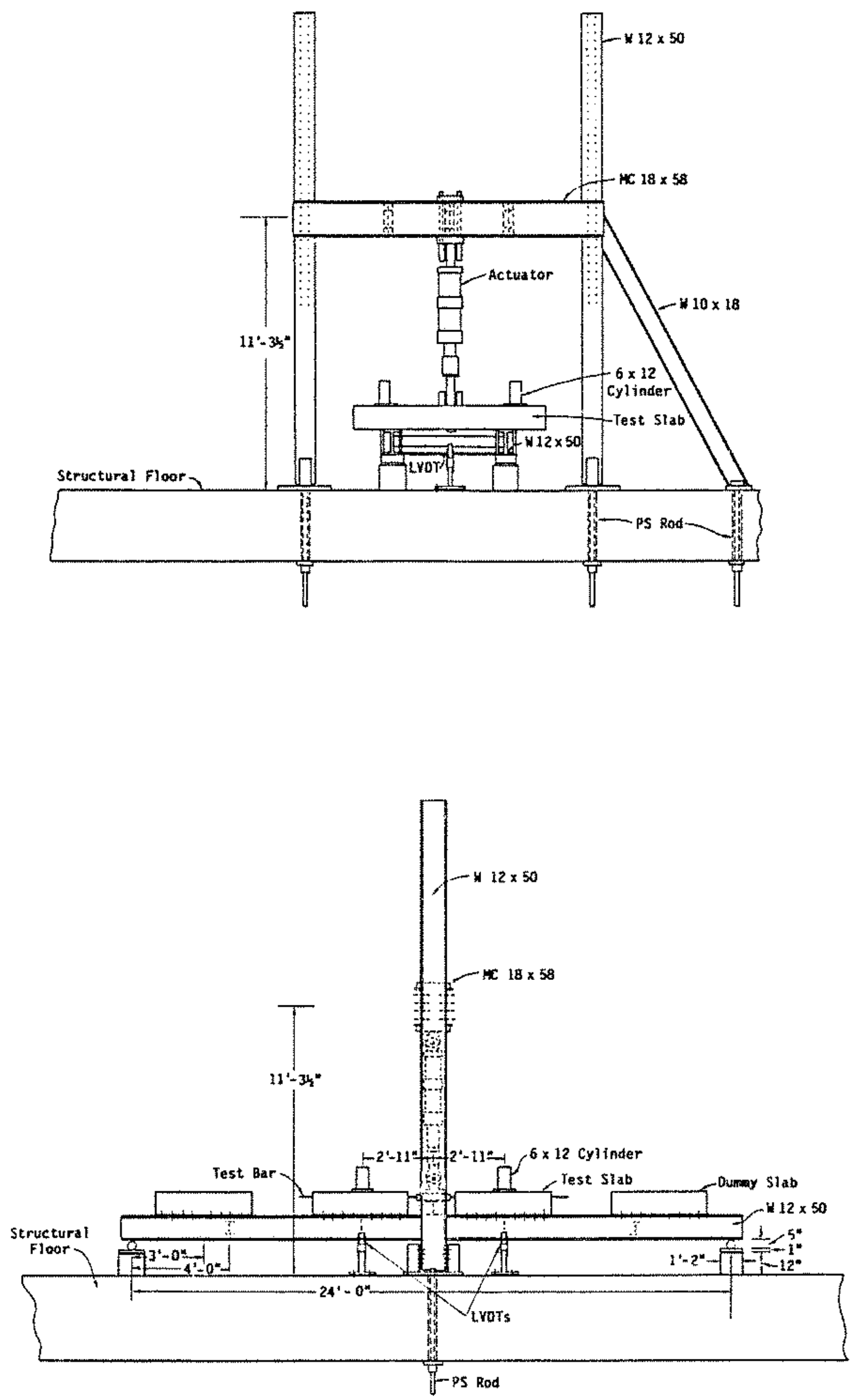
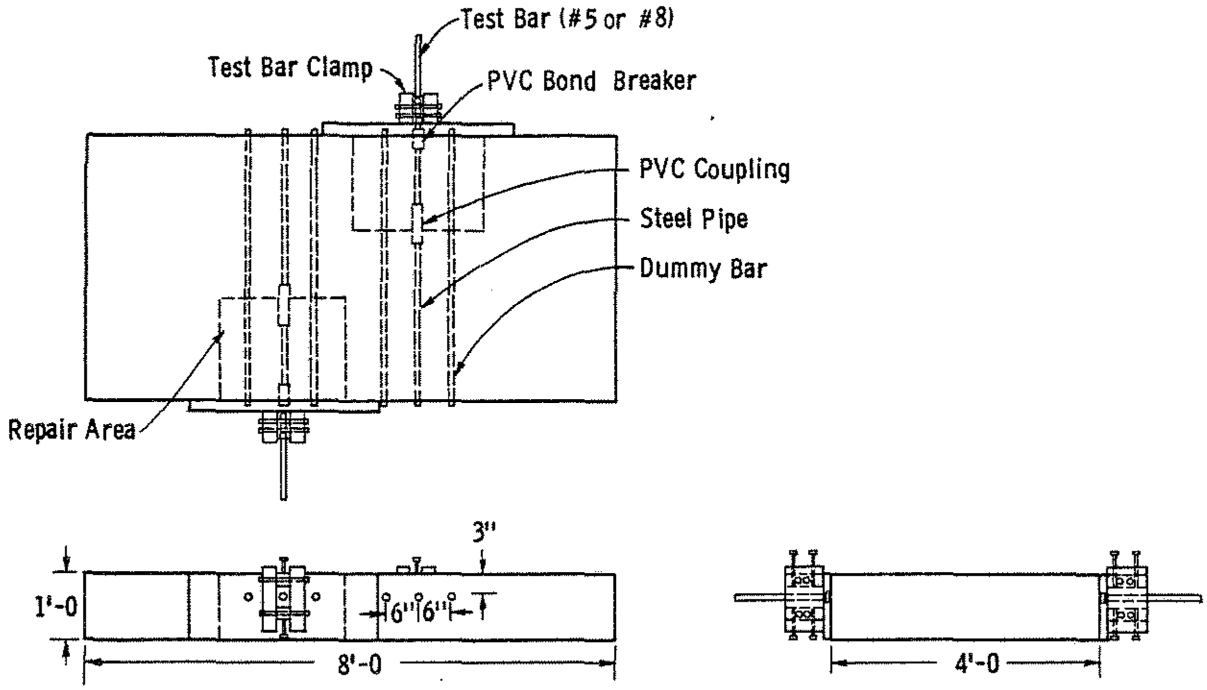
