Tests of 24 in. Square Prestressed Piles Spliced With ABB Splice Units

William L. Gamble
Professor of Civil Engineering
University of Illinois at Urbana-Champaign
Urbana, Illinois

Robert N. Bruce, Jr.
Catherine and Henry Boh Professor of Civil Engineering
Tulane University
New Orleans, Louisiana

Static load tests were conducted on nine 24 x 24 in. (610 x 610 mm) pretensioned concrete piles which were made from two segments joined by ABB pile splices. Three spliced piles were tested in tension, three in bending, and three in compression. The purpose of the investigation was to confirm that large size splices can be designed efficiently and to show that the splice can be applied confidently with a 24 in. (610 mm) square pretensioned pile.

Sections of 24 in. (610 mm) square pretensioned concrete piles were fabricated and spliced together for testing. The splice used was the 24 in. (610 mm) ABB mechanical steel splice manufactured by the A-Joint Corporation.

Tests were performed to determine the behavior of the spliced piles under conditions of axial tension, axial compression, and bending. Similar splices have been used in Europe, but only with much smaller piles. The tests had the purposes of confirming that the larger splice could be designed using the same methods as for the smaller ones, and of producing data to specifically demonstrate the capability of splicing the 24 in. (610 mm) square pretensioned pile.

Static load tests were conducted. It is recognized that the pile handling and driving operations may impose the most severe stress conditions the pile will experience, and that the in-service conditions will lead to various other combinations of moment and axial force. However, laboratory tests will give information about the behavior of the piles not generally available from driving or field tests since structural failures of the pile material (as opposed to the pile-soil system) cannot generally be produced under known, controllable loads under field conditions.

A total of nine tests were performed on spliced members, with three tension tests, three bending tests, and three compression tests. In many respects, these tests were a continuation of work reported in Refs. 1 and 2. Ref. 3 describes tests of a different splice, in a smaller pile, made by A-Joint Corporation.

DESCRIPTION OF SPLICE

General Description of Splice Components

The ABB pile splice uses steel components to transfer tensile forces from one pile section to the other. Compression stresses are transferred by direct contact of full section steel plates cast directly onto the ends of the precast sections. Fig. 1 illustrates the steel components that make up the splice. The steel plate caps, designated A in the drawing, are cast directly onto the pile ends. Welded to the cap plates are carefully oriented machined steel rectangular blocks (B) and dowels (C). Each dowel mates into a block and is secured by a shear pin (D) which is driven into the block through a transverse hole in the block and dowel. Reinforcing bars (E) are threaded into the rectangular blocks and round dowels, and the reinforcing bars transfer tensile stresses to the pile sections by bond.

The cap plate assemblies are supposed to be identical so that any pair of pile segments can be spliced. The air bleed holes on the cap collars must be placed on top during casting of the pile.

Fig. 2 shows how the components are positioned within the ABB splice. The components are labeled with the same letters as in Fig. 1. A lock washer (F) springs open as the shear pin (D) is driven in, and then springs back tight around the pin shoulder to prevent easy withdrawal of the pin. The splice shown in Figs. 1 and 2 has four spliced bars, while the splices used in this test series had eight spliced bars, as shown in Fig. 3. It will be seen that the splice plate is actually 23⅝ in. (600 mm) square but is slightly tapered to facilitate removal from the forms.

The individual steel components for the test pile splices were fabricated in Sweden in order to expedite the test program, using three different grades of steel. The cap plates are ASTM A36 (f_y = 248 MPa) steel, with the 24 x 24 in. (610 x 610 mm) end pieces 5/16 in. (7.9 mm) thick and the collar pieces 1/8 in. (3.2 mm) thick. The A-Joint Corporation plans to produce the steel components in the United States using domestic steels.

The reinforcing bars are ASTM A615 Grade 60 (f_y = 414 MPa) #10 (32.3 mm) bars. The ends of the deformed bars were turned to round and threaded with UNC (Coarse) threads, 1¼ in. (31.75 mm) diameter seven threads per in. (3.63 mm pitch). The threads were cold rolled rather than cut.

The reinforcing anchorage bars were fabricated in two lengths, 48 and 60 in. (1219 and 1524 mm), so all eight bars in a pile splice would not end at the same section. The anchorage bars were placed in the splice plate in a staggered configuration so that there was one 48 in. (1219 mm) and one 60 in. (1524 mm) bar at each corner of the pile. The differences in the sizes of the dowels and blocks resulted in a 14 in. (356 mm) stagger at the far ends of the reinforcing bars.

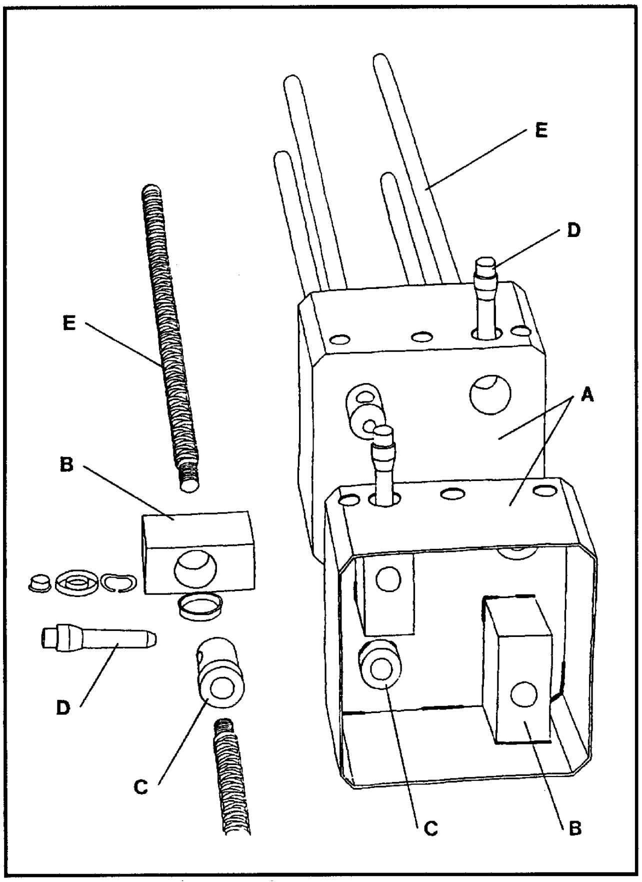


Fig. 1. Details of typical ABB splice.

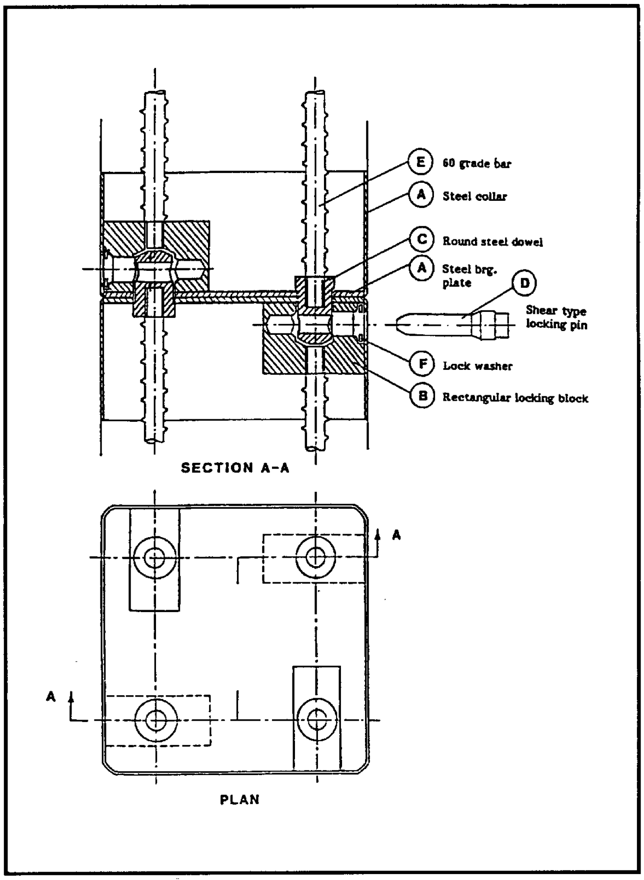
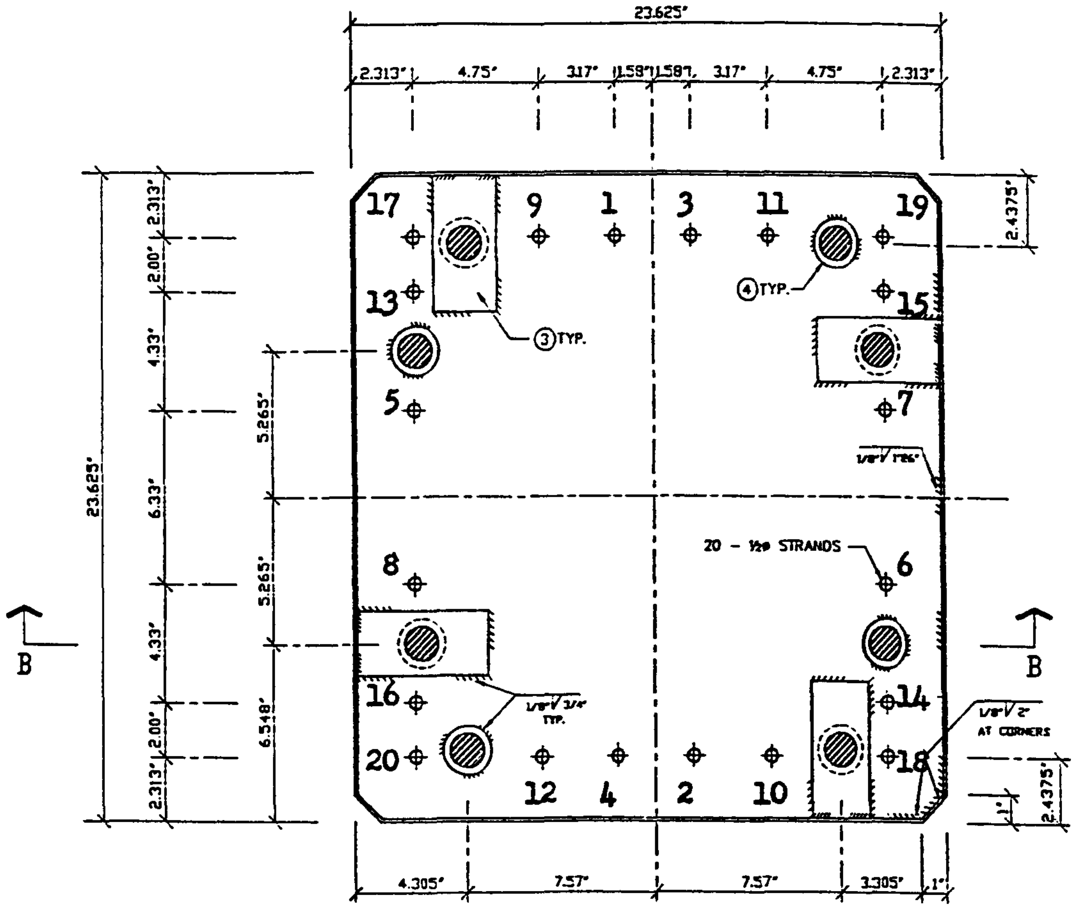


Fig. 2. Typical ABB splice as connected.



Increments of Load Application—Tension Test

Nominal stress in anchorage rebar, ksi	Load per strand, kips	Total load, kips
0	0	0
15	7.65	153
30	15.25	305
40	20.35	407
50	25.40	508
55	28.00	560
60	30.50	610

Fig. 3. Strand numerical identification and stressing sequence.

Problems With Splice Assembly

Two separate problems with the splices were encountered in the bending and compression tests.

