Optimized Post-Tensioning Anchorage in Prestressed Concrete I-Beams

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Anchorage zones in prestressed concrete I-beams are designed to accommodate anchorage hardware and to provide adequate space for the reinforcement needed to distribute the highly concentrated post-tensioning force. Based on analytical and full-scale experimental studies, optimized anchorage zone details have been developed. The proposed standardized anchorage zone is suitable for use with a post-tensioning tendon size up to 15 - 0.6 in. (15 mm) diameter strands. It has a width of 28.5 in. (725 mm), which tapers for a distance of 39.4 in. (1000 mm) to the standard I-beam web width. Standard welded wire reinforcement is proposed for post-tensioning of up to three 15 - 0.6 in. (15 mm) diameter strand tendons. The use of a reduced block size can result in a weight reduction of as much as 80 percent compared to the commonly used anchorage block. A detailed example is included to demonstrate application of the strut-and-tie model to the design of the anchorage zone.

The span limit of precast, pretensioned I-beam bridges is often controlled by handling and shipping limitations.[1] In many regions of the United States, the maximum span and weight ranges are 120 to 150 ft (36 to 46 m) and 60 to 90 tons (54 to 82 t), respectively. Post-tensioning is an efficient method of field splicing of precast concrete I-beam segments to reach spans up to 300 ft (90 m). This can improve bridge economy by providing a structural concrete alternative to span levels that have been in the exclusive domain of structural steel plate girders.

Despite the enormous potential of post-tensioning in increasing the span length of I-beam bridges, several issues need to be resolved before post-tensioning can be implemented more widely. One of these issues is the lack of guidelines for the design and detailing of the post-tensioning anchorage zones.

The 15th Edition of the AASHTO Standard Specifications[2] required that the width of the anchorage zone be as wide as the narrower flanges of the I-beam and as long as three-quarters of the member depth. However, in the current 16th Edition of the AASHTO Standard Specifications[3] and 2nd Edition of the AASHTO LRFD Specifications,[4] the size of the anchorage zone dimensions is not specified. Instead, a somewhat vague statement is given: "...the transverse dimensions (of the anchorage zone) may be taken as the depth and width of the section." This gives the designer little guidance in determining the minimum required anchorage zone dimensions.

This lack of guidance has led designers to over-size the anchorage zone. It should be emphasized that larger concrete dimensions in the anchorage zone do not necessarily produce smaller stresses. They should not be viewed as structurally more conservative. In this paper, the proposed anchorage zone dimensions are compared to the more specific dimensions given in the 15th Edition of the AASHTO Standard Specifications.

Anchorage zones that are unnecessarily bulky and over-reinforced are uneconomical. More importantly, they may cause the precast, prestressed member to be too heavy to transport, thus defeating one of the primary purposes of beam splicing by post-tensioning. The authors have had experience with recently designed bridges where the anchorage zone weight alone was as much as 10 tons (9 t). When weight is critical, it can be a determining factor in the feasibility of the precast concrete alternate.

In the early stages of this investigation,[5] the finite element analysis indicated that increasing the concrete dimensions at the post-tensioning anchorage locations was not a structural design requirement. In other words, if the post-tensioning anchorage hardware were small enough to be fully housed in the web and flanges of the member, the reinforcement could be designed to satisfy the structural requirements.

The reality is that there is no special anchorage hardware and tensioning equipment on the market today that can fit in the 6.9 in. (175 mm) web of an I-beam. Therefore, it was decided to develop an anchorage zone with the smallest possible size that could allow housing of commercially available anchorage hardware.

The most critical stresses in the anchorage zones occur at the time of tendon jacking. Optimizing and standardizing the post-tensioning anchorage zone was based on this condition. Beyond the time of post-tensioning, concrete continues to gain strength and prestress continues to decrease. The study was limited to I-beams with up to three tendons consisting of 15 - 0.6 in. (15 mm) strands per tendon.

This is a relatively large level of post-tensioning and is not expected to be exceeded in most practical applications. Based on the analysis and experimental work, standard concrete dimensions and welded wire reinforcement details are presented in this paper. A detailed example is included in Appendix B to illustrate the design of the anchorage zone using the strut-and-tie model. The method can be used in applications that are not consistent with the assumptions used to develop the standard details.

FLOW OF FORCES IN ANCHORAGE ZONE

The analysis and design of the anchorage zone in post-tensioned members have been studied since the early days of the prestressed concrete industry. In the 1950s and 1960s, extensive research was performed on anchorage zones using a theory of elasticity analysis and small-scale anchor zone tests.[6,7] Later, Gergely and Sozen presented a method of analysis dealing with the effect of transverse reinforcement based on the equilibrium conditions of the cracked anchorage zone.[8]

These studies gave a basic understanding of the flow of forces in simple anchorage zone configurations. However, engineers have had difficulty in extrapolating these results to more complicated anchorage zone configurations. Therefore, the National Cooperative Highway Research Program (NCHRP) initiated a research project in which one of the main research results from that study was the division of the anchorage zone into a local zone and a general zone.[9] It was

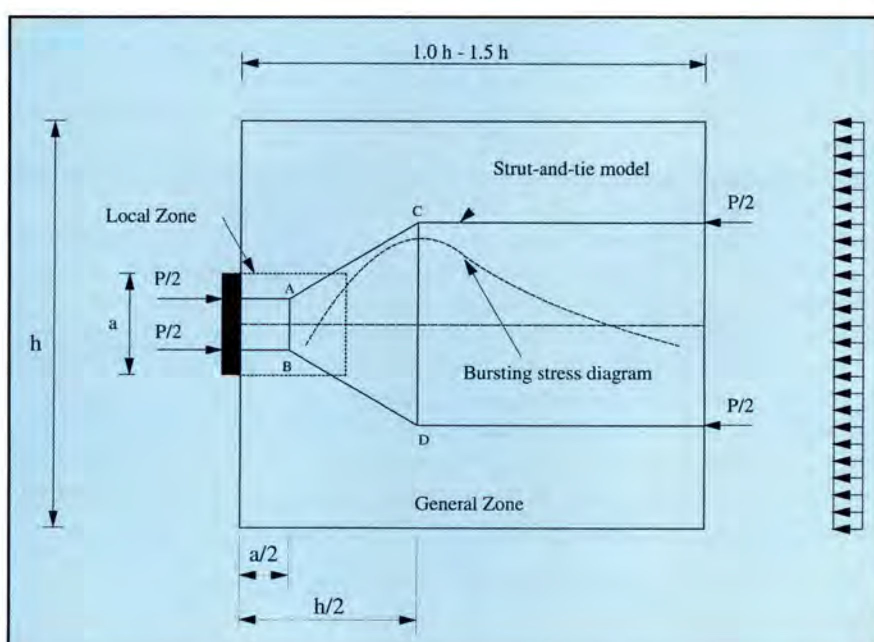

Fig. 1. Force flow of a simple anchorage zone.

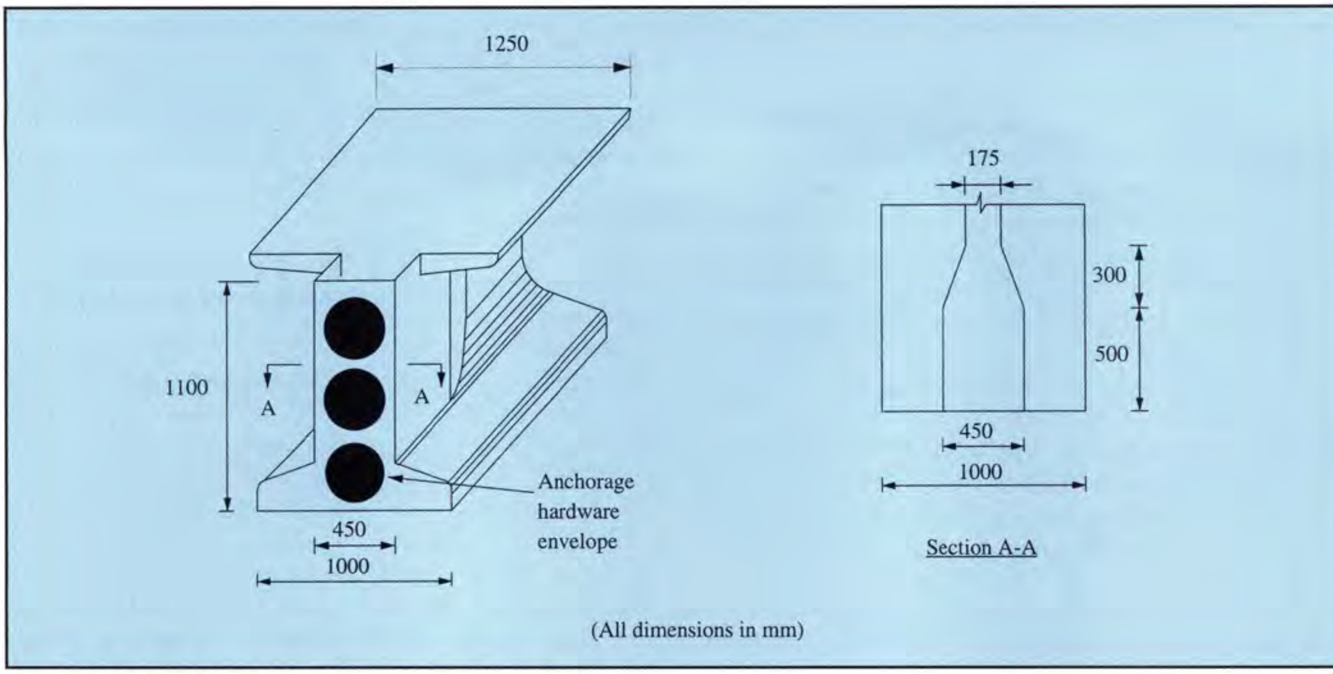

Fig. 2. Detail 1 — Optimized rectangular anchorage zone.

also concluded from that study that the design of the general zone can be based on a strut-and-tie model.

The strut-and-tie model is a general term used to represent the truss model, which was introduced by Ritter in 1899.[10] The model has been used as a conceptual tool in the design of reinforced concrete structures for the last 100 years. Over the years, the model has been modified by several investigators.[11-14]

It has been well understood that cracked reinforced concrete carries load mainly by developing a truss system represented by compressive stresses in the concrete and tensile stresses in the reinforcement. Furthermore, upon the occurrence of significant cracking, the originally curved principal stress trajectories in concrete tend toward straight lines. It is then appropriate to regard the resulting compressive forces as being carried by straight compressive struts. Therefore, the strut-and-tie model is capable of representing such stress flows after the concrete has cracked, and hence indicates a plausible force path for the concentrated post-tensioning force to flow from the anchorage hardware into the member.

Fig. 1 shows a simple strut-and-tie model that represents the force flow in a simple anchorage zone. In the figure, the truss member "AB" represents the compressive strut in the local zone and member "CD" represents the tension tie to resist the bursting force in the general zone. By using the simple strut-and-tie model, considerable insight into the flow of forces in the anchorage zones can be gained. However, the following points need to be emphasized when applying this model to the design of the anchorage zones:

First, it is apparent that if the location of the tension tie member is changed, the bursting force in the tie will be changed. Therefore, one of the keys in using the strut-and-tie model is to locate the tension tie member. Studies[6-9] have shown that a reasonable location of the tension tie is about half of the member height away from the anchorage face for simple rectangular anchorage configurations.

