Experimental Study of Washington State Precast Girders Without End Blocks

UMESH VASISHTH AND RAFIK Y. ITANI

This paper describes the research procedure followed to eliminate the end blocks from simply supported Washington Series 14 and 10 girders. Girders in the former series have 5-in. webs, are over 6 ft deep, and can be up to 140 ft long. The procedure consisted of an analytical study followed by full-scale testing, the monitoring of a girder in a bridge, and implementation of the end block removal. A finite-element analysis was first made of girders with and without end blocks. This was followed by a full-scale test of a 48-ft long, Series 14 girder manufactured without end blocks. A Series 10 girder was manufactured without end blocks and monitored in a bridge under actual service. The findings of this research were implemented by revising the girder standards. This resulted in about a 7 percent saving in the cost of simple span girders. A study is under way to see whether the end blocks can be removed from continuous girders.

An end block is a short section with a thickened web at the end of a prestressed concrete girder. In a posttensioned girder, the end block provides room for the bulky anchorage and the distribution of the prestressing force. All girders are pretensioned these days; posttensioned girders are no longer used. End blocks were eliminated from girders made with the specifications of the American Association of State Highway and Transportation Officials (AASHTO) with 6- and 8-in. webs. Washington series girders have 5-in. webs. The reinforcing steel becomes congested at the ends of these girders if there are no end blocks. There was also some concern about the proper fabrication, transportation, stability, and shear compression capacity of these girders. Series 14 girders can be up to 140 ft long and have a 6-ft 1$^1/_2$-in. depth and 5-in. webs. Series 10 girders have a 4-ft 10-in. depth and are sometimes 120 ft long.

Researchers estimated that about 7 percent of the cost of prestressed girders could be reduced if the end blocks were eliminated. Washington State Department of Transportation (WSDOT) decided to research whether the end blocks were necessary for these girders. Research findings show that end blocks are not necessary for simple span girders.

Phase 1 of the research project, dealing with simple span girders, has just been completed. It consisted of a literature survey, a finite-element study, two full-scale tests, monitoring of the field performance of a girder without end blocks, and implementation of the research findings. A second phase to study the removal of end blocks from continuous girders is now under way.

U. Vasishth, Washington State Department of Transportation, Olympia, Wash. 98504. R. Y. Itani, Department of Civil and Environmental Engineering, Washington State University, Pullman, Wash. 99164.

BACKGROUND

There have been many studies dealing with prestressed concrete girders. Marshall and Mattock (*1*) were among the first investigators whose primary goal was to measure stresses in the end-block region and to study horizontal cracks usually observed in the end regions. From the results of their field survey and experiments, they concluded that end blocks are not necessary and that their presence in pretensioned, prestressed girders does not ensure absence of horizontal cracks. They pointed out that vertical reinforcement close to the end face of such girders will ensure satisfactory performance of the end zones. Furthermore, they suggested that fine, short horizontal cracks will not affect the service performance of the girders.

Gergely et al. (*2*) developed a direct method of designing transverse reinforcement by assuming the initiation of a crack in the end zones. They determined that for an eccentric load, the spalling stresses were larger for a rectangular end block than for an I-shaped one. They also proposed design specifications that could be applied to the anchorage zone of pretensioned members.

In 1965, Arthur and Ganguli (*3*) conducted experimental studies and applied to pretensioned beams theories of posttensioned beams used in the design of end zones. They examined the theories of Magnel et al. (an adaptation of Bleich-Sievers's theory) and Guyon in their prediction of the vertical tensile stresses in the end zone at transfer. Arthur and Ganguli proposed a modification of Magnel's theory so that it could be applied to pretensioned beams, and in 1966, Marshall (*4*) proposed a modification to Sievers's approach (*5*) for determining maximum tensile stress in the web of a posttensioned beam so that the approach could be applied to pretensioned cases.

Hawkins (*6*) in 1966 suggested that rectangular end blocks should not be used for I-beams. Gergely and Sozen (*7*) in 1967 provided a method for designing vertical reinforcement to restrain longitudinal cracks in the end zones of girders. Their method was supported by a series of experimental results that agreed closely with their theoretical values.

Krishnamurthy (*8*) in 1970 developed expressions for obtaining maximum tensile stress on the end face of the beam and the stress distribution in the region. In order to reduce vertical tensile stresses in the end zone and to reduce the transmission length of prestressing wires, gradual transfer should be used in the production of pretensioned beams. Krishnamurthy's equation predicted maximum stresses that compared well with the experimental results by Marshall and Mattock and those by Arthur and Ganguli.

Sarles and Itani (*9*) studied anchorage-zone stresses using linear elastic finite-element models. Their investigations were related, in particular, to the end blocks of WSDOT Series 10 and 14 girders. The study conclusions were that end blocks can safely be removed from such girders and that their use only serves to reduce the congestion of transverse vertical reinforcement.

The Sarles and Itani study was followed by laboratory and in-service testing of full-scale girders. The laboratory test included an evaluation of the performance of a Series 14 girder, whereas the in-service testing concentrated on measuring stresses in a Series 10 girder used in a bridge that was recently opened for traffic. The Series 10 girder was designed on the basis of the results of the laboratory test of the Series 14 girder. As mentioned earlier, the results obtained from both tests are described and presented in this paper.