The first problem encountered was relatively great difficulty in mating some of the pile sections together so that the locking pins could be driven in to lock the two sections together. It must be recognized that this operation was done with the piles in the horizontal position rather than with the segments vertical as would be the case during field driving operations, but there still would have been some problems with this set of pile segments.

These particular splice units were custom made in matched pairs, and they were shipped in assembled matched pairs. However, this matching was not consistently maintained during the casting and shipping process, and some of the specimens were assembled from mismatched sets of splice hardware. When a matched pair of splice-end components were used, the mating of the two segments was done with very little effort.

The splice components in this test series were prototypes, and production type jigging had not been prepared.

The second problem occurred in the prestressing plant. The threaded #10 (32.3 mm) reinforcing bars were screwed into the back sides of the dowels and blocks only after the splice units had been placed in the formwork and the strands strung through them. This was physically very difficult, and at least three of the bars had been screwed in only one to three turns, and this led to problems in the bending test specimens.

The insertion of the bars into the blocks was properly done in each case, and inspection was relatively simple in this case since the end of the bar can be seen in the hole in the block.

TEST SPECIMENS

General Description

Each of the three tension specimens (T-1, T-2, T-3) consisted of a 90 ft (27.43 m) pile made of two 45 ft (13.72 m) pile sections spliced together. Each of the bending specimens (B-1, B-2, and B-3) consisted of a 36 ft (10.97 m) pile made of two 18 ft (5.49 m) sections spliced together. Each of the compression specimens (C-1, C-2, and C-3) was 16 ft (4.88 m) long, with two 8 ft (2.44 m) sections spliced together.

The pile sections were nominally 24 in. (610 mm) square, with chamfered corners. They contained 20 - 0.5 in. (12.7 mm), Grade 270 (1860 MPa) low relaxation, pretensioned strands, spaced around the perimeter as shown in Fig. 3.

Fabrication of Test Specimens

The test specimens were cast at the prestressing plant of Gulf Coast Pre-Stress Co., Inc., in Pass Christian, Mississippi. The specimens were cast in steel forms using the long line method for pretensioning and using normal production procedures. The ABB pile splice assemblies were secured in the steel forms and precisely aligned to the pile axis. The casting tolerance for the deviation of the splice bearing plates from the perpendicular was 1:150 in both directions.

The cement content of the concrete mix was 5.5 sacks per cu yd (307 kg/m³), and the concrete mix was proportioned for a compressive strength of 5000 psi (34.5 MPa) at 28 days. Type I portland cement was used. Pearl River sand and gravel were used, with 1 in. (25 mm) maximum size aggregate. A water-reducing, set-control admixture was used. The mix was weaker than the normally used concrete because of the 3000 kip (13350 kN) limit of the testing machine used for the compression tests.

Each strand was initially tensioned to 30.98 kips (137.8 kN) [202.5 ksi (1396 MPa) = 0.75 f_{pu}]. The tension specimens were fabricated with 15 ft (4.57 m) of free strand protruding from the ends away from the splices, in order to facilitate the subsequent testing.

No. 5 gage [0.207 in. diameter (5.26 mm)] spiral ties were used in all test sections. For the tension and bending specimens, a 6 in. (152 mm) pitch was used along most of the length of the section, with closer spacings near the splices. A 3 in. (76 mm) pitch was used for most of the length for the compression specimens.

All pile sections were cast on October 30, 1987. The prestressing force was transferred November 3, 1987, when f'_c = 4200 psi (29 MPa). The computed steel stress immediately after transfer was 180 ksi (1241 MPa), giving an initial precompression in the concrete of about 950 psi (6.55 MPa). The tension tests were conducted at the prestressing plant after the specimens had been spliced together. The bending and compression tests

Table 1. Concrete compressive strength of specimens.

Specimen	Age, days	f'_c, ksi	No. of cylinders
Tension test specimens			
T-1	19	5.51	3 cylinders
T-2	25	4.99	3 cylinders
T-3	26	5.62	3 cylinders
Bending test specimens			
B-1	32	5.53	5 cylinders
B-2	33	—	—
B-3	34	5.83	4 cylinders
Compression test specimens			
C-1	39	—	—
C-2	40	—	—
C-3	41	5.77	9 cylinders

Metric conversion factor: 1 ksi = 6.895 MPa.

were conducted in the Newmark Civil Engineering Laboratory and Talbot Laboratory, respectively, of the University of Illinois, in Urbana, Illinois, and those specimens were spliced together after they had been shipped to Illinois.

Concrete Strength and Deformation Properties

The compressive strengths of the 6 x 12 in. (150 x 300 mm) cylinders, at the times of the various tests, are summarized in Table 1. Fig. 4 shows measured stress-strain curves from two of the cylinders tested when Specimen B-1 was tested. The peak stresses were reached when the strain was about 0.0016. The initial tangent modulus values were 5.5 to 6 x 10^6 psi (37.9 to 41.4 GPa).

Reinforcing Bar and Splice In-Air Tests

Several individual splice units consisting of #10 (32.3 mm) reinforcing bars, blocks, dowels and shear pins were tested in air, as was a length of the #10 (32.3 mm) reinforcing bar. The results of three of the tests are shown in Fig. 5. The strains were measured with a 10 in. (254 mm) gage length extensometer.

The #10 (32.3 mm) reinforcing bars were Grade 60 (414 MPa), and the measured yield stress was 67.5 ksi (465 MPa). The stress-strain curve shows a sharply defined yield point, with a yield plateau followed by strain hardening. The ultimate stress was 105 ksi (724 MPa).

The stresses in the two splice units are those computed on the basis of the nominal area of the reinforcing bar, 1.27 in.^{2} (819 mm^2), rather than on the reduced section existing at the threaded connection. The stress-strain curves for the two splice tests are similar. There is a little initial set or slack in the system, followed by a range which is nearly linear up to 40 to 50 ksi (276 to 345 MPa) tensile stress, followed by gradual softening. The loadings were increased after the extensometer was removed, and ABB-#1 failed at a nominal bar stress equal to 75.4 ksi (520 MPa). ABB-#1 failed abruptly after large visible deformation of the shear pin, and appears to have broken the shear pin, in pure shear, at its inner end. ABB-#10 failed at 91.5 ksi (631 MPa), with a fracture of a reinforcing bar in its threaded length.

Splice ABB-#1 was from the batch used in the test piles. Splice ABB-#10 was fabricated in the United States using domestic steels. The reinforcing bars were domestic.

Strand Tests

Two samples of strand were obtained by breaking them out of one of the compression test specimens after it had failed. Tension tests were conducted, and the failure stresses were 285 and 288 ksi (1965 and 1986 MPa). The yield stresses, at 0.01 elongation under load, were both 260 ksi (1793 MPa). Fig. 6 contains the stress-strain curve for

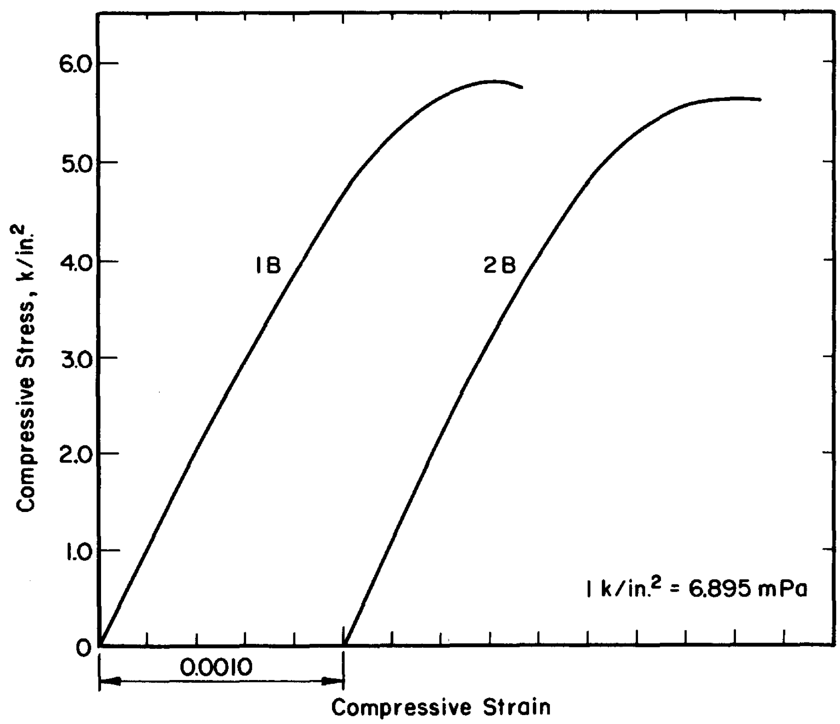


Fig. 4. Measured stress-strain curves for pile concrete.

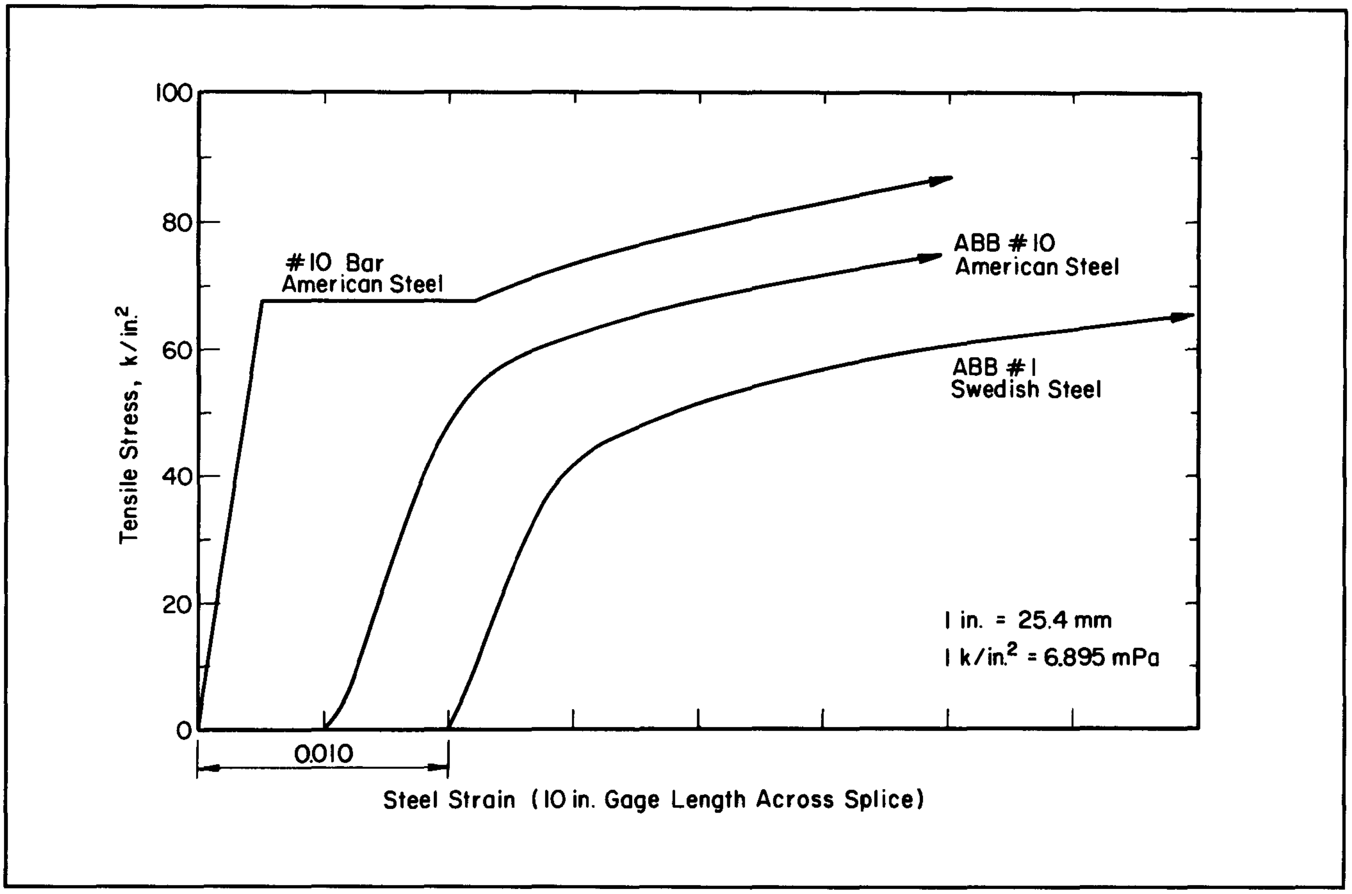


Fig. 5. Stress-strain curves for reinforcing bar and splice units.