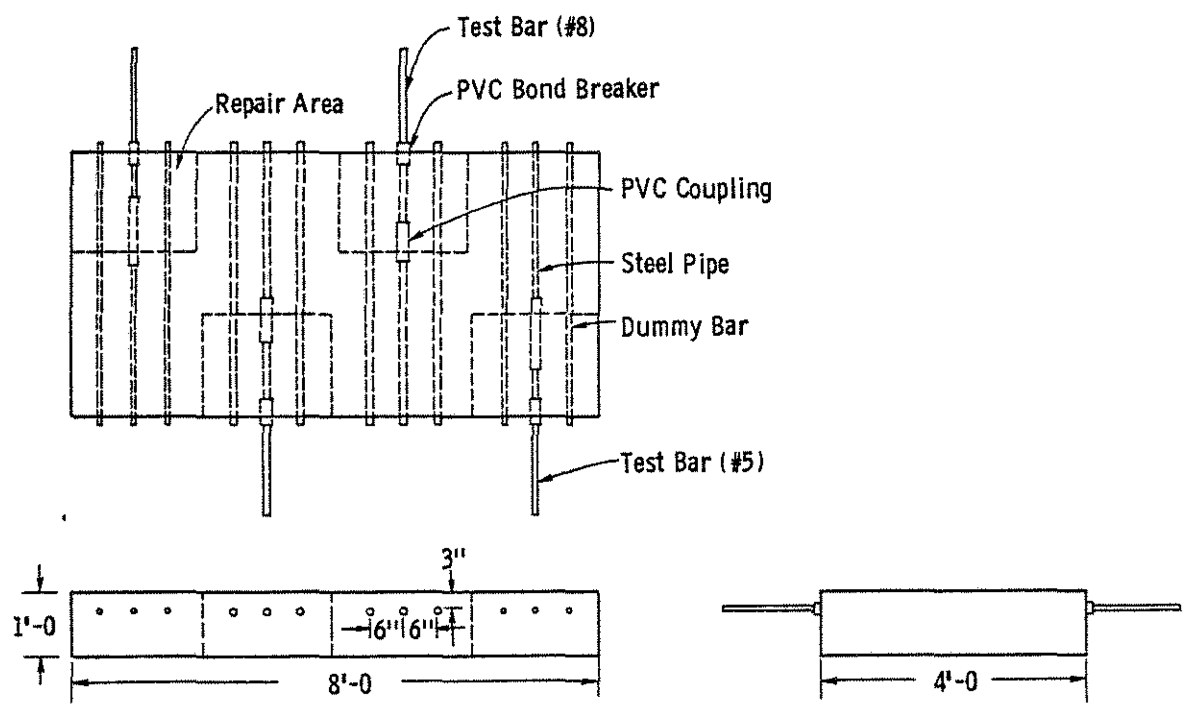