LABORATORY TEST OF SERIES 14 GIRDER

The first part of the study was intended to establish the ultimate shear capacity of Series 14 girders without end blocks. Therefore, an attempt was made in the laboratory to cause shear failure in a 48-ft girder without end blocks. Details of the girder cross section are shown in Figure 1.

Each end of the girder was provided with a different type of vertical reinforcement (Figure 2) so that two separate tests could be conducted. The lightly reinforced end was called the unmarked end and the other end was referred to as the marked end. The reinforcement in the unmarked end was chosen to ensure shear failure whereas the marked end was designed

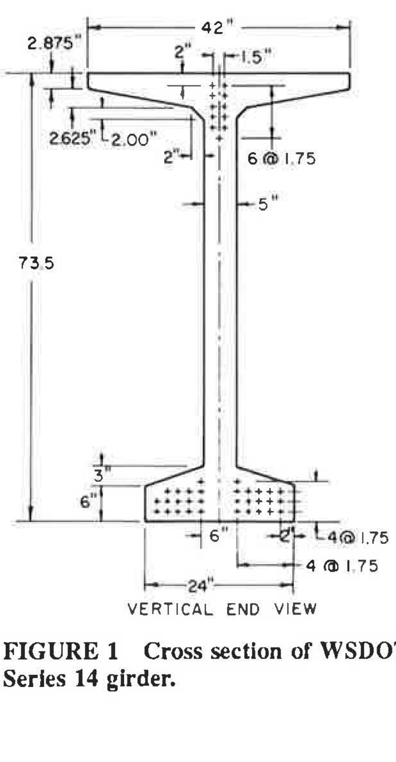

FIGURE 1 Cross section of WSDOT Series 14 girder.

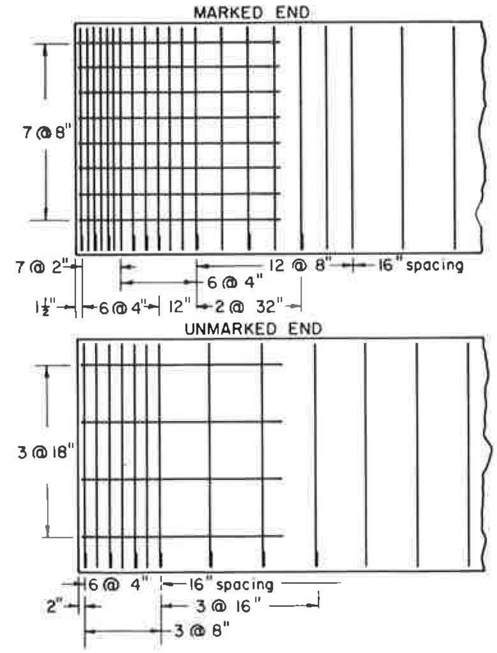

FIGURE 2 Steel reinforcement at the ends of Series 14 test girder.

according to AASHTO specifications and for an ultimate shear capacity of 400 kips. The WSDOT Series 14 girder is recommended for spans between 100 and 140 ft. The number of strands and the arrangement and profile of strands selected for this girder are identical to those used for a 110-ft long girder. Each of the 38 half-inch strands used (28 straight and 10 harped) was jacked to 28.9 kips.

Instrumentation of the girder consisted of both strain rosettes located on the concrete surface and strain gauges on stirrups and ties within the concrete. The gauges were placed so that the stress distribution from both prestress transfer and loading could be determined.

Seventeen SR-4 10-mm gauges were placed on seven stirrups at each end of the the test girder (Figure 3a and b). The gauges were located at points where maximum stresses in the steel stirrups were expected. For the first three stirrups from the ends, the gauges were placed 8, 22, and 36 in. from the bottom of the girder. The remainder of the instrumented stirrups contained only two gauges each, 8 in. and 36 in. from the bottom. For the marked end, the instrumented stirrups were placed 1¼, 6, 12, 22, 38, 70, and 102 in. from the end, whereas for the unmarked end, the instrumented stirrups were placed at 2, 6, 10, 22, 42, 74, and 106 in. Six 10-mm gauges were also mounted on ties confining the straight prestressing strands. Three instrumented ties in the marked end were located 2, 10, and 18 in. from the end, and in the unmarked end they were placed 2, 18, and 42 in. from the end. Reinforcing steel consisted of No. 4, Grade 60 bars.

Because of cost limitations, only the unmarked end of the girder was instrumented with concrete strain rosettes (Figure 3c). Twelve rectangular rosettes were placed in three rows of four, at distances of 2, 11, and 20 in. from the end of the girder. The rosettes were all located on the web region of the girder 10.5, 28.5, 46.5, and 64.5 in. from the bottom, forming a gridlike pattern.