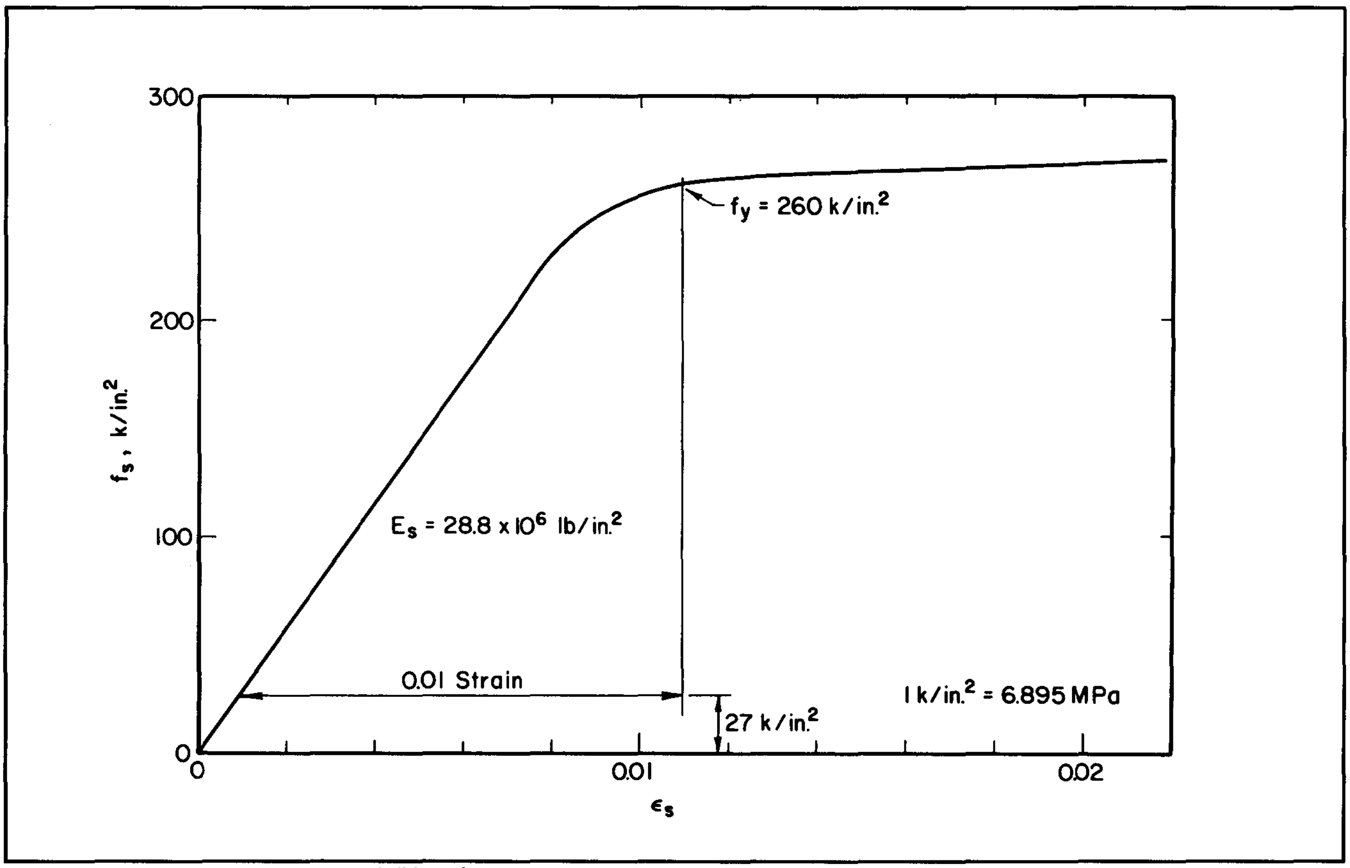


Fig. 6. Measured $f_s - \varepsilon_s$ curve, ½ in. Grade 270 strand.

one strand. Young'smodulus was 28.8 x 10^6 psi (198.6 GPa).

TENSION TESTS

Description of Testing Arrangement

Each 90 ft (27.43 m) long test specimen was assembled from two 45 ft (13.72 m) segments and placed in a 100 ft (30.48 m) long test frame on four support rollers. The stressing ends and the anchor end of the test frame, which was in effect a prestressing bed, reacted against each other by means of four large concrete compression struts.

The test load was applied by pulling on the free strands at one end of the test specimen, one strand at a time. The individual strands were pulled in the numerical sequence shown in Fig. 3, in order to minimize the eccentricities throughout the test.

The total load was applied in seven increments, up to a total load of 610 kips (2713 kN). The load was applied by a strand jack which had recently been recalibrated.

The total test load of 610 kips (2713 kN) represents the design strength, at 60 ksi (414 MPa), of the eight #10 (32.3 mm) reinforcing anchorage bars used to anchor the splice plates.

A dial gage was placed across the splice connection in order to measure any separation between opposing splice plates as the tension load was increased.

Tension Test Results

The tension test results were all similar, and were conducted at the ages shown in Table 1. The measured opening of the joints are plotted versus axial tension in Fig. 7 for all three specimens.

The joint movement for Specimen T-1 was 0.012 in. (0.30 mm). Most of the deformation occurred after the force exceeded 400 kips (1779 kN). No cracks were found anywhere in the specimen.

The measured opening of the joint for Specimen T-2 was 0.011 in. (0.28 mm). The movement is not significantly different than was found in Specimen T-1, although the curve is of somewhat different shape. Again, no cracks were found.

The joint opening for Specimen T-3 was 0.023 in. (0.58 mm) in this test, about twice that in the other two. There is no known reason for the difference. The force was left on the last specimen for 5 days, in an effort to detect any time-dependent cracking. No cracks were found. Fig. 8 is a photograph of the splice region of the test specimen while it was under full load.

Summary of Tension Test Results

The three 24 in. (610 mm) square pretensioned concrete pile specimens tested in tension were fabri-

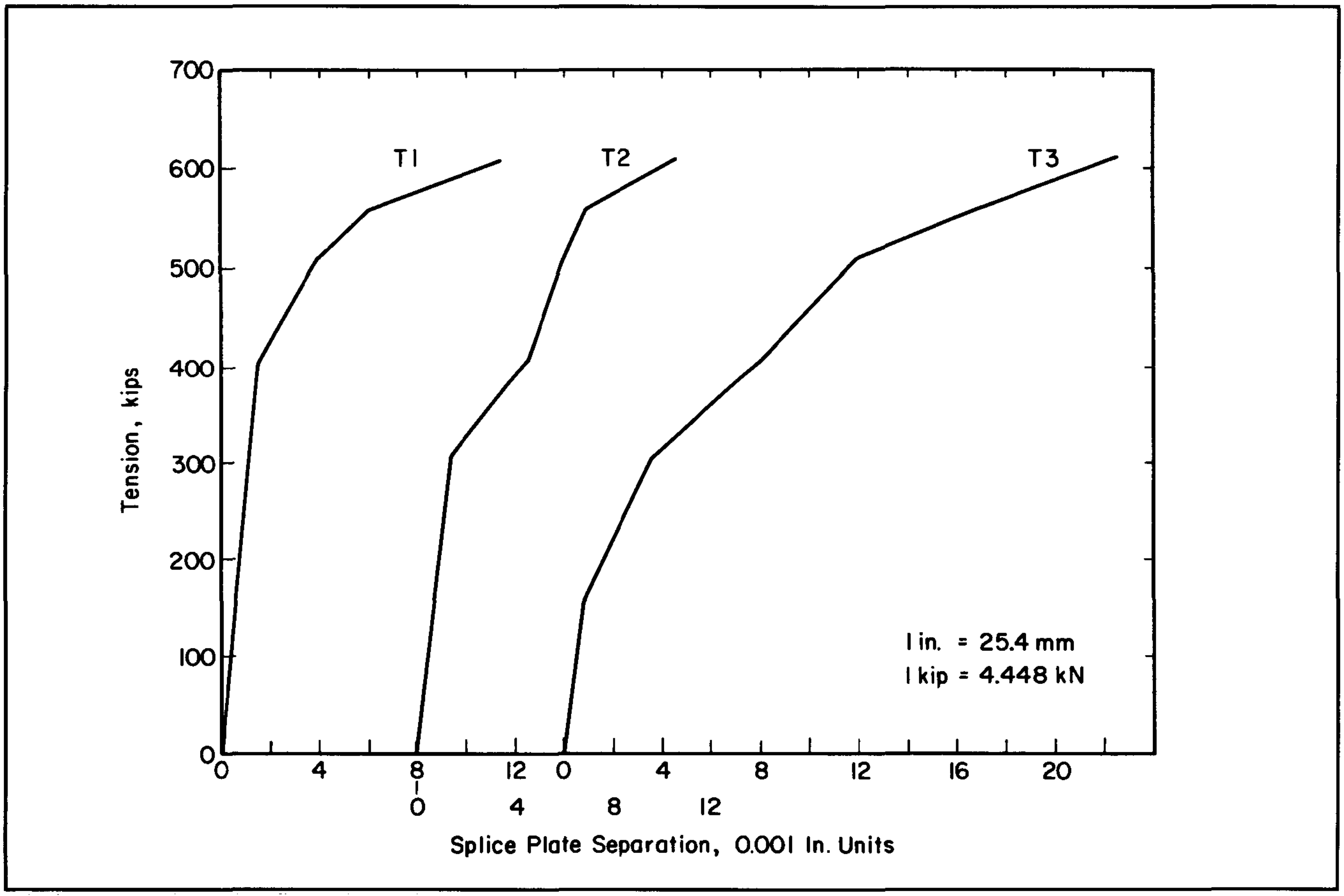


Fig. 7. Tension load vs. separation of splice plates.

Fig. 8. Tension test specimen at maximum load.

cated and spliced by normal production procedures.

Tension tests were performed up to a maximum load of 610 kips (2713 kN) for each of the spliced piles tested. This maximum load was chosen to correspond to the specified yield strength of the reinforcing anchorage bars. As the load was applied in increments, the separation of the splice plates on opposing pile sections was measured, and remained small.

In all three tension tests, the ABB pile splices performed with no visible signs of distress. The splice carried the full tensile load of 610 kips (2713 kN) with no visible cracking of the concrete pile section. The absence of cracking away from the ends of the piles is quite reasonable. The strands were probably stressed to at least 160 ksi (1100 MPa) tension [a time-dependent loss of 20 ksi (138 MPa) after transfer] at the beginning of the tests, for a precompression force in the concrete of about 490 kips (2180 kN). That is, 490 kips (2180 kN) tension applied to the pile section resulted in a net concrete stress of zero.

