Second Progress Report

EXPERIMENTAL INVESTIGATION OF A PRESTRESSED STEEL BEAM-CONCRETE SLAB BRIDGE UNIT

One Year Creep Test Results

by

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CHAPTER I

INTRODUCTION

A research program to study the strength and stiffness characteristics of a full-scale pre-cast, prestressed steel beam composite bridge unit has been undertaken at the Fears Structural Engineering Laboratory, University of Oklahoma, under the sponsorship of the Oklahoma Department of Transportation. The unit is 55 ft. 0 in. long, 6 ft. 9½ in. wide, and consists of a 7½ in. thick reinforced concrete slab and two W21x50 steel beams. The total weight of the unit is approximately 40,500 pounds. A previous progress report, "Experimental Investigation of a Prestressed Steel Beam-Concrete Slab Bridge Unit", Research Report No. FSEL/ODOT 82-02, documents the construction of the unit, reports measured material properties, and reports creep data from the initial 60 days of observation[1]. The primary purpose of that report was to present the first two months of behavior of the unit under sustained loading.

Initially, six months of creep observation under sustained loading were to be made. However, during that six month observation period, the vertical downward deflection of the bridge unit continued to increase, possibly indicating creep phenomenon. Hence, the initial creep observation period was extended to a full year.

One explanation for the creep effects is the large local stresses that exist near the stud shear connectors because of the prestressing.

Stud shear connectors were used to transfer the horizontal shear forces from the concrete to the steel in the bridge unit being studied. Use of connectors with a larger bearing area, such as channel connectors, may minimize creep effects. As a result, a sub-project was undertaken to investigate creep effects associated with both stud and channel shear connectors.

The purpose of this report is to update the creep data from the initial 60 days of observation through the first year and to describe the experimental set-up of the auxiliary creep tests (hereafter referred to as "shear creep tests".) The results of the initial 125 days of observation will also be given for the shear creep tests.

For reference, the first readings for the bridge unit were taken on April 8, 1982 and the last on April 26, 1983. For the auxiliary tests, the first readings were taken on March 25, 1983 and the last on July 30, 1983.

CHAPTER II

BRIDGE UNIT CREEP TEST

2.1 Instrumentation

All components of the bridge unit, with the exception of the shear connectors, were fully instrumented. Instrumentation included electrical resistance strain gages, concrete strain gages, deflection transducers, and thermometers. A detailed description of the instrumentation is given in Reference 1.

2.2 Measurements

The strain measurements were made using a strain indicator and a switch and balance unit which were permanently wired to the strain gages. The deflection transducers were monitored using a d.c. voltage supply and standard voltmeter. The entire set of measuring instruments was stored in a refrigerator (temperature set at 70^{0}F) in a temporary building adjacent to the bridge unit.

Daily readings were taken for the first sixty days of the sustained loading tests. Thereafter, readings were taken on a weekly basis. Weather conditions were recorded each time data was taken. Also, the camber was measured periodically using a surveying level.

2.3 Test Results

The results of the creep data taken for the first year are presented graphically in Figures 2.1 to 2.5. Stress data was obtained from the strain data using Hooke's Law (assuming perfectly elastic material) and an assumed modulus of elasticity of 29,000,000 psi for steel.

Variations in the concrete strain at midspan along the longitudinal axis of the bridge unit together with temperature changes are shown in Figure 2.1. Figure 2.2 depicts the variations in stresses at the top and bottom flanges of the west girder. The average midspan deflection of the unit is shown in Figure 2.3. Table 2.1 compares the change in camber at the centerline of the unit as compared to the change in deflection. Stress variations in both the top and bottom longitudinal reinforcement are illustrated in Figure 2.4. Changes in the neutral axis location in the steel beam versus time are shown in Table 2.2 and Figure 2.5.

2.4 Discussion

An examination of the results shown in Figures 2.1 to 2.4 shows that all the strain values, and consequently all the stress values, along with vertical deflections are sensitive to changes in temperature. Changes in strain on the girder and changes in strain on the rebars appear to be least affected by temperature change while vertical deflection and changes in strain on the concrete surface are the most sensitive. Thus, the variation in temperature must be considered in any interpretation of results.

A study of Figure 2.1 shows that the change in strain at the concrete surface closely follows the change in temperature. As the air temperature increased, the change in strain increased and as the air temperature decreased, the change in strain also decreased. Increases in strain were found to be most rapid during the first few days of observation. The change in strain steadily increased during the warm months (May to August 1982) and fluctuated during the late fall and winter months (December 1982 to February 1983) when the temperature also fluctuated significantly. As the plot indicates, strain values have not yet asymptotically approached a maximum and changes are still occurring. It is noted that the strain is plotted for the top surface at the centerline of the bridge unit.

Figure 2.2 shows the stresses of the bottom and top flanges of the west beam. Stresses were calculated from strain data assuming a modulus of elasticity of 29,000 ksi. A comparison of Figure 2.1 to Figure 2.2 shows that temperature has less effect on the change in stress of the beams than it does on the concrete surface strains. A sharp increase in stress in the bottom flange occurred when the bridge unit was turned over and when the additional dead load was applied. Increases also occurred in the top flange but to a lesser degree. After the application of the dead load, the stresses in the top and bottom flanges gradually increased until about the 265th day (January 1, 1983). Since that time, the stresses in both the top and bottom flanges have somewhat decreased. Initially, the stresses in the top flange increased at a slightly faster rate than those of the bottom flange. This indicated that the location of the neutral axis was moving away from the top flange toward the bottom flange. The location of the neutral axis was determined using the beam flange strain readings and assuming a linear strain distribution within the section.

Figure 2.3 shows the variation in vertical downward deflection at midspan. It is evident that the variations in temperature cause variation in the vertical downward deflection. The figure shows that camber (positive deflection) existed in the bridge unit until the sustained dead load was placed on the bridge (approximately the 20th day). The vertical downward deflection then increased substantially as the temperature increased. The maximum change in vertical downward deflection, 0.87 inches, was reached near the 150th day (first week of September, 1982). Thereafter, the vertical downward deflection decreased (increased upward) as the temperature decreased. As shown in Table 2.1, the change in camber closely followed the change in deflection.

Figure 2.4 shows the behavior of the strain on the reinforcing bars. A comparison of Figures 2.1 and 2.4 shows that the change in strain of the concrete surface is similar to that of the change in strain of the reinforcing steel. The strain (hence the stress) increased and decreased as the temperature increased and decreased. As with the concrete surface strains, the greatest increases in reinforcing steel strains also occurred during the first few days of observation. However, the strains on the reinforcing bars did not increase as rapidly as the strains at the concrete surface, nor as the temperature increased. The location of strain measurements for the two plots shown in Figure 2.4 are top and bottom longitudinal reinforcing bars with strain gages at the centerline.

Figure 2.5 shows a plot of the neutral axis location from the top of girder versus time. It can be seen that initially the neutral axis moved rapidly toward the bottom flange. The rate of increase subsequently decreased considerably, and at the end of the reporting period, appears to be moving back towards the top flange.

Strain gages are located on both sides of the web on the top and bottom flanges, to compare strains on the exterior and interior sides of the flanges. The strains on the underside of the interior side of the lower flange were consistently larger than those on the underside of the exterior side of the lower flange. The differences in strains at the interior and exterior sides of the top flange were negligible. The difference between the strains at top and bottom flanges can probably be attributed to the fact that the top flange of each beam is continuously braced by the concrete slab while the bottom flange is unbraced. The unbraced length of the bottom flange is approximately 18 ft., the distance between the interior diaphragms. As a result, the web and the bottom flange of each beam are relatively free to rotate. As stated in the previous report[1], the addition of dead load on the slab caused bending of the slab in the direction transverse to the beams. The rotation of the slab at the beam location introduces torque into the beams because of the relatively rigid slab to beam connection. This torque tends to bend the bottom flanges outward introducing secondary bending stresses in the flanges. The axis of this bending is in the plane of the web and results in secondary compressive strains in the exterior part of the bottom flange and tensile strains in the interior part of the bottom flange. Thus the existing tensile strain in the exterior part of the bottom flange is decreased and that of the interior part of the bottom flange is increased.