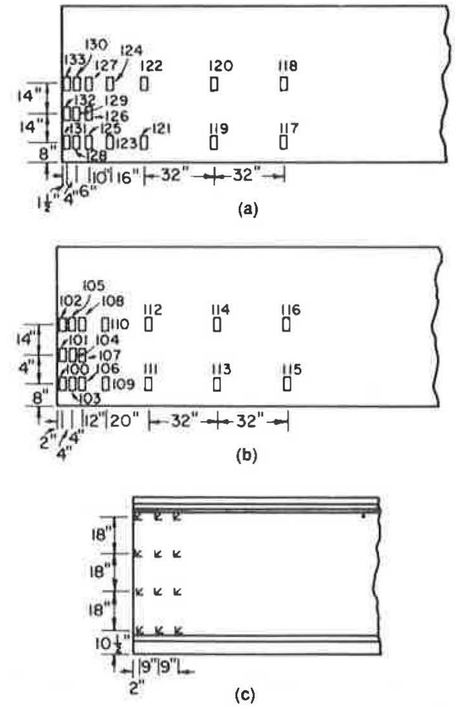

FIGURE 3 Locations of strain gauges and rosettes. (a) Location and number of stirrup strain gauges on marked end. (b) Location and number of stirrup strain gauges on unmarked end. (c) Location of rosettes on unmarked end.

The strains were recorded as the detensioning of the strands began. The only visible results from the prestress transfer test were two horizontal cracks in the unmarked end of the girder near the centroid and midway between the centroid and the bottom flange. These cracks were caused by tensile stresses resulting from the distribution of the concentrated compressive stresses at the end of the girder. The cracks were 0.1 and 0.06 mm wide and around 10 and 15 in. long. Because these cracks were not very long or wide, they were not expected to affect the performance of the girder under loading conditions and, in fact, did close. More than 15 girders with end blocks were inspected at detensioning. All of the girders exhibited cracks that were longer and wider than those observed in the test girder.

For the shear tests, the girder was placed on reinforced-concrete blocks topped with pin supports. On top of the support at the unmarked end was a 6 × 1/4 × 20-in. Fabreeka pad, whereas on the other support was a frictionless Teflon pad, which caused it to behave like a roller. Therefore, the beam was simply supported.

The loading was induced by five hydraulic jacks that could be controlled manually to achieve specified end shears. These jacks were located closer to the end being tested to keep the other end from falling, and were also aligned symmetrically to prevent transverse bending or torsional stresses.

A shear test on the unmarked end was performed first. The 6-in. support was flush with the end of the girder, and the first 100-ton ram was located 10 ft from the end. The rest of the rams were spaced at 3-ft intervals, alternating between 30 and 100 tons. An initial load of 0.1 kip was applied to each jack, which combined with the dead weight of the girder to cause an initial end shear of 18.1 kips. The load was increased in stages, with strain readings taken between loadings (Table 1). At end shears of 44.7, 72.7, 101.0, and 128.5 kips, no changes were visible except that the cracks caused by the prestress transfer began to close. The girder sustained an approximate service load of 156.1 kips without any problems.

TABLE 1 TESTING OF UNMARKED END

Load Stage	Unmarked-End Shear (kips)	Comments
0	18.1	Splitting crack widths 0.004 and 0.0024 in.
1	44.7	No visible change
2	72.7	Splitting crack widths 0.002 and 0.002 in.
3	101.0	No visible change
4	128.5	Splitting crack widths less than 0.002 in.
5	156.1	Approximate service load; no visible change
6	183.9	Splitting cracks closed; diagonal crack appeared in bottom flange
7	212.0	Diagonal crack spread through web to approximately H/2
8	239.3	Diagonal crack grew in length and width; new crack formed parallel to first one
9	263.1	Girder failed in bearing/compression

NOTE: 1 in. = 25.4 mm.

The first sign of cracking occurred at an end shear of 183.9 kips when a diagonal crack appeared at the intersection of the web and bottom flange near the support. At this point, the shear stress was high because of the abrupt reduction in width of the cross section, and the longitudinal compression was reduced by the effect of the applied loads. Therefore maximum principal tensile stresses were produced, causing an inclined crack to form. As the load was increased to 212.0 kips, the diagonal crack extended through the web to a distance of about $h/2$. At 239.3 kips, the first crack continued to grow and a second crack formed parallel to the first one. As the load was increased to an end shear of 263.1 kips, the girder collapsed onto the floor. Although the girder was designed with a nominal shear of 300 kips, a maximum shear of only 260 kips was reached, which exceeded the ultimate design volume for many of the girders used. However, the failure was not caused by shear but rather by crushing of the concrete between the web and the bottom flange where the bearing area was not sufficient to carry the high compressive force. After the two cracks had opened, the load distribution at the edge-loaded short bearing did not represent actual design conditions. Subsequent to this failure, the beam was lifted and supported for the marked-end test.

For the marked-end test, the method of support was changed to simulate actual conditions and prevent failure due to bearing compression. The support width was doubled to 12 in. by using two 6-in. supports. A piece of plywood was used between the beam and the support to provide rotation. The support was also moved in from the end of the girder so that the centerline was 12 in. from the end. The girder was loaded in the same manner as in the unmarked-end test, except that the end shears were slightly less because of the shorter span length. Again, strain readings were taken after each stage of the loading.

The load was increased from an initial end shear of 17.1 kips to 264.4 kips (Table 2) without any cracks or other problems. However, at an end shear of 291.7 kips, the reaction floor of the testing facility began to fail, so the loading was discontinued. No cracks were noticed.

