Generalized Approach to Design of Posttensioned Concrete Anchorage Zones

David H. Sanders and John E. Breen

The current provisions for the design of posttensioned concrete anchorage zones in AASHTO's bridge design specification state only that designers should limit the average bearing pressure ahead of the anchorage device to less than 3,000 lb/in.^{2} or $0.9f'_{ci}$ (f'_{ci} is the initial concrete compressive strength), whichever is smaller. The specification does not give any guidance for other forces in the anchorage zone, which in many cases are critical. A research project funded by NCHRP and conducted at the University of Texas at Austin was initiated in 1984 with the primary objective of developing a rational and systematic approach to anchorage zone design that could be implemented in the AASHTO bridge specification. The project is completed, and the process of submitting provisions to AASHTO is under way. The proposed provisions provide guidelines for both ultimate and service limit states. It is proposed to divide the anchorage zone into a local zone and a general zone, to require an acceptance test for anchorage devices with high bearing stresses, and to implement design procedures for the general zone that can use a strut-and-tie model, finite element analysis, or approximate equations, or all of these. An introduction into the project and those provisions that are being proposed to AASHTO are presented.

In the 1950s and 1960s extensive research on anchorage zones for posttensioned concrete members was conducted (*1–6*). Most researchers used theory-of-elasticity–type analysis and small anchor block tests to investigate anchorage zone forces and stresses. These studies gave a basic understanding of the forces in simple anchorage zone configurations. The theories and charts developed by Guyon (*7*) in the 1950s are still often used in the design of concentric, multiple, and eccentric anchorage zones. However, when anchorage zone configurations are more complex, engineers have difficulty extrapolating from these basic results. In the 1970s the situation improved somewhat as the use of finite element analysis (FEA) became more common. Yet, many times, this type of analysis is too time consuming, expensive, or difficult to translate into reinforcement patterns. In the late 1970s and early 1980s, a study by Stone and Breen (*8,9*) provided empirical equations for the design of single anchorages in thin members but did not provide a generalized approach.

In 1987 an article by Schlaich et al. (*10*) helped to focus attention on the importance of a consistent and rational approach to design. The article introduced the concept of D- and B-regions within a structural system and utilized the strut-and-tie model (STM). B-STM regions are zones in which Bernoulli's theory of plane strain is valid. D-regions (discontinuity, disturbed, or detailed) are zones in which the strain distribution is significantly nonlinear. These regions are near concentrated loads (anchorage zones in posttensioned concrete), corners, and openings. Even though the STM can be used in both D- and B-regions, it is especially useful in D-regions where few rational techniques are available. The use of the STM for anchorage zone design was first introduced by Mörsch in the 1920s but has not been used extensively.

A rational and systematic approach to anchorage zone design is critical for maintaining a consistent factor of safety. A survey of design engineers around the world conducted by the Comité Euro-International du Béton (CEB) in 1987 helped to indicate the wide variability that occurs in the factor of safety (*11*). Engineers were asked to design a beam having six anchorages applying a total force of 2700 kN (607 kips), each according to their own national code or handbook. For instance, asked to calculate the bursting force (that is, the force caused by the spreading of the applied concentrated load), engineers provided results ranging from 49.5 kN (11.1 kips) to 440 kN (98.9 kips), with an average of 192 kN (43.3 kips). Calculations of the length of the bursting zone and the cross-sectional area of the reinforcement needed to carry the bursting force showed similar variations. The survey makes it clear that progress in the current state of the art in design of anchorage zones is not a matter of refining 5 or 10 percent, but rather is at the point of reducing differences of as much as 500 percent.

In the United States there are other issues as well. There is not a consistent division of responsibility for the anchorage zone between the anchorage device supplier, the engineer of record, and the contractor. This inconsistency and the confusion that often results increase the potential of having a design that does not provide adequate section size or reinforcement to ensure a safe force path for the posttensioning force when the final approval is given by the engineer of record.

The current anchorage zone design provisions of AASHTO's bridge design specification (*12*) state only that designers should limit the average bearing pressure ahead of the anchorage device to 3,000 lb/in.^{2} or $0.9f'_{ci}$, whichever is smaller. (The symbol f'_{ci} is the concrete compressive strength at stressing). It gives no guidance for other forces in the anchorage zone, which in many cases are critical. A research project was initiated and funded in 1984 by NCHRP and conducted at the University of Texas at Austin with the primary objective of developing a rational and systematic approach to anchorage zone design that could be implemented in the AASHTO bridge specification. In seeking to properly define responsibilities, both the legal traditions of engineering responsibility and the physical behavior of the anchorage zones were examined. An

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approach was found that closely relates these two seemingly different considerations. The purpose of this paper is to provide a brief review of the findings from that project and an introduction to the provisions that are being proposed to AASHTO.