The remaining 120 kips (534 kN) tension force caused an average tensile stress in the concrete of about 210 psi (1.45 MPa), while the expected direct tensile strength would be expected to be at least 300 psi (2.07 MPa), and perhaps 400 psi (2.76 MPa) or more. Thus, the absence of cracks in the main bodies of the piles was the expected behavior. The prestressing force is not fully developed within the end 2 ft (0.61 m) or more of the pile, and cracking might reasonably be expected in those regions. However, cracks were not found.

BENDING TESTS

Description of Bending Test Equipment and Procedures

Two 18 ft (5.49 m) long segments were assembled to make a 36 ft (10.97 m) long member which was tested in bending. This long member was simply supported on a span of 34.0 ft (10.36 m), as shown in Fig. 9. The end supports were large concrete blocks which were grouted to the laboratory floor, and steel plates and rollers were provided at both ends so that there could be no significant restraint of longitudinal extension. The laboratory floor is a 2 ft (610 mm) thick slab which forms the top flange of a large box girder.

The loads were applied at two points, 6.0 ft (1.83 m) each side of midspan, so that there was a 12 ft (3.66 m) long nominally constant

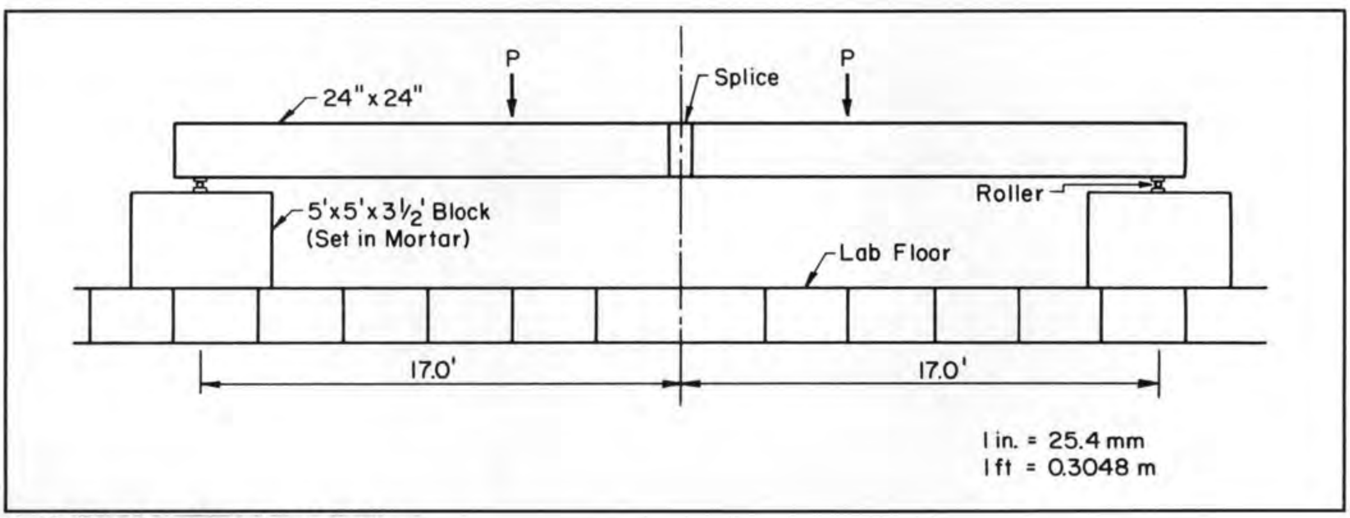


Fig. 9. Elevation of bending test setup.

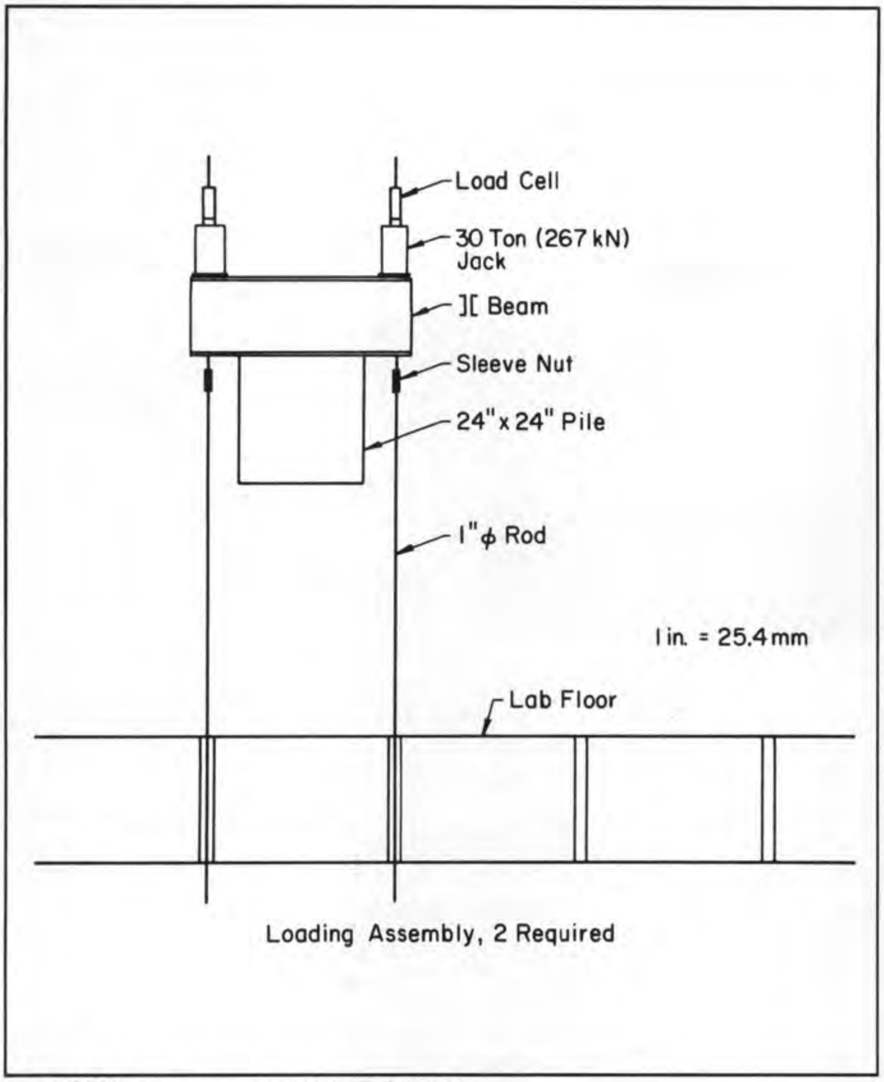


Fig. 10. Pile loading assembly; bending test.

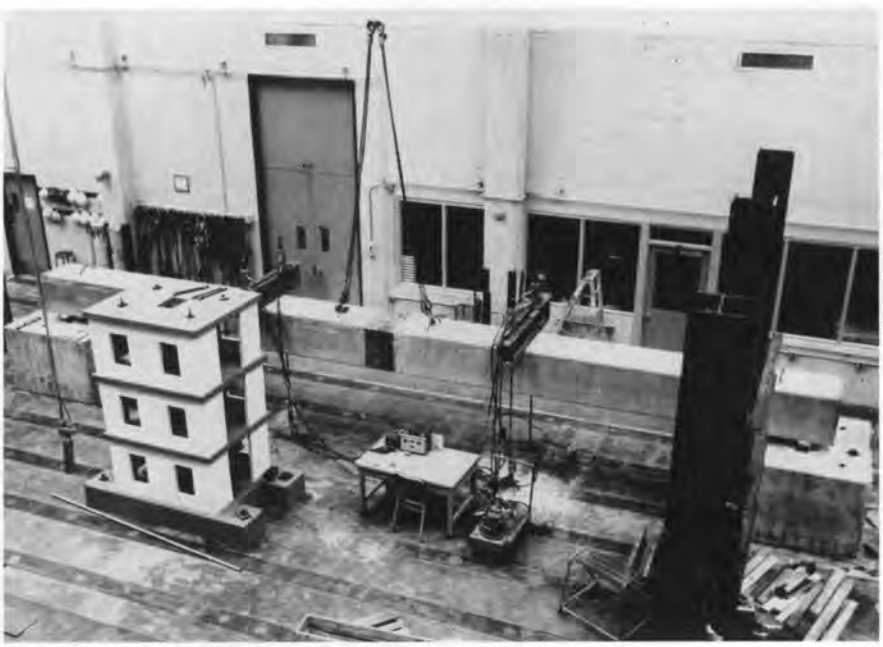


Fig. 11. Flexural test specimen.

moment region. This length was chosen so that the constant moment region would include the full lengths of the #10 (32.3 mm) bars in the splice. Each load was applied by means of two 30 ton (267 kN) center hole hydraulic rams, as shown in Fig. 10.

The rams were connected to the laboratory floor by 1 in. (25.4 mm) diameter high strength steel tension rods, and they bore on a spreader beam made of two 12 in. (305 mm) channel sections which were spaced 1.75 in. (44 mm) apart. The 1 in. (25.4 mm) rods were not long enough, and so were spliced with 1.50 in. (38.1 mm) diameter sleeve nuts.

All four rams were connected to a single hydraulic pump. Each loading assembly, consisting of the double channel beam, two rams, two rods, two load cells and various washers and plates, weighed 605 lbs (2.7 kN force). The force in each ram was measured by means of an aluminum load cell.

Fig. 11 is a photo of the test setup in the Newmark Civil Engineering Laboratory of the University of Illinois at Urbana-Champaign.

Vertical deflections were measured at midspan and at each load point using mechanical dial gages. A dial gage was also mounted across the splice at about 4 in. (100 mm) from the bottom on the north side of the member so that some measure of joint opening could be obtained.

The loads were applied in increments of approximately P = 4.0 kips (17.8 kN) at each load point, so that P = 40 kips (178 kN) should be reached in ten steps.

The ages of the specimens at the time of testing are listed in Table 1.

Bending Test (Specimen B-1)

Fig. 12 is the load-midspan deflection curve, noting that P is the total applied load from the rams at each load point, not including the weight of the loading equipment. The maximum force reached was 49.16 kips (218.7 kN) per load point, at a deflection of about 6 in. (152 mm).