TABLE 2 TESTING OF MARKED END

Load Stage	Marked-End Shear (kips)	Comments
0	17.1	Begin Test 3.1
1	43.5	No new cracks
2	71.3	No new cracks
3	99.5	No new cracks
4	126.7	No new cracks
5	154.3	No new cracks
6	181.9	No new cracks
7	209.3	No new cracks
8	237.0	No new cracks
9	264.8	No new cracks
10	291.7	Loading discontinued due to hold-down failure
0	17.2	Begin Test 3.2
11	153.5	No new cracks
12	181.0	No new cracks
13	208.0	No new cracks
14	235.6	Loading discontinued
0	17.3	Begin Test 3.3
15	152.3	No new cracks
16	179.5	Hairline diagonal crack
17	206.4	No new cracks
18	233.6	No new cracks
19	245.3	No new cracks
0	17.3	Cracks due to detensioning closed up

Stresses were computed from the strain readings. Several assumptions were made for analyzing the stresses. The researchers assumed that both the steel and the concrete were stressed within their elastic ranges and that the concrete remained uncracked. This latter assumption was unrealistic, and one should view the results of high principal stresses with caution. Another assumption was that the concrete behaved as an isotropic and homogeneous material. Because the strains were measured on an unstressed surface of the girder, a condition of plane stress existed. The combination of this condition with the assumption that deformations would be small made the condition of plane strain also valid. These assumptions provided the basis for analyzing the concrete stresses using the mechanics equations associated with linear elastic behavior, plane stress and strain, and Hooke's law. The stresses in the reinforcing steel were calculated by multiplying the strains by the modules of elasticity of the steel (29 × 106 psi).

FIELD TEST OF SERIES 10 GIRDER

The second phase of the study was to design and monitor the performance of a girder without end blocks during prestress transfer and under actual loading conditions. A bridge using Series 10 girders was selected for this purpose. An interior girder in one of the end spans was selected. The simply supported end of the girder was designed without an end block. This end was monitored throughout the construction of the bridge.

Figures 4 and 5 show details of the girder. The particular end without an end block was provided with stirrup reinforcement similar to that used for the marked end of the girder used in the laboratory test.

The girder was instrumented with strain gauges on the reinforcing stirrups and strain rosettes on the surface of the concrete. Eight stirrups were instrumented, with the gauges located at points where cracks were most likely to develop (Figure 6). The first two stirrups, $1^1/_2$ and $5^1/_2$ in. from the end, each carried four strain gauges. The third, at $9^1/_2$ in., had three gauges, whereas the remainder, located at $15^1/_2$, $43^1/_2$, $63^1/_2$, and $91^1/_2$ in., each had two gauges. The gauges near the end were for determining the stresses induced by prestress transfer, whereas the gauges starting $43^1/_2$ in. from the end were provided mainly for monitoring the tensile stresses induced by shear.

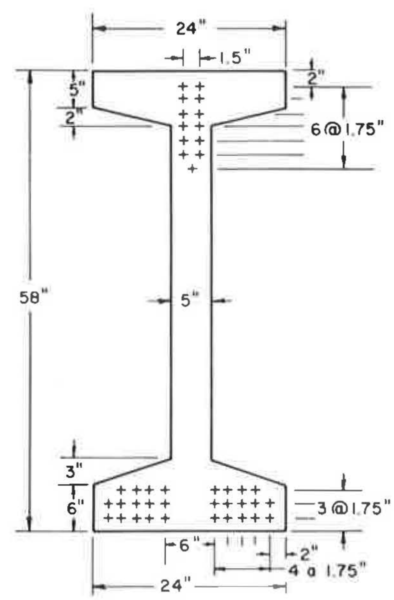

FIGURE 4 Cross section of WSDOT Series 10 girder.

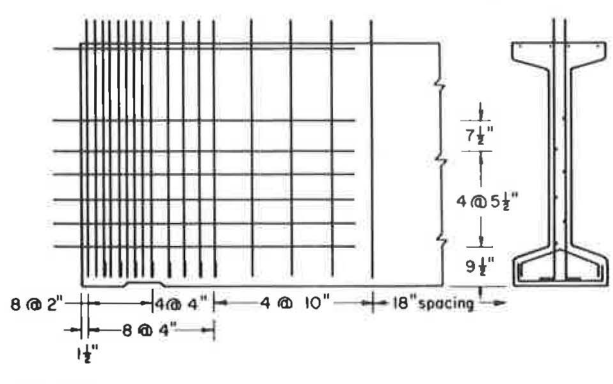

FIGURE 5 Details of steel reinforcement for Series 10 girder.

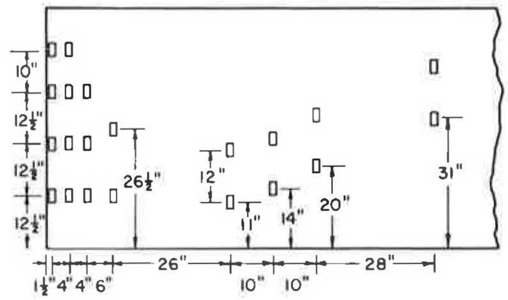

FIGURE 6 Locations of gauges on stirrups.

Strain rosettes were installed on the surface of the concrete (Figure 7) before the prestressed strands were detensioned. Readings were taken during detensioning and after completion of the prestress transfer.

Approximately 28 days later the girder was transported to its proper location. Strain gauge readings were recorded after the 7½-in. concrete slab was placed. The resulting stresses were the most critical when the slab concrete was still wet.