DESIGN APPROACH

A concept central to the design approach developed is the division of the anchorage zone into a local zone and a general zone (see Figure 1). This concept came from the results of the elastic FEA done by Burdet (*13*), which indicated that stresses in the zone immediately ahead for the posttensioning bearing surface and within the lateral dimension of the anchorage device were essentially the same regardless of the type of anchorage configuration (concentric, eccentric, inclined, multiple, etc.).

The local zone is defined as the volume of concrete surrounding and immediately ahead of the anchorage device, throughout which the force applied to the anchorage device is transferred to the general zone. The local zone includes any confining reinforcement required by the specific anchorage device. The local zone must resist the very high local pressures introduced by the anchorage device. Its behavior is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and considerably less influenced by the geometry and loading of the overall structure.

The general zone is defined as the volume of concrete through which the lateral spreading of posttensioning forces occurs from the highly concentrated load at the anchorage device to a more linear force distribution across the entire cross section at some distance from the anchorage device (D-region) (the so-called Saint Vénant Region). In the general zone, the spreading of forces and the induced tensile stresses are the major design considerations. The sections in Figure 1 are two-dimensional representations of the anchorage zone. The anchorage zone is analyzed by analyzing each of the principal planes.

By defining the local zone and the general zone, it is possible to attribute the various responsibilities for the proper performance of the anchorage zone to the engineer of record, the anchorage device supplier, and the constructor. The engineer of record is responsible for (a) the overall design for both the service and ultimate limit states, (b) the approval of the local zone details usually based on test data submitted by the anchorage device supplier, and (c) the design or approval of working drawings for the general anchorage zone, including the specific locations of the tendons, anchorage devices, and general zone reinforcement and for the specific stressing sequence.

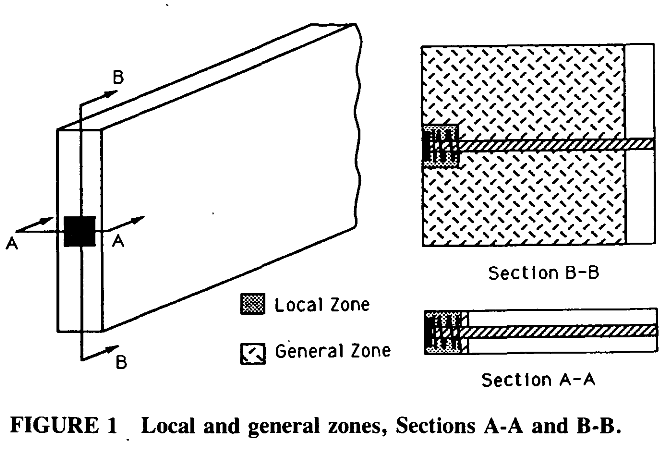

FIGURE 1 Local and general zones, Sections A-A and B-B.

The anchorage device supplier is responsible for (a) furnishing adequate hardware devices; (b) specifying the required auxiliary and confining reinforcement, minimum-edge, and center-to-center distances; and (c) specifying the minimum concrete strength for stressing to ensure the proper performance of the local zone.

The constructor is responsible for (a) the proper placement of all materials and (b) the correct performance of all stressing operations according to the design documents of the engineer of record and the requirements stipulated by the anchorage device supplier.

The posttensioned anchorage zone design provisions proposed for the AASHTO bridge specification incorporate a limit states approach by using the load and resistance factor technique for strength calculations in conjunction with service and detailing checks. A load factor of 1.2 was selected for use with the maximum tendon jacking force ($0.8f'_sA^*_s$) (f'_s is the ultimate strength of the prestressing steel and A^*_s is the area of the prestressing steel). This results in a design force 0.96 times the nominal ultimate strength of the tendon, which is essentially the maximum force that can be applied to a tendon before the grips fail. A ϕ-factor of 0.85 was selected for anchorage zones when normal weight concrete is used and 0.70 for lightweight concrete. The factor reflects the importance of the anchorage zone, the brittle compression failures that can occur, and the relatively wide scatter of the experimental results. The overall factor of safety (load factor/ϕ = 1.2/0.85 = 1.41) is essentially the same as that recommended by the International Federation of Prestressed Concrete (FIP) (*14*).

Local Zone

In the project, to assist in the development of the proposed specification, the large preexisting data base was examined by Roberts (*15*). In addition, 31 tests were performed that investigated the effects of spiral and tie size, spacing and length, anchorage size and type, and concrete strength.

Roberts concluded that to satisfy service and safety requirements, anchorage devices must be divided into two groups: (a) those with reasonable stiffness and an average bearing stress at ultimate of less than $0.7\phi f'_{ci}\sqrt{A/A_g}$ (A is the maximum area of the supporting surface that is similar to and concentric with the loaded area, and A_g is the gross area of the bearing plate) and $2.25\phi f'_{ci}$; and (b) those with higher bearing stresses. Anchorage devices in the first group are called basic anchorage devices. They are limited to a level of bearing stresses that provide good service and ultimate load performance without auxiliary confining reinforcement, and they can be used without further testing. Anchorage devices in the second group are called special anchorage devices, and they must pass an acceptance test. (Most anchorage devices are in this category.) For an anchorage device system to pass the acceptance test proposed, the crack widths must remain below certain limits at several load stages and the anchorage must be able to reach either (a) 1.2 times the ultimate load of the tendons when tested with a long-term (48-hr) testing procedure or a cyclic testing procedure; or (b) 1.3 times the ultimate load of the tendons with a monotonic testing procedure.