Fig. 2.1 Load frame



Traffic Vibrated Slab



Control Slab

Fig. 2.2 Test Slabs

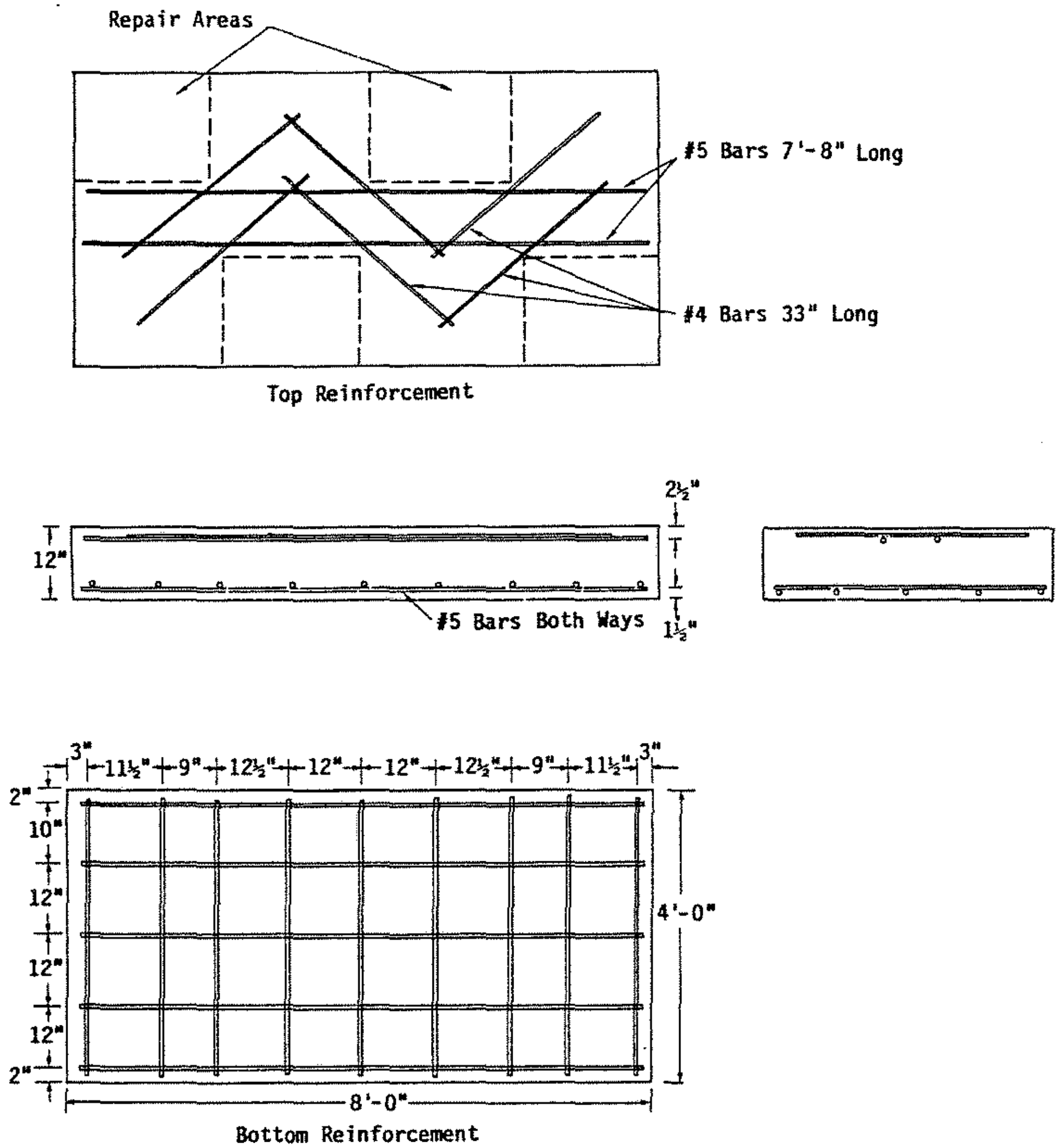
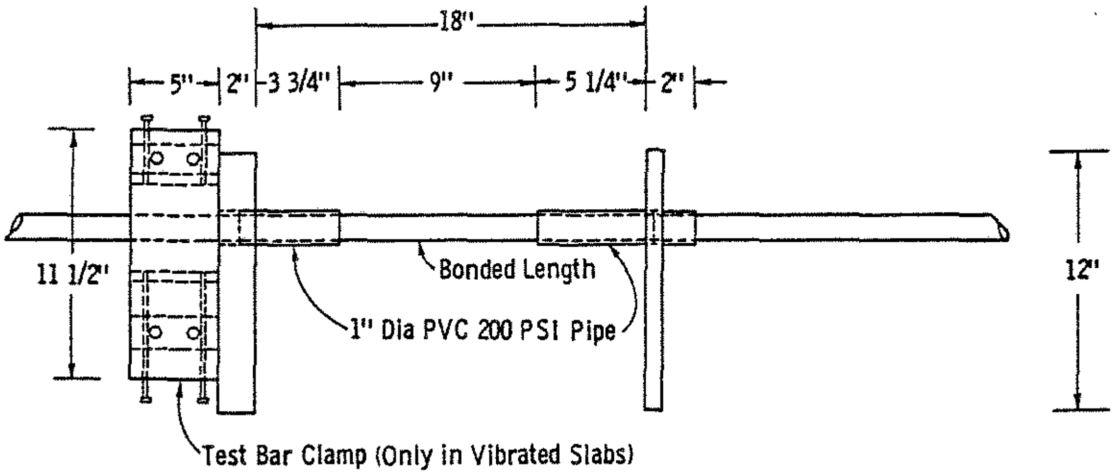
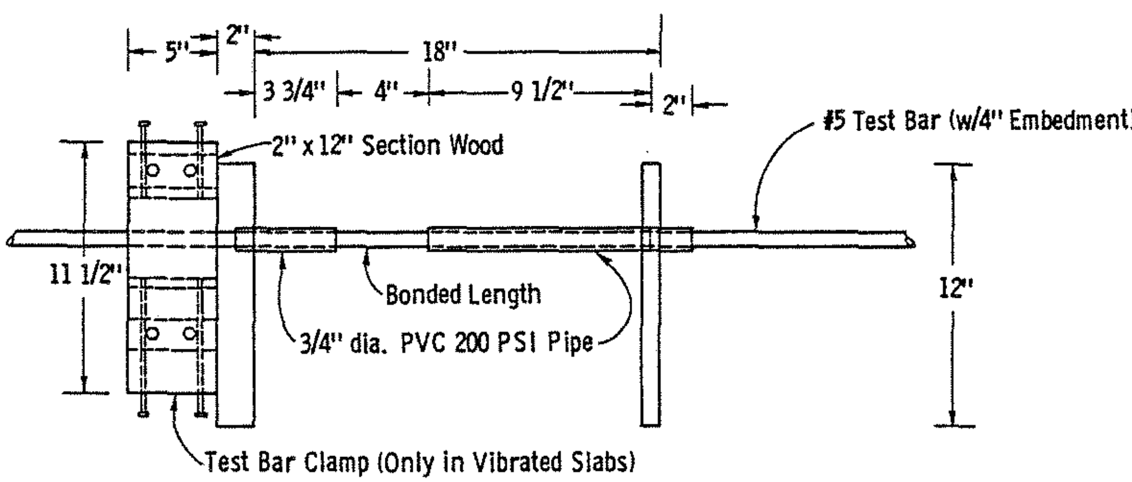