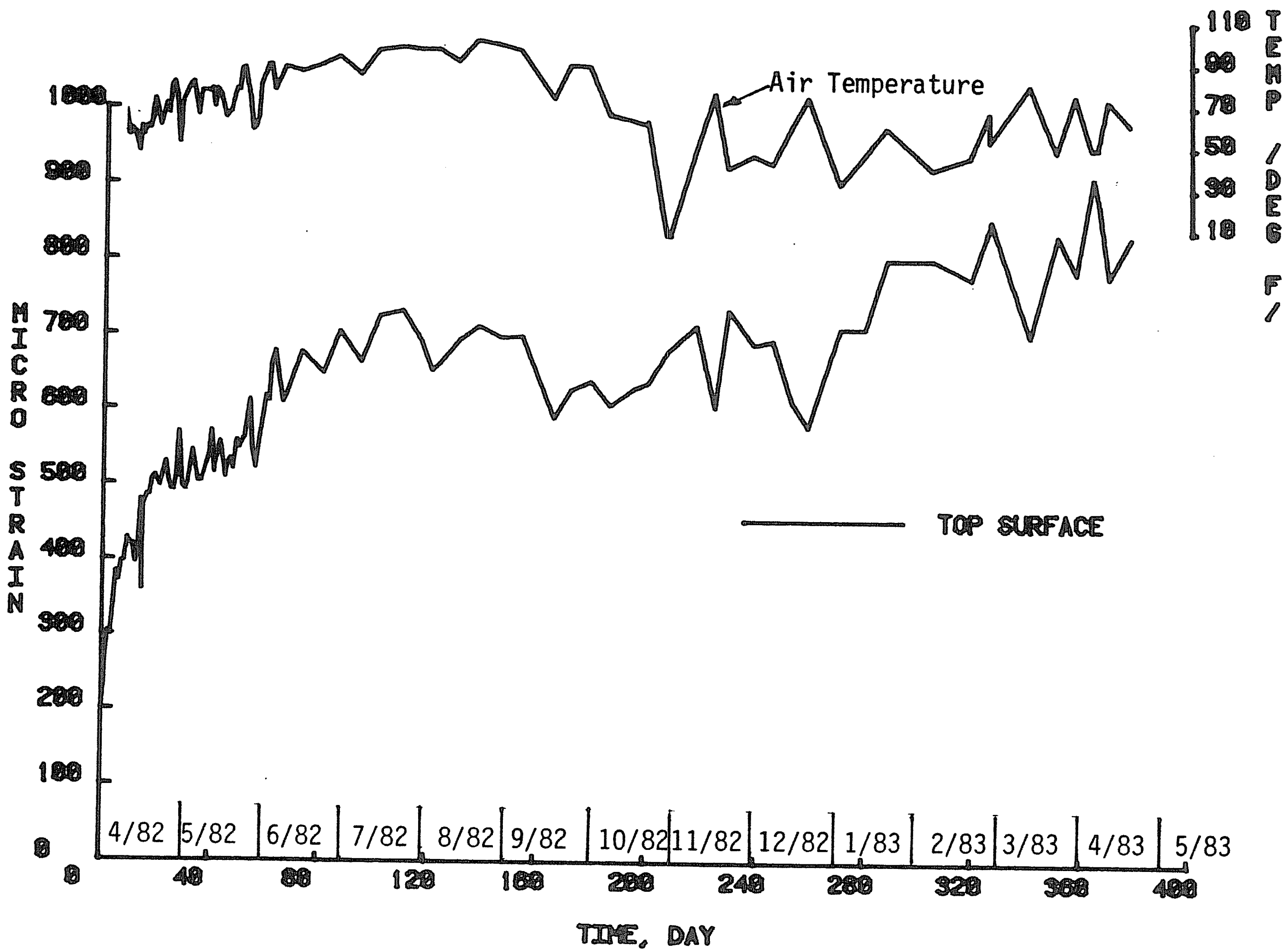


Figure 2.1 Change in Strain of Concrete Surface vs. Time

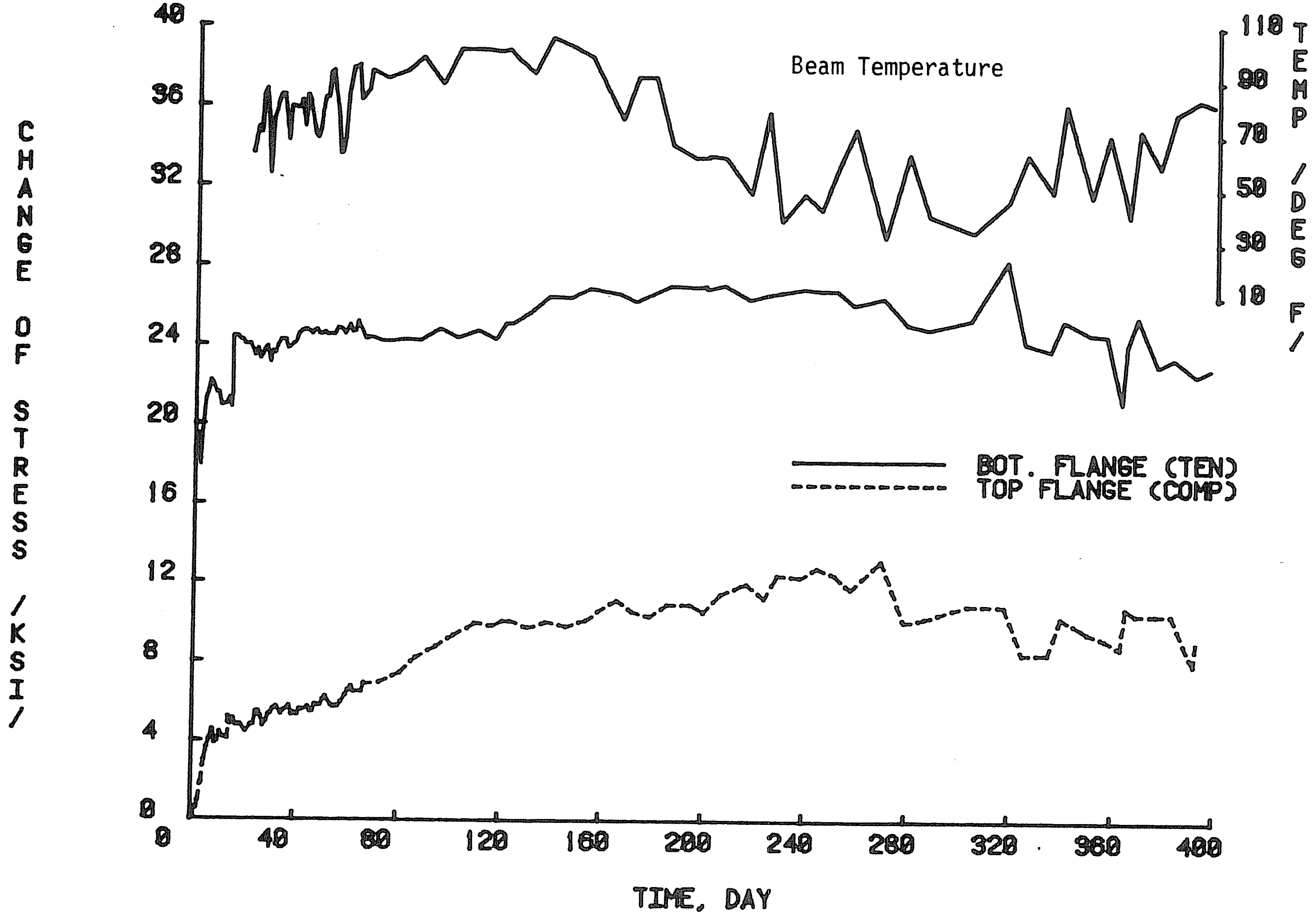


Figure 2.2 Change in Stress on Beam Flanges vs. Time

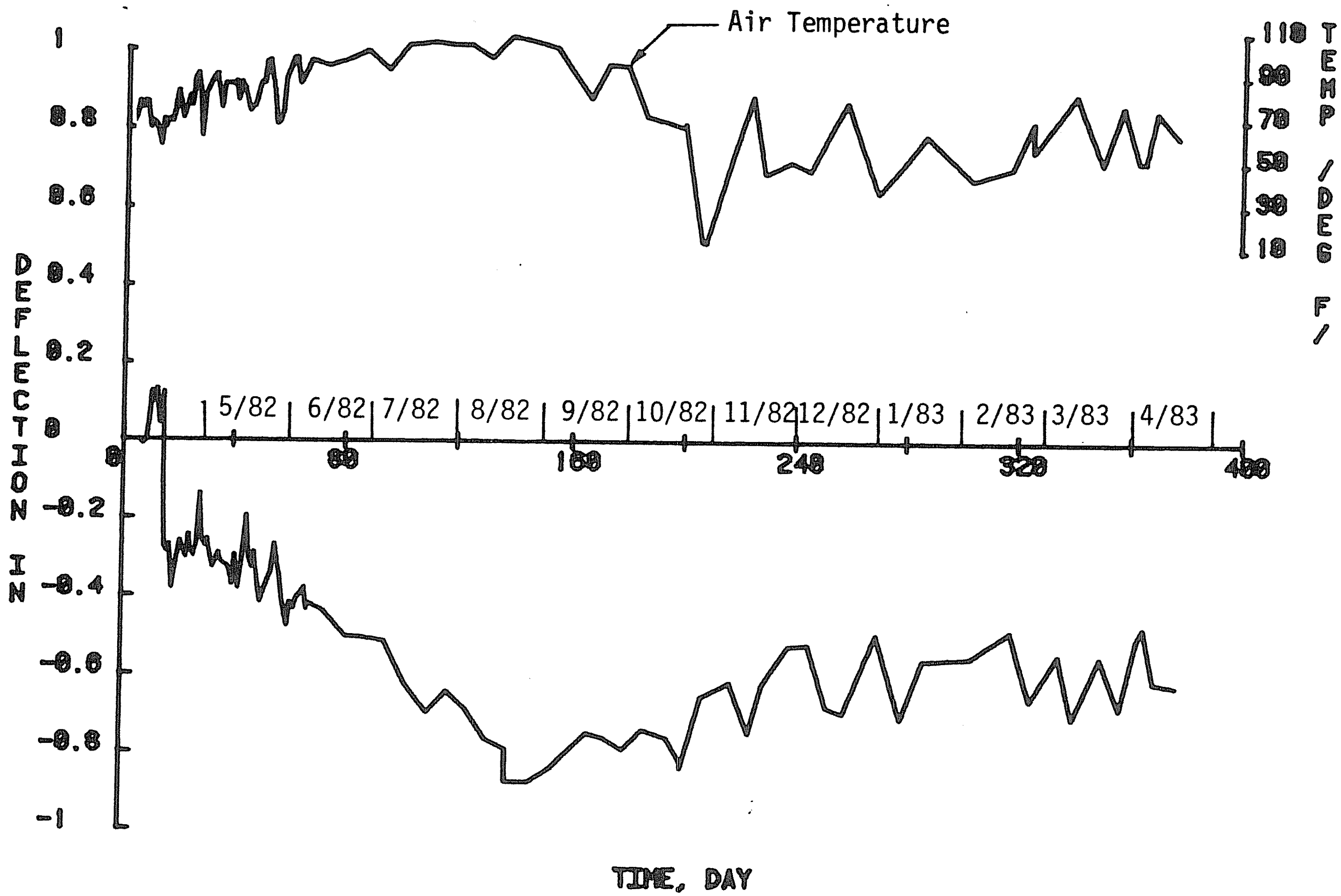


Figure 2.3 Time vs. Change in Deflection

Table 2.1
Comparison of Camber and Deflection in Bridge Unit

Date	No. of Days	Camber (in.)	Change in Camber (in.)	Change in Deflection (in.)
9/16/82	160	0.371	-0.062	-0.032
9/28/82	172	0.309	-0.055	-0.078
10/26/82	200	0.254	+0.195	+0.222
11/12/82	217	0.450	+0.062	-0.006
11/24/82	229	0.512	-0.089	-0.088
3/14/82	340	0.423		

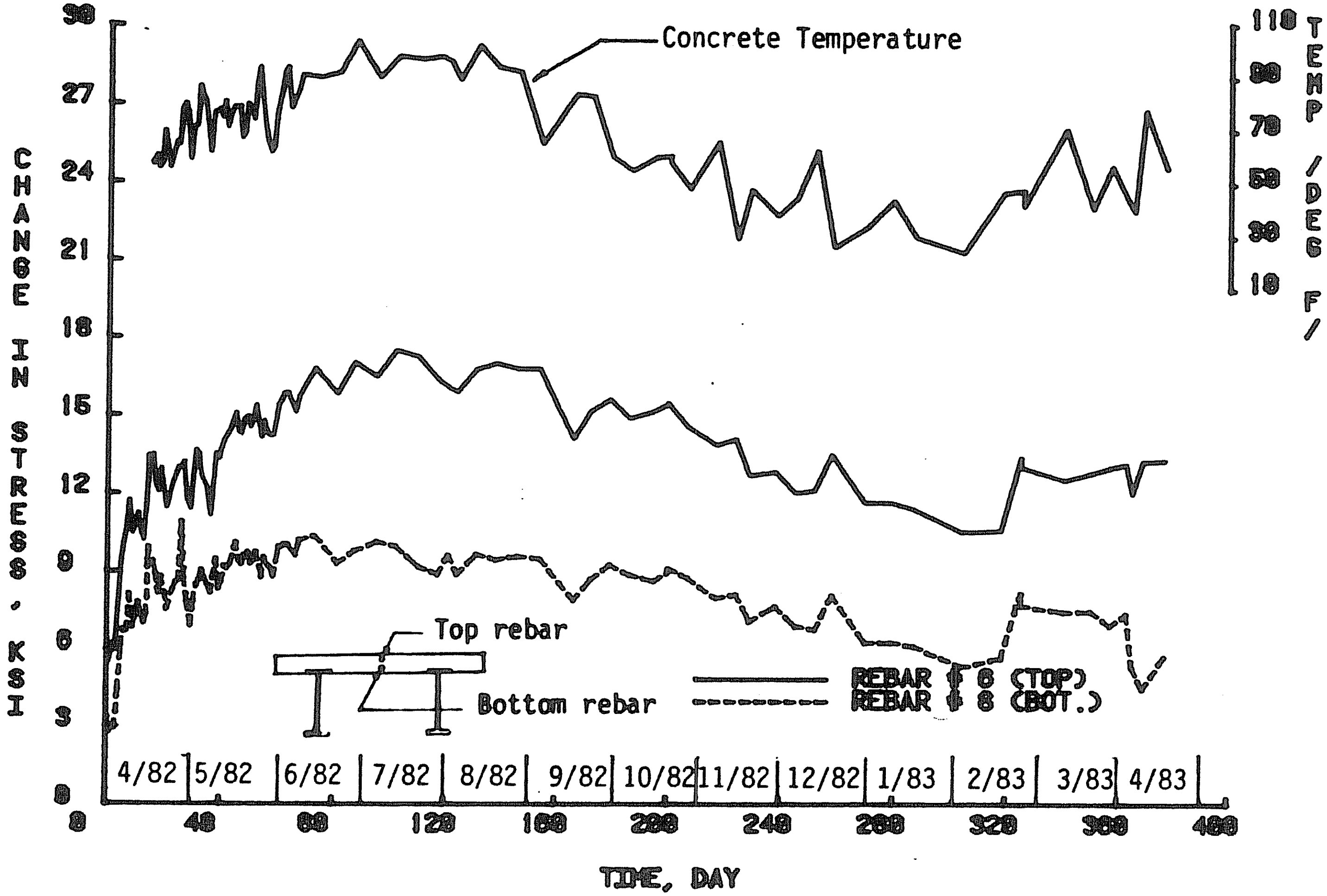


Figure 2.4 Change in Stress of Rebars vs. Time

Table 2.2
Beam Strains and Location of Neutral Axis

Days From Beginning of Observation	West Beam Strains $(10)^{-6}$ in/in				Average Flange Strain		Neutral Axis Location From Top of Beam (in)
	Top of Top Flange		Bottom of Bottom Flange				
	Ext.	Int.	Ext.	Int.	Top	Bottom	
0	-7	-5	626	670	-6	648	0.19
3	-37	-29	649	702	-33	765	0.97
7	-140	-140	728	788	-140	758	3.25
14	-146	-137	684	750	-141	717	3.43
14	-182	-175	716	769	-178	742	4.04
14	-174	-166	813	870	-170	841	3.50
15	-169	-162	810	873	-166	841	3.43
30	-184	-182	778	854	-183	816	3.82
60	-217	-219	816	908	-218	862	4.20
95	-300	-305	807	903	-302	855	5.44
121	-344	-350	813	915	-347	864	5.97
154	-326	-332	881	987	-329	934	5.43
179	-320	-326	873	977	-323	925	5.39
207	-277	-289	892	989	-283	940	4.82
238	-251	-258	890	981	-254	935	4.45
270	-224	-225	876	964	-224	920	4.09
304	-197	-191	889	973	-194	931	3.59
335	-268	-288	832	920	-278	876	5.02
363	-284	-221	801	865	-252	833	4.84
392	-334	-260	804	870	-297	837	5.46