General Zone

The design procedure for the general zone must also satisfy service and ultimate limit states. Unlike the local zone—in which generally there are a limited number of practical configurations—the general zone has a wide range of acceptable configurations. Therefore, it is impossible to use an acceptance test. Design procedures must ensure proper performance. Basing the service limit state on a crack-free performance is unrealistic because of the complex stress state, the effects of shrinkage and restraints, and the reticence of relying on tensile concrete strength. Therefore, in the proposed specification, the service limit state has been addressed by providing guidelines for reinforcement to control cracking to within acceptable limits.

Recommended Design Methods

The proposed provisions describe three methods for determining ultimate capacity of the anchorage zone: (a) FEA, (b) STM, and (c) approximate equations. In each of these procedures, all possible failure modes must be checked, that is, compressive failure of the concrete and tensile or development failure of the reinforcement. The compression failure in the general zone will in most cases occur immediately outside the confined local zone or at a change in thickness within the section.

Finite Element Analysis

FEA allows the calculation of the elastic flow of forces [see Figure 2 (*left*) and paper by Burdet (*13*)], compatibility stresses, and the compression stresses from stress trajectories. The compressive stresses determined are compared with the maximum permissible stress of $0.7\phi f'_{ci}$. The location and magnitude of the bursting force may be obtained by integrating the bursting stresses along the tendon path. However, the time required to input data and the overall cost can make FEA expensive.

Strut-and-Tie Model

STM is a lower-bound plasticity model that has the advantage of showing clearly the design forces necessary for tension reinforcement as well as indicating a plausible force path for the prestress force from the anchorage device into the member. The STM consists of compression struts, tension ties, and nodes (intersection points). Figure 2 (middle) shows a simple STM that crudely represents the force flow, whereas Figure

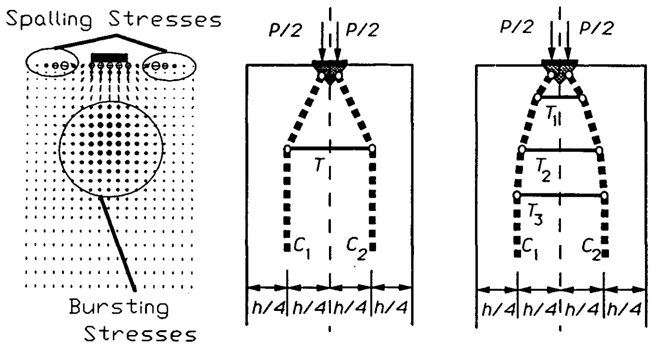

FIGURE 2 Concentric anchorage analysis comparison: *left*, elastic stress; *middle*, single-tie STM; *right*, multiple STM trajectories (*12*).

2 (*right*) shows an STM that more closely models the pattern of the compressive force and tension force distribution. The centroids of tie T in Figure 2 (middle) and group of ties T_1, T_2, and T_3 in Figure 2 (*right*) are approximately at 0.5h. The ties represent the location of the reinforcing bars. Tests showed that either model is acceptable (*16*).

An STM is sketched by following the compression force paths and satisfying equilibrium. The model can be developed by intuition or in complex cases by following the results of an FEA. The model uses the elastic stress distribution at the end of the general zone to determine the location and magnitude of the compression struts. It is not critical that the tension ties be located exactly at the elastic centroid of the tension stresses. The STM allows flexibility in the placement of the tension reinforcement. The STM has the adaptability to be used for the design of many different geometries and loading configurations.

The potentially complex aspect of the STM is the checking of the compression capacity of the struts and nodes. The size of the compression struts and nodes must be estimated. The special anchorage device local zone is assumed to be an acceptable node, if sufficient capacity exists at the local zone-general zone interface. Tension ties must be developed at the other nodes. In addition, since the STM is an equilibrium-based model, it does not accurately model the forces that are needed only to satisfy compatibility conditions [see Figure 2 (*left* and *middle*), spalling stresses]. Rules are given in the proposed specification for estimating the magnitudes of these compatibility forces, which are typically small.

Approximate Methods

The use of approximate equations can be a quick and easy way to design an anchorage zone as long as the assumptions of the equations are met. The extrapolation of an approximate equation to a more complex anchorage zone can produce unconservative designs. The proposed approximate equations are limited to members with (a) a rectangular cross section, (b) an anchorage zone length equal to at least the largest dimension of the cross section, (c) no discontinuities within or ahead of the anchorage zone, (d) only one anchorage device or one group of closely spaced anchorages, (e) straight tendons within the anchorage zone, and (f) a minimum-edge distance for the anchorage in the main plane of the member of at least 1.5 times the corresponding lateral dimension of the anchorage device.