Fig. 2.3 Reinforcement Details for Test Slabs



#8 Test Bar Installation



#5 Test Bar Installation

Fig. 2.4 Test Bar Installation

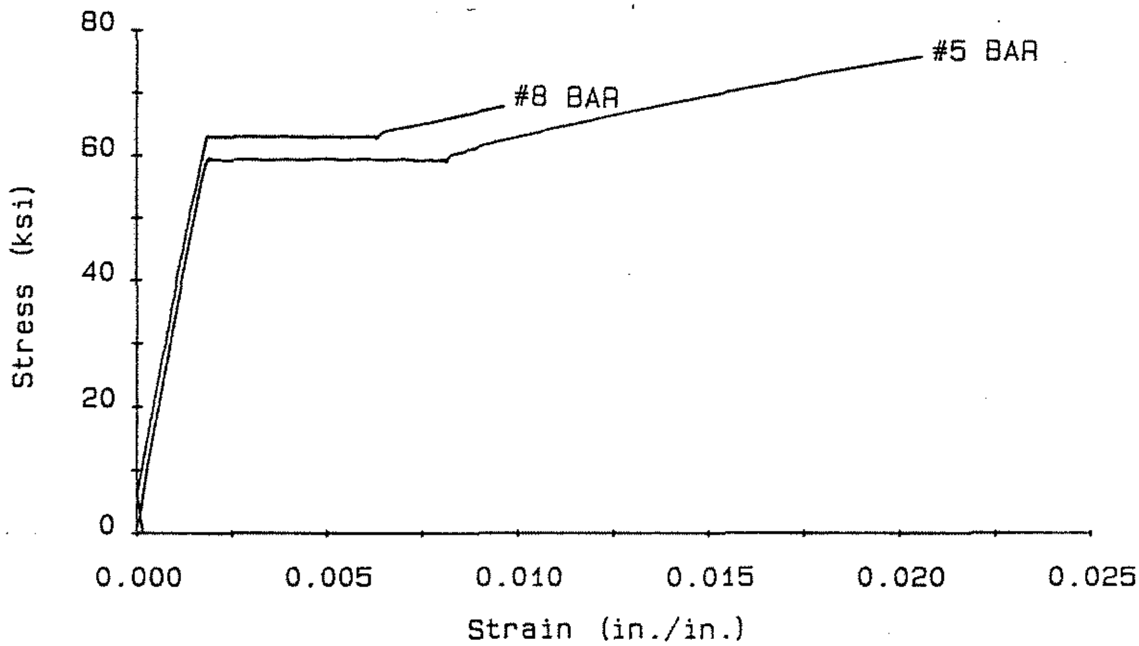


Fig. 2.5 Test Bar Stress-Strain Curves

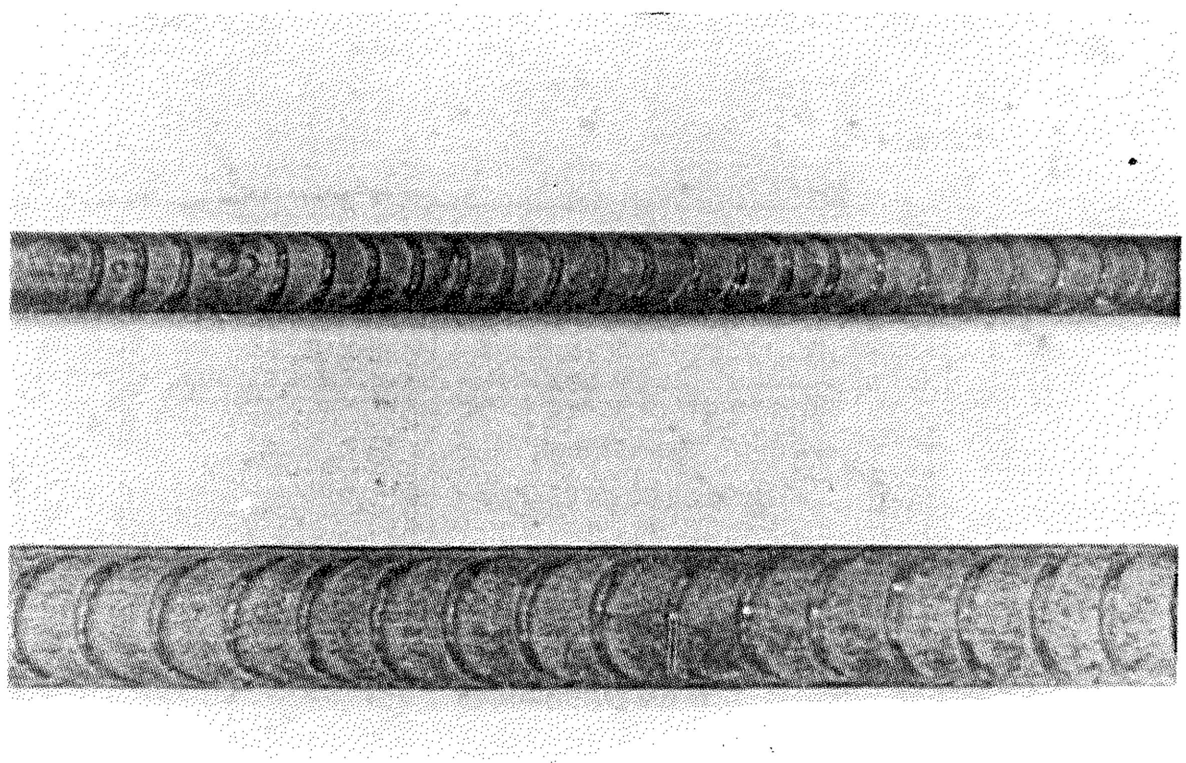


Fig. 2.6 Test Bar Deformation Patterns

