SELECTED PRECAST CONNECTIONS: LOW-CYCLE BEHAVIOR AND STRENGTH

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SYNOPSIS

This paper presents some of the test results related to common precast member connections. The testing program was conducted on full-size panels at Stanley Structures (Denver plant) in 1977.

Due to space limitations the description herebelow will be limited to eight connections. Testing consisted of static, monotonic, or cyclic loading to failure. The number of cycles was limited to three and the duration of each cycle was approximately eight to ten minutes.

The results are summarized in tabular form and provide information on the maximum force attained, secant stiffness and material strengths. No attempt has been made here to derive a general analytical model, but comments are included to allow a better understanding of connection behavior and strength. It is found that connection capacities were higher than the values obtained from current conservative methods since plate bearing (when available) and mesh contribution are usually neglected.

RESUME

Cet article contient les résultats d'essais sur des assemblages pour pièces préfabriquées en béton. Les essais ont porté sur des panneaux en vraie grandeur et, dans cet article, on s'est limité à la description du comportement de huit assemblages qui furent soumis à des chargements statiques et cycliques jusqu'à la rupture. Les spécimens furent soumis à trois cycles de chargement, d'une durée de huit à dix minutes chacun.

Les résultats sont résumés sous forme de tableaux où l'on trouve la charge maximum qui a été atteinte, la rigidité sécante et la résistance des divers matériaux. On n'a pas tenté d'expliquer les résultats à l'aide d'un modèle mais on a inclus des commentaires permettant de mieux comprendre le comportement des assemblages. On a constaté que les assemblages étaient plus résistants que prévu par les méthodes de calcul utilisées couramment en pratique parce que, dans ces méthodes, on néglige la contribution positive de certains éléments comme le grillage d'armature.

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INTRODUCTION

The precast-prestressed industry has experienced a phenomenal growth in the past few years. Parallel to this volume growth a refinement in the stress and strength analysis of monolithic structures was pursued and the pertinent state-of-the-art reached a satisfactory degree of precision.

Precast panel type structures are desirable from an economic point of view; however, the presence of non-grouted vertical and horizontal joints was always a reason for concern in lateral load situations, either in roof/floor diaphragms or in shear walls. Finding the most effective and reliable method to connect prestressed or precast members is one of the most important problems facing the industry today. This problem is especially felt in medium or highrise buildings designed for lateral loads.

Since the floor and wall assemblies are expected to act as diaphragms or shear walls in lateral load resistance, there is a need to know connection capacities, their failure modes and also their stiffness under service loads. However, available information on commonly used welded "dry connections" is lacking in many respects although proper modeling of shear walls and diaphragms requires this kind of information. A conservative capacity estimate can be made using References (4) and (5), but stiffnesses or failure modes are generally not available.

In this report some of the findings of the test program on connections conducted at Stanley Structures (Denver) are presented. They are part of the continuous effort of research and development within the company, (1) to (4). The 1977 research covered several major areas:

a. Precast panel connections
b. Hollow core connections
c. Double tee shear wall and roof diaphragm connections; miscellaneous details, such as corbels and dap ends.

TEST PROGRAM OBJECTIVES

The test program had several objectives:

a. To derive the failure capacities for the most common types of connections.
b. To measure the relative slip of the two sides of the connections under pure shear forces whenever feasible and, consequently, derive the connection stiffness under service loads. The non-linear force-slip relationship was derived up to the maximum capacity and in many instances included the descending branch.
c. To investigate the effect of a biaxial loading situation (shear and pullout) on the stiffness and capacity.
d. To observe the failure mode for the tested connections in order to optimize their design and, later, to derive an analytical expression for the ultimate load.
e. To recommend rational and improved design values for the connection capacities.

SCOPE

All tests were run on full-size panels and connections, each produced using regular materials and production methods. The tests described below are divided into three main categories:

Category I - Connections for double tees (2" flange thickness).
Category II - Connections for precast wall panels, 6" thick.
Category III - Miscellaneous details, such as corbels and dap ends.

Loading was statically applied. In the majority of cases the load was applied monotonically to failure in several steps. In some cases, the load was cycled three times between $\pm P_o$, where P_o is a value smaller than the actual maximum load, and then the connection brought to failure.

In the following pages the words "connection" and "plate" will often be interchanged.

Plates description is detailed in Table 1 and Figures 1 to 5. The approximate average for concrete strengths and rebar yield strengths were 4,900 and 56,000 psi, respectively.

CATEGORY I: DOUBLE TEE CONNECTIONS

A plan view of the test set-up is shown in Figure 6. The precast/prestressed panels were two 8'-0" wide by 21'-0" long double tees set side by side. The section depth was 1'-6" and the flange thickness 2".

In this set-up the shearing force V applied to the tested connection depends on the 1" pins location. In our case, V = 0.516 P, where P is the total jacking force applied to the 1/2" diameter strands. By sliding the pins an amount of 4' -0" after each cycle, an equal and opposite force is applied to the tested connection.

To apply forces normal to the flange plane a steel frame was positioned over the flanges and a small centerhole ram was used to apply a pullout force using a 1/2" strand.

Test Results

Force versus displacement relationships for the various connections are displayed in Figures 7 to 10. Connection D-34 is being used in vertical joints while D-36 and D-40 are generally used in horizontal members connections.

Connection D-34 exhibited a brittle failure at 11.9 kips. This was due to the lack of "truss action" by the rebars. Although the failure load was about double the previous recommended ultimate strength, it was decided to change the design so that the bars will be placed at an angle of 60° with the tee edge as detailed.

Connections D-36 and D-40 exhibited good ductile behavior and stiffness. It is worthy of note that the application of 1.0 kip pullout force normal to the panel caused the D-40 to lose about a third of its ultimate capacity. This latter type of pullout forces whose magnitude is not well known are generated during elevation adjustment of double tees at erection time. Consequently it is desirable to assume a conservative value for the rated capacity.

CATEGORY II: PRECAST WALL CONNECTIONS

The test set-up was similar to Figure 6 except for an additional feature which allowed the application of an in-plane tensile force simultaneously with or independently from the shear force. As usual the panels rested on 4" pipes and teflon pads,Fig. 6A.

Test Results

Figures 11 to 13 display the experimental force/displacement relationships for this category.

The P-8/P-3 combination is a standard connection used in horizontal (grouted) joints. The ultimate capacity was substantially larger than the previously rated capacity of 14.9K. In test "a", hairline cracks initiated around the studs in P-3 at a shear force of 35.7

kips. When the force reached 37 kips brittle weld failure occurred. In test "b", however, a large slip occurred at a level of 35K (due to compression bearing spalling) followed by a high residual force. In both tests no cracks or signs of distress showed up around P-8.

Other combinations P-8/P-3 were tested in a biaxial force situation of shear and in-plane pullout. Under pure pullout, hairline cracking started at 17.8K and maximum force reached was 23.4K while final failure occurred by stud fracture. Under the combined force situation a simultaneous shear of 29.5K and pullout of 20.4K were attained before stud fracture occurred. An elliptical shear/pullout inter-action curve was found to safely represent the behavior of this combination.