For complex I-beams with flanges, however, a greater degree of dispersion is required because a large percentage of the compressive force must find its way into the top and bottom flanges. It is considered conservative to take the bursting distance as half of the member height minus the eccentricity of the tendon group for this type of anchorage zone.

Secondly, the complicated local zone behavior, such as the compressive stress check of member "AB" in Fig. 1, can be separated from the general zone. In general, there are only a limited number of practical configurations of the local zone. Their behavior and design can be handled by a standard acceptance test procedure.[9] In design practice, an engineer who uses an anchorage device type that has passed the acceptance test does not need to check the bearing capacity, node compression capacity, and the node-strut interface capacity in the local zone. This separation makes it acceptable to apply the strut-and-tie model only to the design of the general zone.

Finally, the strut-and-tie model is an equilibrium-based model. It does not accurately model the forces that are needed to satisfy compatibility conditions. Thus, although the strut-and-tie method does not give a unique solution, its use generally produces conservative results. A detailed numerical example is given in Appendix B to demonstrate the design of the anchorage zone using the strut-and-tie model.

OPTIMIZING THE ANCHORAGE ZONE

During the early stages of this investigation, the study focused on the possibility of completely eliminating the increase of the web width at the post-tensioning anchorage locations. Based on a finite element analysis and simple rectangular anchorage zone testing, it was concluded that increasing the con-

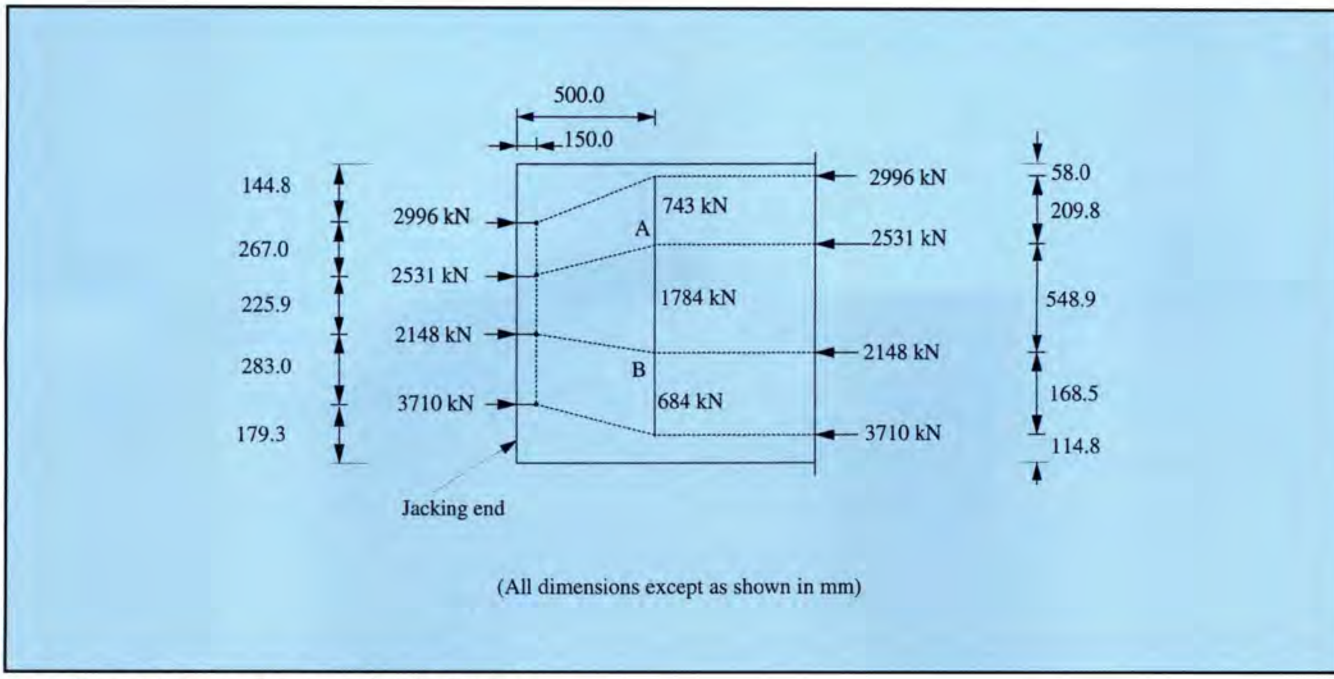

Fig. 3. Strut-and-tie model in beam height direction.

crete dimensions at the post-tensioning anchorage locations was not a structural design requirement.[5]

The problem, however, is that there is no special anchorage hardware in the market today that can fit in a 6.9 in. (175 mm) web of an I-beam. Also the production of new anchorage hardware, which can fit in the thin web, will require producing special jacking equipment to be used with it. This will add an initial cost to the system that might make it an unfavorable alternative. Therefore, it was considered a better alternative to develop an anchorage zone with the smallest possible size that would allow housing of commercially available anchorage hardware.

Detail 1 — Optimized Rectangular Anchorage Zone

In developing the Detail 1 anchorage zone, optimization was focused on minimizing the weight of the anchorage zone. Therefore, the minimum dimensions that can enclose all the available anchorage hardware in the market plus a reasonable concrete cover were chosen.

The 6.9 in. (175 mm) Nebraska University I-beam's web can accommodate a duct with a maximum diameter of 3.5 in. (88 mm). The duct can hold a 15 - 0.6 in. (15 mm) strand tendon. The anchorage hardware with this tendon was then used to estimate the boundaries of the anchorage zone. The resulting anchorage zone has a width of 17.7 in. (450 mm) that is uniform for a distance of 19.7 in. (500 mm) and then tapers for a distance of 11.8 in. (300 mm) to the standard NU I-beam web thickness of 6.9 in. (175 mm), as shown in Fig. 2.

The height of the anchorage zone can vary with the height of the NU I-beam. As an example, the Detail 1 anchorage zone for the NU 2000 I-beam weighs about 2 tons (1.8 t). If this anchorage zone was designed according to the 15th Edition of the AASHTO Specifications,[2] it would weigh about 10 tons (9 t). By comparison, the proposed Detail 1 anchorage zone can reduce the weight of the conventional anchorage zone by as much as 80 percent.

With the chosen concrete dimensions of the Detail 1 anchorage zone, reinforcement for the general zone under the three concentrated post-tensioning forces was designed using the strut-and-tie model. The web of the I-beam provides for the vertical distribution of the three concentrated jacking forces. The factored applied jacking forces were divided into four concentrated forces in the model with two forces flowing into flanges and two into the web, as shown in Fig. 3.

These four forces at the jacking end balance the resulting elastic force resultants at the other end (boundary section) of the anchorage zone. It should be noted that the stress distribution at the boundary section follows the conventional beam theory.

The length of the general zone was taken as one times the depth of the loaded beam, i.e., 3.6 ft (1100 mm). The eccentricity of the three tendons was found to be 2 in. (50 mm). Therefore, the location of the bursting tension tie was chosen as half of the beam depth minus the eccentricity, i.e., 19.7 in. (500 mm) away from the jacking end.

From Fig. 3, the maximum bursting force was calculated to be equal to 401 kips (1784 kN) from Node A and 400.9 kips (1782 kN) from Node B. The required area of reinforcement for the bursting force had been calculated as 6.68 sq in. (4312 mm^2). This reinforcement was then distributed over a length of approximately 3.6 ft (1100 mm). The actual reinforcement provided in the anchorage zone was 0.5 in. (12.7 mm) diameter (#4) reinforcing bar at a spacing of 2.0 in. (50 mm). The resulting area of reinforcement was 8.0 sq in. (5162 mm^2).

Then, the reinforcement in the top flange, bottom flange, and web was designed to allow for the transverse distribution of the three concentrated tendon forces. Strut-and-tie models are shown for the top flange, bottom

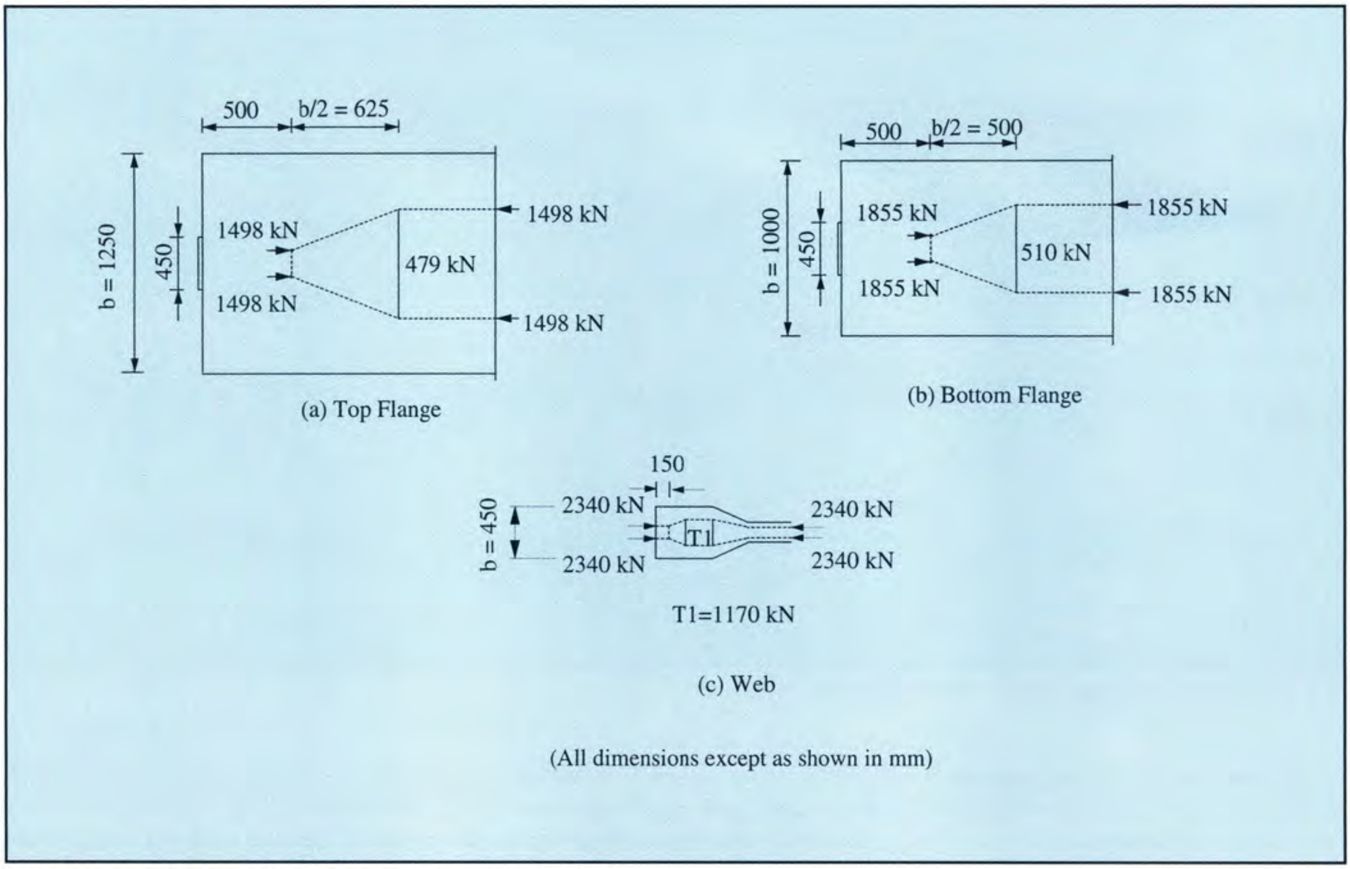

Fig. 4. Strut-and-tie models in transverse direction (Detail 1).

flange, and web in Figs. 4a, 4b, and 4c, respectively.