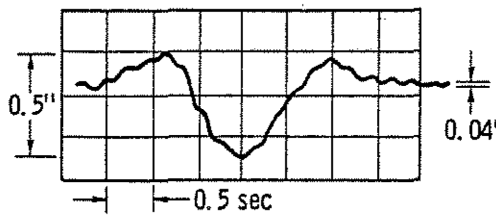


Fig. 2.7 Typical Wave Pattern for Traffic Induced Vibrations

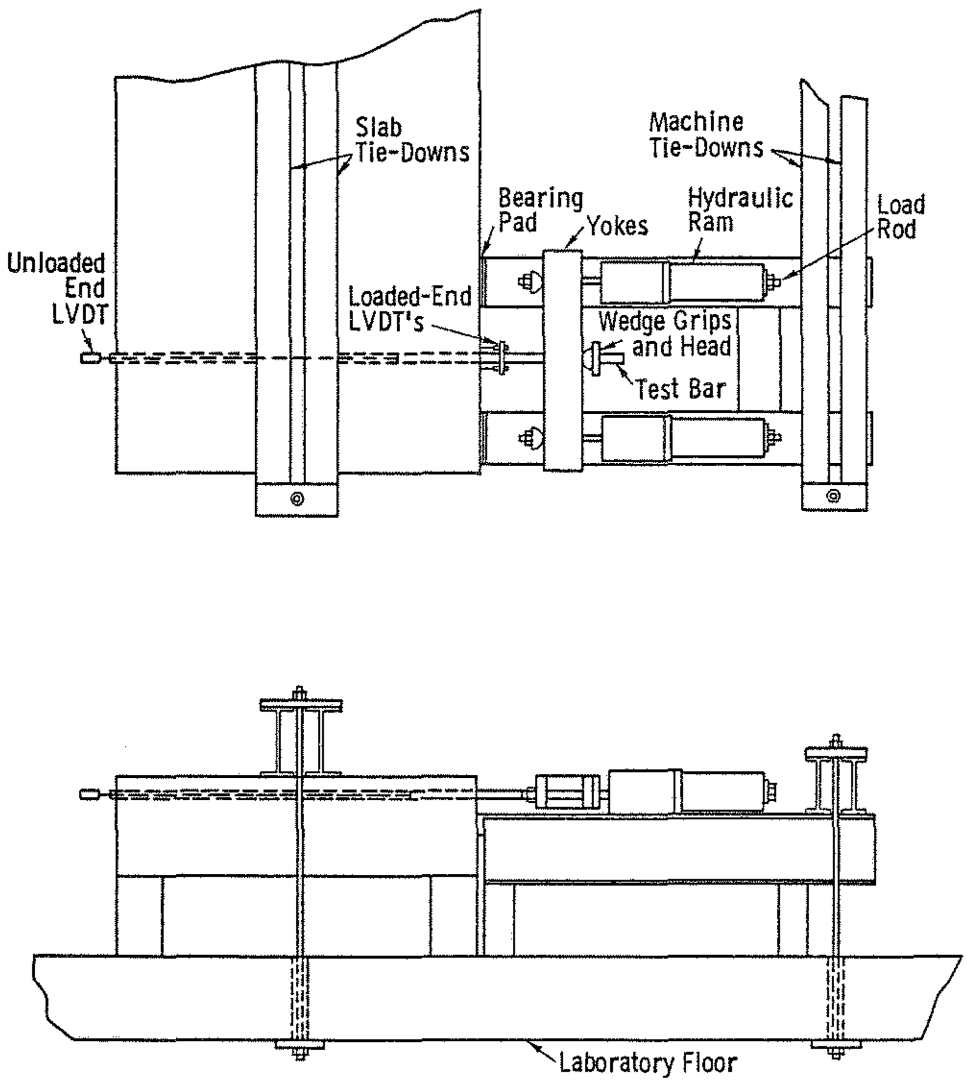


Fig. 2.8 Schematic of Bond Test

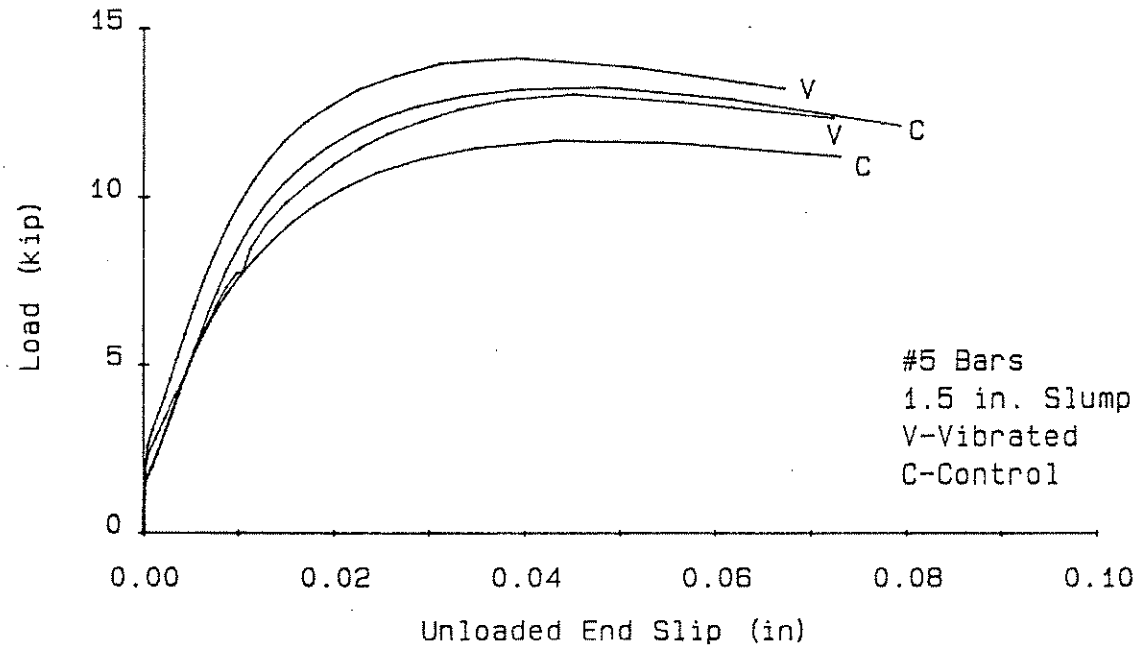


Fig. 2.9 Typical Load-Unloaded End Slip Curves

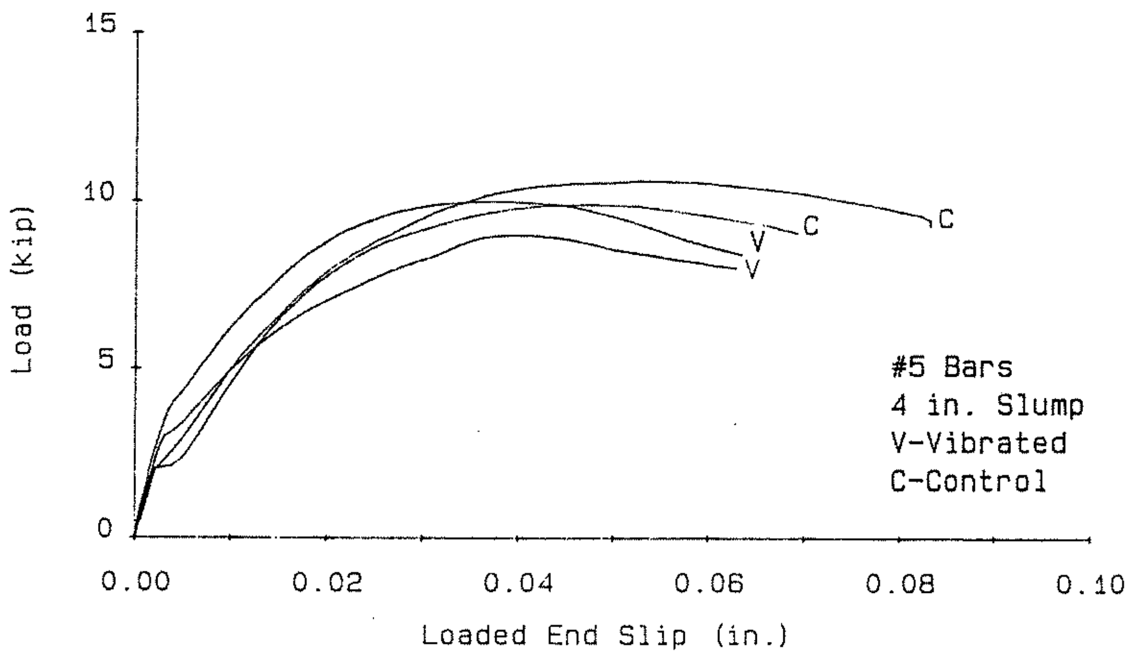