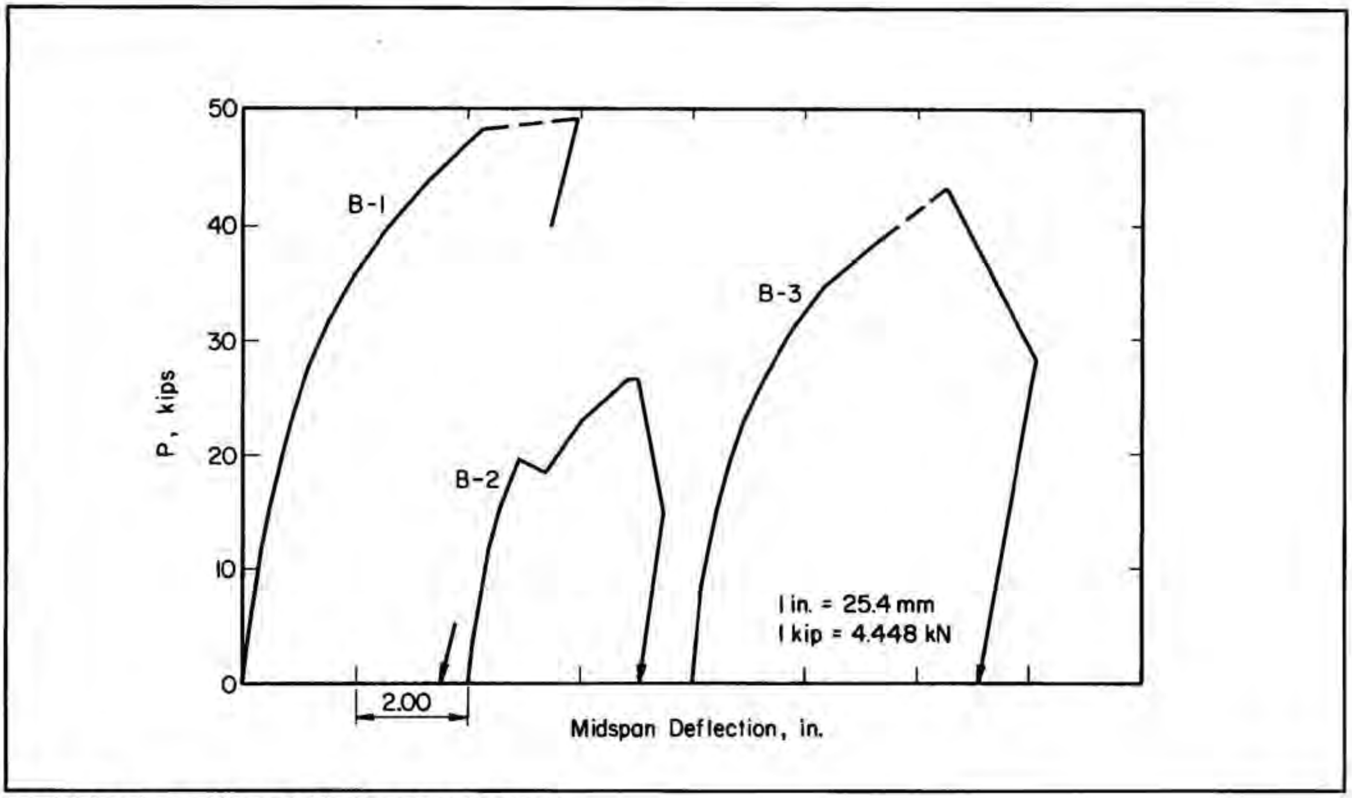


Fig. 12. Load vs. midspan deflections; bending tests.

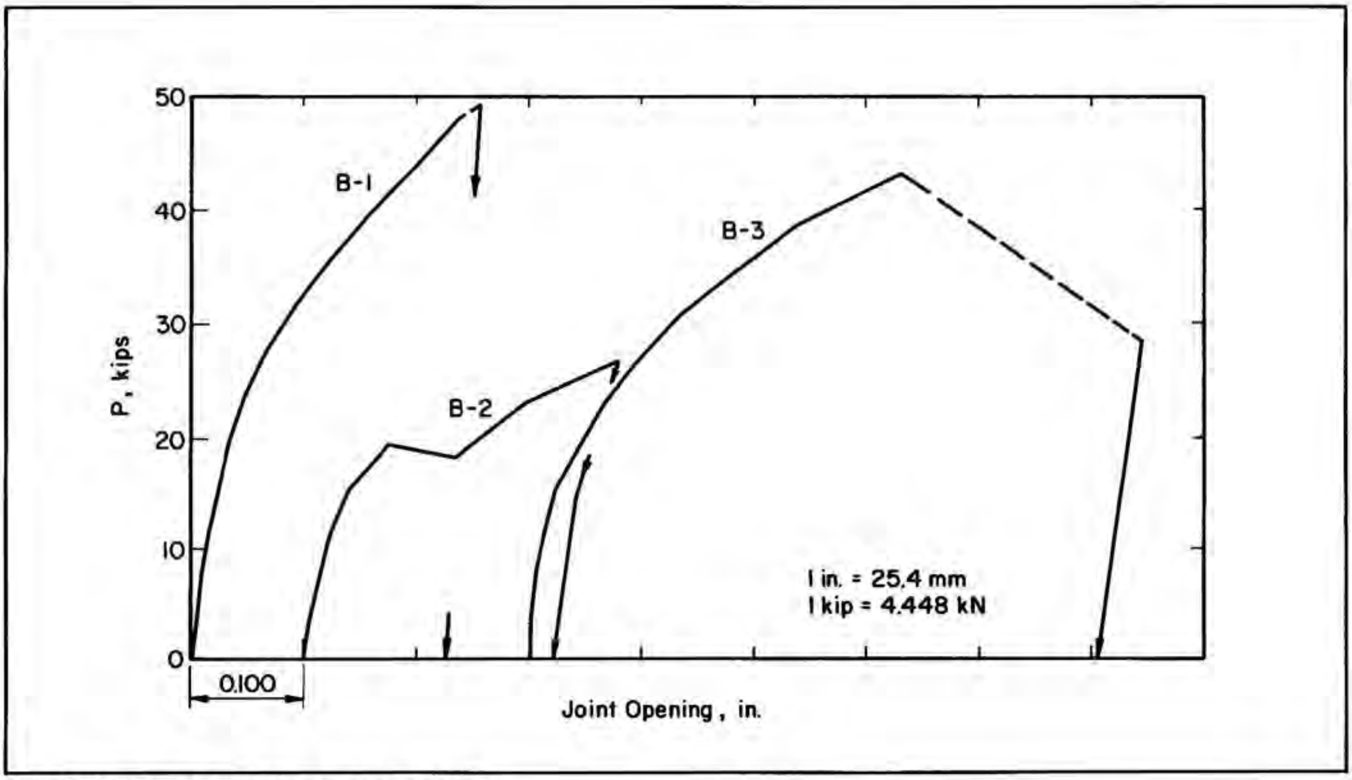


Fig. 13. Load vs. joint opening; bending tests.

Part of the load-deflection curve is shown as a broken line because of some uncertainties about the load in this region. There was a sharp increase in indicated load which apparently occurred when the sleeve nuts coupling the tension rod segments caught on the bottom edges of the channels in the double channel loading beam. The sleeve nuts apparently then slipped past the edge of the channels, leading to the load shown at the end of the load-deflection curves. This final load was 49.16 kips (218.7 kN), and is believed to be reliable.

Fig. 13 is a plot of opening of the joint versus applied load, and it can

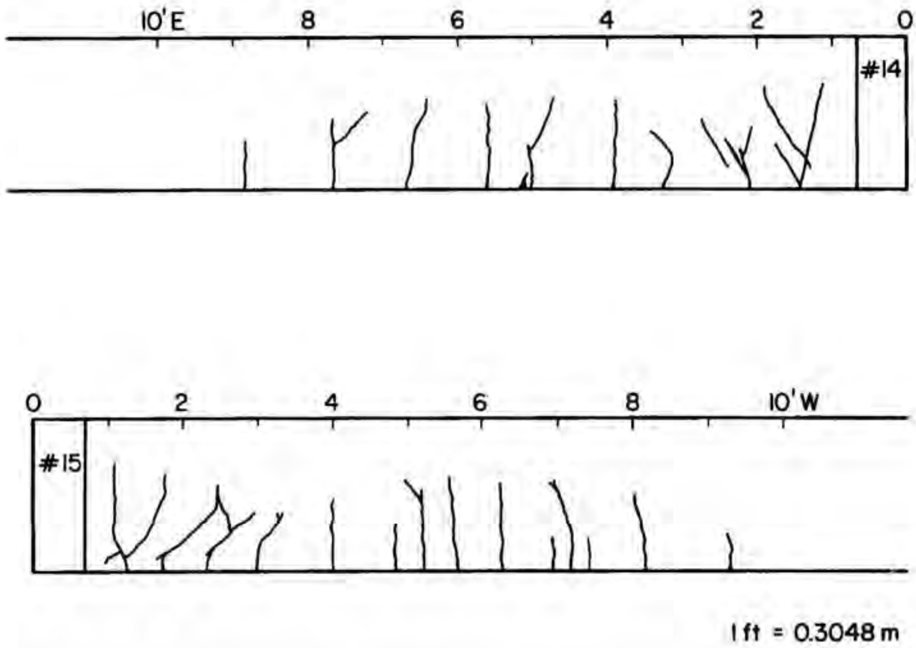


Fig. 14. Final crack pattern in Specimen B-1.

be seen that the movement was eventually about 0.25 in. (6.4 mm).

Fig. 14 shows the crack patterns which developed on the north side of Specimen B-1. The south side was similar but not identical. The first cracks occurred at $P = 20$ kips (89 kN), with one crack at each side of the splice. No additional cracks occurred until $P = 36$ kips (160 kN) was reached, at which time 12 additional cracks formed.

Each additional load increment caused some additional cracking and/or extension of cracks, until the final pattern was reached at $P = 48$ kips (214 kN). Except for the branching of the cracks which occurred near the splices, the crack pattern is normal for pretensioned members.

The branching is apparently related to stress disturbances in the bond transfer areas near the splice. The cracks which were more than about 2 ft (600 mm) from the splice remained very small throughout the test. Those nearest the splice were somewhat larger, reaching 0.25 to 0.35 mm (0.010 to 0.014 in.) when $P = 24$ kips (107 kN).

The discussion of the strength of the member relative to predicted strengths will be delayed until all three bending tests have been described.

Bending Test (Specimen B-2)

Fig. 12 also shows the plot of midspan deflection versus load for Specimen B-2. The load-deflection curve is quite different from that for Specimen B-1 in that the load reached 20 kips (89 kN) and then dropped somewhat, recovered, and then reached a second peak at 26.8 kips (119 kN) and then fell to a much smaller value. There were significant problems with the original assembly of this splice, and it seems likely that the first drop in load occurred when a #10 (32.3 mm) bar which had been screwed only two turns into the dowel failed by stripping of the threads.

This drop in load was accompanied by a noise best described as a "clunk" from the specimen. That left only three bars in the lower half of the member capable of carrying tension, but these three bars were able to accept more load. Then the second thread pullout occurred when $P = 26.8$ kips (119 kN), resulting in the reduction in load to $P = 14.9$ kips (66 kN), and the test was ended. This second drop in load was accompanied by a loud and sharp noise.

Fig. 13 shows the joint opening versus load. It is similar to the load-deflection curves at the time of the first load reduction in that there was a significant increase in deformation accompanying the load reduction. However, at the second load decrease there was a recovery of deformation which is probably related to local bending of the thin steel plate "box" surrounding the end of the concrete.

Fig. 15 shows the crack pattern on the north side of the member. The first cracks were found when $P = 16$ kips (71 kN). The next load, to $P = 20$ kips (89 kN), caused some growth in the cracks, and the second crack in Segment #4 was found when $P = 24$ kips (107 kN).