The P-9/P-9 combination is widely used in vertical joints. Several premature weld failures occurred when undersize field weld plates (2" x 3") were intentionally tried. 3" x 4" x 3/8" weld plates allowed the monotonic failure load to reach 32.7K. Under cyclic loading (P_O = 16.5K) no cracks appeared and maximum load attained was 25.2K when weld failure occurred. In the last test the weld plate was bent to accommodate a 1/2" elevation difference.

CATEGORY III: CORBELS AND DAP ENDS

The test set-ups for corbels (D-50) and dap ends (D-60) are shown in Figures 14 and 15, respectively.

D-50 is a standard corbel used for light to moderate loads on double tee wall panels with an eccentricity of 4 inches (±) from the interior face. D-60 is the lightest standard plate in the series of dap end reinforcements for double tee floor or roof members.

Test Results

Figure 16 shows the cracking pattern at the end of the test for D-50. The wall panel had the usual 1/2" Ø strand 2 inches below the top and 3/8" Ø strand 2 inches above the bottom of the leg. No leg mesh was provided but a standard deck mesh was used in the flanges. In the two tests run on identical corbels, minor cracking started below the flange at 60 kips (±). The tests were stopped at 80K after several 1/16" cracks showed up in the bracket and leg.

Figure 17 shows the cracking pattern for D-60 at ultimate capacity. Initial hairline cracking in each of the tested legs appeared at the re-entrant corners when load reached about 7.7K per leg. The crack formed an angle of 38° (± 2°) with the horizontal reference line. As load increased, cracks propogated until a second substantial crack formed 5 to 7 inches below the re-entrant corner (measured on the 30°

sloping edge). Just before failure both cracks widened to about 1/8" width when test was stopped at 20K per leg. The two cracks indicate two almost simultaneous modes of failure by rotation about either A or B in Figure 17. The appearance of the second crack, away from the re-entrant corner, led us to modify the design so that the middle bar in the plate has enough anchorage beyond that possible crack. The analysis for dap plates has been developed and is explained in detail in Ref. (6). The reinforcement arrangement is very practical from production point of view since the plate comes as one unit (no loose bars). Thousands of these plates have been used in the past two years and showed excellent behavior under different service loads and environments.

CONCLUSIONS

a) When the force applied to a plate was simple shear, ultimate capacities were higher than the ones listed in (4) by up to 100% in some cases. The extra capacity was on the high side whenever the connection included a 3/8" thick plate which increased the bearing portion of the resisting capacity.

b) Ductility of the standard plates tested was good to excellent except for the D-34 with straight rebars, although its actual capacity was much superior to the recommended one in (4). Ductility is enhanced by placing the rebars at 45° to 60° angle with the shear force direction.

c) Rebars' anchorage lengths were sufficient to develop ultimate capacities.

d) Plates subjected to cycled loads showed no major stiffness deterioration after three cycles.

e) Pullout forces <u>normal</u> to panel surface substantially decrease ultimate capacity in shear. Moderate <u>in-plane</u> pullout forces acting simultaneously with shear do not noticeably affect ultimate shear capacity of precast connections, although they reduce the connection stiffness.

f) Size of field weld plates is critical in wall panel connections if premature weld failure is to be avoided.

g) The tests on two D-50 corbels showed that the current design is safe and balanced in the sense the bracket and wall panel start showing signs of distress at about the same load levels.

h) Tests on two legs of dap end D-60 confirmed the safety and correctness of the analysis method outlined in Ref. (6). Due to space limitations this is not repeated here.

Table 2 lists recommended "rated" values for plate capacities assuming $\emptyset$ = 1.0 for the capacity reduction factor, f'_c = 5,000 psi and fy = 48,000 psi. It is worthy of note that in most precast products the latter two strength values are exceeded.

In general, we can conclude that the above (or other) connections if properly detailed would allow for significant inelastic deformation. They could safely be used in Seismic Zones 1, 2 and possibly 3 of the Uniform Building Code. The data base was large enough to reach the conclusion that rebars are a significant factor in obtaining a high ductility level. When joint forces are evaluated accurately, a $\emptyset$ = 0.85 is recommended (except for D-50 and D-60). Further dynamic testing and mathematical modelling along the lines of Reference (9), and using the connections' characteristic behavior is certainly desirable. This is especially true in Seismic Zones 3 and 4.

ACKNOWLEDGMENT

The work described in this paper forms a part of a comprehensive investigation by Stanley Structures, Inc. (Denver) into the behavior of standard connections of precast prestressed structures. The writer wishes to extend his appreciation to F. J. Jacques and Paul Mack for valuable advice, and to Les Kempers whose contribution in conducting the tests was significant.

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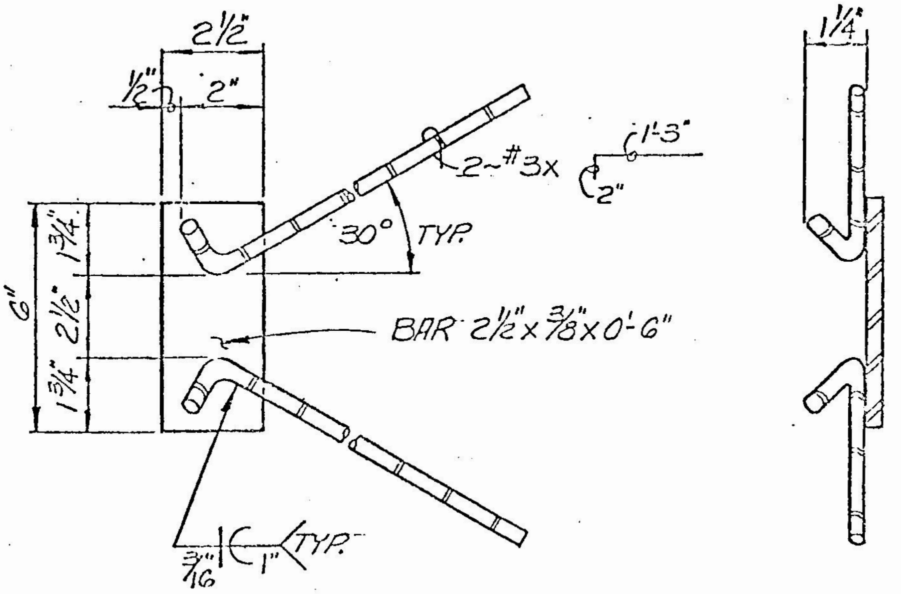
Table 1: Connections Description

Plate No.	3/8" Steel Plate Size	Rebars	Headed Studs
D-34	2-1/2" x 6"	(2) #3	None
D-36	4" x 6"	(2) #4	None
D-40 Tie	None	#4	None
D-50 Corbel	2-#5 top bars, 2-#3 Stir; Fig. 4		None
D-60	6"x7" Vertical 6"x5" Horizontal	(1) # 5 top, (1) # 5 middle, (1) # 4 inclined	None
P-3	5" x 10"	None	(4) 1/2"x8"
P-8	See Fig. 5	(2) #5	None
P-9	See Fig. 3	#5	None