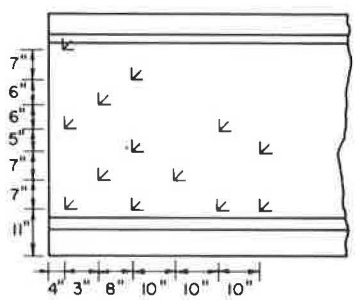

FIGURE 7 Locations of rosettes on concrete.

After the slab was cast, the final test was to place a known live load on the bridge and record the strains. The vehicle used for this purpose consisted of a cab and trailer on five axles. One axle was located under the cab and two sets of axles were under each end of the trailer. The spacing between the axles was 14.6, 4.3, 22, and 4.3 ft. The truck was weighed, and the gross vehicle weight was determined to be 78.6 kips. Each of the back axles weighed 17.1 kips, with the axle under the cab weighing 10.2 kips. The first strain reading was taken with the rear axle of the truck located directly over the end of the girder without an end block. This reading was followed by ones in which the axle was 5, 10, 25, 50, and 75 ft from that end. Strain readings were also taken with the rear axle of the truck located 10 and 5 ft from the opposite end of the girder, as well as 5, 10, and 25 ft from the end of the adjacent continuous span.

DATA REDUCTION

The collection of data from the strain gauges began during detensioning. The only noticeable effect during the prestress transfer was that the girder cambered upward, with a maximum deflection of 1 in. at the midspan. Also, a slight crack formed near the centroid of the girder end. With a length of only 13 cm and a width of 0.1 mm, the crack would not have been detrimental to the service load capacity of the girder.

The stresses in the steel and the concrete were computed from strain data recorded by methods similar to those used in the laboratory tests. The modules of elasticity for concrete were obtained by using the following equation:

$$E_{ct} = 33w^{15} (f_c')t$$

where w was the weight of the concrete, assumed to be 155 pcf, and $(f_c')t$ was the compressive strength at the the time of interest. Poisson's ratio was assumed to be 0.2.

The strain readings recorded after the roadway slab was placed were increased by 6 percent because the zero reading was taken when the steel was already in place.

These stress values were obtained only for the slab load and were later combined with the stresses obtained after the prestress transfer. When these two stresses were superimposed, the latter was reduced by 25 percent to account for a loss in prestress due to creep and shrinkage of the concrete and relaxation of the prestressing steel. The strains recorded for the truck load test were very small.

RESULTS

The stresses computed for the laboratory test and the field test were compared with the appropriate AASHTO specifications for limited stresses in concrete and steel.

The variation of maximum tensile stress in the stirrups versus their distance from the end of the girder in the laboratory test is shown in Figure 8 for the marked end of the girder. The plot for the unmarked end showed a maximum stress of around 22,000 psi at the end, which decreased to zero at the approximate transfer length. The plot of the marked end indicates that the tensile stress was much less at that end of the girder, around 15,000 psi. However, it did not follow the same pattern as that of the unmarked end, perhaps because of the heavy vertical reinforcement.

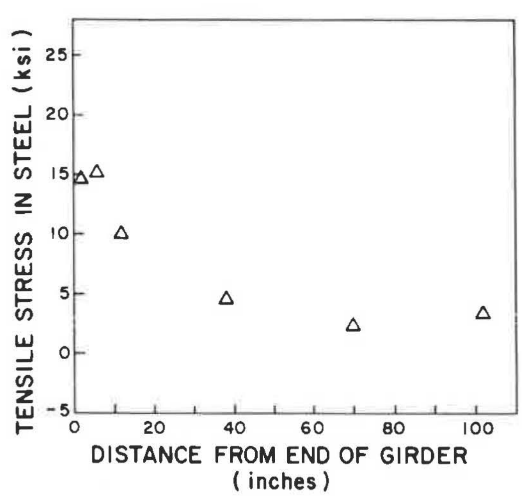

FIGURE 8 Stirrup stresses in marked end caused by prestress transfer.

During the shear test of the unmarked end, high tensile stresses were recorded for gauges 106, 109, 111, and 114 at an end shear of 239.3 kips, the final reading before failure. These agreed well with the locations of the cracks that formed (Figure 3 and Table 3). A compressive stress of around 35 ksi was indicated by the gauges located closest to the lower corner of the girder. This high stress confirmed that failure of the unmarked end was caused by the concrete at the support.

The shear test of the marked end did not produce any critical stresses in the end region of the girder (Figure 3 and Table 4). Only Gauge 121 measured tensile stress, which was well within the allowable tensile stress for the service load design of 24,000 psi for Grade 60 reinforcement. The rest of the gauges measured compressive stresses that were well above the support, but not of critical magnitude.

During the prestress transfer, the highest tensile and compressive principal stresses in the concrete were located near the bottom flange and slightly in from the end of the girder (Figure 9). (Unreliable stress values are indicated in Figures 9–12 by an asterisk.) These stresses were probably the combined result of bursting stresses produced by the straight prestressing strands and compressive stresses produced by the vertical reaction from the dead load of the girder. The magnitude of the compressive stress was well within the allowable limit (3,249 psi); however, the tensile stress exceeded the allowable limit (552 psi) by almost 800 percent. With that much stress, a microcrack forms, and the bonded steel reinforcement takes up the tensile stresses. Computations of stresses in the concrete became meaningless.

Figure 10 shows the principal stresses induced by the final stage of loading before the unmarked end of the girder failed. Tensile and compressive stresses exceeding their allowable values (557 and 3,249 psi) were indicated by the two gauges located near the bottom flange of the girder. The high principal compressive stress above the support again confirmed that failure of the unmarked end was caused by the crushing of the concrete in this area.