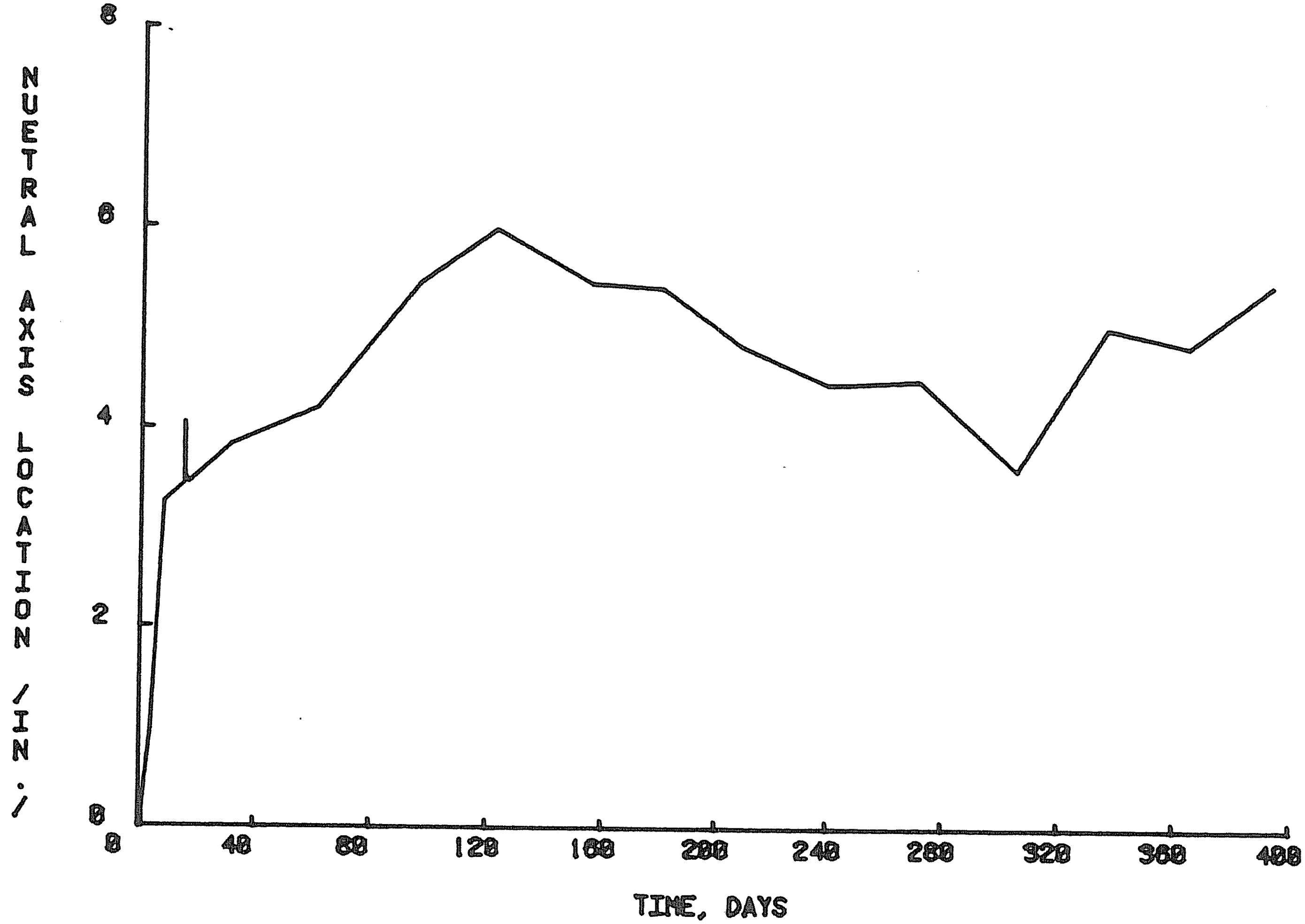


Figure 2.5 Neutral Axis Location from Top of Beam vs. Time

CHAPTER III

SHEAR CREEP TESTS

3.1 Test Description

Four test specimens were constructed at the Fears Structural Engineering Laboratory to investigate movement of the concrete slab near the shear connectors under conditions similar to that found in the bridge unit. As shown in Figure 3.1, each test specimen consists of 27 in. long, W12x35 steel wide flange section with two 7 in. thick reinforced concrete slabs cast symmetrically on each flange of the steel section. Each slab is 24 in. wide, 30 in. long, and is attached to the steel section by shear connectors. Two of the specimens have steel stud shear connectors and the remaining two have steel channel shear connectors. The stud shear connectors are 4 in. long and 3/4 in. in diameter. They were welded to the steel section in two rows, (two studs per row), at a distance of 12 in. apart. The channel connectors are 4 in. long, C3x4.1 hot rolled sections, also spaced 12 in. apart. Figure 3.2 shows the details of each type of specimen.

The specimens were constructed to match the construction of the bridge as closely as possible. An air-entrained concrete was obtained from the same local ready-mix plant as before with the same mix design and the same design strength (4000 psi). The design slump was between 1 and 3 in. The slabs for the four specimens were cast simultaneously, one side at a time, with two days between casting, to allow time to strip the forms and turn the specimens over and set them in the forms for the next pour.