The required steel in the top flange was found to be 0.5 in. (12.7 mm) diameter (#4) reinforcing bar at a spacing of 6.1 in. (156 mm), which was smaller than the steel provided to resist the loading during deck placement. The steel required in the bottom flange was 0.5 in. (12.7 mm) diameter reinforcing bar at a spacing of 4.3 in. (110 mm), which was more than that normally provided for strand confinement. The actual steel provided for the top and bottom flange was the same, i.e., 0.5 in. (12.7 mm) diameter reinforcing bar at a spacing of 2.0 in. (50 mm).

Based on calculations, the required steel in the web was equal to 0.63 in. (16 mm) diameter (#5) reinforcing bar at a spacing of 2.5 in. (64 mm). Because there was not enough length in this direction to develop the 0.63 in. (16 mm) diameter reinforcement, threaded rods of 0.63 in. (16 mm) diameter with anchoring plates at the ends were used. The actual spacing of the rod was 2.0 in. (50 mm). Fig. 5 shows the completed general zone reinforcing cage.

Detail 2 — Optimized Tapered Anchorage Zone

During the optimization process of the Detail 2 anchorage zone, several other factors were considered besides the minimum weight requirement. First, the tapered part of the Detail 1 anchorage zone was extended to the end of the beam to allow for more space for the concrete consolidation in this area. Fig. 6 shows the comparison between the Detail 2 anchorage zone and Detail 1 anchorage zone.

The slope of the tapered anchorage zone was calculated based on the following conditions: (1) the width at the section 19.7 in. (500 mm) away from the jacking end was at least 17.7 in. (450 mm) to accommodate the available anchorage hardware; and (2) the chosen slope would result in minimum weight of the anchorage zone.

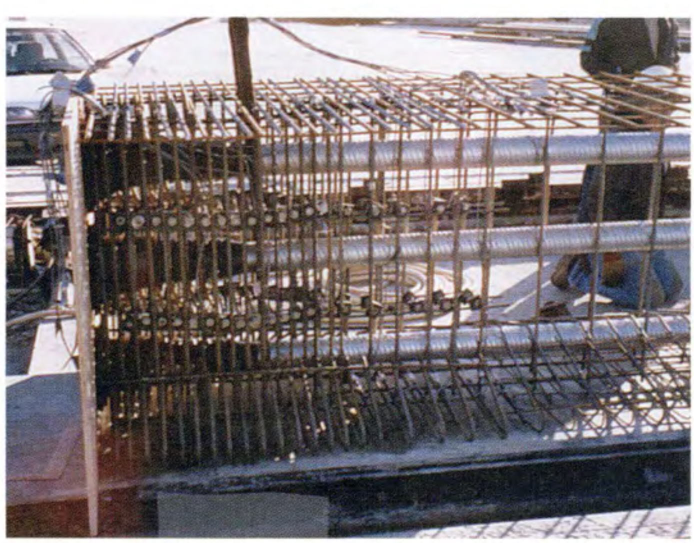

Fig. 5. The general zone reinforcing cage (Detail 1).

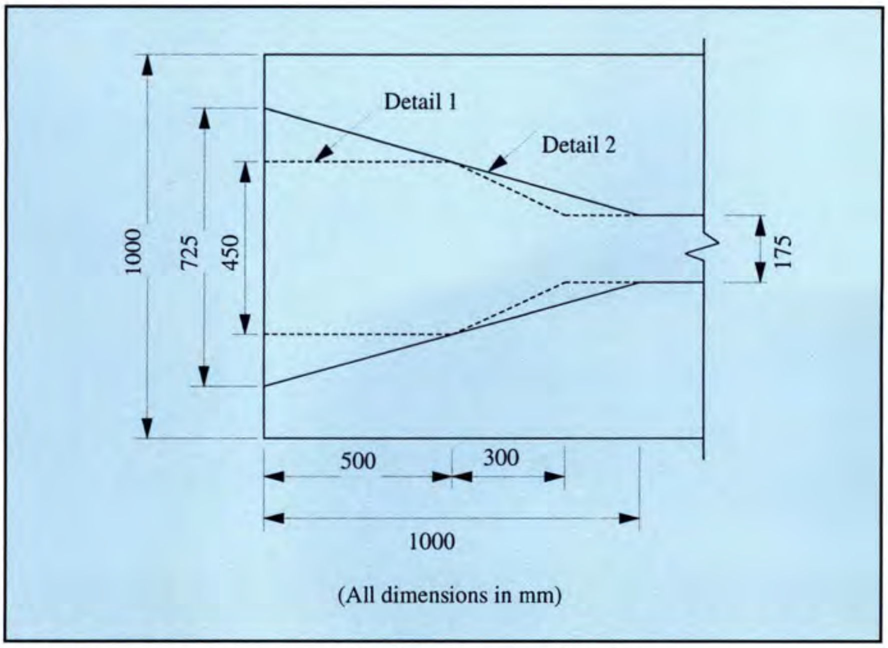

Fig. 6. Comparison between two details in transverse direction.

As a result, the Detail 2 anchorage zone has a width of 28.5 in. (725 mm) that tapers for a distance of 39.4 in. (1000 mm) to the standard NU beam web thickness of 6.9 in. (175 mm). The height of the anchorage zone can vary with the height of the I-beam.

Once the concrete dimensions of the Detail 2 anchorage zone were chosen, reinforcement was designed based on the strut-and-tie model similar to the design of the Detail 1 anchorage zone. However, the decision was made to change the threaded rod details to a prefabricated reinforcing bar welded cage to save production time. This cage has the same shape as the tapered anchorage zone in the transverse direction for easy placement, as shown in Fig. 7. The final design results are shown in Fig. 8.

INTEGRAL VS. SEPARATE ANCHORAGE ZONE

In applying the post-tensioning splicing technique, producers might be concerned about the possible modification of available standard prismatic steel forms. One of the objectives of this testing program was to explore how to minimize modifying the current constant section steel forms. Considering the fact that the current prismatic steel forms have a standard length of about 40 ft (12 m), there are two possible alternatives to produce post-tensioned concrete I-beams.

The first alternative is to cast both anchorage zones simultaneously with a prismatic I-beam piece. In this case, producers need not only to add steel forms for anchorage zones, but also to cut the standard 40 ft (12 m) long prismatic form to fit the designed length of the precast beams.

The other alternative is to cast one anchorage zone with the prismatic beam component. In this second alternative, producers only need to add a small steel form for the anchorage zone to the available standard prismatic steel forms. The other anchorage zone can be cast separately or using a longer beam segment with an anchorage segment cast integrally with it. The separately-cast precast components can then be post-tensioned together by match-casting or by a wet joint.

FULL-SCALE TEST

In contrast to the conventional laboratory testing in which specimens are loaded proportionally until failure, the

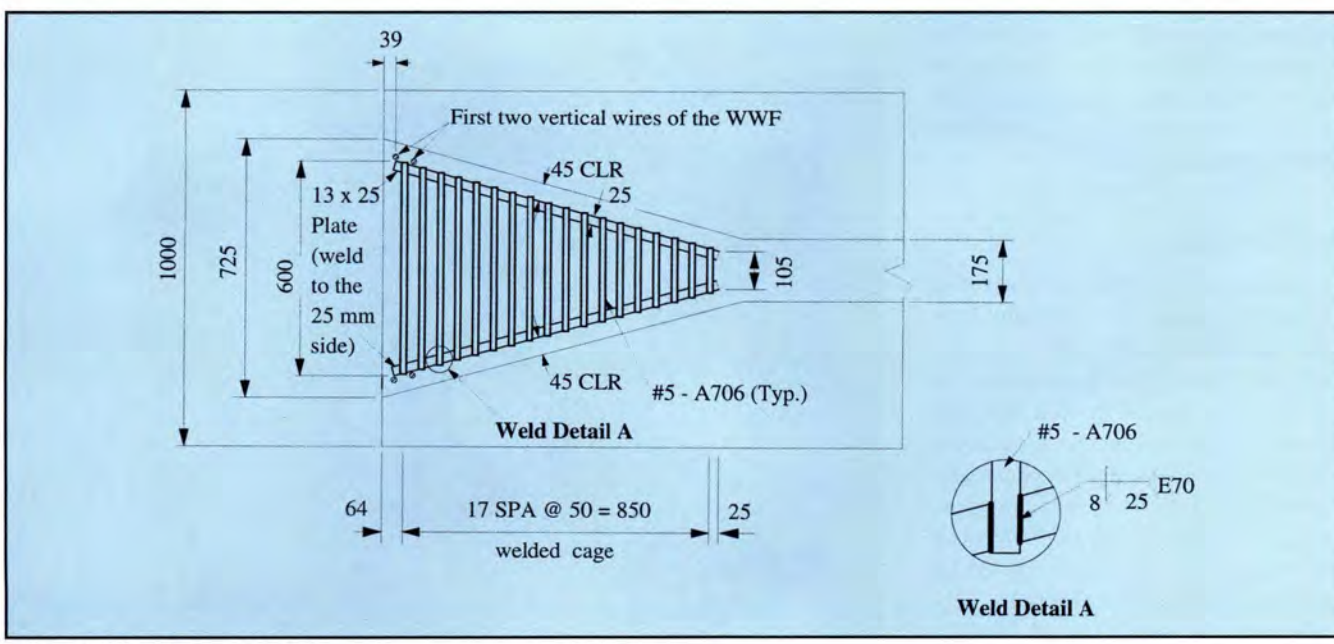

Fig. 7. Welded bar cage detail.