Bending Test (Specimen B-3)

Fig. 12 also includes the graph of midspan deflection versus load for Specimen B-3. The same problem caused by the sleeve nuts striking the double channel loading beam existed in this beam, leading to the

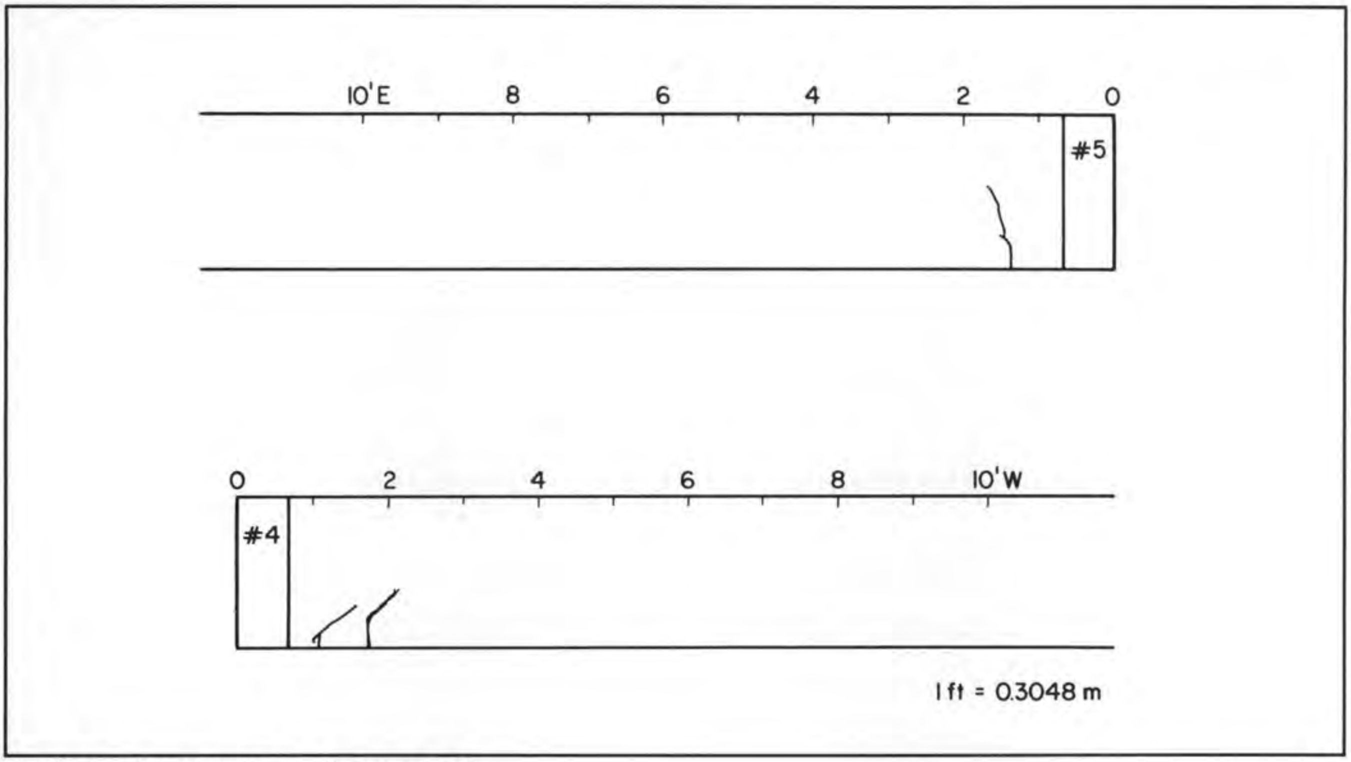


Fig. 15. Final crack pattern in Specimen B-2.

uncertainties indicated by the broken line. However, the decrease in load capacity from the peak force of 43.1 kips (192 kN) to the force of 28.4 kips (126 kN) was very real and was accompanied by a very loud, sharp noise from the specimen.

When this specimen was disassembled after the test, it was found that three of the four shear pins had failed in shear near their outer ends. The fourth pin, from the lower north face joint, was virtually undeformed, indicating that it had not carried any significant load. On further disassembly, it was found that the #10 (32.3 mm) dowel bar had engaged only one or two threads.

In the remainder of this paper, it will be assumed that the maximum force resisted by Specimen B-3 was 43.1 kips (192 kN), with full recognition that it might have been slightly higher.

Fig. 13 shows the splice gap opening versus load. The movements were considerably larger than in Specimen B-1, at least partially because the dial gage was mounted very near the dowel block unit which was not transmitting its proper share of the load.

Fig. 16 is a photo looking along the length of Specimen B-3 at the end of the test. The large deflection existing at the end of the test is quite evident.

Fig. 17 shows the cracks found on the north side of Specimen B-3. The first crack, in Segment #6, was found when P = 16 kips (72 kN), and there were two cracks in each segment when P = 27 kips (120 kN). Several additional cracks were found when P = 31 kips (138 kN), and each additional load increment caused some extension in length and some new cracks. Minor crushing was found on the top surface near the edges of the splice "box" slightly before the failure of the shear pins, and significant crushing was found in Segment #6

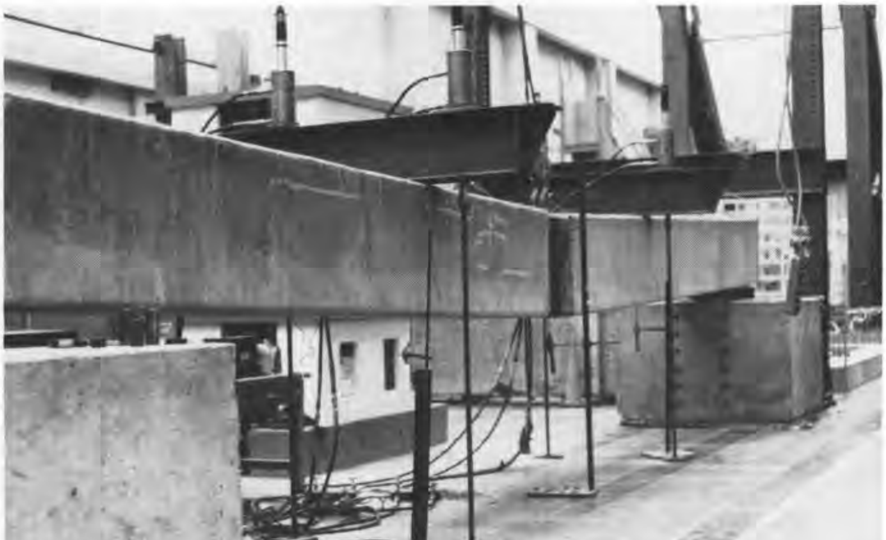


Fig. 16. Specimen B-3 at end of test.

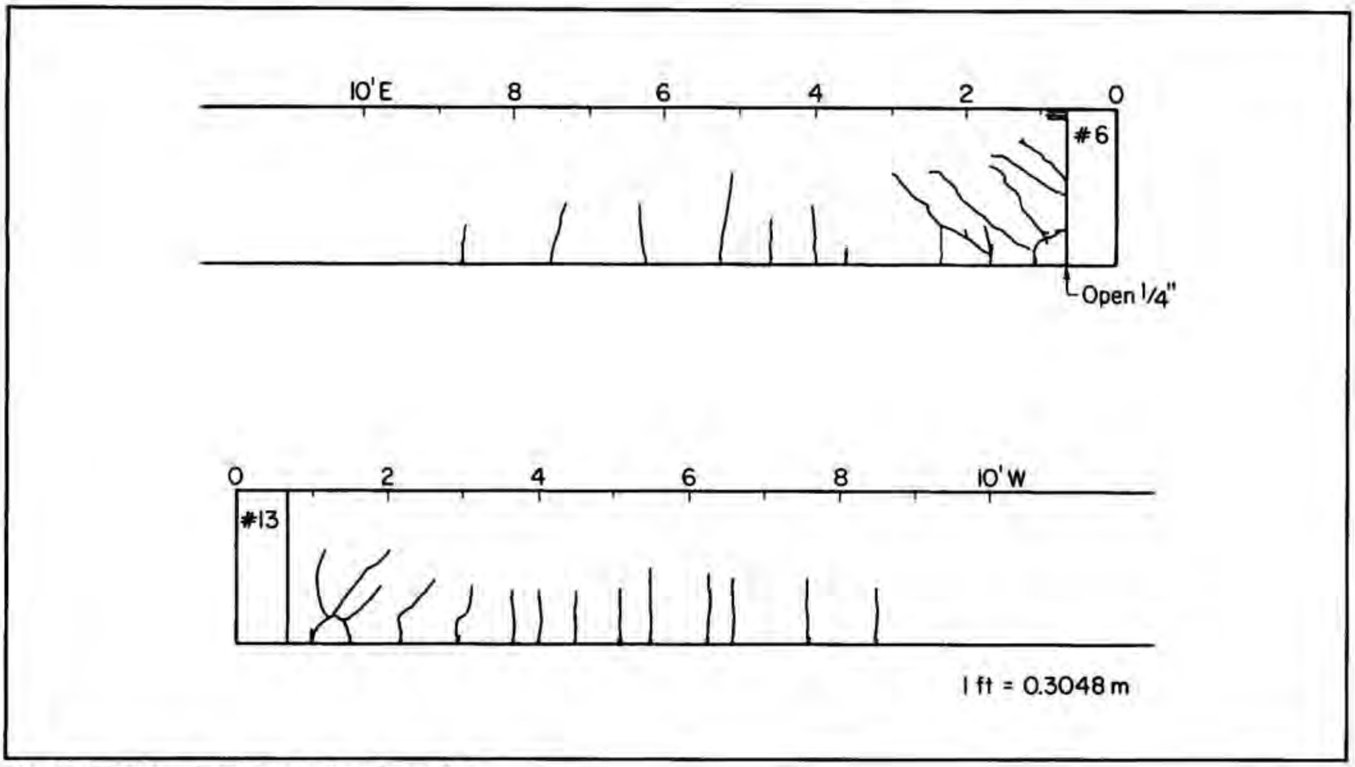


Fig. 17. Final crack pattern in Specimen B-3.

at this location after the failure.

At the maximum load, it was noted that the crack located 5 ft (1.52 m) east of the splice, in Segment #6, was significantly larger than other cracks in the same region. This is at or near the end of the #10 (32.3 mm) reinforcing bars, and may be an indication of distress at the discontinuity of the bar, or the first sign of a strand bond failure. Several of the inclined cracks near the splice in Segment #6 were found only after the failure occurred.

Summary and Discussion of Bending Test Results

The observed moment capacities will be presented and then compared with various expected strengths. The measured moment capacities are listed in Table 2, where the effects of the dead load moment plus the moment caused by the weight of the loading equipment are explicitly taken into account. These are specifically midspan moments. The moments at the load points are 10.8 kip-ft (14.6 kN-m) smaller.

Several different theoretical moment capacities must be considered in this evaluation. The following values of M_n were all determined using strain compatibility analyses which are capable of utilizing specific stress-strain data for the steel if it is available. All analyses assumed a limiting concrete strain of 0.003 compression.

One limit of the strength at the splice is the strength of the #10 (32.3 mm) reinforcing bars, ignoring the fact that they are threaded and have reduced sections. With f'_c = 5.5 ksi (37.9 MPa) and f_y = 67.5 ksi (465 MPa), M_n = 548 kip-ft (743 kN-m). Strain hardening was taken into account, but added only about 2 ksi (14 MPa) beyond f_y for the two lowest bars.

Another limit of the strength at the splice is the strength of the section considering the stress-strain curve for the splice unit. The stress-strain curve ABB-#1 shown in Fig. 6 was used in the analysis, and with this highly nonlinear curve the moment capacity was found to be M_n = 437 kip-ft (592 kN-m). This is an unduly pessimistic assessment of the strength, however, since it is assumed that the entire bar has the same stress-strain curve in both tension and compression, while in the beam the actual response at the splice must be somewhere between that corresponding to the splice unit and that corresponding to the full section #10 (32.3 mm) bar.

The analysis with the nonlinear stress-strain curve led to a steel stress of about 57 ksi (393 MPa), and a strain of 0.0151, in the lower

Table 2. Measured moment capacities of spliced pile sections.

Specimen	Jack P +	0.6 kips	Moment, kip-ft	Dead load moment, kip-ft	Total moment, kip-ft
B-1	49.16 +	0.6	546.3	86.7	633.0
B-2	27.05 +	0.6	304.1	86.7	390.8
B-3	43.14 +	0.6	480.0	86.7	566.7

Metric conversion factors: 1 kip force = 4.448 kN; 1 kip-ft = 1.3558 kN-m.

two bars, on the basis of an assumed linear distribution of strain over the section depth in conjunction with a concrete failure strain of 0.003. However, from the deformation which occurred in the lower four pins of Specimen B-1, it was clear that all four were close to shear failure and the associated 75 ksi (517 MPa) tensile stress. A flaw in the analysis is possibly in the 0.003 assumed strain, since the plates in the splice assembly may be able to provide some triaxial restraint and therefore significantly greater failure strain in the concrete.