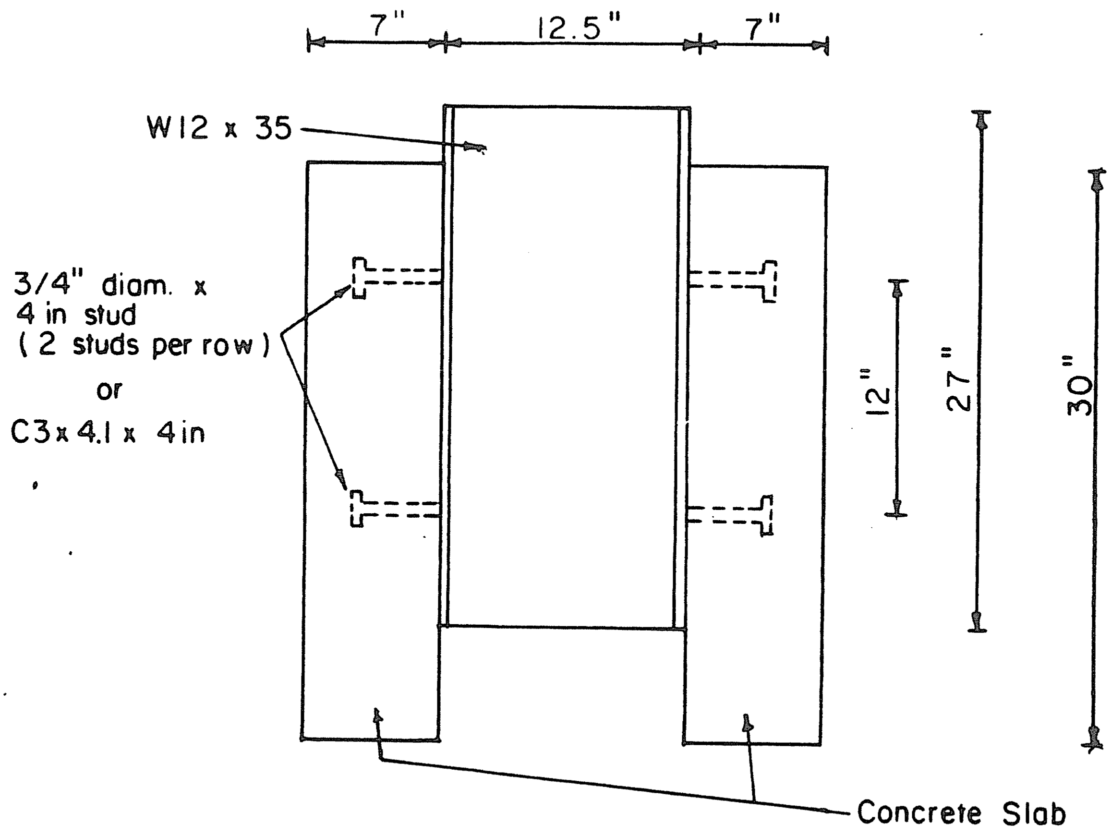


Figure 3.1 Shear Connector Creep Test Specimen

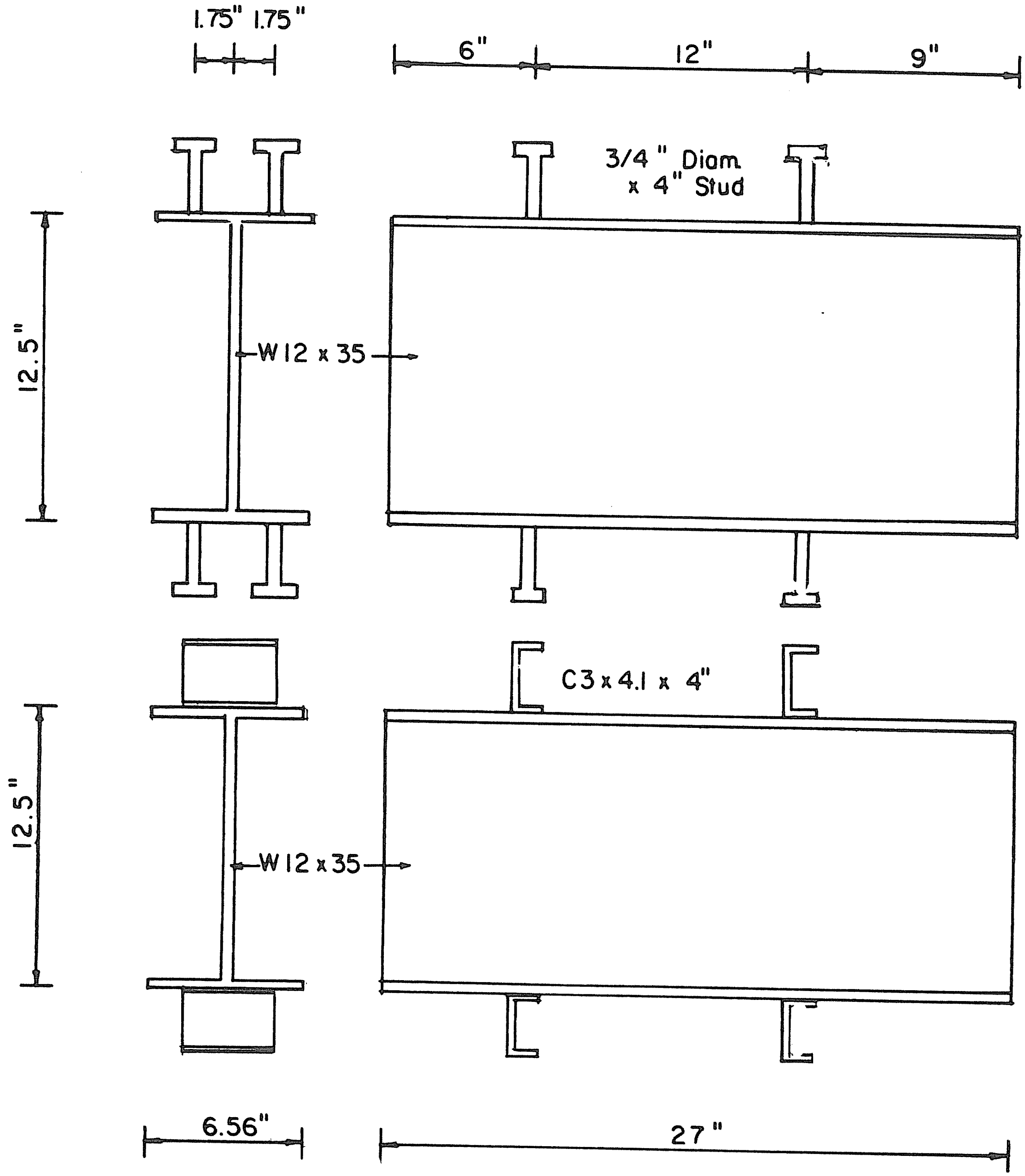


Figure 3.2 Shear Connector Details

During each concrete pour, a total of twelve 6 in. diameter by 12 in. high control cylinders were cast from the mixing truck. Slump tests were performed before the cylinders were cast. The air content was determined using a "Roll-a-Meter". The slump, air content, age and strengths of concrete are given in Table 3.1. The influence of age on the compressive strength of concrete is shown in Figure 3.3.

The four steel beams were standard W12x35 sections of A36 steel. Shear connectors were welded to the beams at the fabrication shop. All fabrication was done by Robberson Steel Company, Oklahoma City, Oklahoma, the fabricator of the bridge unit. No. 4 deformed reinforcing bars of Grade 60 steel were used in both directions in the concrete slab. The spacing between the bars was the same as that used in one section of the test unit's slab. Details of the reinforcement spacing are shown in Figure 3.4.

After casting, the concrete slabs were moist-cured for seven days. The exposed surface of the concrete was covered with wet burlap and polyethylene sheets during the curing period. After the curing period, the composite sections were placed in the specially designed test set-ups, Figures 3.5 and 3.6. The specimens were loaded approximately 20 days after the curing period (28 days after pouring). Each specimen was loaded using a combination of hydraulic ram and springs. The hydraulic ram was used to compress the springs to the desired deflection.

Prior to placing in the test set-ups, the springs were preloaded in a universal testing machine several times to 25.0 kips. Average deflections were measured at load increments and from this data a stiffness value was determined for each set of springs. With the known stiffness values, the loading on each specimen is determined periodically from measurements of the spring height. Each specimen was then loaded to 48 kips or 24 kips

Table 3.1
Concrete Properties - Shear Connector Test

Properties	Age	First Truck	Second Truck
Slump (in.)	-	2.5	2
Air Content (%)	-	4.5	5
Compressive Strength (psi)	7	4525	4128
	21	5405	5051
	28	5586	5168

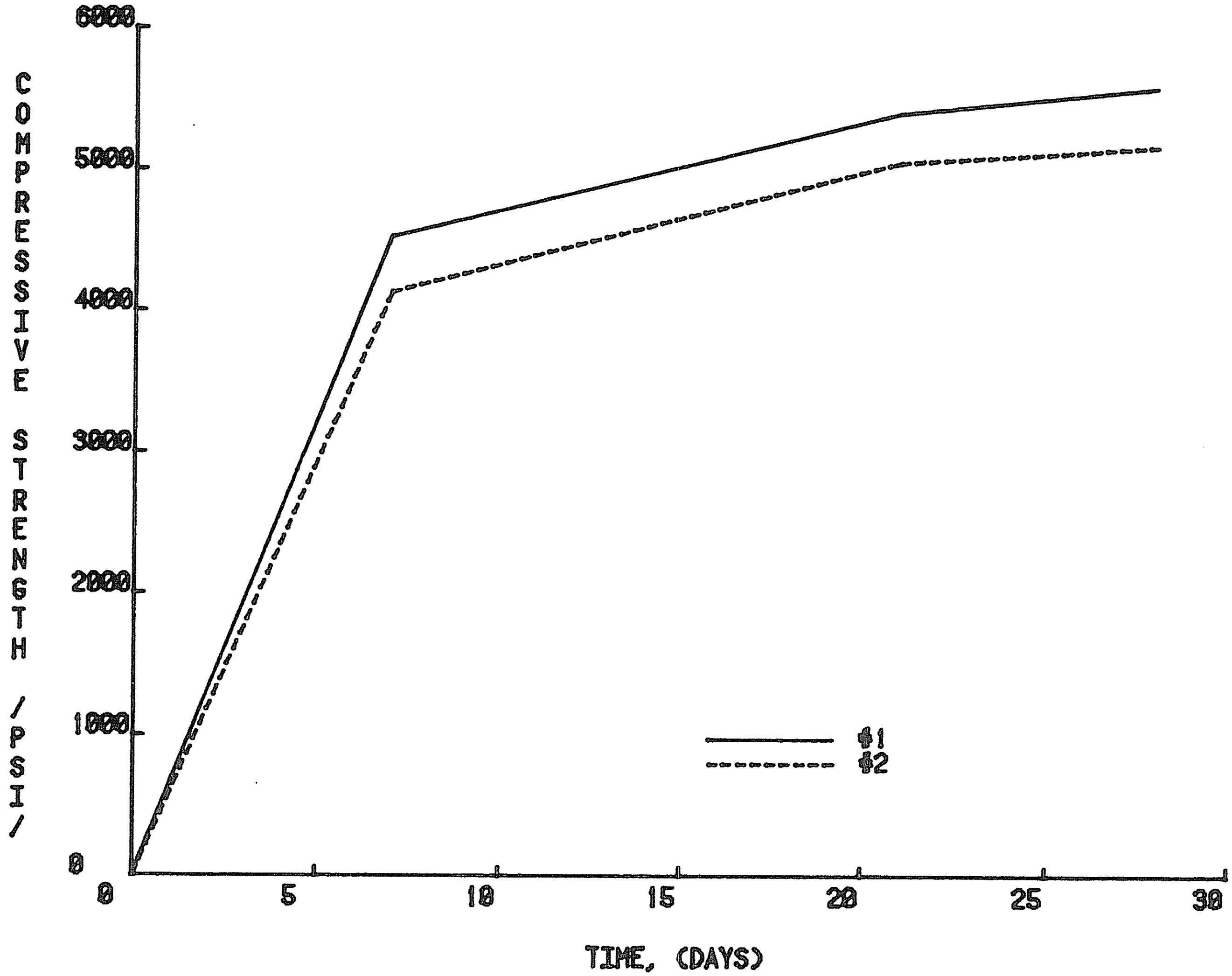
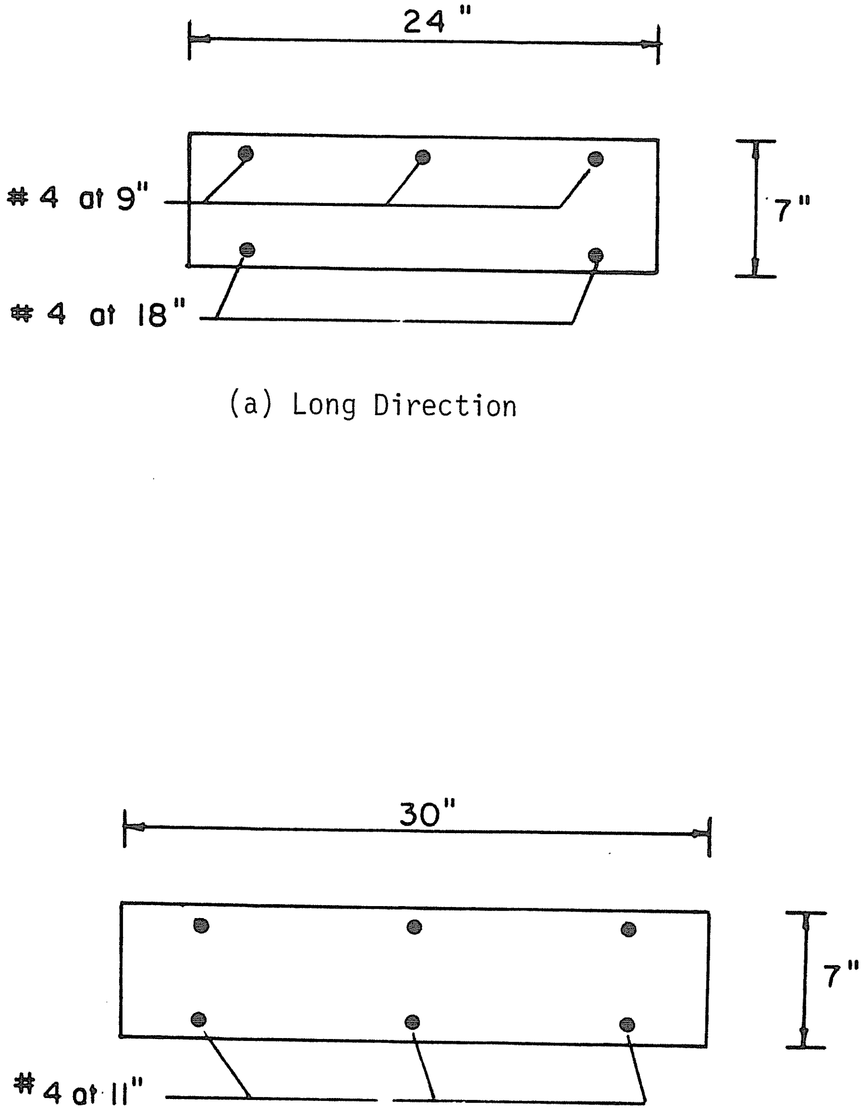


Figure 3.3 Compressive Strength of Concrete vs. Age, Shear Connector Test Specimen