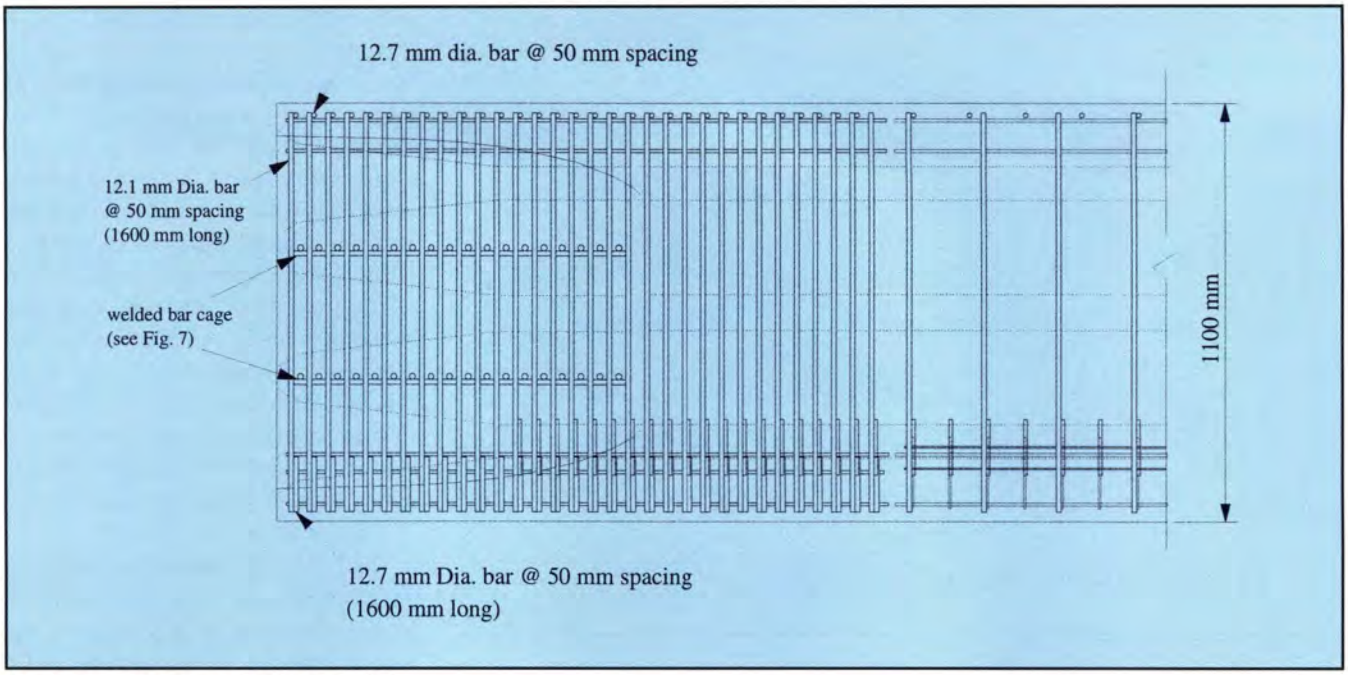

Fig. 8. Reinforcement details (Detail 2).

full-scale experimental program[15] in this research project was designed to simulate the field post-tensioning process. Before the full-scale specimens were produced, anchorage hardware available in the market was installed in the specimens. The post-tensioning technicians from that particular hardware supplier were invited to do the actual post-tensioning work.

The main objectives of the program were to study the behavior of the proposed anchorage zones under factored post-tensioning forces, to verify the adequacy of the design based on the strut-and-tie model and the overall performance of the local zone as well as the general zone, and to investigate casting the anchorage zone separately from the prismatic beam.

First Test Series

The first series of tests consists of two Detail 1 specimens (B1 and B2). Each specimen is 20 ft (6.1 m) long with two Detail 1 anchorage zones at the ends. Of the 20 ft (6.1 m) long specimen, each anchorage zone is 2.6 ft (0.8 m) in length and the remainder is 14.8 ft (4.5 m) long standard prismatic NU1100 I-beam segment. The anchorage hardware was provided by Dywidag Systems International.

The special steel form to produce the Detail 1 anchorage zone shape was first ordered. This form has the same length as the anchorage zone, i.e., 2.6 ft (0.8 m). Then, it was attached to the standard NU1100 prismatic steel form to make one end of the anchorage zone plus the 14.8 ft (4.5 m) prismatic I-beam segment. Finally, the other end of the 2.6 ft (0.8 m) anchorage zone was match-cast, as shown in Fig. 9.

Once the two pieces of the same specimen were ready to receive the post-tensioning force, the match-cast faces were lubricated with epoxy to reduce possible local stress concentration. This procedure proved to be satisfactory.

To account for the load factor of 1.2 in the actual post-tensioning stage, it was found that the tendon with 19 - 0.6 in. (15 mm) low-relaxation strands should be used. These tendons can give a load factor of 19/15 = 1.27 for the tendon with 15 - 0.6 in. (15 mm) strands used in the design of the optimized anchorage zone dimensions based on the AASHTO LRFD Specifications.[4]

Fig. 9. Match-cast joint surface.

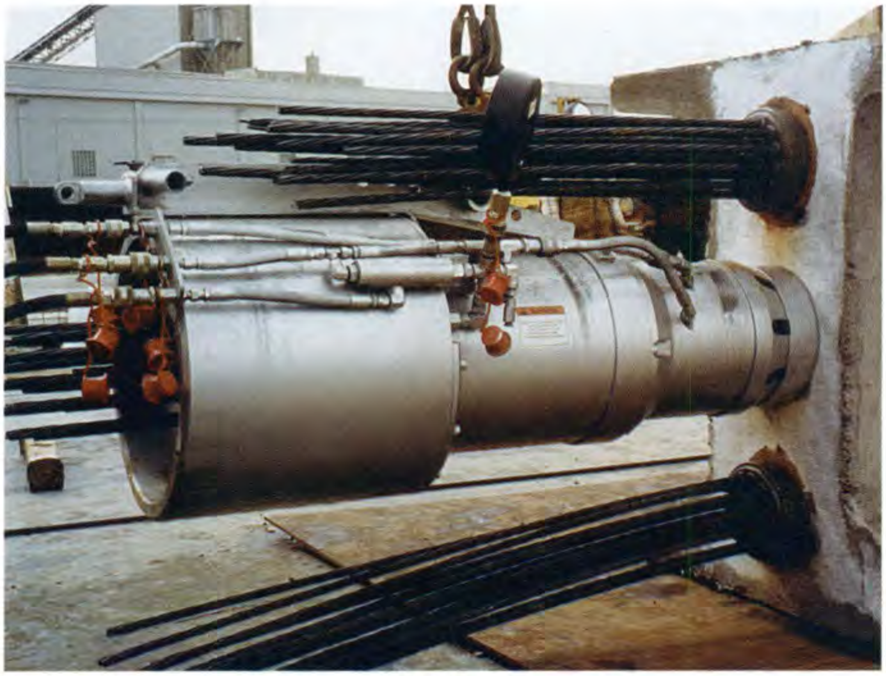

Fig. 10. Middle tendon during post-tensioning.

A specified concrete strength of 6000 psi (41.4 MPa) at the stage of post-tensioning and a 28-day strength of 8000 psi (55.2 MPa) were chosen for design calculations. The actual cylinder strength at the time of the post-tensioning, which was applied about two months after the specimens were produced, was 9600 psi (66.2 MPa) for Specimen B1 and 10,500 psi (72.4 MPa) for Specimen B2, respectively.

The reinforcement of the web, top flange and bottom flange in the center of the predicted bursting zone was instrumented to evaluate the behavior of the design model.

During the post-tensioning process, the actual post-tensioning sequence in the field was followed:

Step 1 — The middle tendon of the three designed tendons was post-tensioned to $0.85A_{ps}f_{pu}$ = 946.2 kips (4205 kN). This was equivalent to (0.85/0.81) (1.27) = 1.33 times the designed post-tensioning force level. Then, the possible concrete cracking in the anchorage zone was observed and marked. Fig. 10 shows the middle tendon during post-tensioning. No cracking was found.

Step 2 — The bottom tendon was post-tensioned to the same force level as the middle tendon.

Step 3 — The last top tendon close to the centroid of the composite section, which was designed to be post-tensioned after the deck was cast, was post-tensioned to the same force level. To keep the testing simple, the last tendon was post-tensioned on the non-composite specimen. This procedure was believed to be on the safe side, as compared to the procedure in which the third tendon was post-tensioned after the deck was cast.

According to this testing procedure, the non-composite specimen had actually received 1.33 times the designed maximum jacking force. This was believed to be similar to ultimate strength testing. For ultimate strength testing, only minor hairline cracks were found in the anchorage zone, as shown in Fig. 11. No signs of other distress were noticed. According to the strain gauge readings after the three tendons had been post-tensioned, all the reinforcement stress in the predicted bursting zone reached about the yield stress level. The performance of the specimens was believed to be satisfactory.

Second Test Series

The second series consists of two Detail 2 anchorage zone specimens (B3 and B4). Each specimen is again about 20 ft (6.1 m) long with two optimized tapered anchorage zones at the ends. The anchorage hardware was provided by CCS Special Structures.

The special steel form of 3.3 ft (1.0 m) in length to produce the Detail 2 anchorage zone shape was ordered. Fig. 12 shows the form and its attachment to the standard NU1100 prismatic steel form to make half of the 20 ft (6.1 m) long specimen. Once the other half was produced, the two pieces were connected together by a wet joint about 4 in. (102 mm) long at the middle of the specimen instead of the match-cast joint in the Detail 1 anchorage zone.

This change was made based on the fact that some producers may prefer the wet joint option. To make the wet

Fig. 11. Hairline cracks after post-tensioning (Detail 1).

Fig. 12. Steel form for tapered anchorage zone.

joint, the following procedure was used:

1. Prepare a rough joint surface. In the testing program, the "panel pad" (see Fig. 13A) was attached to the steel end plate to make the rough surface. Fig. 13B shows the roughened joint surface using the "panel pad."

2. Align two pieces of the same beam together. Care has to be taken to make the two pieces straight. The gap between the two pieces is recommended to be about 4.0 in. (102 mm).

3. Use a non-shrink high performance grout to pour the wet joint. The "Sure-Grip High Performance Grout" from the market was used in the testing program. A mechanical mixer with rotating blades was used. Up to 45 percent of washed pea gravel in a maximum size of $^3/_8$ in. (9.5 mm) was added to the grout mix. Testing has shown that this procedure works well.

For the transverse bursting force, tapered welded bar cages were used and produced by welding the bars to flat

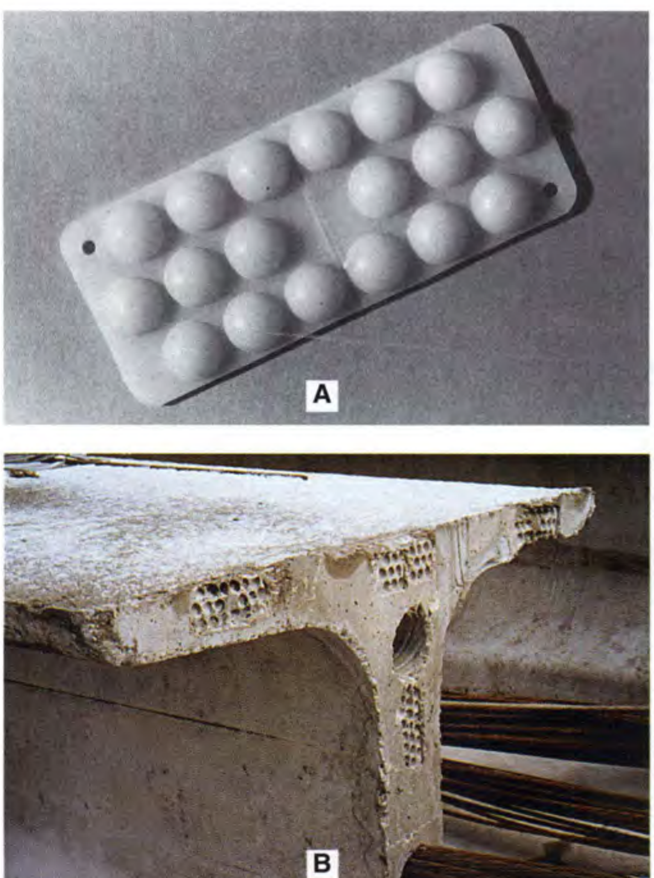

Fig. 13. The "panel pad" used to roughen the joint surface. (A) The "panel pad"; (B) Roughened wet joint surface.