Fig. 2.10 Typical Load-Loaded End Slip Curves

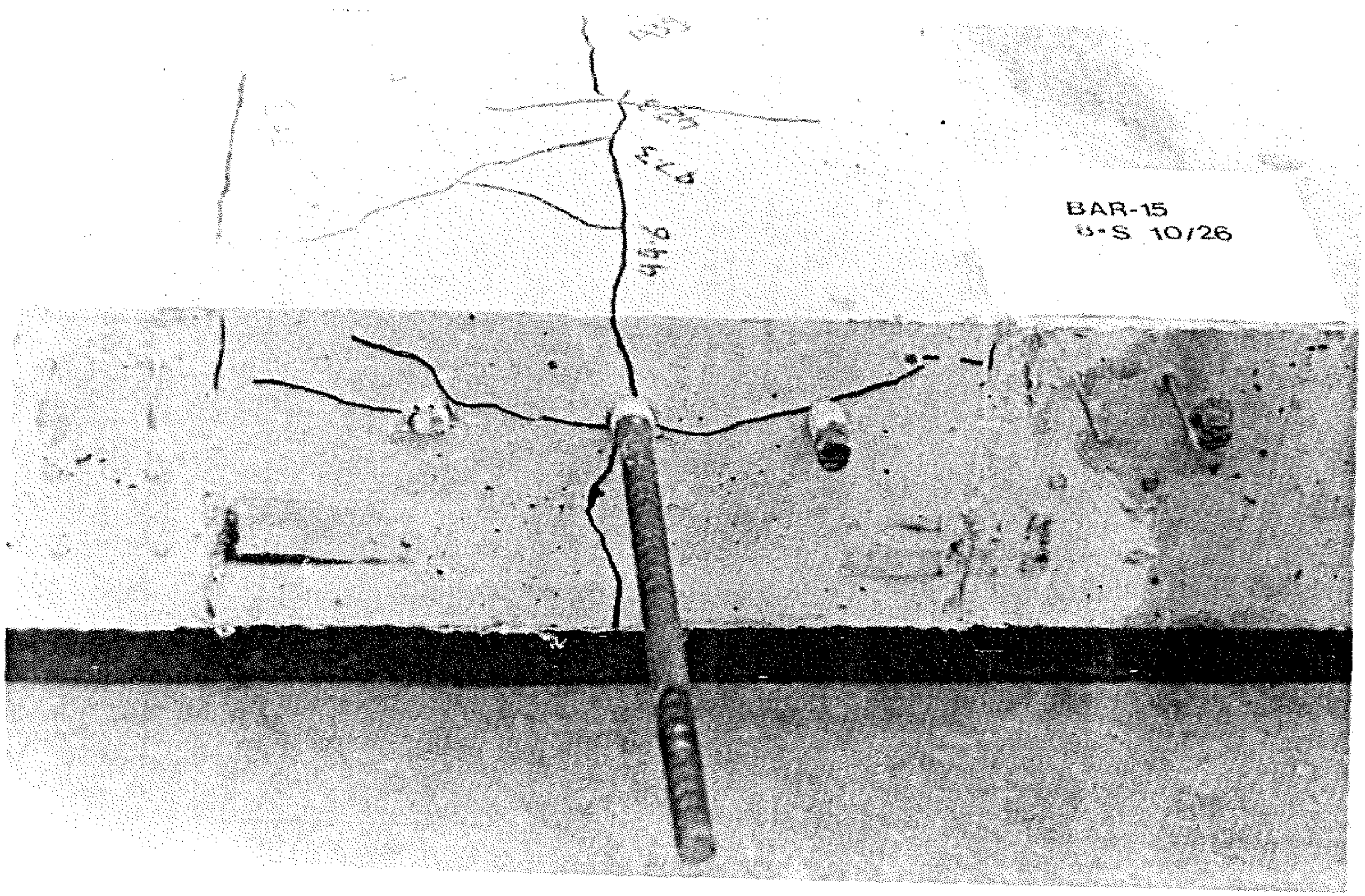


Fig. 2.11 Typical Cracking Pattern after Test Bar Pullout

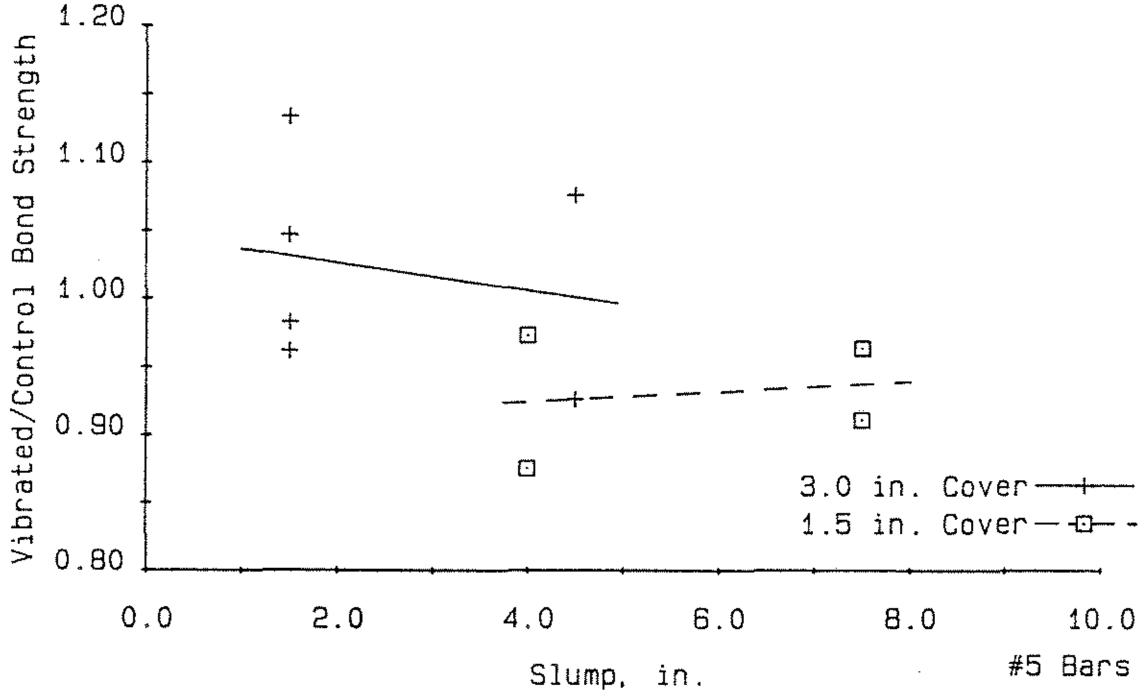


Fig. 3.1 Ratio of Traffic Vibrated to Control Bond Strength versus Slump (#5 Bars)