Table 2: Capacities Summary

Plate No.	Force Type	Maximum Test Force in KIPS	Recommended Rated Capacity (Ø = 1), KIPS
D-34	Shear	11.9	7.5
D-36	Shear	16.1	14.0
D-40	Shear	12.3	8.0, incl. pull ⊥ to panel
D-50	Shear	80.0	67.0 at 4" eccentricity (±)
D-60	Vert. reaction	20.0	10.6 (Recommend. Ø = 3/4)
P-8/P-3	Shear	36.0	29.0
P-8/P-3	Shear and in-plane pullout	29.5 & 20.4	20 (shear) + 16 (pull); or 12 (shear) + 20 (pull)
P-9/P-9	Shear	32.7	24.0 with 3"x4"x3/8" weld plate



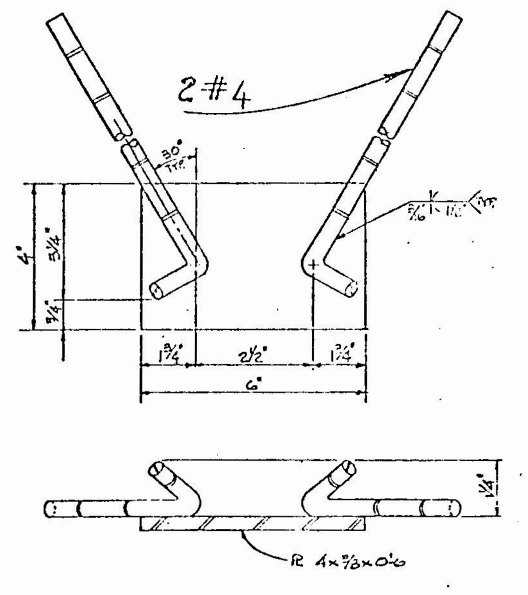


FIGURE 1.

D-34 (TOP),

AND

D-36 (RIGHT)

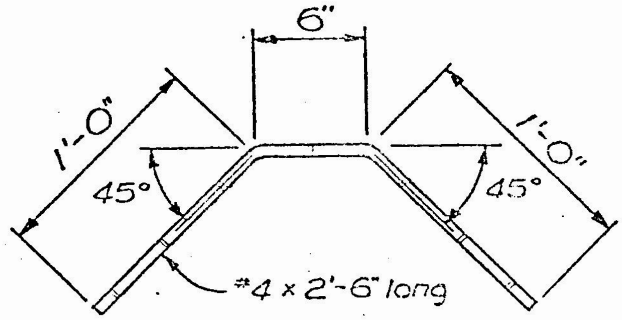


FIGURE 2. D-40 TIE

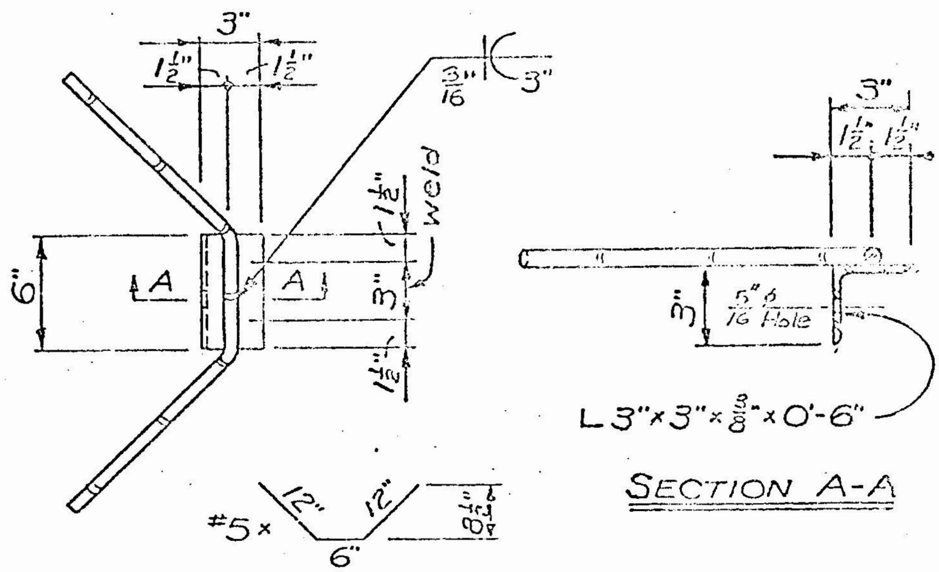


FIGURE 3. P-9

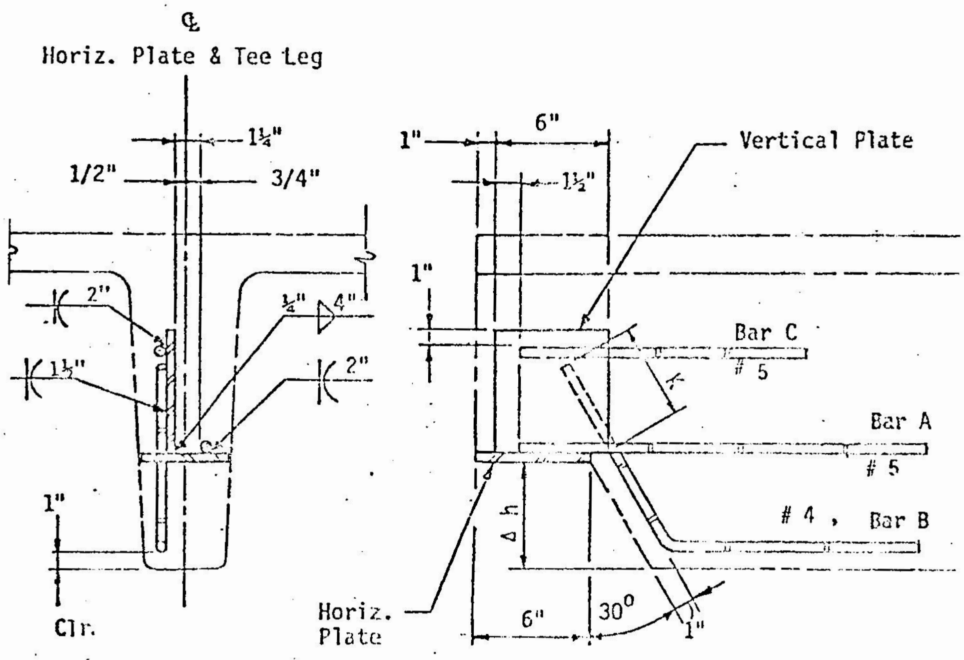
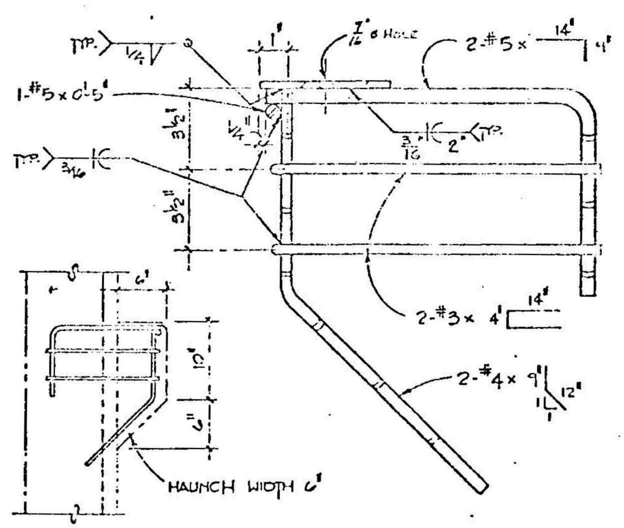


FIGURE 4.