Another limit is the strength of the pretensioned section at sections remote from the splice and where no bond problems can develop. An analysis assuming $f_c' = 5.8$ ksi (40 MPa) and using the measured stress-strain curve for strand shown in Fig. 6 led to $M_n = 612.6$ kip-ft (831 kN-m).

Specimen B-1 reached a maximum midspan moment of $M = 633$ kip-ft (858 kN-m), which is considerably higher than the computed capacity of the splice section and slightly higher than the computed capacity of the pretensioned pile section. Although the pretensioned section was extensively cracked at sections away from the bond transfer zone, the deformation and damage were not great, and there was no suggestion that failure was imminent.

In addition, Specimen B-3 also reached a moment exceeding the computed capacity of the splice section despite the fact that only three of the four #10 (32.3 mm) bars were effectively spliced. The stress developed in the remaining three bars was apparently significantly greater than the yield stress, which requires rather large steel strains.

Flexural Strengths of Pile Segments

The 18 ft (5.49 m) long segments used in the splice bending tests were damaged only near the splice location, and the individual segments were later tested as simply supported beams in order to obtain the strength of the pile section.

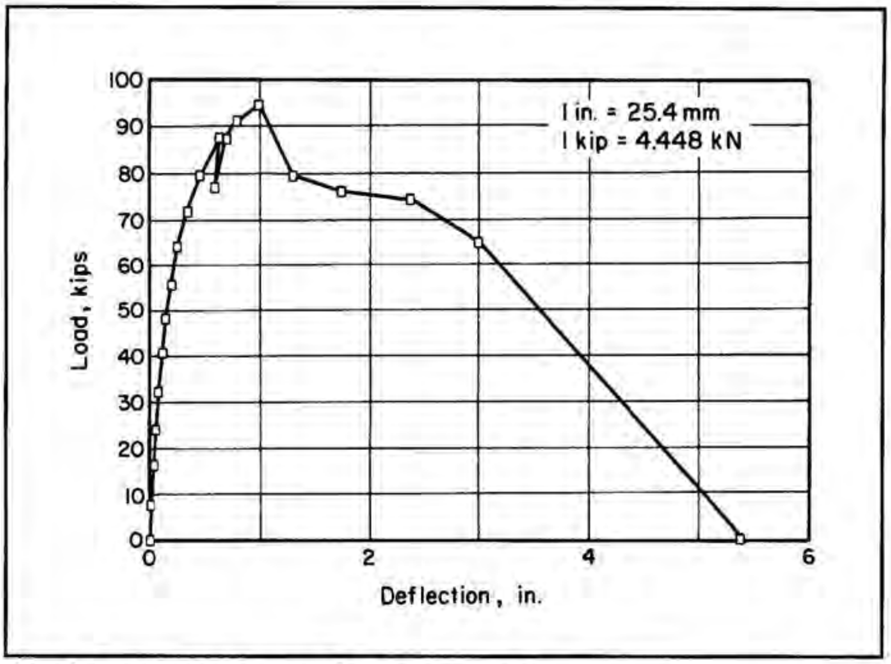


Fig. 18. Load-deflection curve for pile segment 15.

Each segment was simply supported on a span of 17.0 ft (5.18 m), and was loaded by two concentrated loads placed 1.5 ft (457 mm) each side of midspan. The shear span from the support ot the load was 7.0 ft (2.13 m). The loading arrangements were similar to those already described, except that the constant moment region was very short.

Fig. 18 shows the load-midspan deflection for Segment 15, and the others were all very similar. The curve was nearly linear to about 50 kips (222 kN) force at each load point. Previously existing cracks started reopening when P exceeded 32 kips (142 kN), and new cracks formed near midspan when P exceeded 40 kips (178 kN). Crushing of the concrete was found when P = 95.6 kips (425 kN), and the load capacity then decreased to about 80 kips (356 kN) and then gradually reduced as the deformation was increased. Crushing developed to about the level of the top layer of strand and then stabilized. This stability was probably the result of relieving the remaining tension out of the upper layer of strands.

After additional deformation, several strands fractured at one section near midspan. All six tests ended with strand fracture. In most cases a few strands broke at a time, while in one case at least ten strands fractured at the same instant.

The maximum bending moments resisted are summarized in Table 3. The maximums correspond to initiation of major crushing in the top

Table 3. Measured moment capacities of spliced pile sections.

Specimen	Jack P +	0.6 kips	Moment, kip-ft	Dead load moment, kip-ft	Total moment, kip-ft
4	95.7 +	0.6	674.1	21.7	695.8
5	92.7 +	0.6	653.1	21.7	674.8
6	94.8 +	0.6	667.8	21.7	689.5
13	91.6 +	0.6	645.4	21.7	667.1
14	95.1 +	0.6	669.9	21.7	691.6
15	95.6 +	0.6	673.4	21.7	695.1

Metric conversion factors: 1 kip force = 4.448 kN; 1 kip-ft = 1.3558 kN-m.

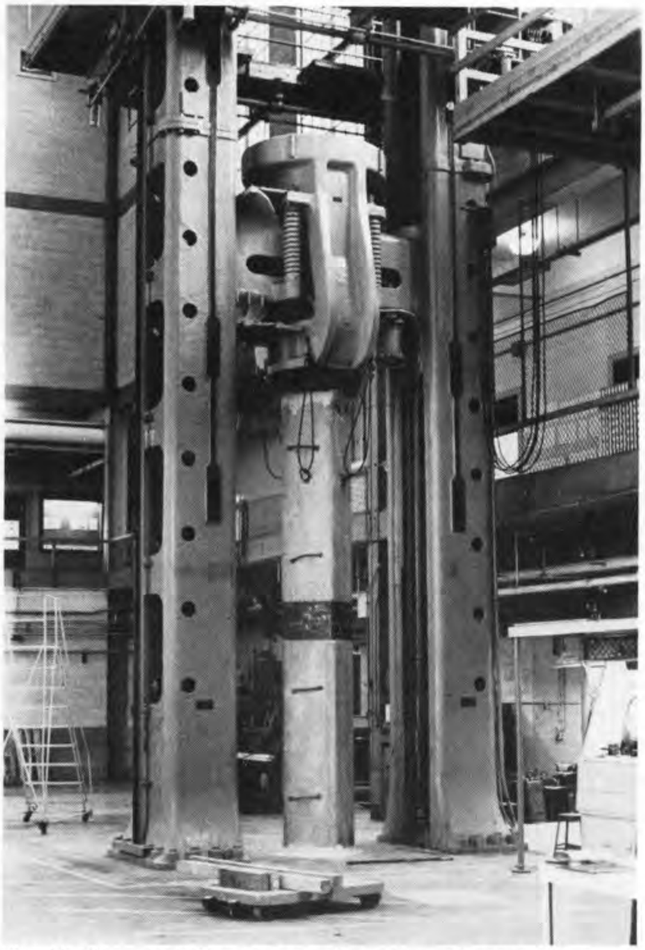


Fig. 19. Compression test specimen in testing machine.

Fig. 21. Lower part of Specimen C-1.

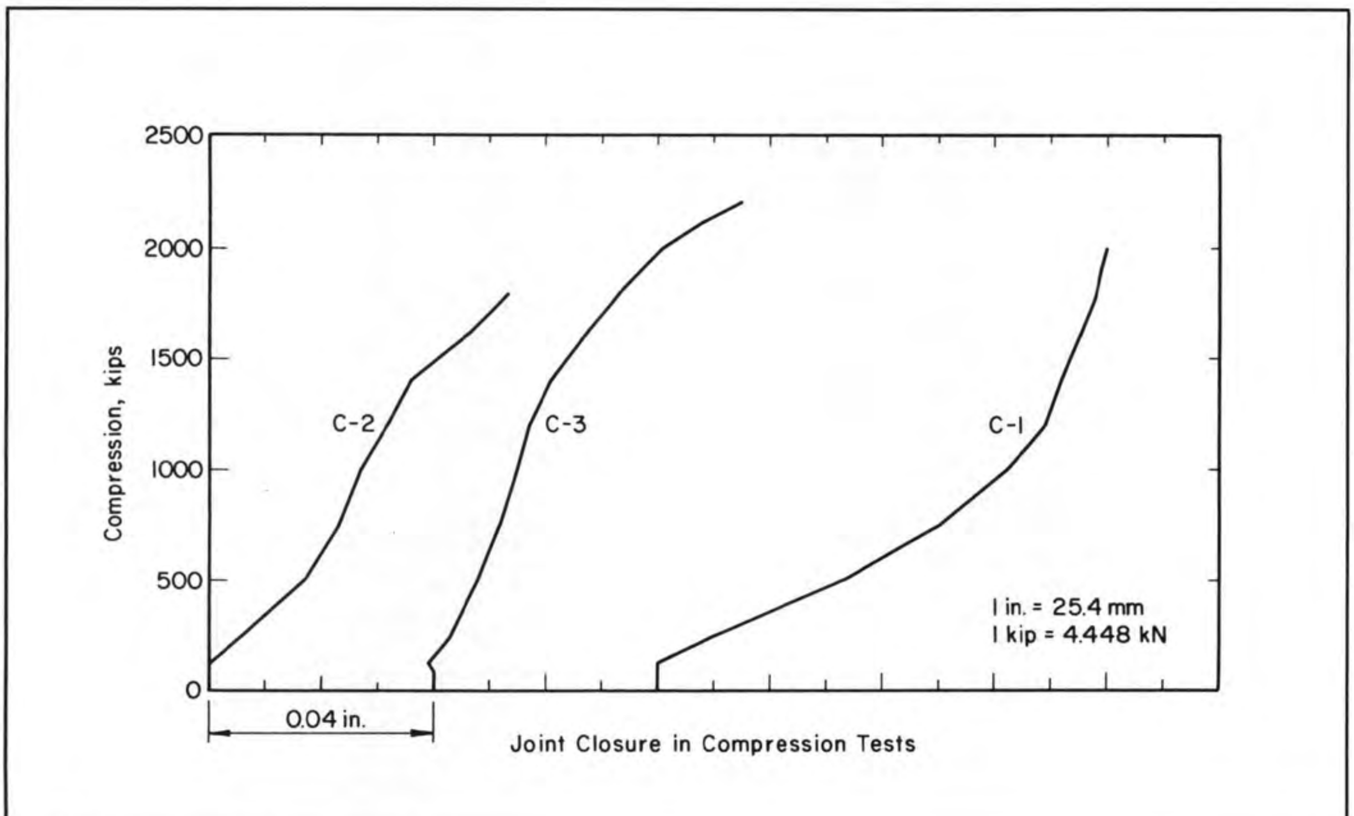


Fig. 20. Joint closure vs. compression force.

fibers of the members in all test cases.

COMPRESSION TESTS

Testing Arrangements and Setup

The compression tests of the three spliced pile members were conducted in a 3,000,000 lb (3000 kip) (13,350 kN) capacity Baldwin hydraulic testing machine located in Talbot Laboratory of the University of Illinois in Champaign.

The 16 ft (4.88 m) long specimens were assembled from two 8 ft (2.44 m) lengths, with one specimen being assembled in the machine and two outside of the machine. After each complete 16 ft (4.88 m) long unit was in the machine, it was seated in fresh Hydrocal plaster. Plaster was then put on the top of the pile and the cross head set down on it. A dial gage was attached to measure closure of the joint gap. After the plaster hardened, the test started with the application of the load in steps which were initially large and were gradually reduced to steps of 50 to 100 kips (222 to 449 kN).

Fig. 19 shows one of the specimens in the 3000 kip (13,350 kN) testing machine.