Finally, stresses caused by prestress transfer and the stresses at a simulated maximum service load shear of 156.1 kips were superimposed to give the principal stresses. The only principal stress that exceeded the allowable value was the tensile stress of 2,687 psi. However, this did not take into account losses due to creep and shrinkage of the concrete, or relaxation of the prestressing strands. Because the total losses were assumed to be 25 percent of the initial stresses, the horizontal, vertical, and shearing stresses (x, y, and xy) during the prestress transfer were reduced accordingly. These were again combined with the service load stresses to produce the principal stresses shown in Figure 11. This caused the highest principal tensile stress to decrease to 1,687 psi. Although it still exceeded the allowable value, this stress was greatly reduced from its initial

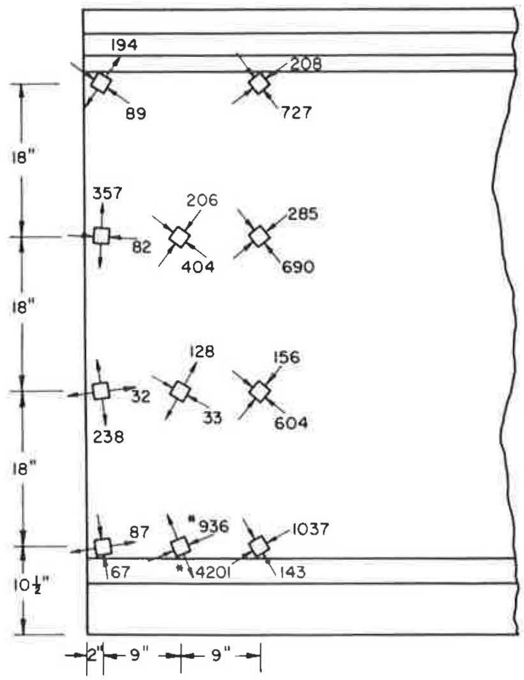

FIGURE 9 Principal stresses in concrete caused by prestress transfer (asterisk indicates unreliable value).

TABLE 3 STRESSES IN STIRRUPS DURING UNMARKED-END TEST

	Stress (psi) by End Shear (kips)								
Gauge	18.1	44.7	72.7	101.0	128.5	156.1	183.9	212.0	239.3
100	0	−3,538	−7,308	−11,165	−15,254	−19,720	−30,305	−40,339	−54,549
101	29	−3,161	−7,540	−10,556	−13,311	−15,660	−19,720	−22,707	−25,781
102[a]	–	–	–	–	–	–	–	–	–
103	87	−3,552	−5,278	−7,946	−10,614	−12,644	−16,530	−20,445	−25,230
104	0	−3,538	−7,482	−10,005	−12,122	−13,978	−17,225	−19,633	−22,069
105	58	−2,813	−6,612	−9,715	−12,093	−14,065	−17,110	−18,966	−20,735
106	29	−1,624	−2,929	−3,886	−4,524	−3,248	−5,394	10,266	10,382
107	29	−2,233	−4,872	−6,554	−8,149	−9,454	−11,745	−13,137	−14,442
108	29	−1,798	−4,524	−6,873	−8,584	−9,860	−11,890	−13,311	−14,442
109	0	−2,610	1,305	−4,350	−1,305	435	11,774	27,492	55,419
110	87	−841	−1,914	−2,956	−3,451	−3,799	−4,031	−3,712	−3,393
111	0	−203	−145	−145	−116	203	58	203	25,143
112	29	−203	−261	−522	−551	−174	−696	−464	696
113	29	−261	−145	−377	−464	−406	−406	−406	−377
114	29	−116	0	−87	−29	145	145	10,034	35,409
115	928	783	6,815	4,582	4,350	8,249	2,291	1,537	5,974
116	58	−203	−116	−145	−203	−145	−464	−406	2,958

[a]Not functioning.

TABLE 4 STRESSES IN STIRRUPS DURING MARKED-END TEST

	Stress (psi) by End Shear (kips)										
Gauge	17.1	43.4	71.3	99.5	126.7	154.3	181.9	109.3	237.0	264.8	291.7
117	–58	–522	–638	–551	–725	–667	–812	–696	–580	–1,160	–667
118	29	0	0	0	–174	–145	–116	–145	–116	–261	–551
119	0	0	–145	–203	–435	–522	–522	–638	–609	–5,220	–580
120	0	0	0	0	0	0	0	0	0	0	29
121	0	174	0	0	–174	–116	0	145	290	377	725
122	–58	0	–232	–435	–696	–783	–783	–986	–1,044	–1,131	–1,131
123	0	–580	–1,769	–3,074	–4,727	-6,003	–8,207	–7,859	–8,671	–9,570	–1,044
124[a]	–	–	–	–	–	–	–	–	–	–	–
125[a]	–	–	–	–	–	–	–	–	–	–	–
126	0	–1,479	–3,509	–5,075	–6,641	–7,540	–8,460	–8,512	–10,353	–11,368	–12,470
127	–29	–406	–1,044	–1,769	–2,668	–3,219	–3,886	–4,437	–4,959	–5,597	–6,380
128	–29	–1,769	–4,321	–6,090	–7,917	–9,077	–10,498	–11,774	–1,305	–14,297	–16,240
129	0	–1,421	–3,741	–5,684	–7,511	–8,044	–9,396	–10,556	–11,571	–12,702	–14,036
130[a]	0	0	0	0	0	0	0	0	0	0	0
131	–58	–1,856	–4,437	–6,119	–7,424	–8,625	–9,280	–10,237	–11,049	–12,064	–13,311
132	–29	–1,450	–3,828	–5,771	–7,453	–8,613	–9,918	–11,136	–11,716	–13,166	–14,500
133	–116	–1,102	–2,552	–3,915	–5,452	–6,264	–7,395	–8,352	–8,961	–9,918	–10,904

[a]Not functioning.