D-60 (TOP),

AND

D-50 (RIGHT)



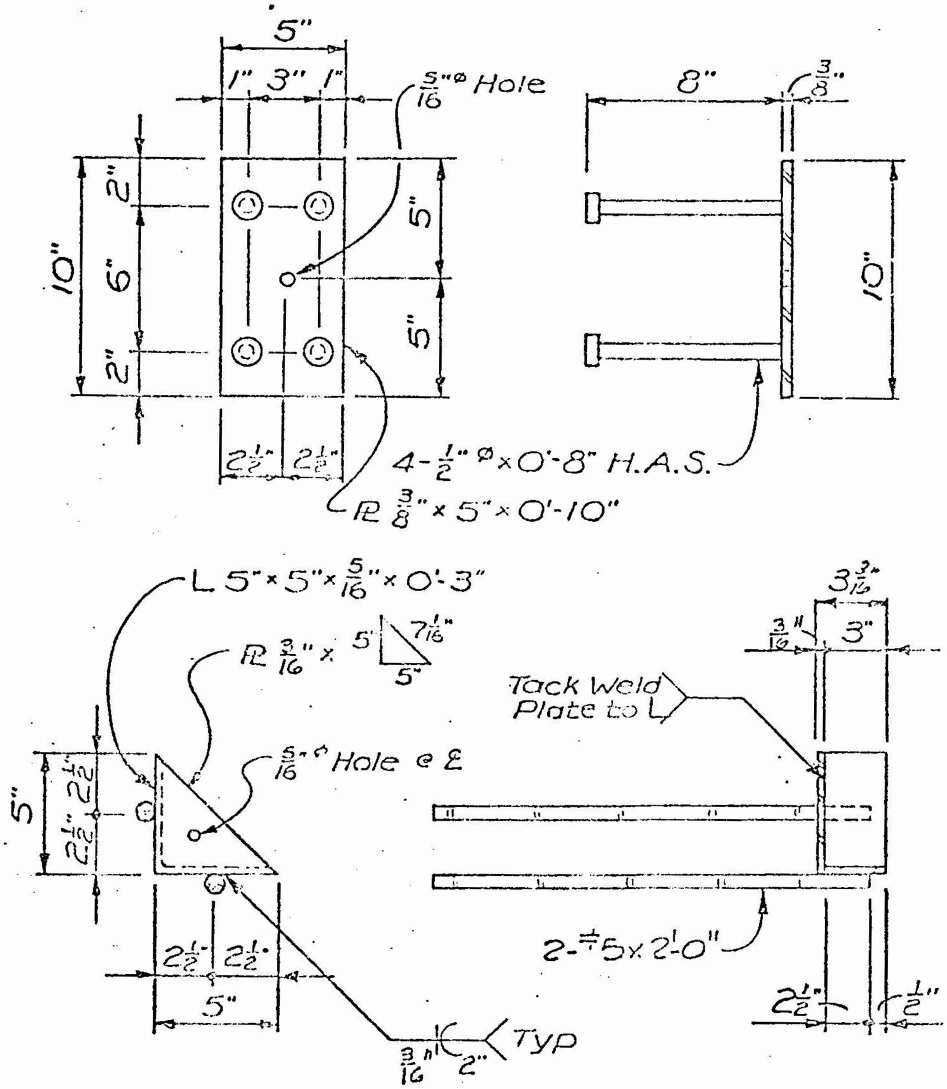


FIGURE 5. P-3 (TOP) AND P-8 (BOTTOM)