Test Results

The results of the tests were so nearly identical that they wll be described and discussed as a group. The tests were similar to all other tests of reinforced concrete columns under concentric load in that there was no visible damage until the loads were nearly at the failure point. The specimens complained audibly, with faint cracking sounds which apparently originated in or near the splice, when the load exceeded 1000 kips (4450 kN). The joint gaps closed slightly, as shown by the dial indicator, with somewhat more movement on Specimen C-1 than the other two, because of a poorer initial fit. Fig. 20 shows the closure of the joint gap versus axial load, up to forces of about 2000 kips (8900 kN), for the three specimens.

All three tests ended with the failure of the pile section just at the ends of the embedded #10 (32.3 mm) reinforcing bars. In each case the failure was sudden, with little warning, and it was complete. The concrete cross section was completely destroyed for about 3 ft (0.9 m) of the length of the pile. Fig. 21 shows the failure section of Specimen C-1.

Table 4. Measured strengths of compression specimens.

Specimen	Maximum force, kips
C-1	2735
C-2	2750
C-3	2800

Metric conversion factor: 1 kip force = 4.448 kN.

The forces are listed in Table 4. The compression failure forces varied only from 2735 to 2800 kips (12170 to 12450 kN), a very small range.

About the only thing to distinguish between the three tests was a small amount of spalling in Specimen C-2, just above and below the "box" of the joint, starting at 2000 kips (8900 kN) force. This remained a shallow surface spall, and it played no part in the eventual failure.

Discussion of Test Results

There is relatively little test data on the axial load capacity of pretensioned columns, so a little extra space will be given to theoretical considerations. For an ordinary reinforced concrete column, the axial load capacity is commonly computed as:

$$P_o = 0.85 f_c' (A_g - A_{st}) + A_{st} f_y \quad (1)$$

This is a well recognized equation, and can be traced to tests conducted at the University of Illinois and Lehigh University in the early 1930s. The 0.85 factor was related to differences between the test cylinders and the concrete in the column, and was attributed to migration of water toward the top of a vertically cast column. For horizontally cast columns, this factor was found to be 1.0 since there was no tendency for water to migrate toward either end and the water-cement ratio should be the same in all sections of the column and in the test cylinders as well. However, no tests were conducted on horizontally cast members as large as these 24 in. (610 mm) thick piles so there may be some question about whether the 0.85 or the 1.0 factor is most appropriate.

The A_{st} area of steel is assumed to be in compression, which will be true for a reinforced concrete column, but this is not correct for a pretensioned column because there will still remain a significant residual tension when the concrete fails in compression. If the strength equation is rewritten to take this into account, we find:

$$P_o = 0.85 f_c' (A_g - A_{ps}) - A_{ps} f_{pu} \quad (2)$$

where f_{pu} is the residual tensile stress.

Assuming a change in concrete strain of 0.002 compression from zero load to failure, and with E_s = 28,000 ksi (193 GPa) for prestressing strand, one can compute:

$$\begin{aligned} f_{pu} &= f_{se} - 0.002 E_s \\ &= f_{se} - 56 \text{ ksi} \\ &= f_{se} - 386 \text{ MPa} \quad (3) \end{aligned}$$

This leaves a residual tensile stress of 104 ksi (717 MPa) if f_{se} = 160 ksi (1100 MPa). Then, with f_c' = 5.77 ksi (39.8 MPa), A_g = 576 in.^{2} (371,600 mm^2), and A_{ps} = 20 x 0.153 = 3.06 in.^{2} (1974 mm^2), we compute:

$$\begin{aligned} P_o &= 0.85\ (5.77)\ (576 - 3.06) - 3.06\ (104) \\ &= 2492 \text{ kips } (11084 \text{ kN}) \end{aligned}$$

If the 0.85 factor is replaced with 1.0, P = 2988 kips (13290 kN).

Strains were not measured so the 0.002 strain which was assumed cannot be confirmed, but even without this data it is clear that the computed failure forces were of the correct general magnitude.

The part of the pile near the

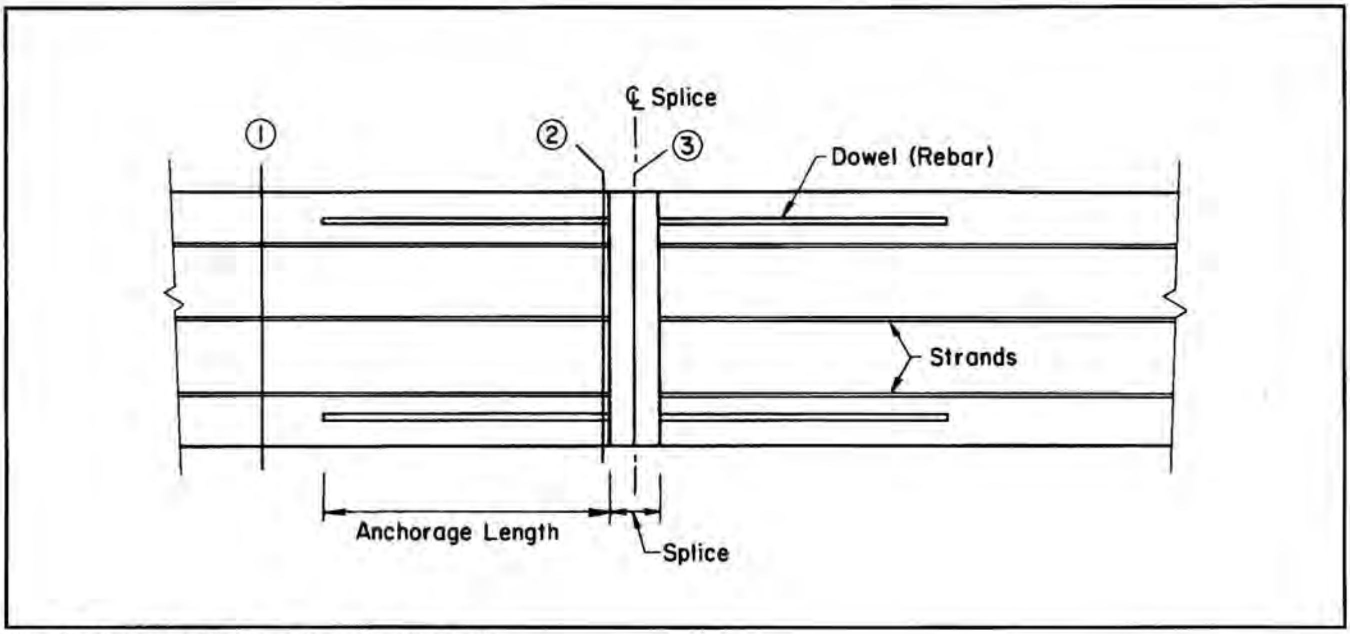


Fig. 22. Various critical sections of spliced pile.

splice has both strands (in tension) and reinforcing bars (in compression), and combining Eqs. (1) and (2) would lead to the conclusion that the section with the 8-#10 (32.3 mm) bars is just over 600 kips (2670 kN) stronger than the section with strands alone. This is an adequate explanation for having the failure sections right at the ends of the #10 (32.3 mm) bars.

DISCUSSION OF CRITICAL SECTIONS

The critical section in direct compression has just been discussed, and this section will consider the possible critical sections in bending. A case of compression plus major bending, of tension alone, or of tension combined with bending would be similar to the bending case. A member with compression and only minor bending would be similar to the pure compression case.

Fig. 22 is an elevation of an idealized, simplified pile splice. Critical section #1 is in the pretensioned pile section, either away from the ends of the dowel bars or right at the dowel termination. The moment capacity of the undisturbed pile cross section is in effect the force for which the splice must be proportioned. However, if the dowels are too short, a possible mode of failure is in flexure at the end of the dowels with the stress in the strands limited by the bond capacity existing between the end of the pile segment and the end of the dowel bars.

This will be seen in the pile as the development of a single large crack at this point, followed by a premature compression failure in the concrete. Given the right (wrong) combination of short dowels and poor workmanship leading to pretensioning with dirty strands, the reduction in strength from the nominal moment capacity can be remarkable.

Critical section #2 is at or near the plate on the end of the pile segment, but slightly back in the segment. At this section the strength comes from the dowels alone, and the section is a reinforced concrete pile rather than a pretensioned pile since the strands must be stress-free where they penetrate the end plate on the segment. The design job is to supply adequate bars of adequate length, although the anchorage length for the dowel bars will ordinarily be much less critical than the required development length of the strands.

Critical section #3 is between the splice plates on the ends of the segments, and in this case the ABB connectors directly join the #10 (32.3 mm) bars across the splice. Since it is clear from the tests in air that the shear pin in the connector is the critical component, and that the #10 (32.3 mm) bars themselves can yield but will not break, the strength of critical section #3 is lower than that of critical section #2 for this ABB pile splice.

With the other pile splice systems, other mechanical components may be involved in the capacity of critical section #3. In such cases, the strengths of the plates on the ends of the splice units, or the strengths of welds holding dowels to the end plates, may be critical. However, the breakdown of the critical sections as suggested here will always help the designer identify the appropriate limiting conditions.

SUMMARY AND CONCLUSIONS

This report describes the results of static tests of nine 24 x 24 in. (610 x 610 mm) pretensioned piles which were made from two segments joined by ABB pile splices. For the 24 in. (610 mm) square pile, the ABB splice contains 8-#10 (32.3 mm) Grade 60 (414 MPa) reinforcing bars which are connected so that they can transmit both tension and compression. Three spliced piles were tested in tension, three in bending, and three in compression.

The tension tests all reached the nominal yield capacity of the 8-#10 (32.3 mm) reinforcing bars contained in the splice, 610 kips (2713 kN) total force, without causing cracking or any other visible damage. Movement across the splice joint was measured, with joint openings ranging from 0.011 to 0.023 in. (0.28 to 0.58 mm) at maximum load.

The bending tests were conducted with the entire length of the splice within a constant moment region, which is a relatively severe test. Specimen B-1 resisted an applied moment of 633 kip-ft (858 kN-m), which is slightly greater than the computed nominal capacity of the pile section and significantly greater than the computed strength of the joint if strain hardening in the 8-#10 (32.3 mm) bars which are coupled by the ABB splice units is ignored.

The other two members were not as strong, because some of the threaded #10 (32.3 mm) reinforcing bars had not been properly screwed into the dowel parts of the splices. Despite the fact that three of the four bars in the lower half of Specimen B-3 were active (the fourth bar was screwed in only 1 to 1.5 turns), the maximum moment reached, 566.7 kip-ft (768 kN-m), was greater than the 548 kip-ft (743 kN-m) value computed, ignoring strain hardening. The joint will work properly, but adequate quality control in the assembly process is necessary.

The three members tested in compression failed at loads within the very narrow range of 2735 to 2800 kips (21160 to 12450 kN), which is in satisfactory agreement with theoretical values. The failures all occurred at the ends of the #10 (32.3 mm) reinforcing bars extending from the splice, as would be expected since this is theoretically the weakest section of the pile.

In all three types of tests the splices behaved essentially as would be expected from mechanics of material principles as applied to reinforced and prestressed concrete members. It seems apparent that ABB mechanical splices having different sizes and configurations can be analytically designed using force and strain values determined from "in air" tests of reinforcing bar splice assemblies.

REFERENCES

1. Bruce, R. N., Jr., and Hebert, D. C., "Splicing of Precast Prestressed Concrete Piles: Part 1 — Review and Performance of Splices," PCI JOURNAL, V. 19, No. 5, September-October 1974, pp. 70-97.
2. Bruce, R. N., Jr., and Hebert, D. C., "Splicing of Precast Prestressed Concrete Piles: Part 2 — Tests and Analysis of Cement Dowel Splices," PCI JOURNAL, V. 19, No. 6, November-December 1974, pp. 40-66.
3. Venuti, W. J., "Efficient Splicing Techniques for Precast Prestressed Concrete Piles," PCI JOURNAL, V. 25, No. 4, July-August 1980, pp. 102-124.