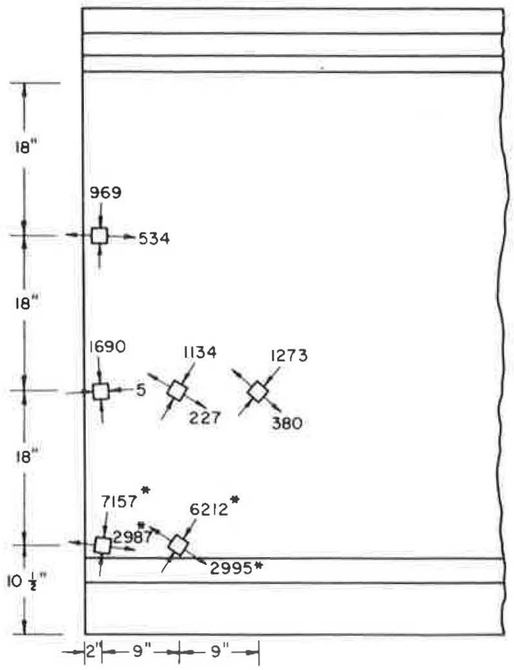

FIGURE 10 Principal stresses in concrete caused by a shear of 260 kips (asterisk indicates unreliable value).

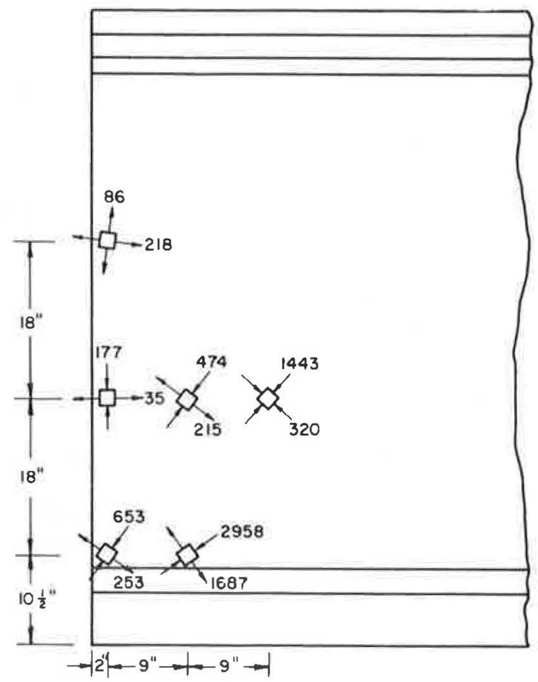

FIGURE 11 Principal stresses in concrete caused by superposition of prestress transfer and service load with reduction for losses.

value after detensioning of 4,201 psi. A tensile stress of this magnitude indicates that a crack has probably formed and that the bonded steel reinforcement is being stressed.

The highest stress in the steel stirrups was 13.5 kips for the field test girder. The highest stress was located in the stirrup closest to the end face of the girder and decreased to a minimum at a distance approximately equal to the transfer length from the end of the girder. A maximum stress of around 15,000 psi was recorded when the girder was picked up; however, this was well within allowable limits, and it also decreased when the girder was placed on supports.

The principal stresses due to prestress transfer and their orientations were recorded at the end of the girder. Many of the principal tensile stresses exceeded the allowable value. This was the result of microcracks forming in the concrete because of tensile stresses. However, visual inspection of the end of the girder indicated no visible cracks in these areas. The principal compressive stresses were well within the allowable limit, indicating that the end of the girder was capable of handling the transfer of prestress effectively, without the need for an end block.The principal stresses induced by the addition of the slab were rather small, except at three of the gauges. The high compressive stresses are probably caused by the reaction of the bearing, which was 13 in. wide, with the centerline located 15 in. from the end of the girder. However, these stresses did not represent the actual condition. To get an idea of the actual stress condition, it was necessary to combine these stresses and the stresses at prestress transfer.

Figure 12 shows the principal stresses due to a combination of prestress transfer stresses and stresses from the addition of the roadway slab. The principal stresses in Figure 12 are adjusted for a 25 percent loss in prestressing. A comparison of these with the AASHTO allowable stresses of 3,112 psi compression and 529 psi tension revealed that a few of the stresses exceeded the limits. According to Lin (*10*), the compressive stresses on which the girder design is based are horizontal stresses, although the principal compressive stresses at that point are somewhat greater in magnitude. Whenever the principal tensile stress indicated by the rosette gauge was greater than 1,058 psi [$= 12 (f_c')^{1/2}$], a surface tension crack probably existed under the strain gauge. In such a case both the principal stresses, tension and compression, obtained from that rosette gauge were unreliable. The tensile stresses that exceeded the allowable value are not critical because when the concrete cracks, the tensile force is transferred to the stirrups.