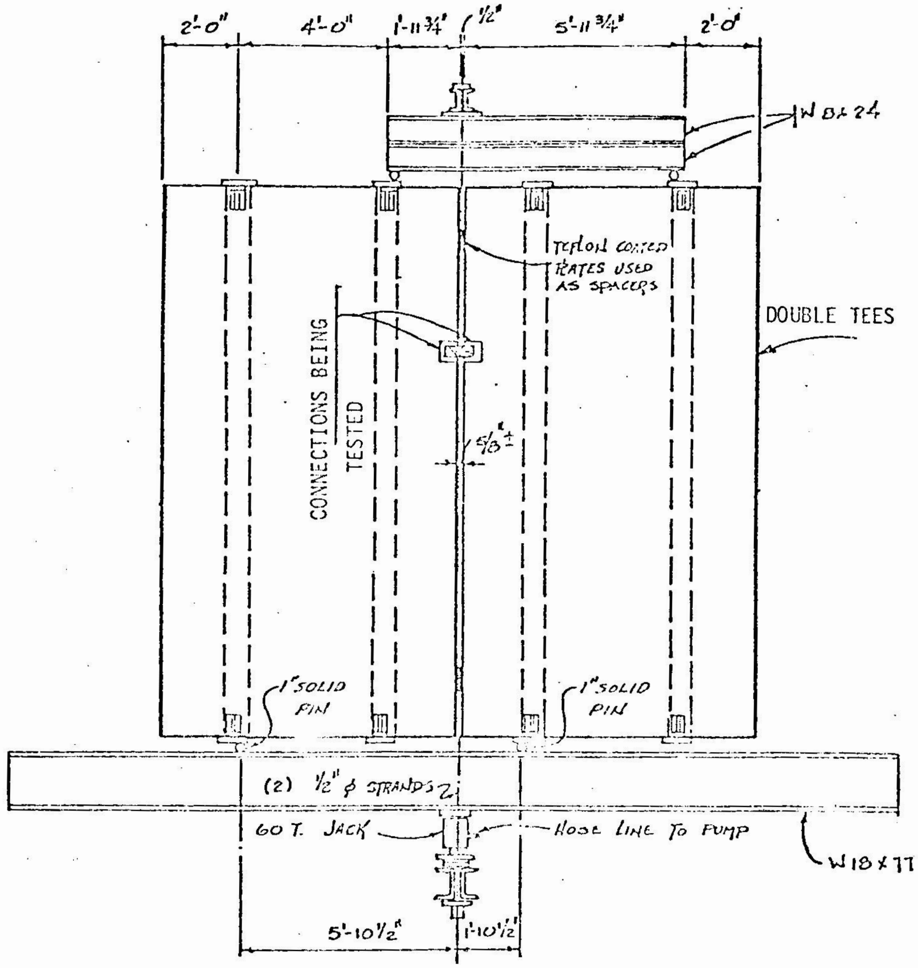


FIGURE 6. TEST SET-UP FOR DOUBLE TEES

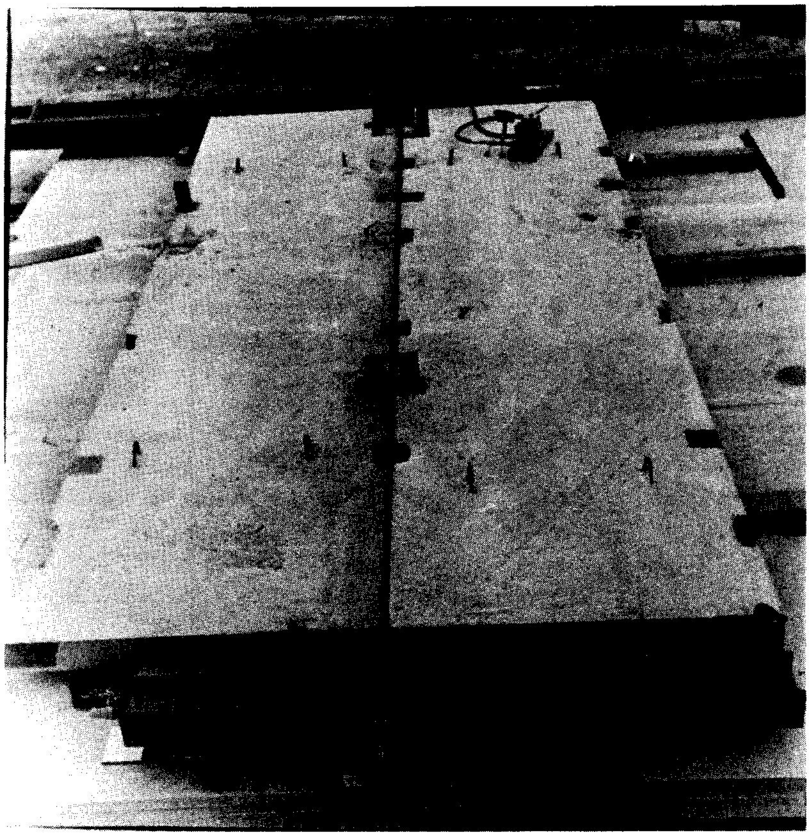


FIGURE 6A. TEST SET-UP FOR PRECAST PANELS

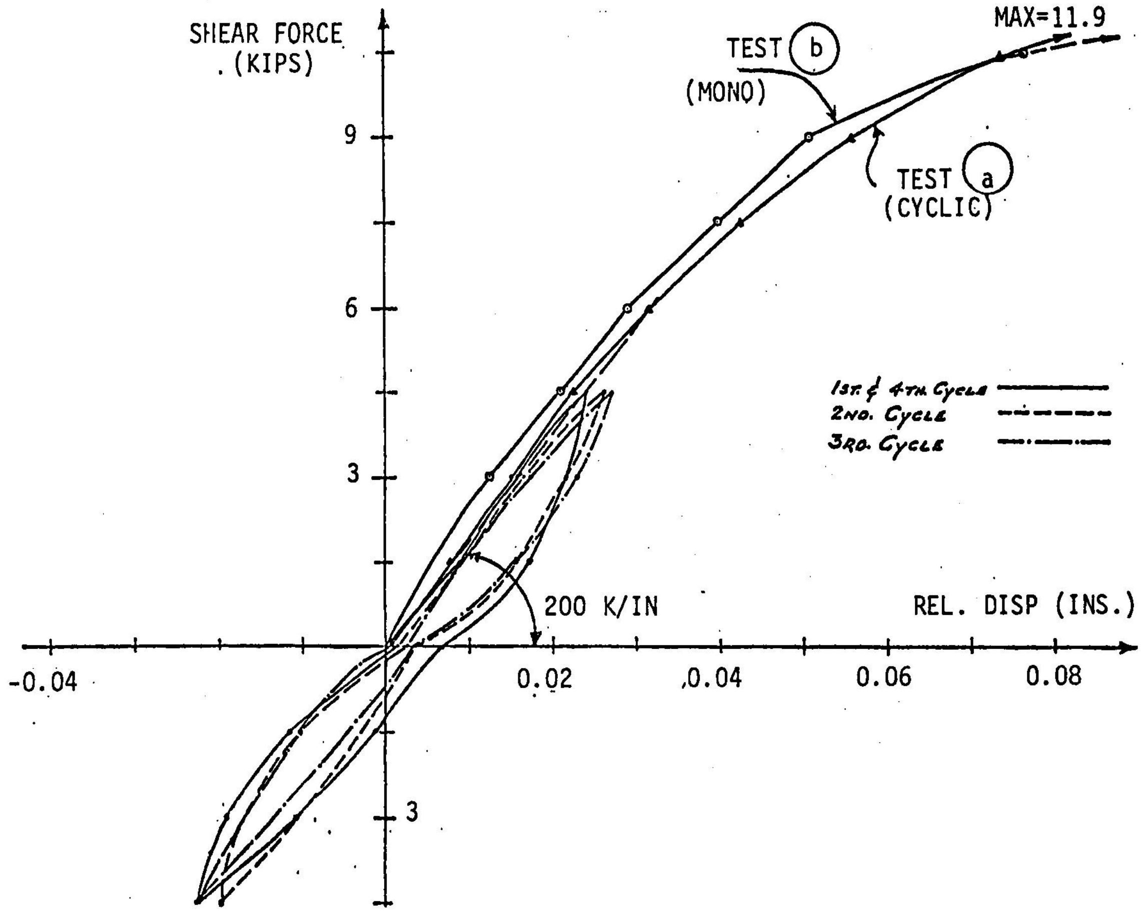


FIGURE 7 . SHEAR FORCE VS. RELATIVE SLIP FOR D-34/D-34