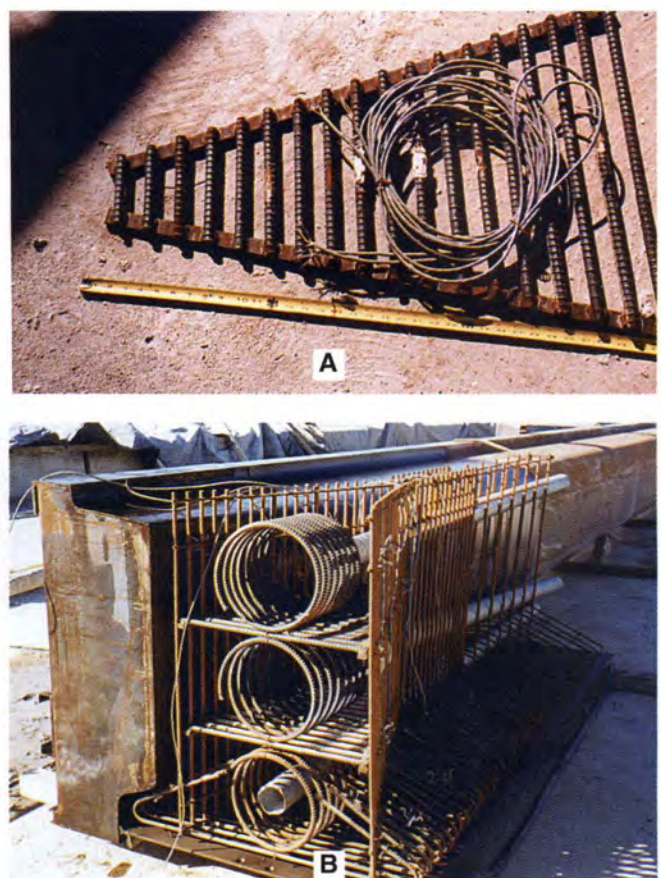

Fig. 14. Anchorage zone reinforcement details (Detail 2). (A) Tapered welded bar cage; (B) Reinforcement detail.

plates (see Fig. 14A). Two such cages were used in the anchorage zone, as shown in Fig. 14B.

To account for the load factor of 1.2 in the actual post-tensioning stage, the tendon with 19 - 0.6 in. (15 mm) low–relaxation strands was again used, as in the first series.

A specified concrete strength of 6000 psi (41.4 MPa) at the stage of post-tensioning and a 28-day strength of 8000 psi (55.2 MPa) were again chosen for the design calculations. However, the actual cylinder strength at the time of the post-tensioning was 10,170 psi (70.1 MPa) for Specimen B3 and 10,050 psi (69.3 MPa) for Specimen B4, respectively. Again, the reinforcement of the web, top flange and bottom flange in the center of the predicted bursting zone was instrumented.

As in the first test series, the actual post-tensioning sequence in the field was followed. However, the following changes were made as compared to the first series:

1. The tendon was only post-tensioned to $0.81A_{ps}f_{pu}$ = 901.7 kips (4010 kN). This is equivalent to 1.27 times the designed post-tensioning force level, which is smaller than the load level used in the first series but still larger than the ultimate load factor specified in the AASHTO LRFD Specifications.[4]

2. After the middle tendon, the top tendon was post-tensioned before the bottom tendon. This sequence was different from the first test series.

Not even minor cracks were found after the middle tendon was tensioned to the maximum load level in this test series. After the top tendon was post-tensioned to the load level, small minor cracks were found and no further cracking was found at the post-tensioning of the third tendon. As shown in Fig. 15, the minor cracks were limited within a much smaller range than the crack found in the first test series. No signs of other distress were observed.

It should be emphasized that minor cracking in the anchorage zone is quite normal and to be expected.[16] It has nothing to do with the onset of anchorage zone failure. In adequately reinforced tendon anchorage zones, such cracking is not necessarily critical, and there often is significant strength beyond the first appearance of cracks.

Fig. 15. Hairline cracks for Specimen B4.

According to the strain gauge readings, after the three tendons were post-tensioned, the maximum reinforcement stress in the predicted bursting zone had reached about 35 ksi (241.3 MPa), which is smaller than the stress level in the first test series. Based on the test results of this test series, the concrete dimensions of the Detail 2 anchorage zone, the reinforcing details used, the wet joint detail, and the local zone performance all proved to be adequate.

STANDARD ANCHORAGE ZONE DETAILS

Based on the full-scale NU1100 specimen test results, the two optimized anchorage zones have been proven to perform well. It is important to point out that the test results were based on specimens with a limited height of 43.3 in. (1100 mm). It is impractical to test all the specimens with different sizes. However, the tests on the NU1100 specimens have shown that the strut-and-tie model can give a good prediction on the bursting reinforcement requirement. Therefore, it was used as a design tool for the other I-beam sizes.

Although both Detail 1 and Detail 2 performed well in the full-scale tests, the optimized Detail 2 anchorage zone design was chosen as the standard concrete dimensions based on the following:

1. Minor cracks were limited to a smaller range in Detail 2 than in Detail 1 during the full-scale tests.

2. It is easier to consolidate the concrete near the anchorage hardware in Detail 2 than in Detail 1.

3. The smooth transition shape of Detail 2 makes the placement of transverse bursting reinforcement more convenient.

4. Detail 2 is slightly more aesthetically pleasing than Detail 1.

In the design, attention was focused on adopting standard anchorage details for different I-beam sections varying from NU1100 to NU2000. To this end, three 15 - 0.6 in. (15 mm) diameter low-relaxation strand tendons were chosen for all the sections. The different force flow at the anchorage zone due to the changing of the section height was adjusted by extending the vertical bursting reinforcement.

Table 1 summarizes the bursting reinforcement required in the anchorage zone for different NU I-beam sections. Based on the strut-and-tie model design, the following conclusions can be made:

1. The standard tapered anchorage zone (Detail 2) with the same concrete dimensions can be used for all the I-beam sections, as shown in Fig. 16.

2. As shown in Fig. 17, the same reinforcement details in the top flange, web, and bottom flange with the exception of the vertical web reinforcement can be used for all sizes of I-beams.

3. In the vertical direction, the reinforcement size, D18, and spacing of 2 in. (50 mm) can be used for all beam sizes in the NU I-beam series, which ranges from 43.3 to 78.7 in. (1100 to 2000 mm) deep. However, the extent of this reinforcement size and spacing into the beam depends on the beam depth, as shown in Table 1.

IMPACT OF SHEAR

One important issue may be that the anchorage zone might need more reinforcement to resist the high shear force at the member ends. Considering the possible reinforcement congestion in this zone, it is very important to investigate the impact the high shear force would have on this region. Because the NU2000 I-beam section has the longest anchorage zone, the impact of shear was studied for this section using the following two examples.

Example 1

The first example is a single span of 155 ft (47.2 m). NU2000 I-beams are spaced at 8.0 ft (2.4 m). Concrete strength is 8000 psi (55.2 MPa) at service and 6000 psi (41.4 MPa) at the time of post-tensioning. HS-25 truck loading is used. Deck thickness is 7.5 in. (191 mm) with a concrete strength of 5000 psi (34.5 MPa). Based on the AASHTO Standard Specifications,[3] the required shear reinforcement at the critical section is 0.42 sq in./ft (0.89 mm²/mm).

From Table 1, the required transverse reinforcement at the section to resist the bursting post-tensioning force is 11.48 sq in. (7406.4 mm²)/72 in. (1828.8 mm) = 1.91 sq in./ft (4.05 mm²/mm). If the prestress loss of the post-tensioning tendons is assumed to be 20 percent, then the actual required bursting reinforcement at service is 1.91 sq in./ft (4.05 mm²/mm) (1 – 0.2) = 1.53 sq in./ft (3.24 mm²/mm).

The actual reinforcement provided at this section is 14.17 sq in./ft (9141.9 mm²/mm)/78.7 in. (2000.0 mm) = 2.16 sq in./ft (4.57 mm²/mm), which is greater than 0.42 sq in./ft (0.89 mm²/mm) + 1.53 sq in./ft (3.24 mm²/mm) = 1.95 sq in./ft (4.13 mm²/mm). Therefore, the reinforcement provided at the anchorage zone

Table 1. Post-tensioning anchorage reinforcement for NU I-beams.

Action	Beam size	Force		Required reinforcement				Provided reinforcement			
				Area		Distance		Area		Distance	
		kips	kN	sq in.	mm²	in.	mm	sq in.	mm²	in.	mm
Transverse bursting* (WWF1)	NU 1100	401.4	1784.0	6.69	4316.1	40.0	1016.0	11.34	7316.1	63.0	1600.0
	NU 1350	559.5	2486.7	9.33	6019.3	52.0	1320.8	11.34	7316.1	63.0	1600.0
	NU 1600	633.2	2814.2	10.55	6806.4	60.0	1524.0	11.34	7316.1	63.0	1600.0
	NU 1800	665.3	2956.9	11.09	7154.8	64.0	1625.6	12.76	8232.2	70.9	1800.0
	NU 2000	688.8	3061.3	11.48	7406.4	72.0	1828.8	14.17	9141.9	78.7	2000.0
Top flange bursting in thin direction (WWF6)	NU 1100	107.7	478.7	1.80	1161.3	48.0	1219.2	6.30	4064.5	63.0	1600.0
	NU 1350	99.2	440.9	1.65	1064.5	48.0	1219.2	6.30	4064.5	63.0	1600.0
	NU 1600	91.5	406.7	1.52	980.6	48.0	1219.2	6.30	4064.5	63.0	1600.0
	NU 1800	85.7	380.9	1.43	922.6	48.0	1219.2	7.09	4574.2	70.9	1800.0
	NU 2000	80.7	358.7	1.35	871.0	48.0	1219.2	7.87	5077.4	78.7	2000.0
Web bursting in thin direction (WBC)	NU 1100	263.2	1169.8	4.39	2832.3	18.0	457.2	10.97	7077.4	35.4	900.0
	NU 1350	296.0	1315.6	4.93	3180.6	18.0	457.2	10.97	7077.4	35.4	900.0
	NU 1600	323.4	1437.3	5.39	3477.4	18.0	457.2	10.97	7077.4	35.4	900.0
	NU 1800	342.2	1520.9	5.70	3677.4	18.0	457.2	10.97	7077.4	35.4	900.0
	NU 2000	358.9	1595.1	5.98	3858.1	18.0	457.2	10.97	7077.4	35.4	900.0
Bottom bursting in thin direction (WBF1)	NU 1100	114.9	510.7	1.91	1232.3	40.0	1016.0	6.30	4064.5	63.0	1600.0
	NU 1350	104.3	463.6	1.74	1122.6	40.0	1016.0	6.30	4064.5	63.0	1600.0
	NU 1600	95.9	426.2	1.60	1032.3	40.0	1016.0	6.30	4064.5	63.0	1600.0
	NU 1800	90.4	401.8	1.51	974.2	40.0	1016.0	7.09	4574.2	70.9	1800.0
	NU 2000	85.5	380.0	1.42	916.1	40.0	1016.0	7.87	5077.4	78.7	2000.0

* Reinforcement was designed to resist post-tensioning in combination with vertical shear.
WWF1, WWF6, WBC and WBF1 are shown in Fig. 17.