The approximations for the compressive stresses at the interface between the local zone and the general zone are calculated using Equations 1 and 2, whereas the approximations for the bursting tension force needed are calculated using Equations 3 and 4.

$$f_{ca} = \kappa \frac{0.6P_u}{A_{\text{eff}}} \frac{1}{1 + l_c \left(\frac{1}{b_{\text{eff}}} - \frac{1}{t}\right)} \tag{1}$$

$$\kappa = 1 + \left(2 - \frac{s}{a_{\text{eff}}}\right)\left(0.3 + \frac{n}{15}\right) \qquad \text{for } a_{\text{eff}} \leq s < 2a_{\text{eff}}$$

$$\kappa = 1 \qquad \text{for } s \geq 2a_{\text{eff}} \tag{2}$$

$$T_{\text{burst}} = 0.25 \sum P_u \left(1 - \frac{a}{h}\right) + 0.5\ V_a \tag{3}$$

$$d_{\text{burst}} = 0.5(h - 2e) + 5e \sin\alpha \tag{4}$$

where

- $\sum P_u$ = sum of factored tendon loads,
- V_a = shear force in anchorage zone at time of stressing,
- a = lateral dimension of anchorage device or closely spaced group of anchorages,
- e = eccentricity of the anchorage device or group from section centroid (always positive),
- h = lateral dimension of cross section being considered, and
- α = angle of inclination of resultant of tendon force with respect to the centerline of the member, positive for concentric tendons or if anchor force points toward the centroid of section (must be between −5 and 20 degrees).

VERIFICATION OF GENERAL ZONE DESIGN PROCEDURES

A physical testing program was developed to verify the design procedures that use the STM, FEA, and approximate equations. Using tests to verify procedures instead of to develop empirical expressions enabled the researchers to test a wide variety of specimens with few replicates. This seemed to be the best use of limited resources.

Testing Procedures

The program was divided into two phases. Phase A focused on end-type anchorages and consisted of 36 general zone physical specimens reported by Sanders (*16*). Anchorage location and reinforcing patterns were varied (concentric anchorage with both prestressed and nonprestressed reinforcement, eccentric anchorage, multiple anchorages, and anchorages with inclination and tendon curvature). In Phase B, a total of 14 general zone specimens (specimens with intermediate anchorages, anchorages in beams with support conditions perpendicular to the posttensioning force, and anchorages in blisters, ribs, and diaphragms) were reported by Wollmann (*17*) and 6 slab specimens (each with multiple anchorage tests) were reported by Falconer (*18*). The test specimens were designed using the STM to proportion the general zone reinforcement after using FEA (*13,17*) to indicate the stress fields. The critical reinforcement in each test specimen was instrumented. Crack development and ultimate loads were recorded. The specimen results were compared with the design models.

Basic STM

Examples of STM appear in Figures 3 and 4. These were constructed assuming an elastic stress distribution at the end of the general zone. For eccentric anchorages with the resultant force outside the kern [see Figure 3 (*left*)], a tension force occurs on the outer longitudinal edge fibers farthest from the anchorage device (tension tie T_2). To keep crack sizes small, the spalling force (T_3) is located very close to the exterior surface, a distance from the loading surface approximately equal to the width of the anchorage plate divided by four. The use of tendons with curvature adds radial forces along the tendon path. These forces act as a distributed load perpendicular to the tendon path with a value equal to the tendon load divided by the radius of curvature of the tendon. The radial forces can be modeled as forces being applied to the compression struts [see Figure 3 (*right*)]. Tie-back reinforcement used to carry the radial forces back across the tendon path are modeled as tensile forces applied to the compression strut [see left compression strut in Figure 3 (*right*)]. An example of an STM when multiple anchorages are close together is shown in Figure 4 (*left*), and Figure 4 (*right*) shows two anchorages that are far enough apart to cause a major spalling tension force (T_3) to develop along the loaded surface in the concrete. As with the spalling force in Figure 3 (*left*), the forces C_7 and T_3 are very close to the loading surface.

Once the designer determines the basic shape of the STM, which implicitly assumes a particular reinforcement distribution, the equally important task of defining the size and capacity of the compression struts and nodes must be done. The local zone surrounding the anchorage device contains the node that transfers the tendon force onto the concrete. In this and several previous studies, a pyramid or cone was observed under the loading plate when a bearing failure occurred; therefore, a pyramid was assumed for the shape of the local zone node. The idealization of the pyramid shape in a principal direction for a concentric anchorage zone appears in Figure 5. The compression struts initiate from the local zone node.