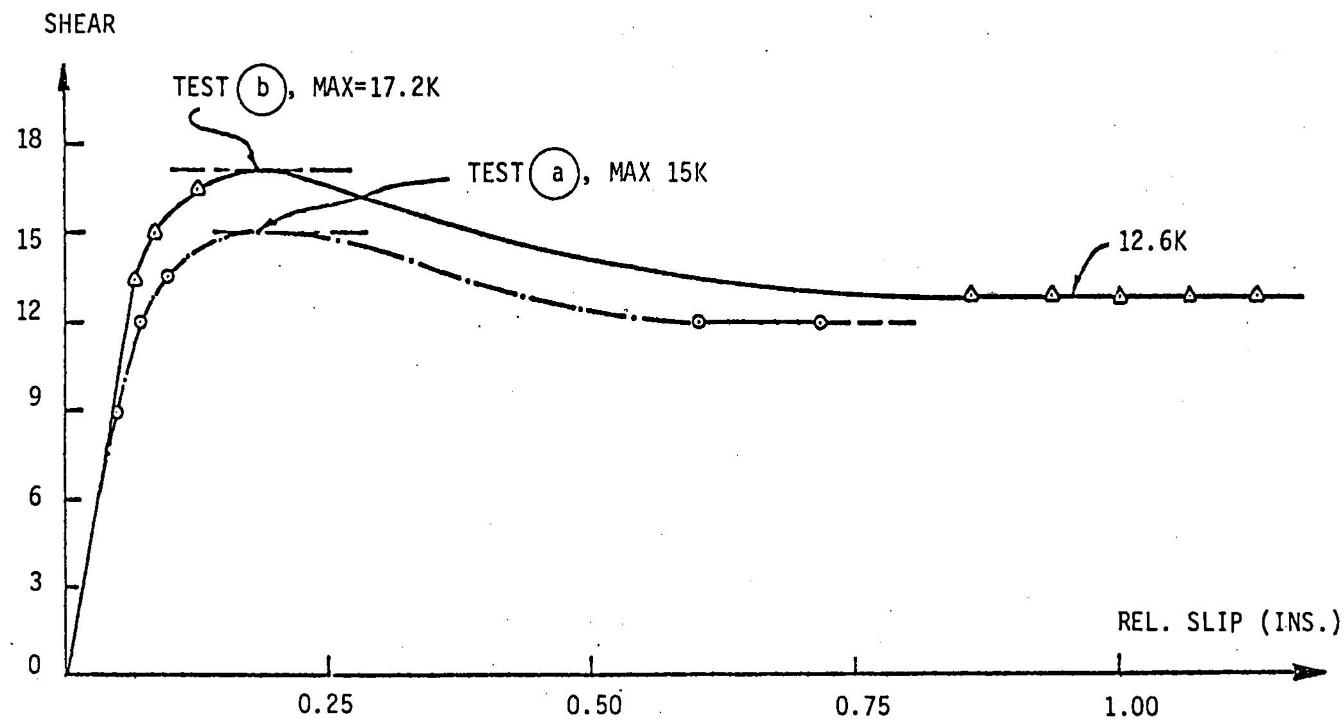


FIGURE 8 . SHEAR FORCE VS. RELATIVE SLIP FOR D-36/D-36
(INCLUDING DESCENDING BRANCH)

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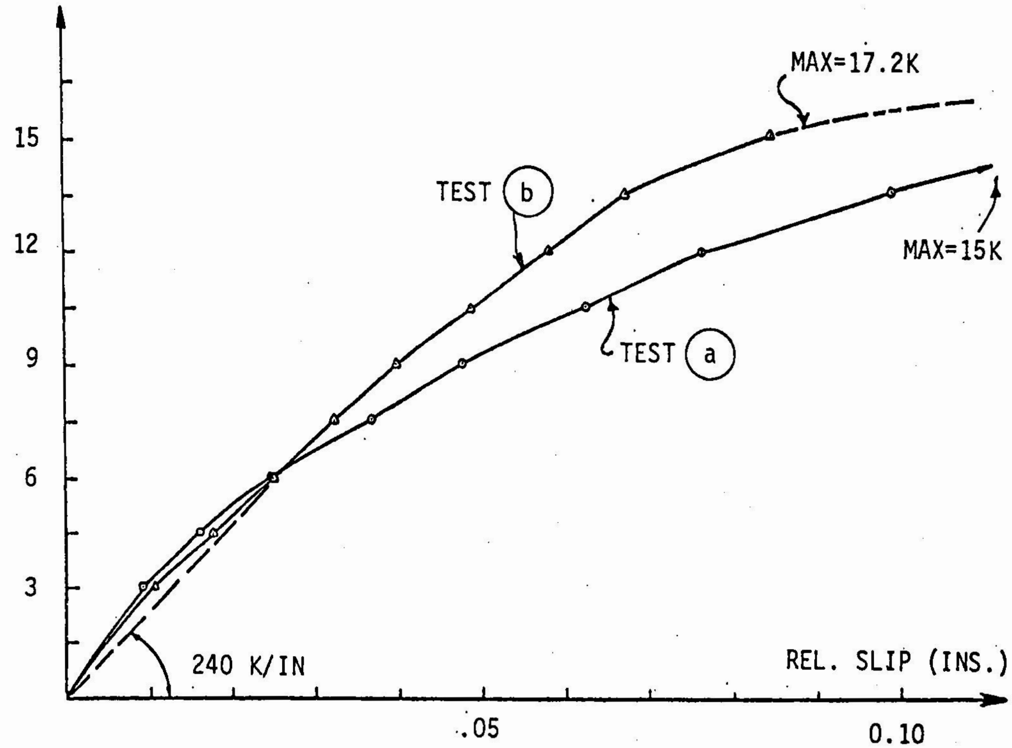