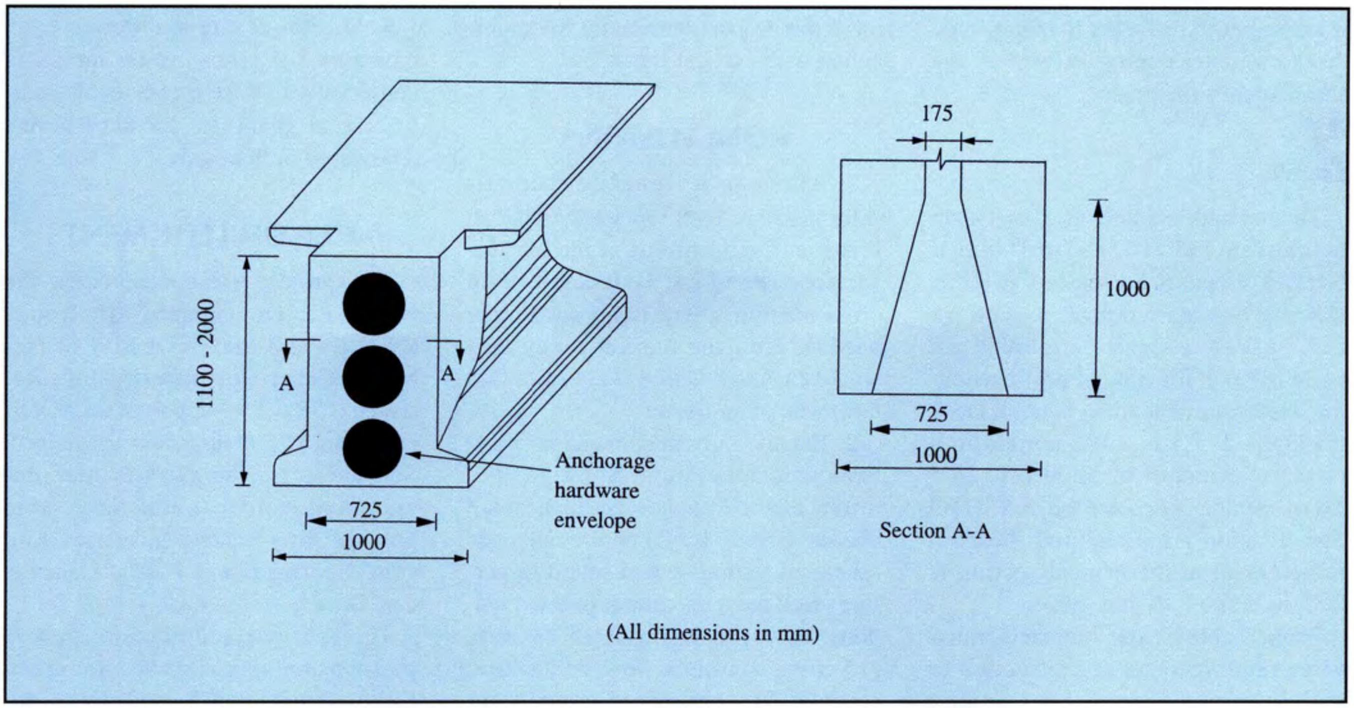

Fig. 16. Standard NU I-beam post-tensioning anchorage (concrete dimension).

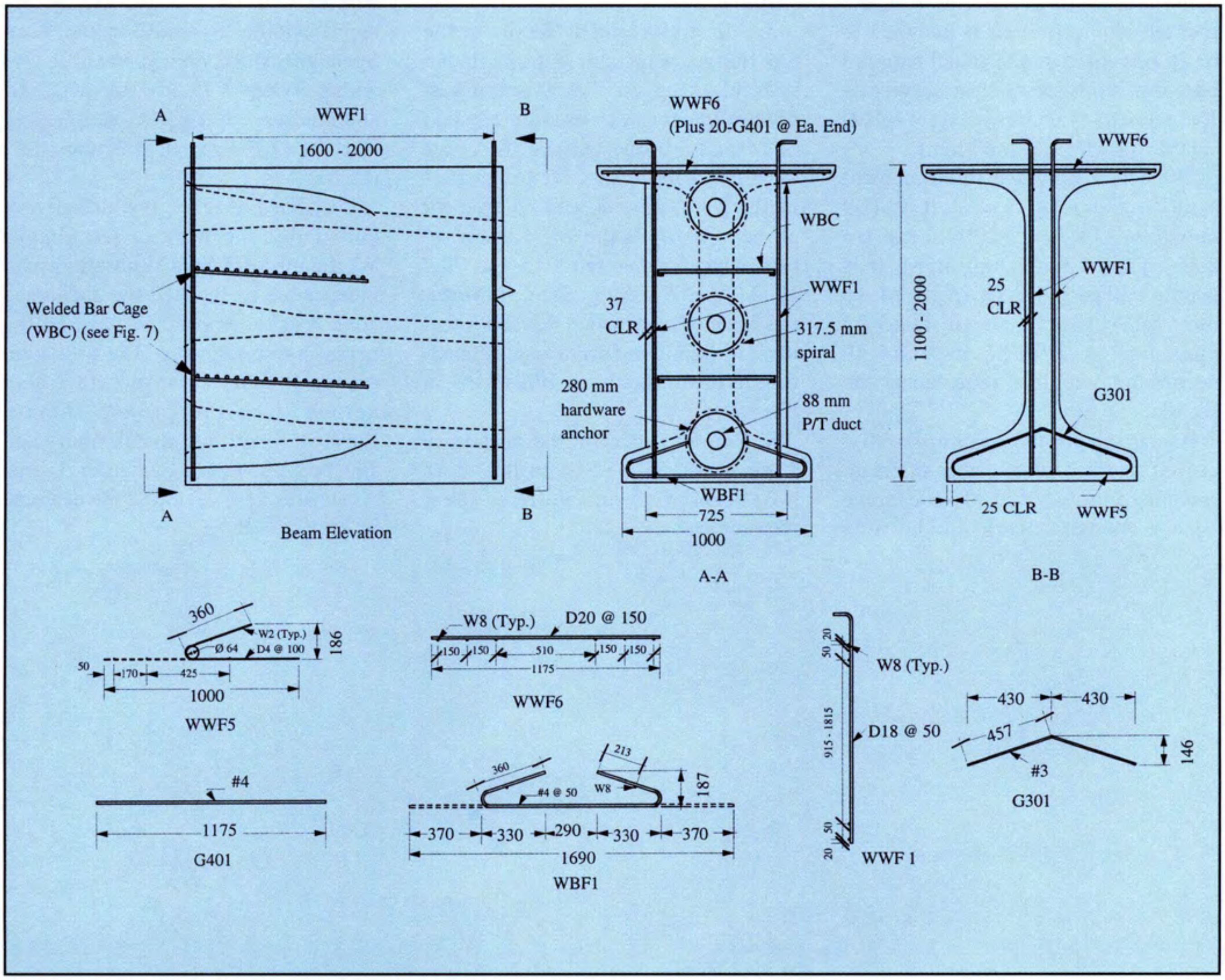

Fig. 17. Standard NU I-beam post-tensioning anchorage (reinforcement).

is sufficient to resist the bursting force due to post-tensioning as well as the shear force at ultimate.

Example 2

The second example is a two-span bridge of 165 ft (50.3 m) spans. NU2000 I-beams are spaced at 8.0 ft (2.4 m). Concrete strength is 8000 psi (55.2 MPa) at service and 6000 psi (41.4 MPa) at the time of post-tensioning. HS-25 truck loading is used. Deck thickness is 7.5 in. (191 mm) with a concrete strength of 5000 psi (34.5 MPa). Again, based on the AASHTO Specifications,[3] the required shear reinforcement at the critical section is 0.55 sq in./ft (1.16 mm^2/mm).

From Table 1, the required transverse reinforcement at the section to resist the bursting post-tensioning force is 11.48 sq in. (7406.4 mm^2)/72 in. (1828.8 mm) = 1.91 sq in./ft (4.05 mm^2/mm). If the prestress loss of the post-tensioning tendons is assumed to be 20 percent, then the actual required bursting reinforcement at service is 1.91 sq in./ft (4.05 mm^2/mm) (1 – 0.2) = 1.53 sq in./ft (3.24 mm^2/mm).

The actual provided reinforcement at this section is 14.17 sq in./ft (9141.9 mm^2/mm)/78.7 in. (2000.0 mm) = 2.16 sq in./ft (4.57 mm^2/mm). It is greater than 0.55 sq in./ft (1.16 mm^2/mm) + 1.53 sq in./ft (3.24 mm^2/mm) = 2.08 sq in./ft (4.40 mm^2/mm), which is required at the section.

Based on the two examples discussed, it is concluded that the reinforcement provided at the anchorage zone is enough to resist the bursting force due to post-tensioning in combination with vertical shear.

CONCLUSIONS

1. Although it would be theoretically possible to eliminate the change in concrete dimensions at the anchorage zone and to use a constant I-beam cross section, there is no anchorage hardware on the market today that would fit into a 7 to 8 in. (178 to 203 mm) web of an I-beam.

2. For the currently available post-tensioning hardware systems, an optimized tapered anchorage zone detail was developed. Based on the full-scale specimen testing, it was found to perform well for a maximum of three tendons, each consisting of 15 - 0.6 in. (15 mm) diameter low-relaxation strands. The optimized tapered anchorage zone dimensions are applicable for all currently existing I-beam shapes and sizes.

3. All reinforcement details in the top flange, web, and bottom flange with the exception of the vertical web reinforcement can be used for all sizes of I-beams. In the vertical direction, the reinforcement size, D18, and spacing of 2 in. (50 mm) can be used for all beam sizes in the NU I-beam series, which ranges from 43.3 to 78.7 in. (1100 to 2000 mm) deep. However, the distance over which this reinforcement is used in the beam ends depends on the beam depth, as illustrated in this paper.

4. Reinforcement in the anchorage zone was designed to satisfy both post-tensioning and vertical shear requirements.

5. Design of the anchorage zone based on the strut-and-tie model is conservative. It gave reasonable reinforcement quantities for the I-beams considered in this study.

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APPENDIX A — NOTATION

a = width of anchorage plate
A = area of noncomposite gross section
A_{ps} = total area of strands per tendon
b = width of transverse direction
F_i = node force in ith node
f'_{ci} = specified concrete strength at time of post-tensioning
f_{pu} = specified tensile strength of prestressing strands
h = overall depth of beam
h_b = equivalent bottom flange height
h_t = equivalent top flange height
h_w = equivalent web height
I = moment of inertia of noncomposite gross section
P = maximum jacking force per tendon
P_T = total factored tendon force in horizontal direction
P_u = factored tendon force per tendon
R = reaction force
S_b = noncomposite section modulus for extreme fiber of section where tensile stress is caused by externally applied loads
S_t = noncomposite section modulus for extreme fiber of section where compressive stress is caused by externally applied loads
V = shear force
w_1 = beam self-weight
w_2 = deck weight
y_b = distance from neutral axis to extreme tension fiber
y_{bottom} = location of resultant tendon force in bottom flange
y_t = distance from neutral axis to extreme compression fiber
y_{top} = location of resultant tendon force in top flange
y_{web} = location of resultant tendon force in web
α = angle of tendon relative to longitudinal axis
ϕ = resistance factor

APPENDIX B — DESIGN OF POST-TENSIONED ANCHORAGE ZONE

Design of the post-tensioned anchorage zone based on the AASHTO LRFD Specifications[4] by the strut-and-tie model is illustrated using the following example. The example is for a two-span continuous bridge constructed with AASHTO Type IV I-beams. The spans are each 122 ft (37.2 m) long. The beams are spaced at 10.7 ft (3.25 m) and have 28 - 1/2 in. (12.7 mm) low-relaxation pretensioning strands and three post-tensioning tendons of 12 - 0.6 in. (15 mm) strands.