In most anchorage zone configurations, the struts will widen and the stresses will decrease with distance from the anchorage device. The spreading of the struts is why Schlaich et al. (*10*) describe some nodes as discrete and others as smeared. The two nodes [see Figure 2 (*middle*)] near the bearing surface are discrete nodes; they are confined to a small area and have high concentrated forces. The two nodes shown further away from the anchorage device are shown as two dots but are actually smeared nodes spread over a much larger area. Proper reinforcement detailing is necessary for both node types to ensure that reinforcement can be fully developed at the node.

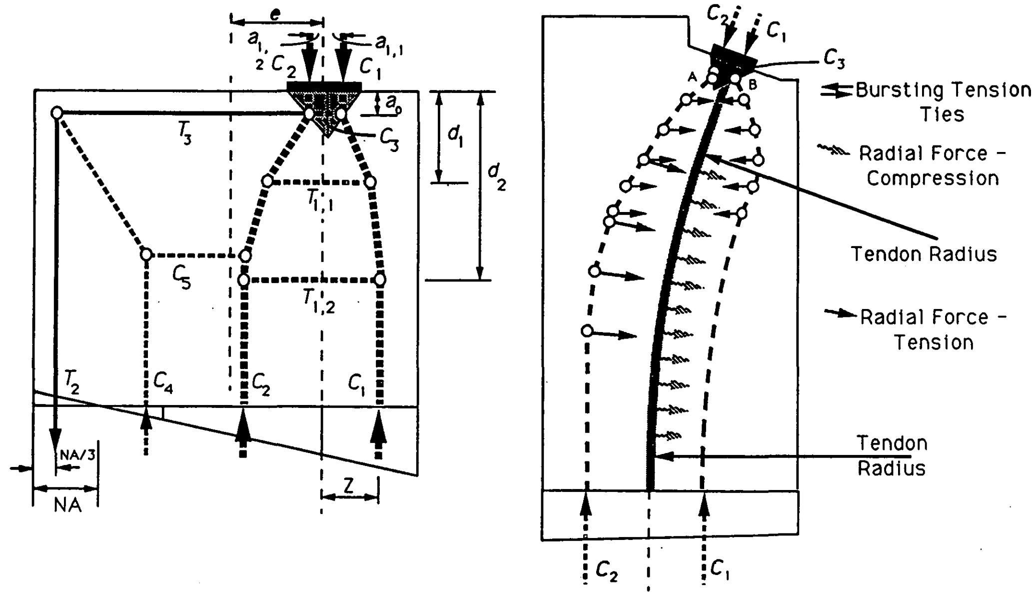

FIGURE 3 Eccentric anchorage STM: ***left*****, straight tendon outside kern;** ***right*****, curved tendon with anchorage inclination.**

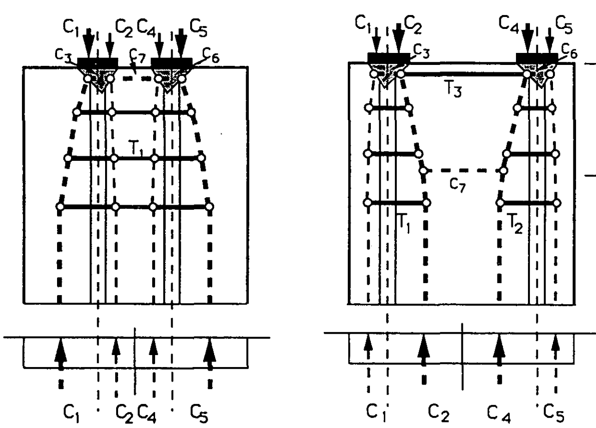

FIGURE 4 Multiple anchorage STM: ***left*****, closely spaced anchors;** ***right*****, widely spaced anchors.**

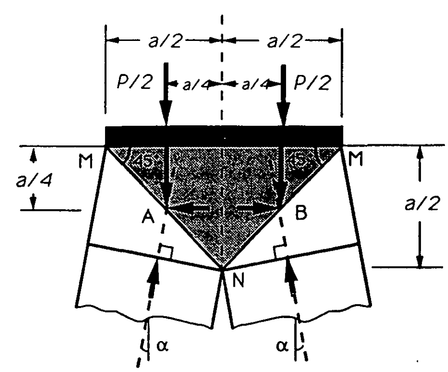

FIGURE 5 Idealized concentric node.

However, discrete nodes must also be checked for concentrated compressive stresses, whereas smeared nodes have distributed compressive stresses and do not need to be checked.