FIGURE 9. SHEAR FORCE VS. RELATIVE SLIP FOR D-36/D-36

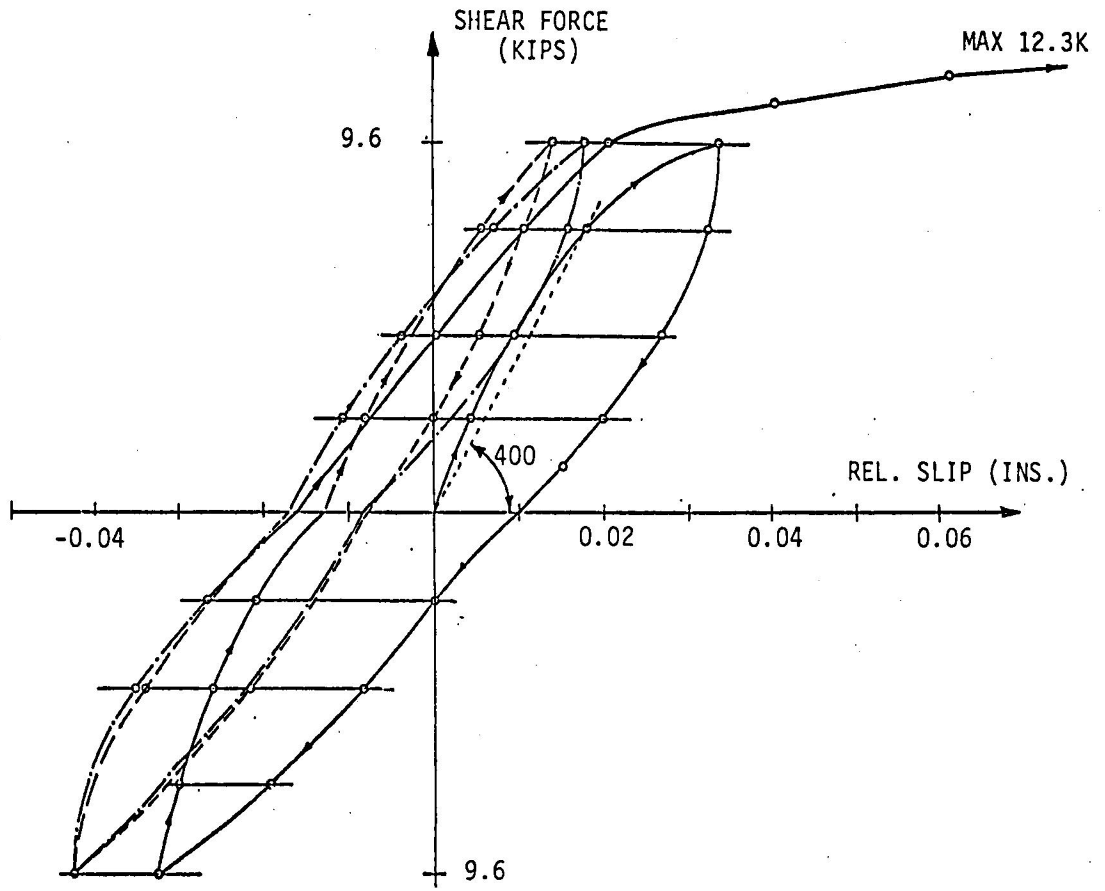

FIGURE 10. CYCLIC SHEAR FORCE VS. RELATIVE SLIP FOR D-40/D-40 TIES

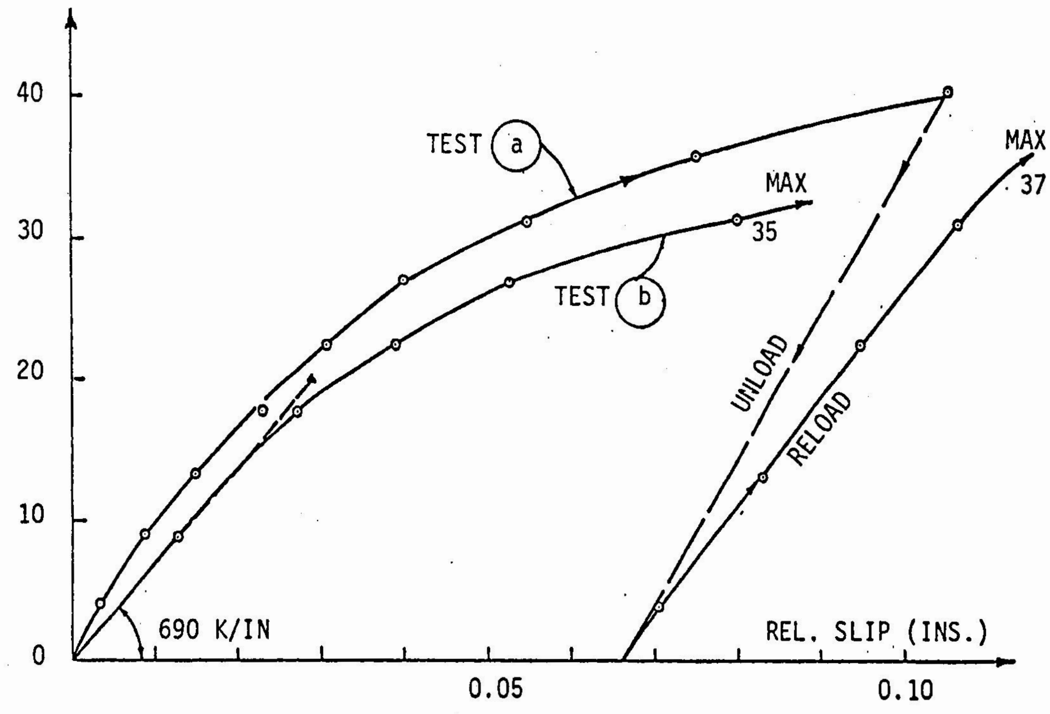

FIGURE 11. SHEAR FORCE VS. RELATIVE SLIP FOR P-8/P-3 COMBINATION

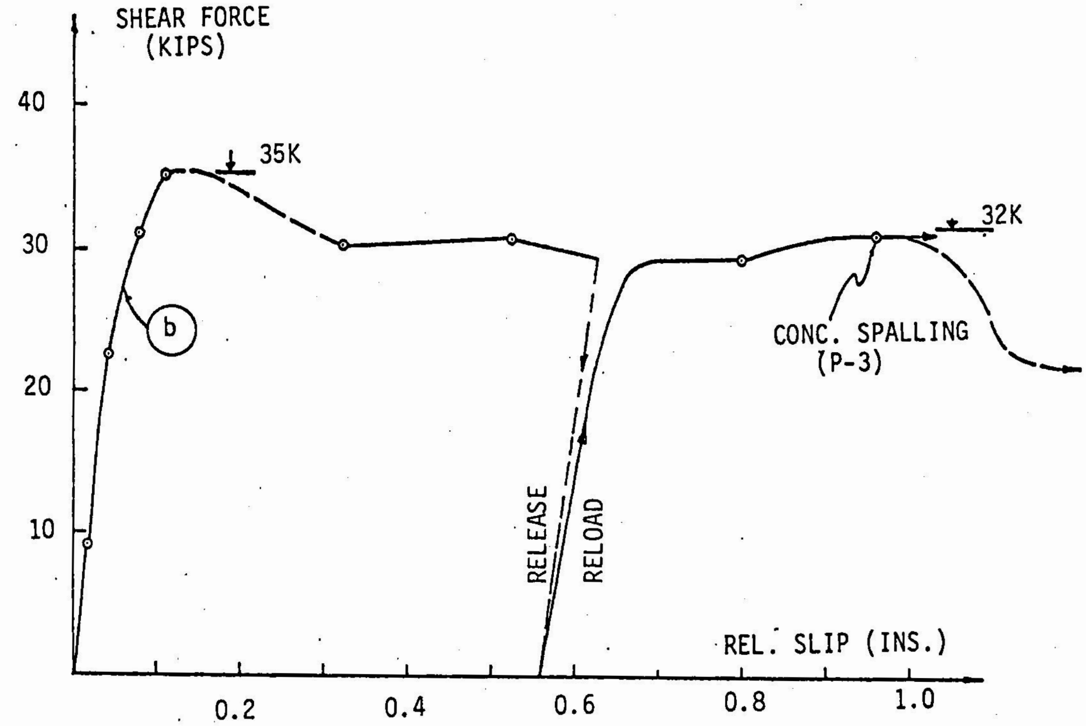

FIGURE 12. SHEAR FORCE VS. RELATIVE SLIP FOR P-8/P-3 COMBINATION
(INCLUDING DESCENDING BRANCH)