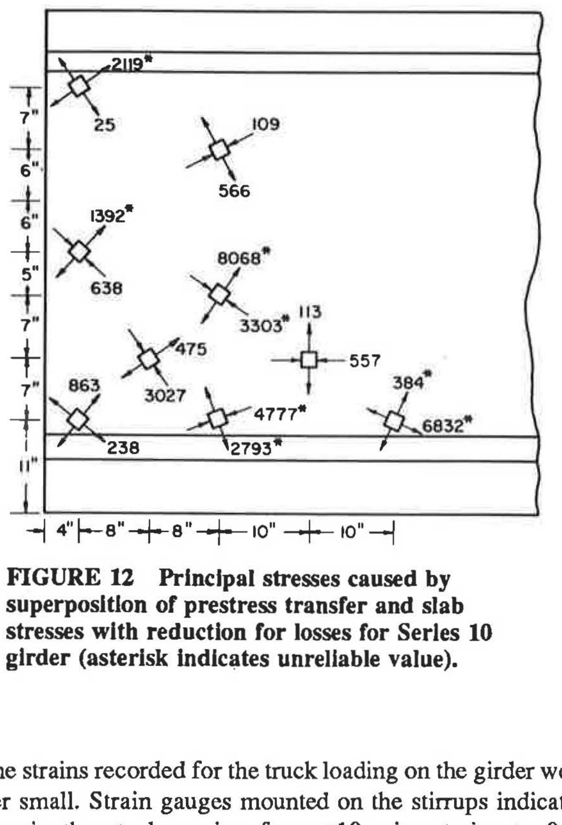

FIGURE 12 Principal stresses caused by superposition of prestress transfer and slab stresses with reduction for losses for Series 10 girder (asterisk indicates unreliable value).

The strains recorded for the truck loading on the girder were rather small. Strain gauges mounted on the stirrups indicated strains in the steel ranging from +10 microstrains to 0.35 microstrain, with 70 percent indicating compression (–). This converts to a range in stress from 290 psi tension to 1,015 psi compression for the stirrups.

Similarly, the strains indicated by the rosettes on the concrete ranged from a maximum tensile strain of +20 microstrains to a maximum compressive strain of 0.12 microstrain. Conversion of the strains into principal stresses produced values ranging from 183.9 psi in tension to 98.1 psi in compression; both extremes occurred when the rear of the truck was located at or near the end of the girder.

CONCLUSION

The results of the ultimate load test indicated that the modified ends perform effectively under both prestress transfer and service load conditions. The vertical stirrup reinforcement designed according to AASHTO shear and anchorage-zone requirements is an effective replacement for end blocks. The ties and longitudinal reinforcement are not affected by the change, and are therefore provided according to standard practice.

Production of the Series 10 and 14 girders for this project proved that steel placement and compaction of concrete in the end regions were possible with a 5-in. web.

Of interest to this project is the investigation of horizontal cracking in the end regions of WSDOT series girders. The perception that end blocks reduce horizontal cracking caused by detensioning of the prestressing strands is incorrect. However, examination of a random sample of approximately 15 Series 10 and 14 girders with end blocks showed that the end regions of all the girders contained as many as two to five cracks, with widths ranging from 0.2 to 0.3 mm (0.008 to 0.012 in.). The Series 10 and 14 girders without end blocks each had only one or two cracks 0.1 mm wide (0.004 in.) or less. Therefore, the modified design is effective in reducing the creation of cracks during prestressed transfer.

Finally, the removal of end blocks from WSDOT's simple span, pretensioned concrete girder series is not only possible and desirable but also economical, and would result in thousands of dollars in savings each year.

IMPLEMENTATION

As a result of this research project, end blocks have been eliminated in simple span, prestressed concrete girders and at the free end of continuous girders. Details of the reinforcing steel in the current, standard WSDOT Series 14 girder are similar to those used for the test girders.

FUTURE RESEARCH

A research project to determine whether the end blocks from continuous girders can be eliminated is currently under way. A full-scale specimen has been tested to a force of 500 kips. The results are not yet available.

ACKNOWLEDGMENTS

The authors wish to express their gratitude and appreciation to the Washington State Department of Transportation and its engineers, who showed sincere interest in this project and contributed their technical assistance. Acknowledgment is also due Rick Anderson and Concrete Technology Corporation of Tacoma for manufacturing and testing a Series 14 girder. Anderson's technical assistance and comments were extremely valuable in data collection and reduction.

Appreciation is extended to Central Pre-Mix of Spokane for their assistance in manufacturing the Series 10 girder for use on the Sullivan Road overpass.

Special gratitude is due the Washington State Transportation Center (TRAC) for their support of this research. Appreciation is also due FHWA for support of the research program under which this project was conducted.

Finally, the efforts and help of Ron Galbraith, a former Washington State University graduate student, are gratefully acknowledged. Galbraith worked on this project as a research assistant in pursuit of his master's degree. Others whose help is appreciated include Fumio Kamiya of the Forestry and Forest Products Institution of Tsukuba, Japan, and Girish Hiremath, a doctoral graduate student at Washington State.

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Publication of this paper sponsored by Committee on on Concrete Bridges.