In this example, the anchorage zone will be designed to resist three post-tensioning tendons of 15 - 0.6 in. (15 mm) strands. The effect of the pretensioning strands is ignored. Fig. B1 shows the beam details near the anchorage zone. All post-tensioning forces are assumed to be applied to the non-composite precast beams. The gross section properties at Section B-B in Fig. B1 are as follows:

A = 789 sq in. (0.509 m^2)
y_b = 24.73 in. (0.63 m)
y_t = 29.27 in. (0.74 m)
I = 260,741 in.^{4} (0.109 m^4)
S_b = 10,543 cu in. (0.173 m^3)
S_t = 8908 cu in. (0.146 m^3)

In this example, the DSI Multiplane Anchorage (MA) is used. The anchorage size is 15 - 0.6 in. (15 mm) which can accommodate 15 - 0.6 in. (15 mm) GR 270 strands. The maximum jacking force is $P = 0.9\,A_{ps}f_{py} = 0.81A_{ps}f_{pu}$ = 712 kips (3164 kN) (AASHTO LRFD Article 5.9.3), where A_{ps} is the total area of strands per tendon, f_{py} is the yield strength of prestressing strands, and f_{pu} is the specified tensile strength of prestressing strands.

According to AASHTO LRFD Article 3.4.3, the design force for post-tensioning anchorage zones is taken as 1.2 times the maximum jacking force. The resistance factor ϕ is taken as 0.80 for compression in anchorage zones and 1.00 for tension in steel in anchorage zones (AASHTO LRFD Article 5.5.4.2.1). Hence, the tendon force used in design is $P_u = 1.2P$ = 854 kips (3797 kN). The uniform dead load from the deck and the beam bearing reaction tend to reduce the bursting force. For this reason, a load factor of 1.0 is used for these loads.

The anchor plate size is taken approximately as the spiral size plus the concrete cover. According to the hardware supplier (DSI), the outside diameter of the spiral is 12.5 in. (318 mm). Since the spiral is round, it is easier to convert the spiral into an equivalent square to compare it with rectangular struts. Therefore, the width of plate can be taken as a = 12.0 in. (305 mm).

Fig. B1. Example anchorage zone.

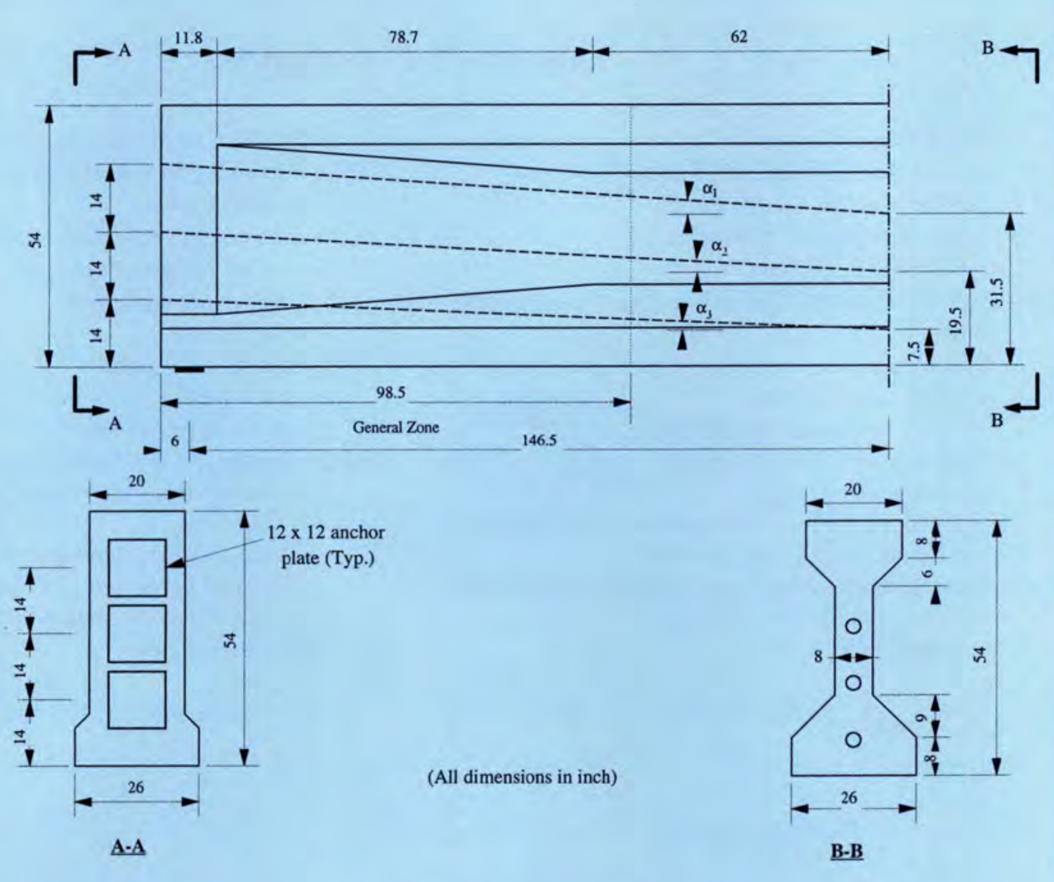


Fig. B1. Example anchorage zone.

Determine Extent of General Zone

There are two types of discontinuities in the end region of the beam, which disturb the stress distribution based on the beam theory: (1) loading discontinuities, such as post-tensioning force and reaction force; and (2) geometric discontinuity.

According to AASHTO LRFD Article 5.10.9.1, the longitudinal extent of the anchorage zone in the direction of the tendon is to be not less than the greater of the transverse dimensions of the anchorage zone and not more than one and one-half times that dimension.

In this example, the height of the noncomposite beam is 54 in. (1.4 m). Therefore, the length of the general zone is not to exceed 1.5 x 54 in. (1.5 x 1.4 m) = 81 in. (2.1 m). However, considering the geometrical discontinuity, the boundary of the general zone will be assumed to be located one web width ahead of the end of the change in web geometry. As a result, the extent of the general zone is taken as 11.8 in. (0.3 m) + 78.7 in. (2.0 m) + 8.0 in. (0.2 m) = 98.5 in. (2.5 m) as shown in Fig. B1.

Determine Stress Distribution at End of General Zone

The locations of the three tendons are shown in Fig. B1. From this figure, the calculated angles of the three tendons relative to the longitudinal axis are $\alpha_1 = 3.94°$, $\alpha_2 = 3.19°$, and $\alpha_3 = 2.44°$. At the end of the general zone, the center of gravity of the three tendons is 22.5 in. (572 mm), and the eccentricity is 2.23 in. (56.6 mm).

The total factored design tendon force in the horizontal direction is $P_T = P_u(\cos\alpha_1 + \cos\alpha_2 + \cos\alpha_3) =$ 2559 kips (11372 kN). Thus, the bot-

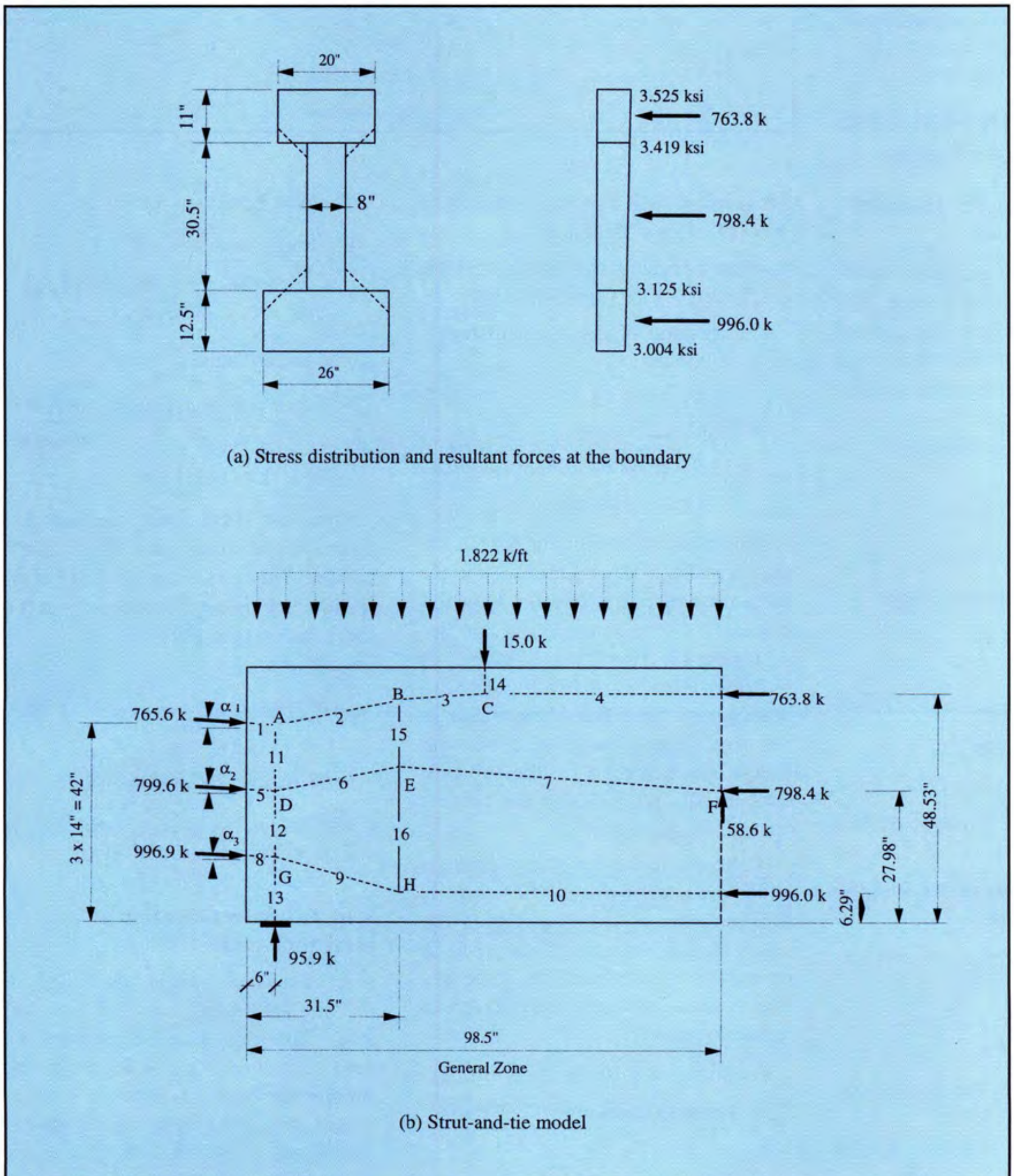


Fig. B2. Strut-and-tie model. (a) Stress distribution and resultant forces at boundary; (b) Strut-and-tie model.

tom fiber and top fiber stresses at the end of the general zone due to this tendon force are 3.784 and 2.602 ksi (26.1 and 17.9 MPa) respectively.