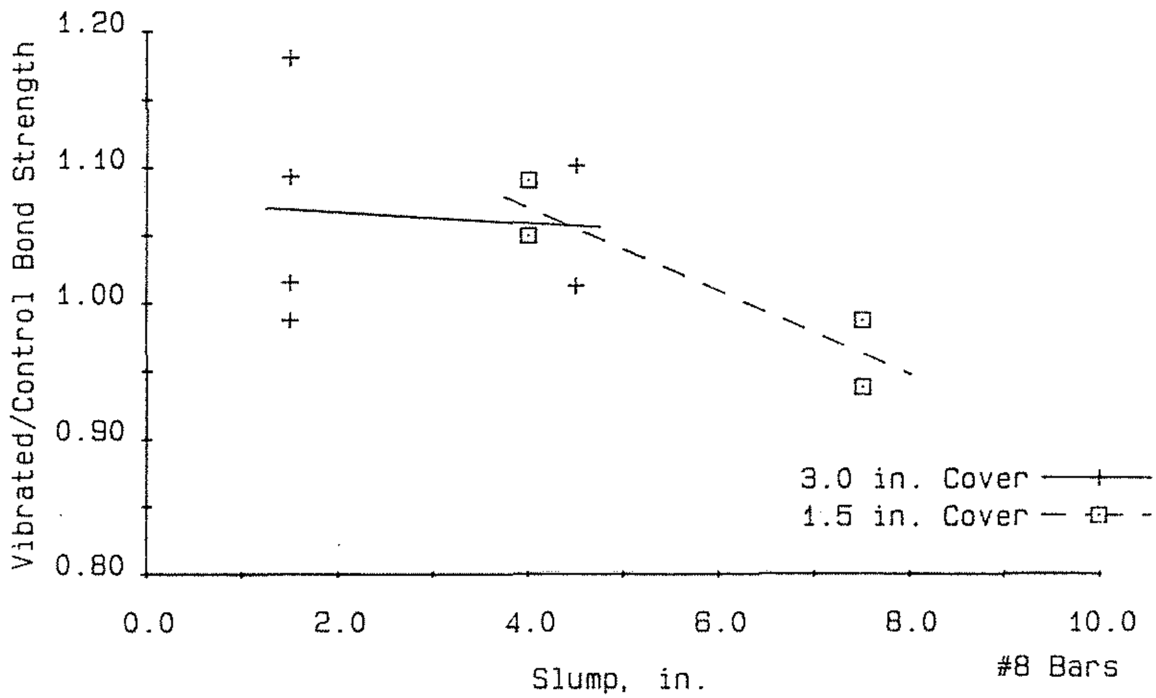


Fig. 3.2 Ratio of Traffic Vibrated to Control Bond Strength versus Slump (#8 Bars)

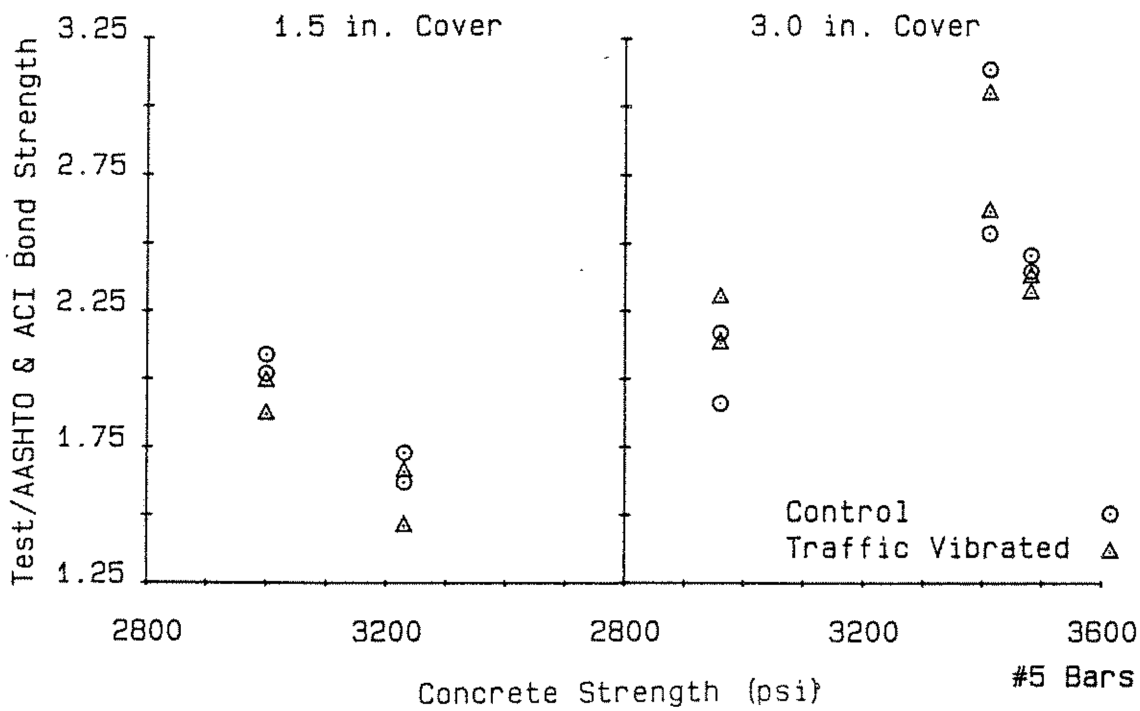


Fig. 3.3 Comparison of Experimental Bond Strengths to AASHTO and ACI Bond Strengths (#5 Bars)