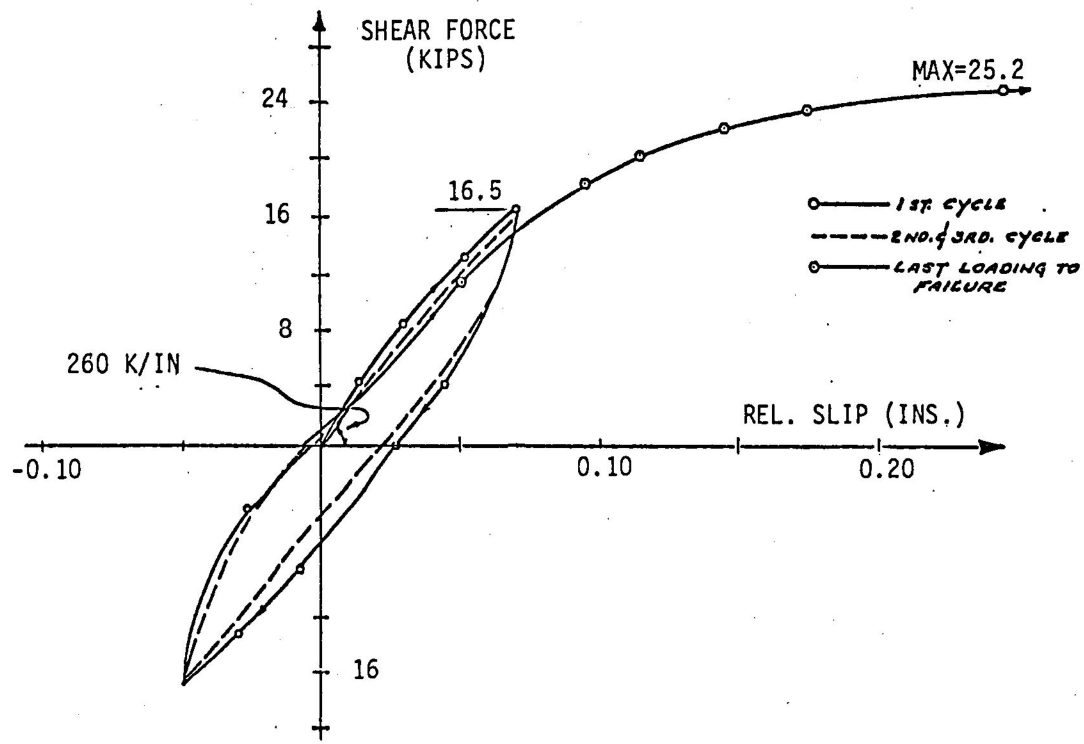

FIGURE 13. CYCLIC SHEAR FORCE VS. RELATIVE SLIP FOR P-9/P-9
(CONNECTIONS PREVIOUSLY LOADED IN SHEAR TO 15.0 K)

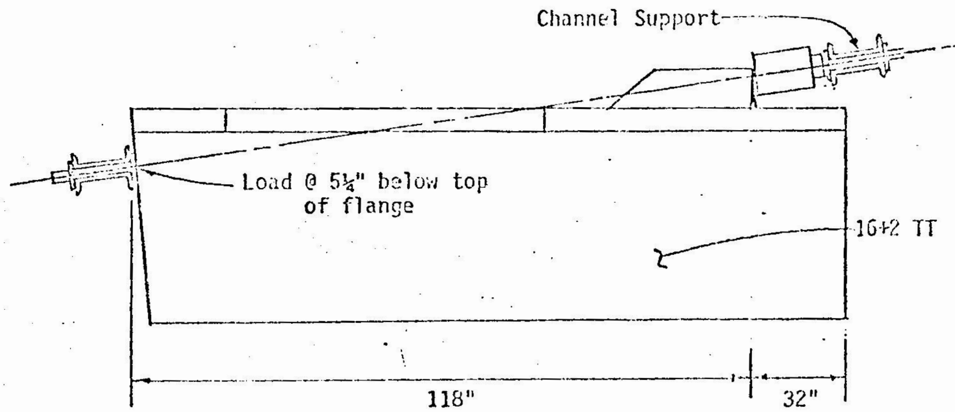

FIGURE 14. TEST SET-UP FOR CORBEL D-50

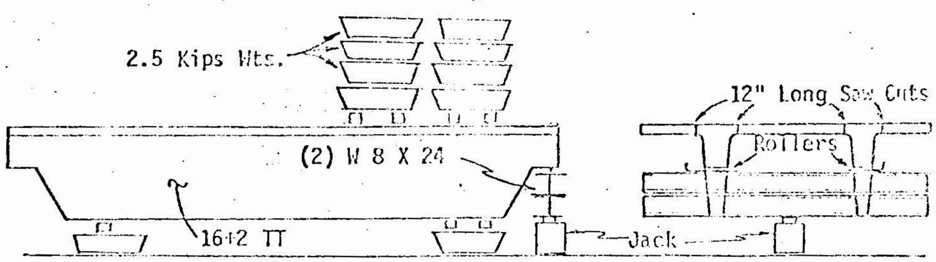

FIGURE 15. TEST SET-UP FOR DAP END (D-60)

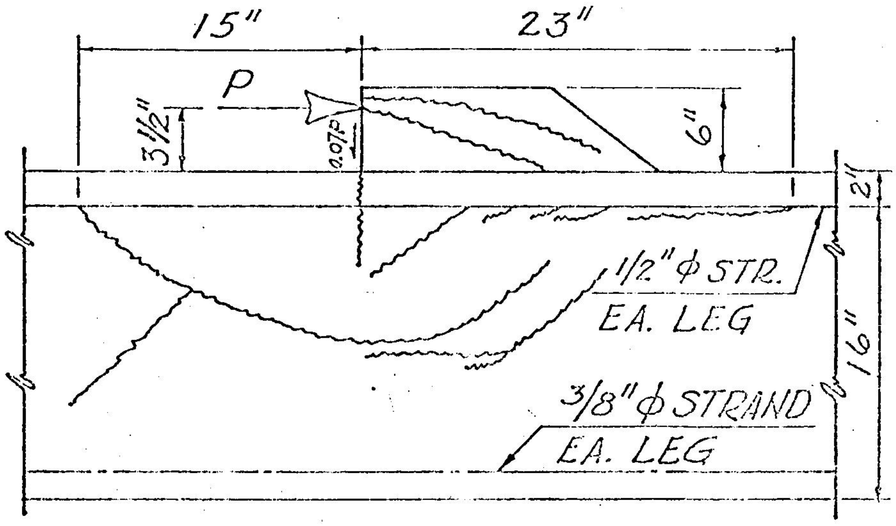

FIGURE 16. CRACKING PATTERN FOR CORBEL (D-50)

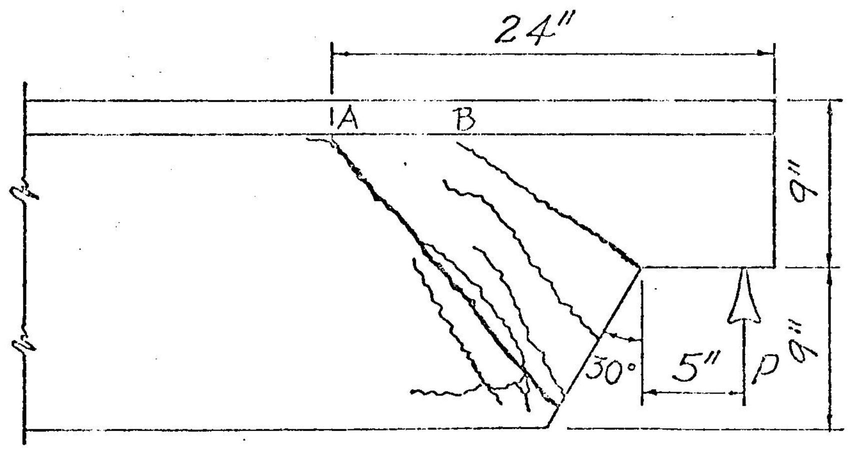

FIGURE 17. CRACKING PATTERN FOR DAP END (D-60)