The width of the compression strut is assumed to be equal to twice the distance from the tendon axis to the centerline of the strut as measured normal to the strut axis (see Figure 5). This is assumed because (a) the two struts cannot overlap at the tendon axis; and (b) a uniform stress distribution is assumed for the strut, which defines the exterior boundary since the centroid of the strut is known. Three locations are critical for verifying the strut capacity: (a) the interface between the node and the strut where a portion of the strut is typically unconfined; (b) the interface between the local zone and the general zone where the strut leaves the local zone and enters the general zone (the strut is assumed fully unconfined); and (c) at sections where thickness changes and increases in the strut stresses can occur. For design purposes, the certification of the special anchorage device provided by the anchorage device supplier is used to check the adequacy of the interface between the node and the strut. The engineer of record must check the compressive stresses at the local zone-general zone interface. The location of the local zone-general zone interface can be assumed to be at a distance equal to one times the lateral dimension of the anchorage device away from the bearing surface or the end of the confining reinforcement, whichever is closer. This assumption provides the best correlation with the experimental results when using a concrete compressive stress limit of $0.7f'_{ci}$.

The STM is a lower-bound model based on the theory of plasticity and was found to be a conservative way of estimating the ultimate strength of the specimens. The average of the test ultimate strength divided by the STM prediction for the specimens in Phase A (end-type anchorage zones) was 1.50 (see Table 1) with a coefficient of variation of 0.32 (*16*) when assuming the critical depth for the local zone-general zone interface at the end of the local zone confining spiral. Setting aside Specimen B5, which had no general zone reinforcement and hence is really outside the scope of the STM, only one specimen was unconservatively predicted. This was Specimen B2, where the prediction was one percent unconservative, possibly because its bursting reinforcement was very deep within the section. (The distance from the loaded surface to the centroid of the bursting reinforcement divided by the section width was equal to 0.84.) If the critical depth for the local zone–general zone interface is placed at a depth equal to the lateral dimension of the anchorage device, then the average of the test ultimate divided by the STM prediction for the specimens only changes slightly to 1.52 with a coefficient of variation of 0.31. More details on reasons for this assumption are given in the next section.

Care should be taken not to place reinforcement, which is depended on to carry bursting tension, too far from the anchorage device. If reinforcement is placed too deep within the section, additional cracking must occur for this reinforcement to become active. This increased cracking could cause serviceability problems or reduce compressive strength, or both.

Modified STM

The strength of several specimens exceeded the basic STM prediction, in one case by a factor of 3.3. The cause was sought by examining cracking patterns from the physical tests and using a step-by-step elastic analysis. It was determined that as the anchorage zone cracked along the tendon axis, the anchorage zone extended further into the section and the compression struts began to shift closer to the tendon axis (see Figure 6). In the physical specimens, this created a more plastic stress distribution at the base of the specimens. As the struts shift in toward the tendon axis, the posttensioning axial capacity (as governed by the tension ties) increases because the moment arm of the compression force C_1 about Point A decreases. The shift of the strut reduces the width of the strut at the local zone-general zone interface [Plane a-a in Figure

TABLE 1 Phase A: Basic STM Results (*16*)

		LZ-GZ Interface at End of Spiral			LZ-GZ Interface at Plate Width from Bearing Surface		
Test Name	Ultimate	STM Prediction Mode	Load	Test/STM	STM Prediction Mode	Load	Test/STM
A1	298	N-S	195	1.52	N-S	195	1.49
A2	275	N-S	190	1.45	N-S	190	1.45
A3	265	N-S	204	1.30	N-S	204	1.30
A4	437	N-S	306	1.43	N-S	306	1.43
B1	366	T-Tie	299	1.22	T-Tie	299	1.22
B2	290	T-Tie	292	0.99	T-Tie	292	0.99
B3	331	T-Tie	296	1.12	T-Tie	296	1.12
B4	337	T-Tie	277	1.22	T-Tie	277	1.22
B5[a]	212	LZ-GZ	218	0.97	LZ-GZ	201	1.05
B6	297	Bearing	218	1.36	Bearing	218	1.36
B7	296	T-Tie	269	1.10	T-Tie	269	1.10
B8	276	T-Tie	252	1.09	T-Tie	252	1.10
C1	370	T-Tie	192	1.93	T-Tie	192	1.93
TPT1	310	T-Tie	180	1.72	T-Tie	180	1.72
TPT2	300	T-Tie	253	1.19	T-Tie	253	1.19
TPT3	370	T-Tie	247	1.50	T-Tie	247	1.50
TPT4	332	T-Tie	235	1.41	T-Tie	235	1.41
E1	475	Bearing	404	1.17	Bearing	404	1.18
E2	500	Bearing	445	1.12	LZ-GZ	425	1.18
E3	522	Bearing	453	1.15	LZ-GZ	438	1.19
E4[b]	500	Bearing	434	1.15	LZ-GZ	407	1.23
E5	332	T-Tie	238	1.39	T-Tie	238	1.39
E6	348	T-Tie	259	1.34	T-Tie	259	1.34
M1	304	T-Tie	189	1.61	T-Tie	189	1.61
M2	401	LZ-GZ	322	1.25	LZ-GZ	245	1.64
M3	364	T-Tie	290	1.26	LZ-GZ	244	1.49
M4	411	T-Tie	180	2.28	T-Tie	180	2.28
M5	339	T-Tie	202	1.68	T-Tie	202	1.68
M6	300	T-Tie	104	2.88	T-Tie	104	2.88
ME1	350	T-Tie	226	1.55	T-Tie	226	1.55
ME2	370	T-Tie	261	1.42	T-Tie	261	1.42
F1	248	T-Tie	133	1.86	T-Tie	133	1.86
I1	423	T-Tie	281	1.51	T-Tie	281	1.51
I2	437	T-Tie	223	1.96	T-Tie	223	1.96
I3	375	T-Tie	262	1.43	T-Tie	262	1.43
I4	420	T-Tie	126	3.33	T-Tie	126	3.33
	Average			1.50			1.52
	Standard Deviation			0.49			0.48
	Coef. of Variation			0.32			0.31