(a) Long Direction

(b) Short Direction

Figure 3.4 Spacing of Reinforcement in Shear Connector Test Specimen

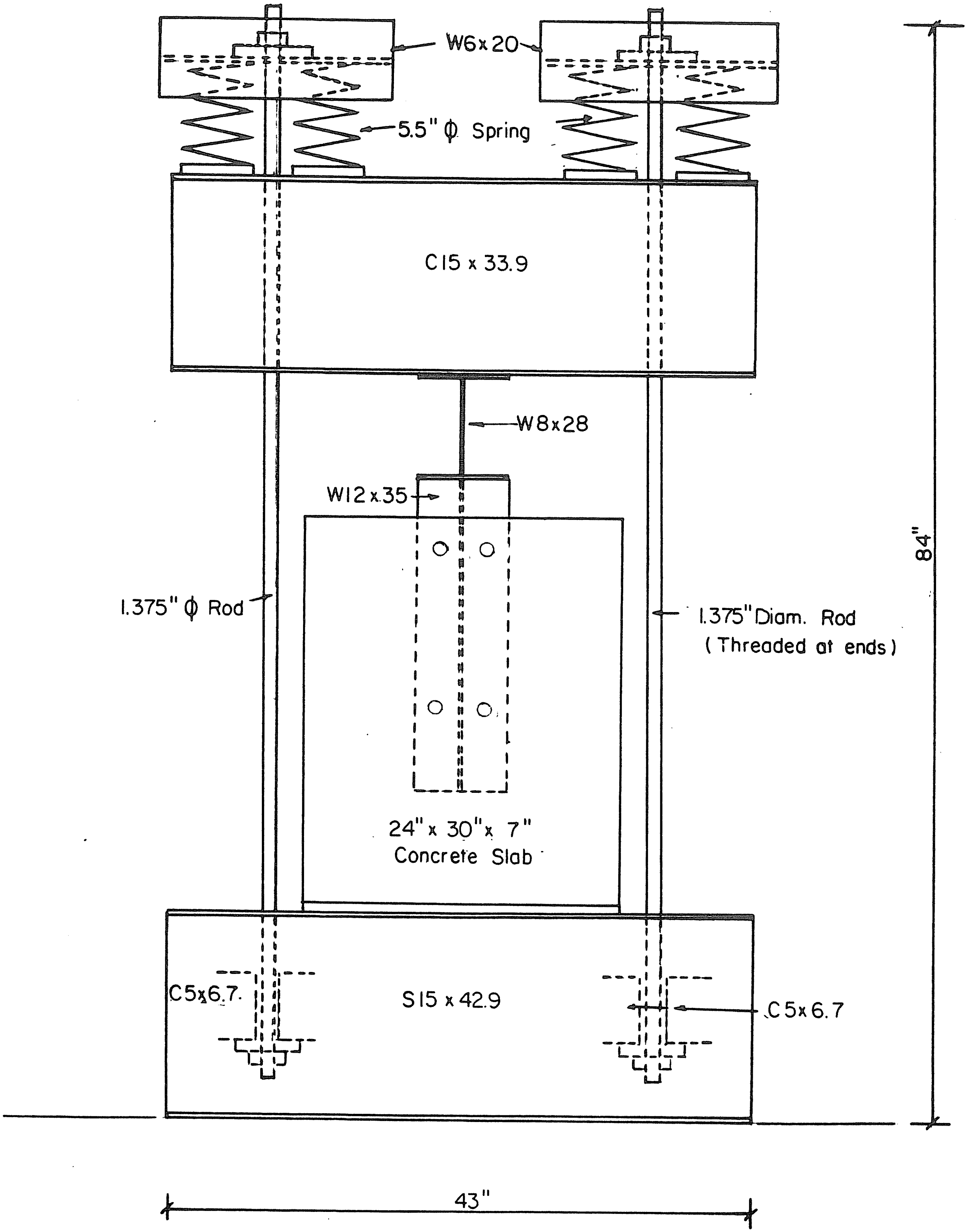


Figure 3.5 Front View of Shear Connector Test Set-up

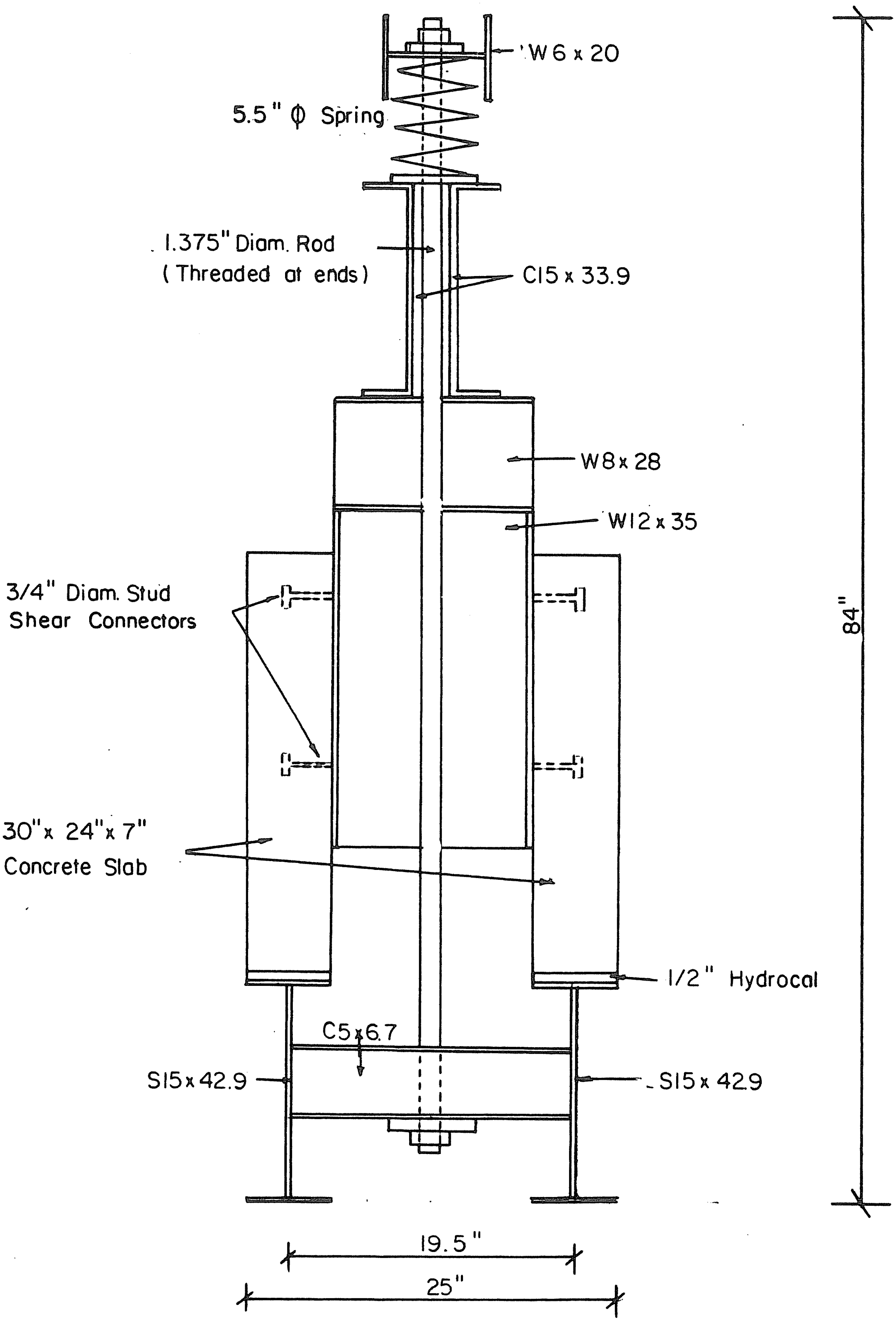


Figure 3.6 Side View of Shear Connector Test Set-up

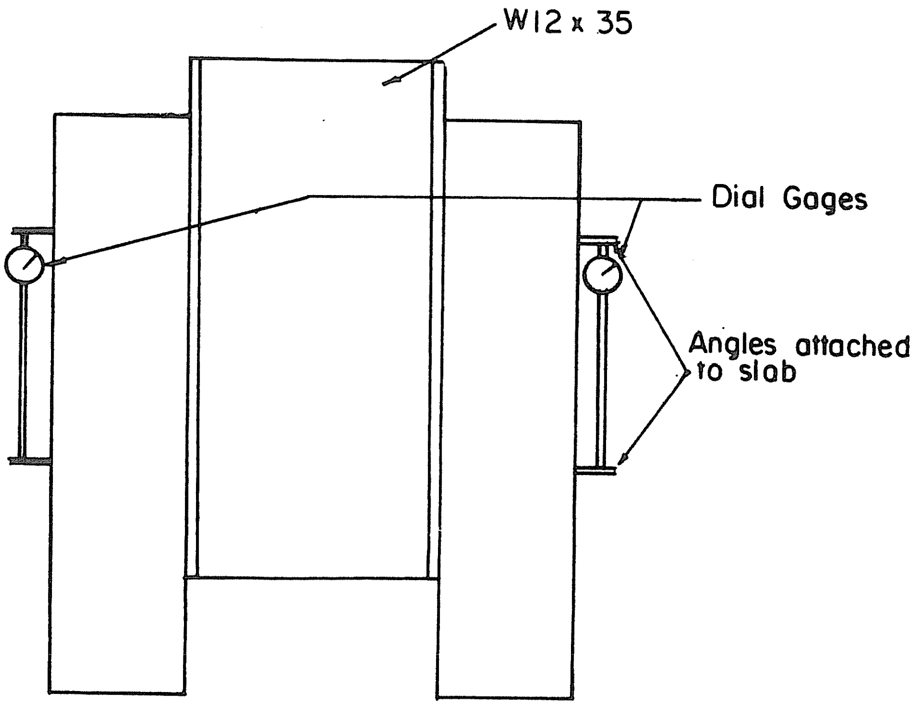
per rod using the hydraulic ram. The nuts were then tightened on the threaded rods to maintain the compression once the hydraulic ram was removed. Spring height measurements were made periodically to verify that the compression is being maintained.

Instrumentation for each specimen consists of four 0.0001 in. count dial gages to measure concrete creep displacements and two 0.0001 in. count dial gages to measure slip displacements between the concrete and the beam flange. The instrumentation is divided equally between the slabs of each specimen. Each dial gage is attached to a small angle and has a 10 in. rod extension that extends to another angle mounted 12 in. away in the vertical direction (either above or below the dial gage location). The angles are either bolted to small coupling nuts that were placed in the concrete at the time of pouring or welded to the beam flanges. Figure 3.7 shows the instrumentation for the test specimens.

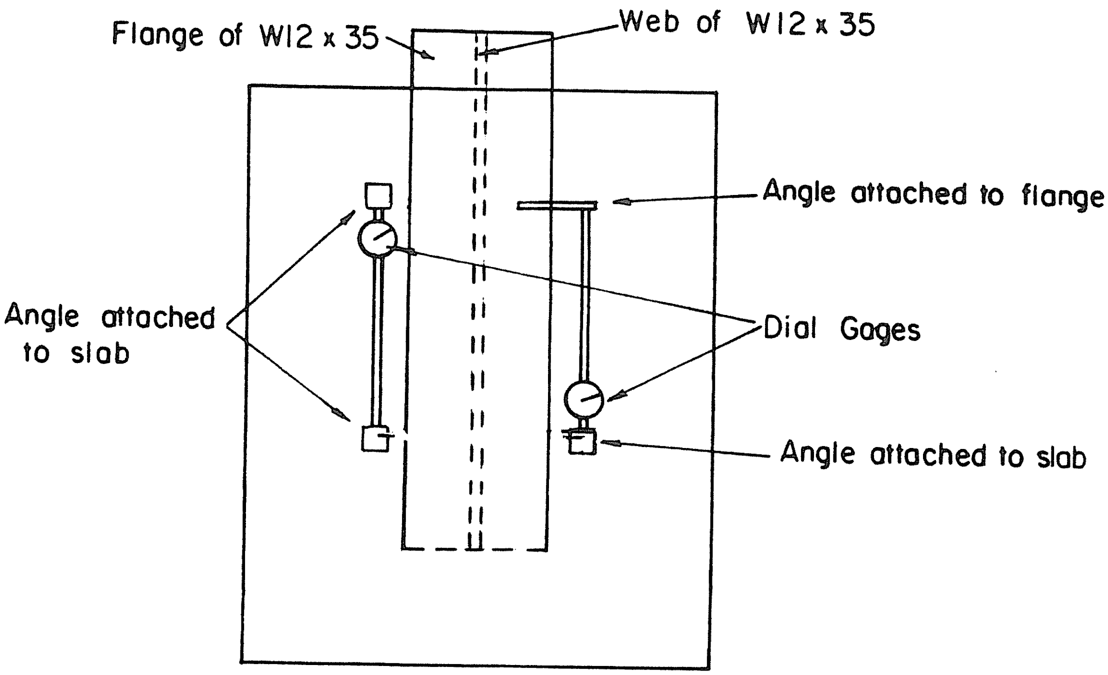
3.2 Preliminary Results

Dial gage recordings were taken on a daily basis for the first 30 days. Between 30 and 60 days, recordings were taken every second or third day. After 60 days, weekly readings were taken. Temperature and humidity readings were also recorded each time dial gage readings were taken. Time versus displacement curves for the first 125 days of data are shown in Figures 3.8 to 3.15. The curves shown are average values for all similar data, e.g. four creep displacement measurements (two per specimen and two specimens) and four slip displacement measurements. The variation in temperature is also shown on each plot.