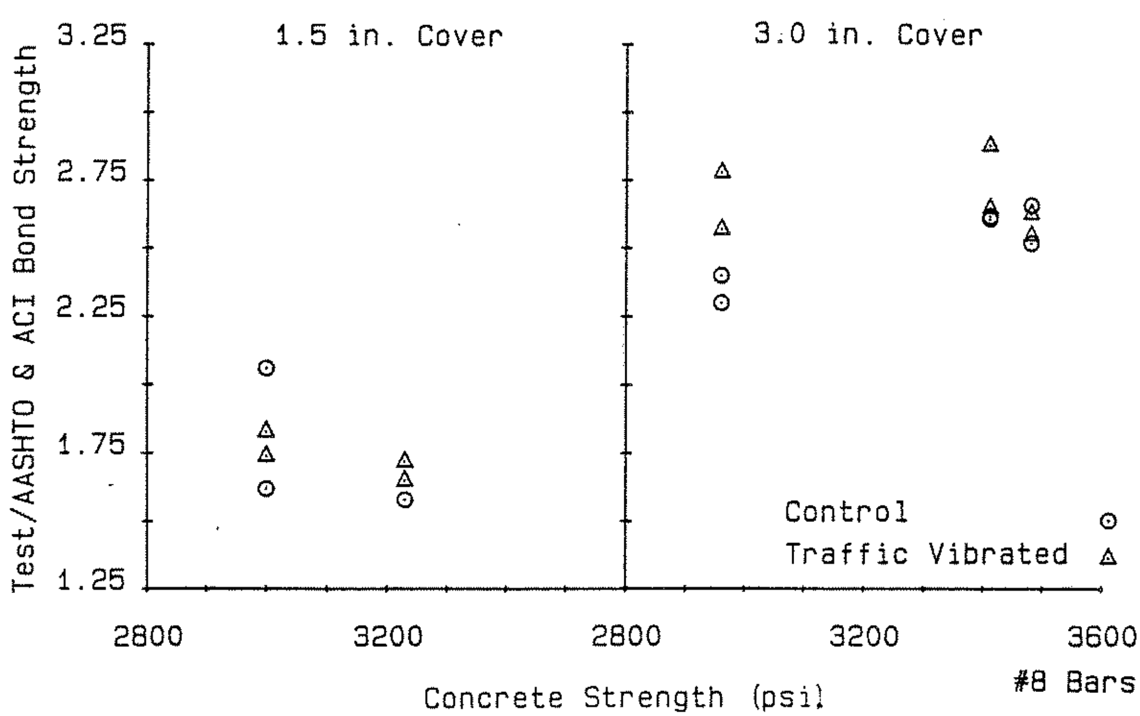


Fig. 3.4 Comparison of Experimental Bond Strengths to AASHTO and ACI Bond Strengths (#8 Bars)

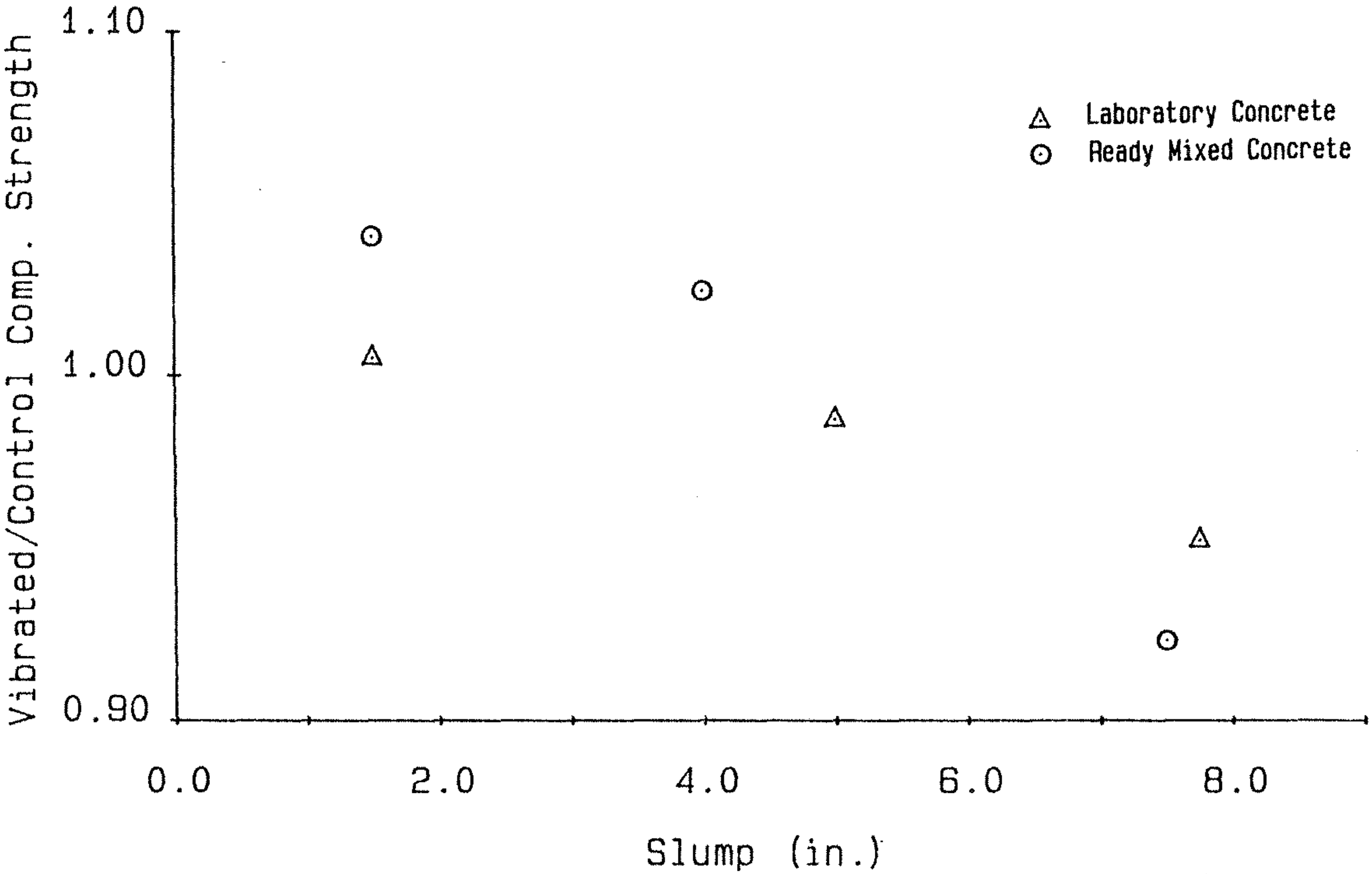


Fig. 3.5 Ratio of Traffic Vibrated to Control Compressive Strength versus Concrete Slump