T-Tie = Tension Tie Capacity Governs
N-S = Node-Strut Interface Capacity Governs
LZ-GZ = Local Zone-General Zone Interface Capacity Governs
Bearing = Bearing Capacity Under Anchorage Device Governs
All Units in kips

[a] *no bursting steel, zero tension capacity ignored*
[b] *longitudinal edge tension and spalling tension capacity*

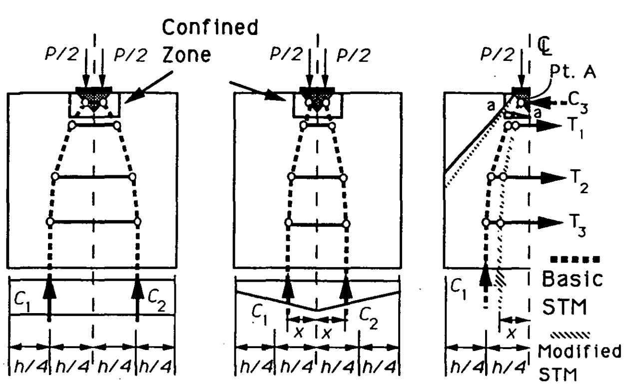

FIGURE 6 Basic STM (*left*), modified STM (*middle*), and comparison (*right*).

6 (*right*)], reducing the posttensioning axial capacity (as governed by the compression struts).

The configuration of the modified STM (*16*) is determined by locating the strut C_1 [distance x in Figure 6 (*right*)] at a position where the STM-predicted ultimate load, assuming that the bursting tension ties control, is the same as the STM-predicted ultimate load, assuming that the compression struts control. The location is determined by shifting the assumed centroid of the compression strut toward the tendon axis until the two loads match.

Even though the modified STM was much more accurate for modeling behavior, it predicted unconservatively the ultimate strength for 14 out of the 35 specimens (setting aside Specimen B5). The average of the ratio between the ultimate test load and the predicted load for all the specimens was 1.05 with a coefficient of variation of 0.20 (see Table 2) (*16*) when

TABLE 2 Phase A: Modified STM Results (*16*)

		LZ-GZ Interface at End of Spiral			LZ-GZ Interface at Plate Width from Bearing Surface		
Test Name	Ultimate	STM Prediction Mode	Load	Test/ STM	STM Prediction Mode	Load	Test/ STM
A1	298	N-S	199	1.49	N-S	199	1.49
A2	275	N-S	204	1.35	N-S	204	1.35
A3	265	N-S	200	1.33	N-S	200	1.33
A4	437	LZ-GZ	335	1.30	LZ-GZ	335	1.30
B1	366	LZ-GZ	440	0.83	LZ-GZ	343	1.07
B2	290	LZ-GZ	381	0.76	LZ-GZ	305	0.95
B3	331	LZ-GZ	416	0.80	LZ-GZ	326	1.02
B4	337	LZ-GZ	429	0.79	LZ-GZ	330	1.02
B5 [a]	212	LZ-GZ	218	0.97	LZ-GZ	201	1.05
B6	297	Bearing	218	1.36	Bearing	218	1.36
B7	296	LZ-GZ	421	0.70	LZ-GZ	327	0.91
B8	276	LZ-GZ	405	0.68	LZ-GZ	303	0.91
C1	370	LZ-GZ	419	0.88	LZ-GZ	328	1.13
TPT1	310	LZ-GZ	346	0.90	LZ-GZ	290	1.07
TPT2	300	LZ-GZ	376	0.80	LZ-GZ	311	0.96
TPT3	370	LZ-GZ	432	0.86	LZ-GZ	351	1.05
TPT4	332	LZ-GZ	401	0.83	LZ-GZ	328	1.01
E1	475	Bearing	404	1.18	Bearing	404	1.18
E2	500	Bearing	445	1.12	Bearing	445	1.12
E3	522	Bearing	453	1.15	Bearing	453	1.15
E4 [b]	500	Bearing	434	1.15	Bearing	434	1.15
E5	332	Bearing	343	0.97	LZ-GZ	315	1.05
E6	348	LZ-GZ	339	1.03	LZ-GZ	333	1.05
M1	304	LZ-GZ	293	1.04	LZ-GZ	231	1.32
M2	401	LZ-GZ	336	1.19	LZ-GZ	256	1.57
M3	364	LZ-GZ	355	1.03	LZ-GZ	236	1.54
M4	411	LZ-GZ	345	1.19	LZ-GZ	280	1.47
M5	339	LZ-GZ	280	1.21	LZ-GZ	245	1.38
M6	300	LZ-GZ	206	1.46	LZ-GZ	206	1.46
ME1	350	LZ-GZ	410	0.85	LZ-GZ	328	1.07
ME2	370	LZ-GZ	374	0.99	LZ-GZ	330	1.12
F1	248	LZ-GZ	227	1.09	LZ-GZ	205	1.21
I1	423	LZ-GZ	349	1.21	LZ-GZ	302	1.40
I2	437	LZ-GZ	381	1.15	LZ-GZ	335	1.30
I3	375	LZ-GZ	350	1.07	LZ-GZ	277	1.35
I4	420	LZ-GZ	343	1.22	LZ-GZ	288	1.46
	Average			1.05			1.20
	Standard Deviation			0.21			0.19
	Coef. of Variation			0.20			0.16