Figure 3.8 shows the average creep displacement at the shear connectors and the creep displacement at approximately 5 in. away from the



(a) Surface Creep



(b) Shear Displacement (SLIP)

Figure 3.7 Instrumentation of Shear Connector Specimens shear connector for the stud-connector specimens. Figure 3.9 depicts the average slip displacement for the same specimens. For the channel-connector specimens, the average creep displacements at the shear connectors and at approximately 5 in. away from the shear connectors is shown in Figure 3.10. Figure 3.11 shows the average slip displacement for the channel-connector specimens.

Figures 3.12 to 3.15 compare the creep and slip displacements between the stud and channel specimens. The creep displacements at the shear connectors and the creep displacements away from the shear connectors are compared in Figures 3.12 and 3.13, respectively. Figure 3.14 compares the slip displacement between the two types of specimens. Figure 3.15 compares the creep displacements at and away from the shear connectors for each type of specimen.

3.3 Discussion

An examination of the results shown in Figures 3.8 to 3.11 reveals that all the displacement values (creep and slip) are sensitive to changes in temperature and any interpretation of the results must take into consideration this variation. Figure 3.8 shows that creep displacements at the stud shear connectors is considerably larger than the creep displacements away from the stud shear connectors. The change in creep displacements at the shear connectors closely follows the change in temperature with the peaks and valleys for each variable (displacement and temperature) occurring on the same days. The creep displacement away from the stud shear connector also follows the temperature variations but to a lesser extent. The largest value of creep displacement measured for the 125 days of data is 0.0030 in. at the stud shear connectors and 0.0014 in. away from them. Both values occurred on the day the highest temperature, 92^{o}F, was recorded.

Figure 3.9 shows the average slip displacement for the stud shear connector specimens. The slip displacements also follow the temperature variations closely. The largest value of slip displacement was 0.0026 in. and occurred at the time of the highest temperature.

Figure 3.10 shows the average creep displacements at the shear connectors and at approximately 5 in. (approximately) away the shear connectors for the channel-connector specimens. The creep displacement at the channel shear connectors is substantially larger than that away from them. Both curves vary with change in temperature. The largest displacements recorded were 0.0030 in. of creep displacement at the channel shear connectors and 0.0013 in. of creep displacement away from the channel shear connectors.

Figure 3.11 shows the average slip displacement for the specimens with channel shear connectors. It can be seen from Figure 3.11 that these displacements are also sensitive to changes in temperature. The largest displacement recorded was 0.0033 in., again occurring simultaneously with the highest temperature.

A comparison of average creep displacements at the shear connectors for stud and channel specimens is shown in Figure 3.12. The figure shows that this data is nearly identical for the two types of shear connectors. Figure 3.13 compares the average creep displacements away from the shear connectors for each type of specimen. Again, the results are nearly identical.

Figure 3.14 compares the average slip displacements for the two types of specimens. The results show that the slip in the channel specimens is slightly greater than that in the stud specimens. Figure 3.15

compares the average creep displacements at the shear connectors and away from the shear connectors for both types of specimens. It is clear that the creep displacements are greater near the shear connectors than it is away from the shear connectors for both types of constructions.