After calculating the stresses due to beam self-weight (w_1 = 0.822 kip/ft or 12.0 kN/m) and deck weight (w_2 = 1.000 kip/ft or 14.6 kN/m), the total bottom fiber and top fiber stresses at the end of the general zone are 3.004 and 3.525 ksi (20.7 and 24.30 MPa) respectively, as shown in Fig. B2 (a).

The reaction force at the support near the jacking end is 50.1 kips (222.8 kN) due to beam self-weight. The reaction force due to deck weight on the two-span continuous beams is 45.8 kips (203.3 kN). Thus, the total reaction force at the support near the jacking end is R = 95.9 kips (426.1 kN).

Table B1. Member forces in the strut-and-tie model.

Member	Force, kips (kN)	By which node balance?
1	765.6 (3403)	Known
2	788.6 (3505)	Node B
3	763.9 (3395)	Node C
4	763.8 (3395)	Known
5	799.6 (3554)	Known
6	800.8 (3559)	Node D
7	800.5 (3558)	Node F
8	996.9 (4431)	Known
9	1040.5 (4624)	Node H
10	996.0 (4427)	Known
11	248.7 (1105)	Node A
12	354.7 (1576)	Node G
13	95.9 (426)	Known
14	15.0 (67)	Known
15	-181.1 (-805)(Tension)	Node B
16	-301.2 (-1339)(Tension)	Node H

Draw Strut-and-Tie Model and Determine Member Forces

Step 1 — Calculate force resultant at end of general zone

When considering possible strut-and-tie model of the general zone, it is easier to start at the end of the general zone. The top and bottom flanges of the beam are transferred into equivalent rectangular shapes:

Equivalent top flange height:

$$h_t = 8 + \frac{2 \times (0.5 \times 6 \times 6)}{20 - 8}$$
$$= 11.0 \text{ in. } (279.4 \text{ mm})$$

Equivalent bottom flange height:

$$h_b = 8 + \frac{2 \times (0.5 \times 9 \times 9)}{26 - 8}$$
$$= 12.5 \text{ in. } (317.5 \text{ mm})$$

The height of the web:

$h_w = 54.0 - h_t - h_b = 30.5$ in. (774.7 mm)

The stress at the bottom of the equivalent top flange is:

3.004 + (3.525 – 3.004)(12.5 + 30.5)/54 = 3.419 ksi (23.57 MPa)

The stress at the top of the equivalent bottom flange is:

3.004 + (3.525 – 3.004)(12.5)/54 = 3.125 ksi (21.55 MPa)

Resultant forces at the end of the general zone are as follows:

At top flange = 0.5 (3.525 + 3.419) (11) (20) = 763.8 kips (3395 kN)

At web = 0.5 (3.419 + 3.125) (30.5) (8) = 798.4 kips (3548 kN)

At bottom flange = 0.5 (3.125 + 3.004) (12.5) (26) = 996.0 kips (4427 kN)

The location of the resultant forces are as following:

y_{top} = 12.5 + 30.5 + 5.53 = 48.53 in. (1233 mm)

y_{web} = 12.5 + 15.48 = 27.98 in. (711 mm)

y_{bottom} = 6.29 in. (160 mm)

The calculated resultant forces are shown in Fig. B2(a). Once all the resultants and their locations on the boundary of the general zone are determined, the final "destinations" of the post-tensioning force paths are determined. The force resultants in the top flange, web, and bottom flange are balanced by the top tendon, middle tendon, and the bottom tendon, respectively.

If the compressive stress directly under the anchor plate, whose size is the diameter of the spiral plus the cover in this example, is assumed to be uniformly distributed, the three tendon forces can be re-calculated to maintain equilibrium.

The top tendon force:

$$P_u^1 = \frac{\text{Force in top flange}}{\cos\alpha_1} = \frac{763}{\cos 3.94°}$$
$$= 765.6 \text{ kips } (3403 \text{ kN})$$

The middle tendon force:

$$P_u^2 = \frac{\text{Force in web}}{\cos\alpha_2} = \frac{798.4}{\cos 3.19°}$$
$$= 799.6 \text{ kips } (3554 \text{ kN})$$

The bottom tendon force:

$$P_u^3 = \frac{\text{Force in bottom flange}}{\cos\alpha_3} = \frac{996.0}{\cos 2.44°}$$
$$= 996.0 \text{ kips } (4431 \text{ kN})$$

Resultant shear force could be assigned based on the shear stress distribution, but it is simpler and sufficiently accurate to assign all of the shear force to the web. That is:

Shear force:

$$V = P_u^1 \sin\alpha_1 + P_u^2 \sin\alpha_2 + (98.5/12)(w_1 + w_2) - R$$
$$= 58.6 \text{ kips } (260.0 \text{ kN})$$

where, w_1 and w_2 are beam self-weight and deck weight, respectively.

Step 2 — Select location of local zone node

As discussed earlier, the width of the anchor plate a = 12.0 in. (305 mm). The closer to the anchors the local zone nodes are located, the smaller the bursting force. In this example, the local zone nodes are selected at: $(a/2)$ = 6.0 in. (152 mm) ahead of the anchor bearing plates.

Step 3 — Select location of bursting tie

For the bursting reinforcement, a uniform arrangement in the general zone is envisioned. To be on the conservative side, however, the calculation will be based on the uniformly distributed reinforcement for a distance of beam height.

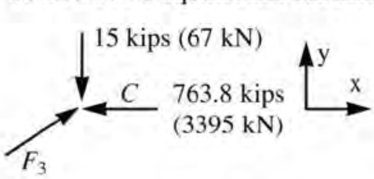
Assume the right edge of the bearing plate to the end face of the beam is 9 in. (229 mm). The beam height is 54 in. (1372 mm). Then, the location of the bursting tie is taken as 9 + (54 – 9)/2 = 31.5 in. (800 mm) away from the end face of the beam. The final calculated reinforcement spacing within the range of (54 – 9) = 45 in. (1143 mm) will be used for the entire general zone.

Step 4 — Draw strut-and-tie model and calculate member force

Based on the above steps, the strut-and-tie model can be built as shown in Fig. B2(b). As shown in Fig. B2(b), there are eight named nodes (i.e., A through H) and sixteen members (i.e., 1 through 16) in the strut-and-tie model. The forces in members 1, 4, 5, 8, 10, 13, and 14 can be observed from the model directly. The other member forces can be calculated from node force equilibrium condition.

If F_i is the force in the ith member, then for example: F_1 = 766 kips (3403 kN).

From Node C equilibrium condition:



F_{3x} = 763.8 kips (3395 kN)

F_{3y} = 15.0 kips (67 kN)

$F_3 = \sqrt{\left(F_{3x}^2 + F_{3y}^2\right)}$ = 764 kips (3395 kN)

Note that at Node C, the 15 kips (67 kN) load represents the beam self-weight and deck weight for the general zone. The slope of the Member 3 can also be determined from Node C's force equilibrium condition.

Similarly, all member forces can be determined as shown in Table B1.

From Table B1, it can be found that all nodes are used to calculate member forces except Node E. Therefore, this node can be checked to verify the model accuracy.

$$\sum F_x = F_{6x} - F_{7x} = 0 \qquad \text{(ok)}$$
$$\sum F_y = F_{6y} + F_{7y} + F_{16} = 0$$

Similar steps can be used to draw a strut-and-tie model in the transverse direction. Then the transverse bursting forces can also be calculated.

Check Compression Stresses

Compression stresses may be critical immediately ahead of the anchor plates (bearing pressure), immediately outside the locally confined region (i.e., local zone – general zone interface), and the node compression strut capacity perpendicular to the tendon path. However, the bearing pressure and node compression strut capacity are considered the local zone check. They are guaranteed by the anchorage device supplier through standard acceptance test procedures.

From Fig. B2, the maximum compression stress at the boundary is 3.525 ksi (24.30 MPa). Therefore, the minimum required concrete strength at the time of post-tensioning is (AASHTO LRFD Article 5.10.9.3.1):

$$f'_{ci} = \frac{3.525}{0.7\phi} = 6.295 \text{ ksi (43.4 MPa)}$$

For the local zone - general zone interface capacity, the following simplified method is used. From Fig. B1, the anchor plate size is 12 x 12 in. (305 x 305 mm). From hardware supplier (DSI), the depth of the local zone is 14.75 in. (375 mm). The physical width at the end of the local zone is therefore:

$$8 + (20 - 8)\frac{78.7 - (14.75 - 11.8)}{78.7}$$
$$= 19.55 \text{ in. (497 mm)}$$

Suppose the slope of the compressive strut is one transverse to three longitudinal, then the width of the bearing area at the local zone – general zone interface is:

$$12 + 2\frac{14.75}{3} = 21.8 \text{ in.} > 19.55 \text{ in.}$$
$$(554 \text{ mm} > 497 \text{ mm})$$

Therefore, the width of the bearing area at the interface is 19.55 in. (497 mm).

The height of the bearing area assuming one to three distribution is:

$$(6 + 2 \times 14 + 6) + 2\frac{14.75}{3}$$
$$= 49.83 \text{ in. (1266 mm)}$$

The bearing stress at the local zone - general zone interface is, therefore:

$$\frac{P_T}{19.55 \times 49.83} = 2.627 \text{ ksi (18.11 MPa)}$$

which should be less than (AASHTO LRFD Article 5.10.9.3.1):

$0.6\phi f'_{ci}$ and $\phi = 0.8$

Thus, the required concrete strength at the time of post-tensioning f'_{ci}:

$$f'_{ci} \geq \frac{2.627}{0.6 \times 0.8} = 5.473 \text{ ksi (37.7 MPa)}$$

In conclusion, the concrete strength (f'_{ci}) at time of post-tensioning must be greater than 6.295 ksi (43.4 MPa).

Select Bursting Reinforcement

Table B2 lists the tensile bursting forces in the anchorage zone, the corresponding reinforcement requirements, and the reinforcing bars selected.

When selecting the reinforcement required, the following data were used: the yield strength = 60 ksi (414 MPa); the strength reduction factor ϕ = 1.0; transverse bursting length = 2(31.5) = 63.0 in. (1600 mm); web bursting length in thin direction = 2(10) = 20 in. (508 mm); flange bursting length in thin direction = 2(13) = 26 in. (660 mm).

Table B2. General zone reinforcement.

Action	Force kips (kN)	Reinforcement, sq in. (mm²)	
		Required	Selected
Transverse bursting	301.2 (1339)	5.02 (3239)	9.30 (6000)(#5 @ 4 in.)
Bursting in thin direction of web	214.8 (955)	3.58 (2310)	3.72 (2400)(#5 @ 4 in.)
Flange bursting	112.2 (499)	1.87 (1206)	2.17 (1400)(#5 @ 4 in.)