N-S = Node-Strut Interface Capacity Governs
LZ-GZ = Local Zone-General Zone Interface Capacity Governs
Bearing = Bearing Capacity Under Anchorage Device Governs
All Units in kips

[a] *no bursting steel, zero tension capacity ignored*
[b] *longitudinal edge tension and spalling tension capacity*

the local zone-general zone interface was assumed to be at the end of the confining spiral. This coefficient of variation is similar to that found when estimating the bearing capacity of anchorage device systems using compression capacity equations developed by Roberts (*15*). Possible reasons for the unconservative prediction are (a) a reduction in the concrete compression capacity because of the excessive and large cracking, (b) inaccuracies in the strut width approximation, or (c) inaccuracies in the assumed location of the critical local zone-general zone interface, or all of these.

When the basic STM was used to predict the capacities of the specimens, only 6 of the 36 specimens were controlled by compression strut failure. It was difficult to refine the compression model using the basic STM. Initially, the location of the local zone-general zone interface was assumed to be at the end of the local zone-confining spiral, which varied between one and two times the anchorage width in the specimens tested. When the depth of the interface was limited to the width of the anchorage, the average of the modified STM comparison ratio increased to 1.20 and the coefficient of variation dropped to 0.16 (see Table 2). The ultimate strength was unconservatively predicted for only four specimens. Those that were unconservatively estimated had predicted to actual strength ratio values greater than 0.9. Therefore, in the proposed specification, the location of the local zone-general zone interface was assumed to be at a distance from the bearing surface equal to one times the lateral dimension of the anchorage device or at the end of the confining reinforcement, whichever was closer.

The design of the anchorage zone must provide some degree of ductility and plenty of warning before failure. In this regard, an analogy between the modified STM and the basic STM can be seen with reinforced concrete beam design. To provide ductility and warning before failure, reinforced concrete beam design has an upper limit on the amount of flexural reinforcement; the design load is based on the yielding of the tensile reinforcement occurring before the compression failure of the concrete. In the anchorage zone tests, extensive cracking was typically observed between the failure load predicted by the basic STM and the actual failure load when the bursting reinforcement controlled the design. Thus the basic STM provides a warning of failure by predicting the ultimate capacity before the extension of the anchorage zone occurs and additional redistribution of forces. The modified STM predicts the final failure state. At this stage of design procedure development, the basic STM is desirable for safety.

CONCLUSIONS

The major characteristics of the proposed design procedure for anchorage zones are

1. Clear division of criteria into those associated with the anchorage device (local zone) and those having to do with the overall spread of the anchor forces (general zone);
2. Flexibility for the engineer of record in choosing an analysis based on the STM, FEA, or, for certain simpler cases, approximate equations;
3. A rational and systematic approach that examines the flow of forces from the anchorage device until the forces have spread over the entire cross section;
4. A lower-bound plasticity method (basic STM) that is highly transparent and provides reasonably accurate results while being conservative; and
5. A clear and logical division of responsibility among the anchorage device supplier, engineer of record, and the constructor.

The current research has established the ground rules for a generalized approach to the design of posttensioned concrete anchorage zones. The design method an engineer selects will depend on the designer's preference as well as the characteristics of the anchorage zone being designed. Each method can yield slightly different results, but all will be conservative when applied properly and when used with proper reinforcement detailing techniques.

Detailed reports (*12,15–18*) are available that document the experimental study and the various methods used. A series of shorter articles is being developed that will discuss in more detail the topics examined in this paper. Several examples were presented by the authors at the American Segmental Bridge Institute (ASBI) Meeting in Miami, Florida, in December 1990. Handouts from this meeting are available through ASBI (*19*). Although some small changes have been made in the code since they were published, these examples should provide a good reference.

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