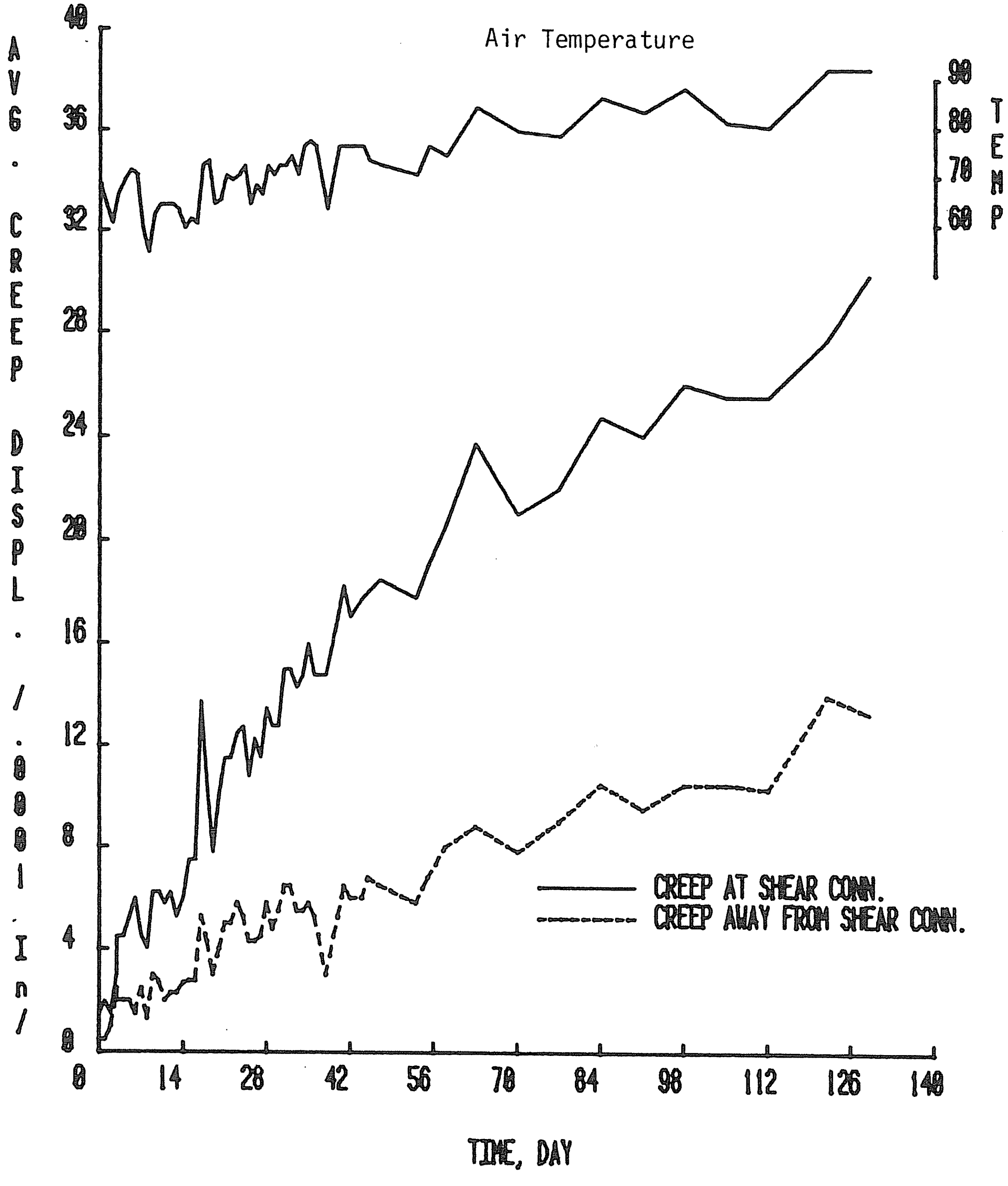


Figure 3.8 Average Creep Displacement in Specimens with Stud Connectors

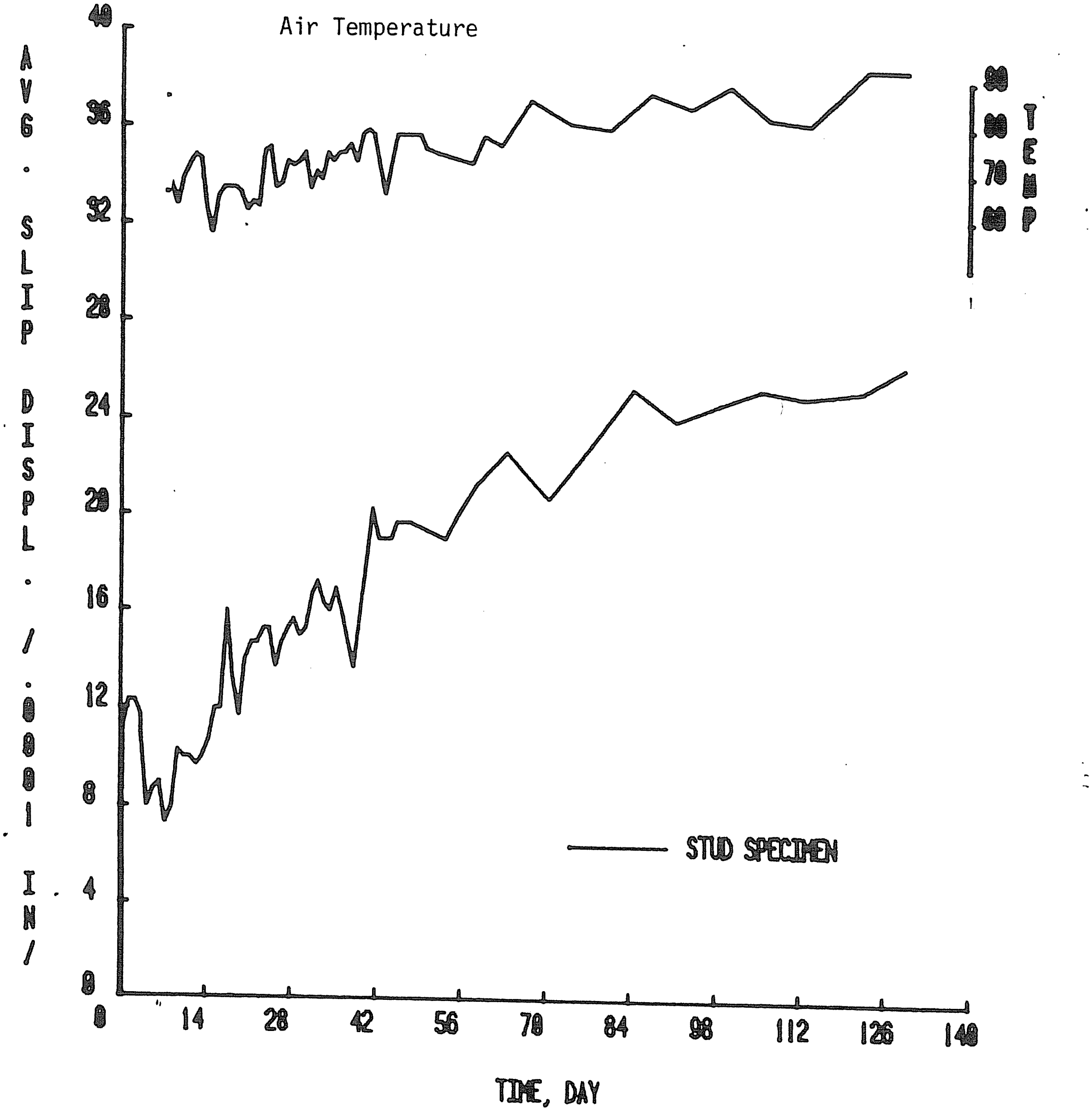


Figure 3.9 Average Shear Slip Displacement in Specimens with Stud Connectors

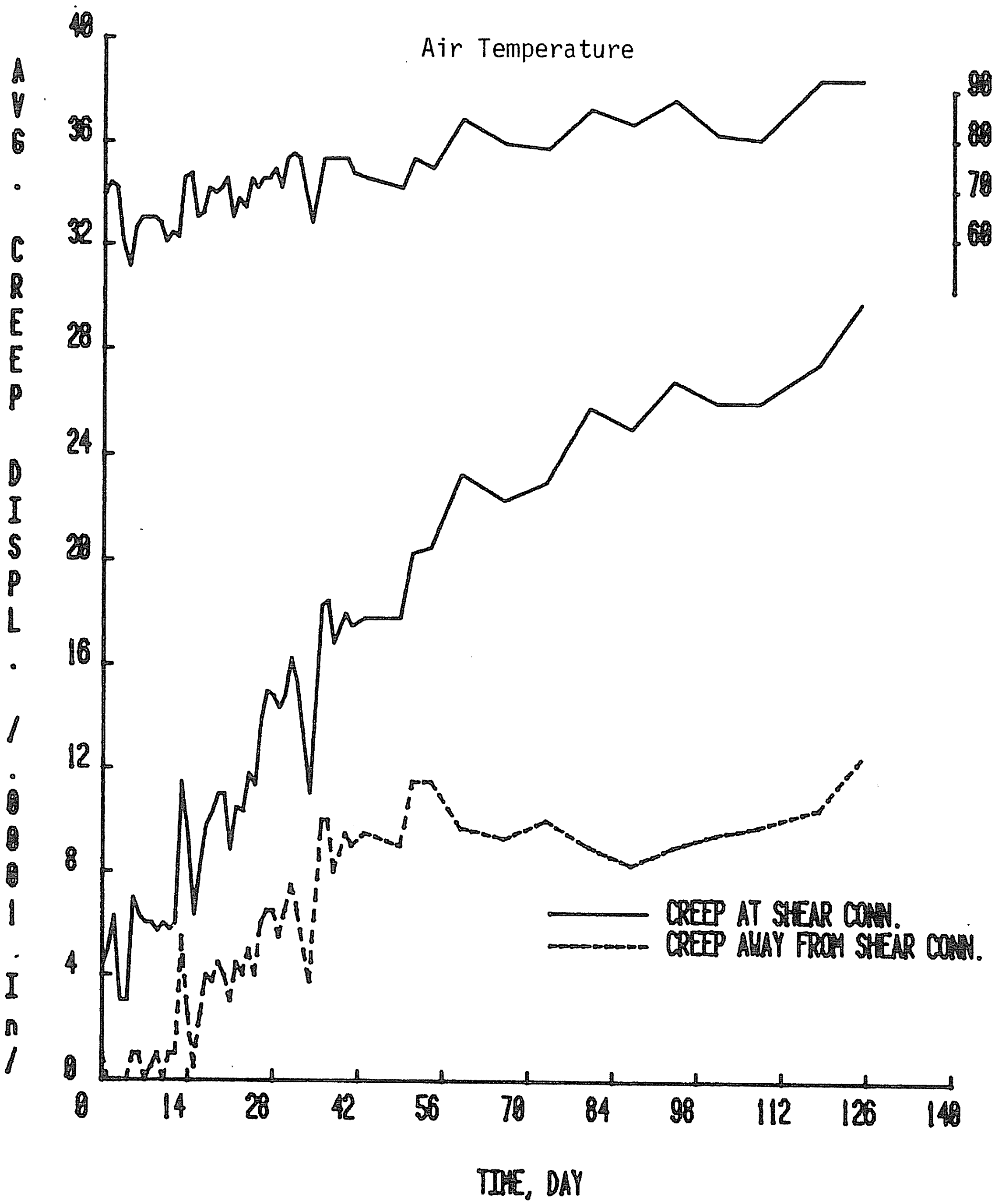


Figure 3.10 Average Creep Displacement in Specimens with Channel Connectors

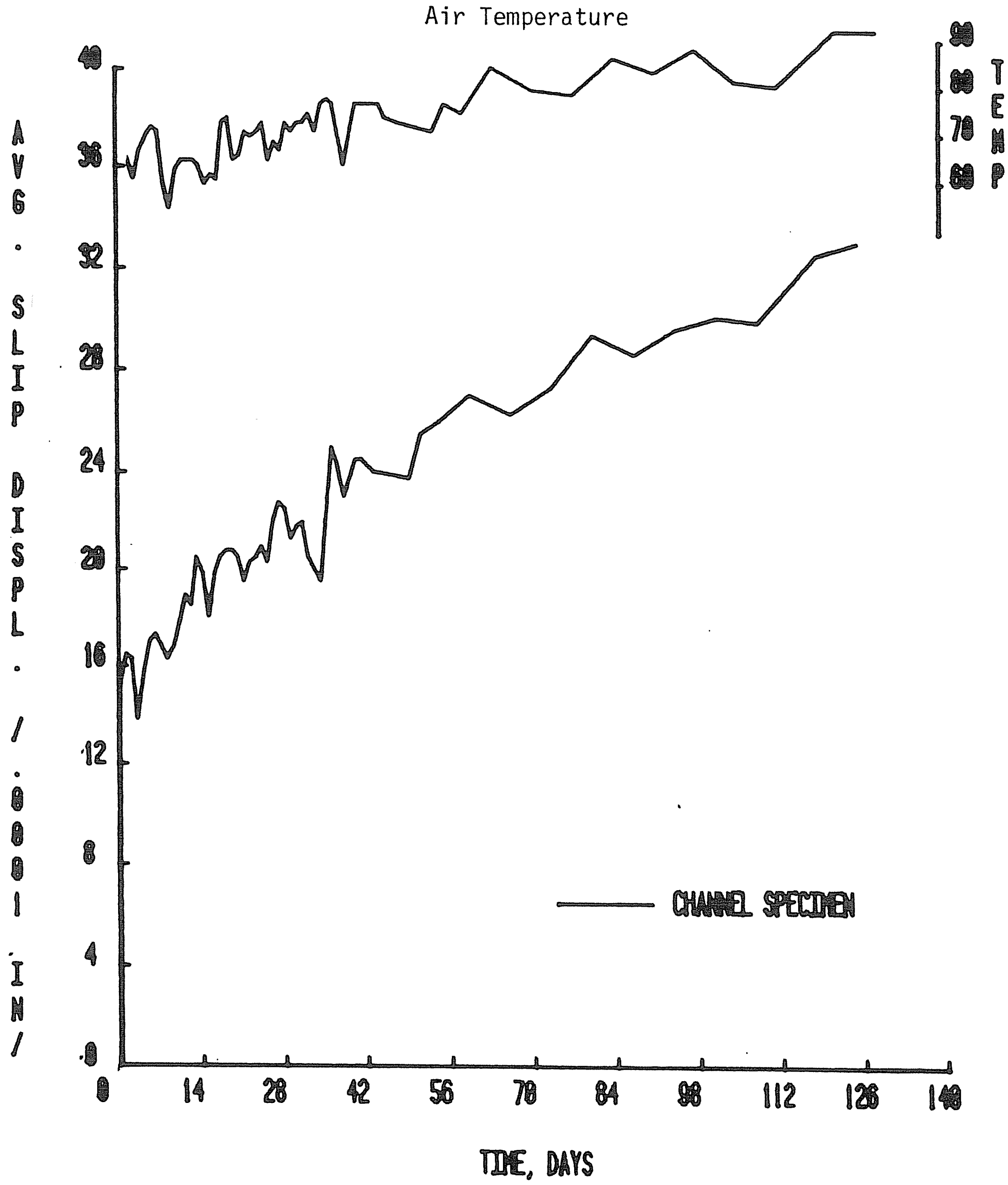


Figure 3.11 Average Shear Slip Displacement in Specimens with Channel Connectors

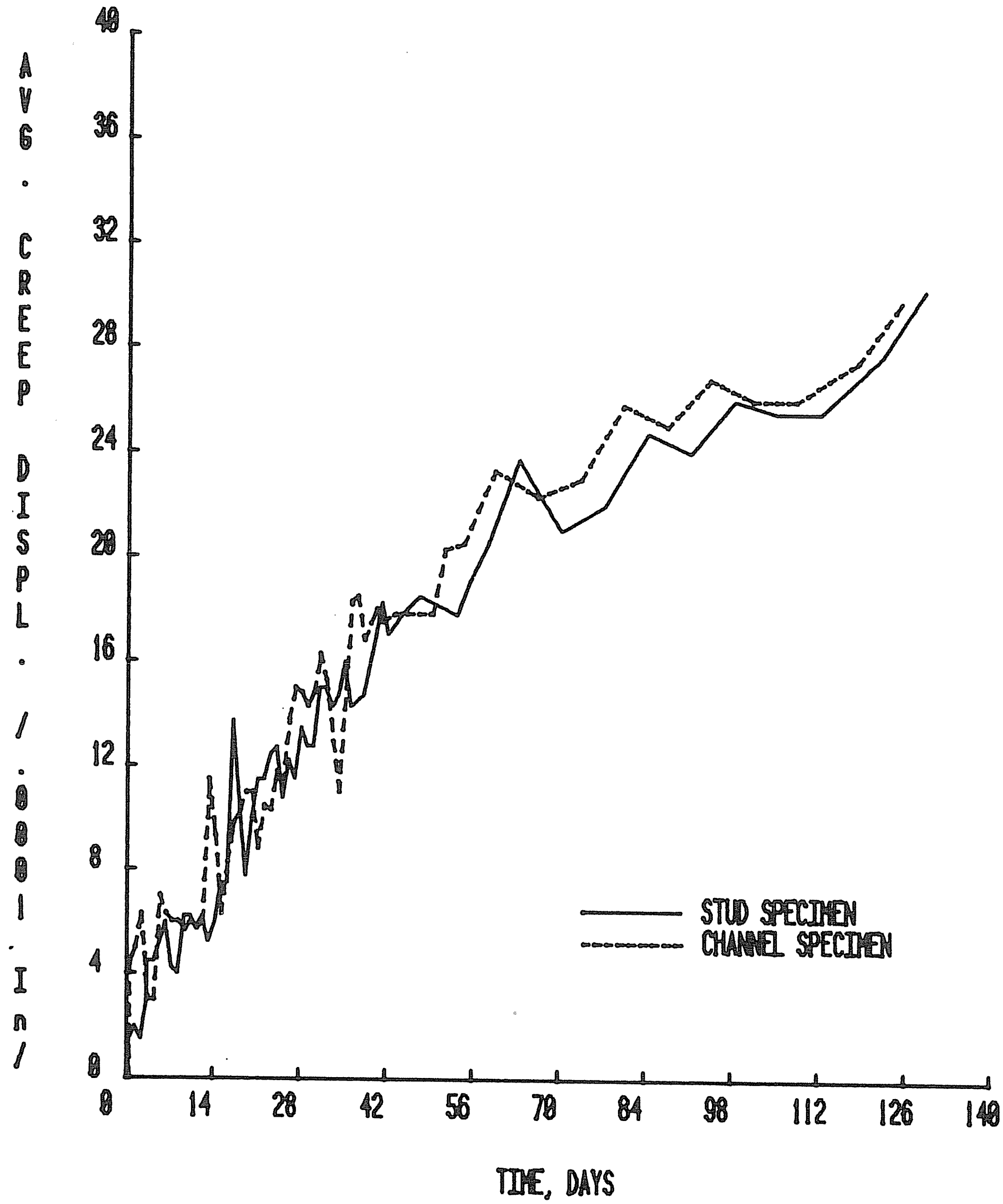


Figure 3.12 Comparison of Average Creep Values at Shear Connectors

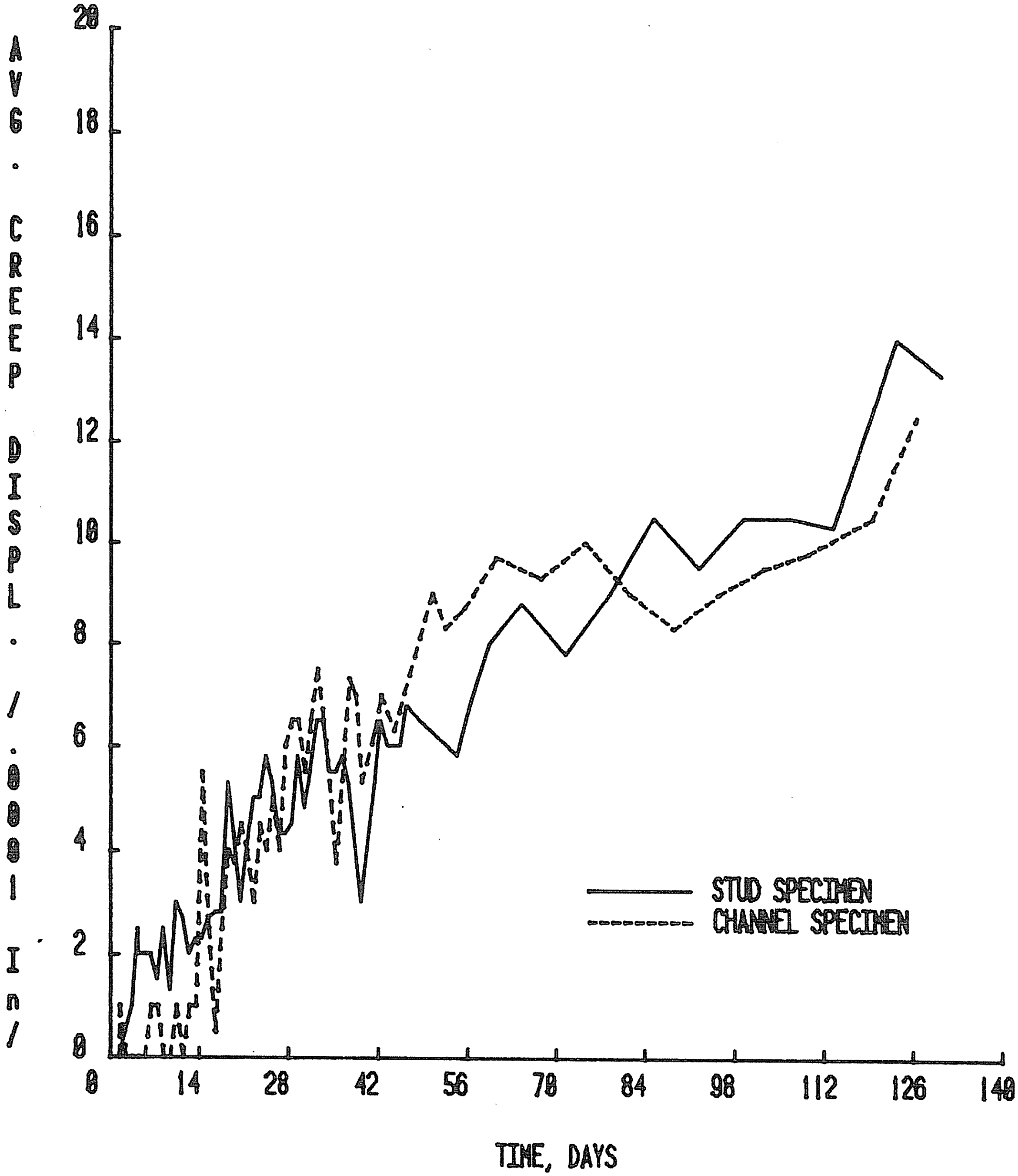


Figure 3.13 Comparison of Average Creep Values Away from Shear Connectors

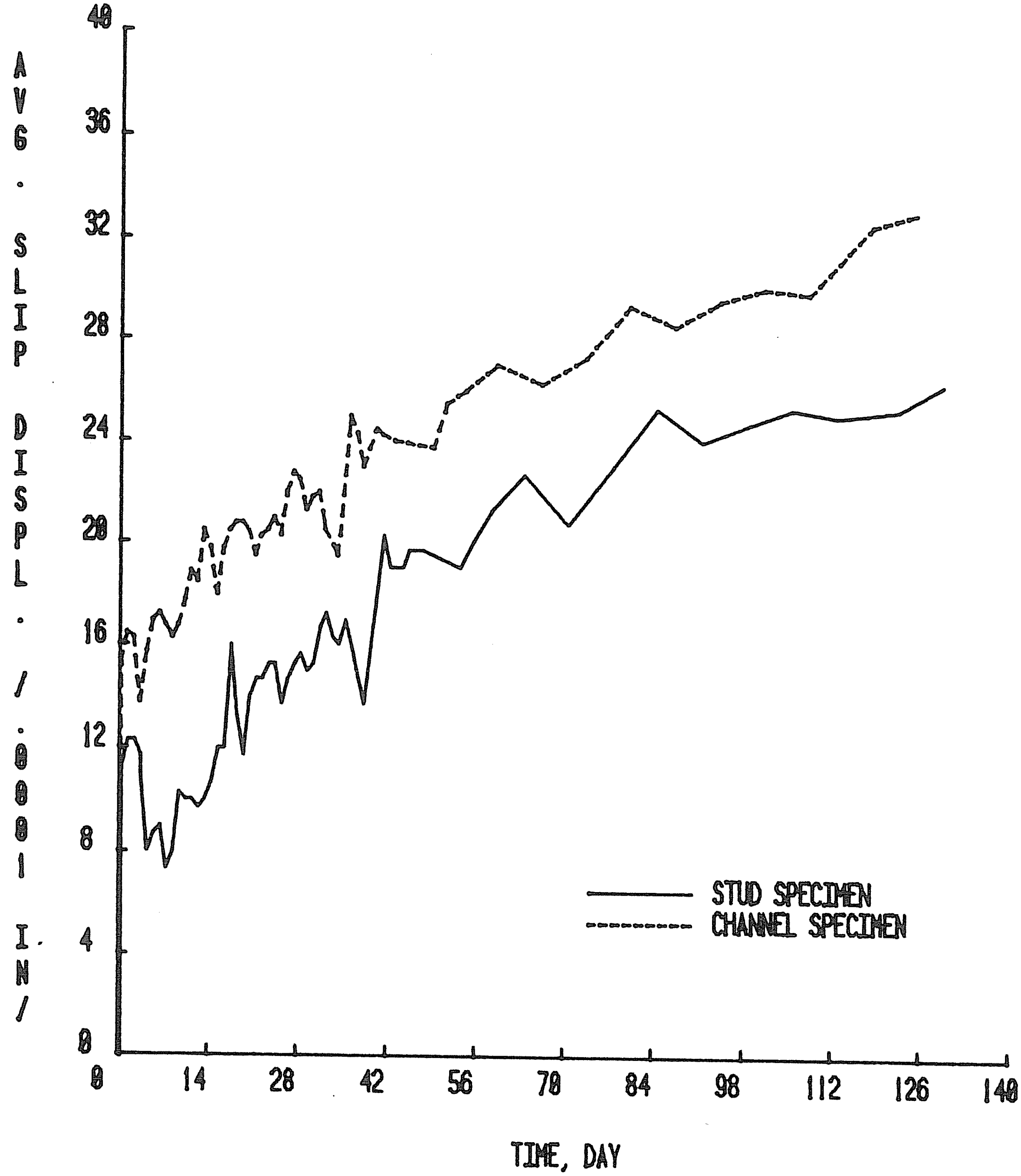


Figure 3.14 Comparison of Average Slip Values of Shear Connectors

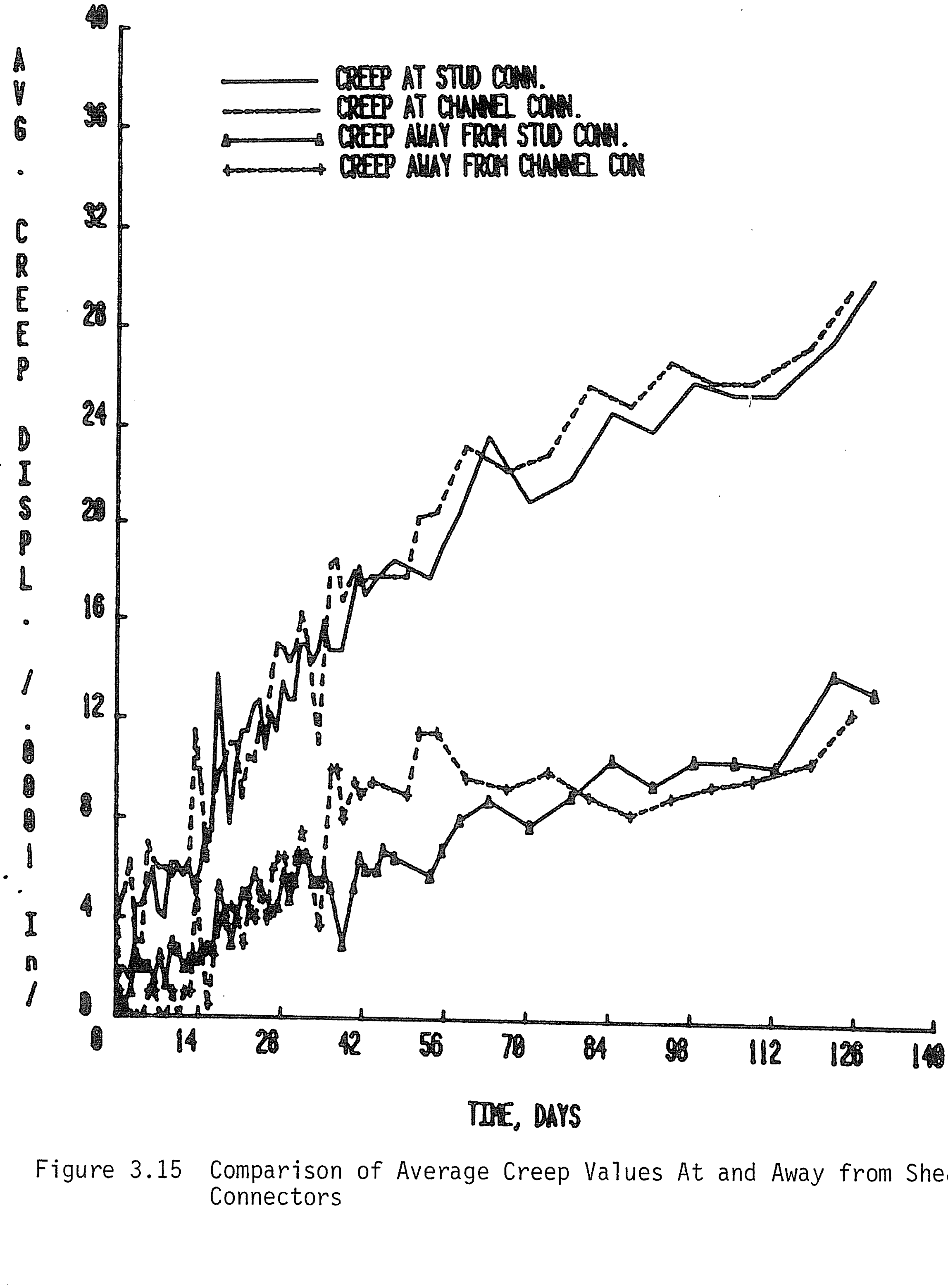


Figure 3.15 Comparison of Average Creep Values At and Away from Shear Connectors

CHAPTER IV

CONCLUSIONS

Bridge Unit. At the end of approximately one year of observation under sustained loading, strain values on the concrete slab of the bridge unit did not asymptotically reach a maximum value. However, stresses, calculated assuming elastic material behavior from strain measurements, on the steel beam flanges did reach asymptotic values. Similarly, the stresses (strains) on the longitudinal reinforcing steel in the concrete slab approached constant values.

The net vertical deflection of the bridge unit increased downward 0.77 in. between the time of application of the additional sustained loading (36 psf) to the end of the observation period. Further, the vertical deflection of the unit varies considerably with change in temperature.

Shear Creep Tests. Although no firm conclusions can be drawn from the results of shear creep tests at this stage of the investigation, the following observations can be made. The creep displacements near the shear connectors for both types of specimens is considerably more than the creep displacements away from the shear connectors. The creep and slip displacements for both types of specimens appear to be sensitive to changes in temperature. Comparison of stud connector results to channel connector results shows no distinct difference in the amount of creep displacement, however, the slip displacements for the channel connector specimens are larger than the slip displacements for the stud connector specimens.

REFERENCES

1. Clottey, C., and Thomas M. Murray, "Experimental Investigation of a Prestressed Steel Beam-Concrete Slab Bridge Unit", Research Report submitted to Oklahoma Department of Transportation, Research Division, July, 1982, 29 pages.