# A PRAGMATIC APPROACH TO STRUT-AND-TIE MODELING

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## ABSTRACT

*Accelerated bridge construction may benefit from simplified strut-and-tie modeling (STM) procedures that can be routinely implemented in the design of the deep beam regions of precast concrete bridge components (e.g., precast pier/pile caps). Although STM provisions were introduced into the AASHTO LRFD Bridge Design Specifications nearly two decades ago, uncertainty faced by engineers during implementation of STM has impeded its routine application. A guidebook has been created to aid practitioners with the use of updated STM provisions that were recently developed at The University of Texas at Austin. The proposed recommendations are simpler and more accurate than existing provisions. The STM guidebook includes an overview of the new provisions as well as detailed design examples for common bridge components.*

*A brief introduction to the updated strut-and-tie modeling procedure will be presented along with a description of the STM guidebook. Implementation of the proposed provisions will be demonstrated through the design of a precast concrete bent cap representative of those commonly encountered in accelerated bridge construction.*

**Keywords:** Strut-and-Tie Modeling, Deep Beams, Bent Cap, Accelerated Bridge Construction

## INTRODUCTION

Strut-and-tie modeling (STM) is a versatile, lower-bound (i.e., conservative) design method for reinforced concrete structural components. STM is most commonly used to design regions of structural components disturbed by a load and/or geometric discontinuity. Load and geometric discontinuities cause a nonlinear distribution of strains to develop within the surrounding region. As a result, plane sections can no longer be assumed to remain plane within the region disturbed by the discontinuity. Sectional design methodologies are predicated on traditional beam theory, including the assumption that plane sections remain plane, and are not appropriate for application to disturbed regions, or D-regions. The design of D-regions must therefore proceed on a regional, rather than a sectional, basis. STM provides the means by which this goal can be accomplished.

Strut-and-tie modeling provisions were introduced into the AASHTO LRFD Bridge Design Specifications in 1994 and the ACI 318: Building Code Requirements for Structural Concrete in 2002. Although two decades have passed since their introduction into American design specifications, uncertainty faced by engineers during implementation of the STM provisions has impeded their routine application.

In response to the concerns expressed by design engineers and a growing inventory of distressed in-service bent caps exhibiting diagonal cracking within Texas, an extensive experimental program focusing on the behavior of deep beams was conducted. A total of 37 tests were performed on specimens that were some of the largest beams ever tested in the history of shear research. Existing STM code provisions were then calibrated and refined based upon the experimental results and a complementary database of 142 tests from the literature, as detailed in Birrcher et al.[1]. Three of the most significant revisions to existing STM code provisions that were proposed are presented below:

- The concrete efficiency factors, $v$, for the nodal faces were revised based on the tests within the experimental database.
- The triaxial confinement factor, $m$, can be applied to all faces of a node that abuts a bearing area that has a width that is smaller than the width of the member.
- The design of a strut is not explicitly performed but is accounted for in the design of the strut-to-node interface, encouraging the engineer to focus on the most critically stressed regions of the structural member (i.e., nodes).

To facilitate adoption of the design recommendations, an STM guidebook[2] was created to aid practitioners with their implementation and to clarify any remaining uncertainties associated with STM. As demonstrated through the five design examples provided within the guidebook (five-column bent cap of a skewed bridge, cantilever bent cap, inverted-T straddle bent cap – moment frame and simply supported, and drilled-shaft footing), the updated STM provisions can be extended to the design of a wide variety of reinforced concrete components.

## DESIGN OF A PRECAST CONCRETE BENT CAP USING PROPOSED STM PROVISIONS

Accelerated bridge construction may benefit from simplified strut-and-tie modeling procedures that can be routinely implemented in the design of deep beam regions of precast bridge components. The STM design recommendations presented in Birrcher et al.[1] and Williams et al.[2] provide a means to design such elements using provisions that are simpler and more accurate than existing STM specifications. The key elements of the recommended provisions are demonstrated below through the design of a precast concrete bent cap representative of those commonly encountered in accelerated bridge construction. A step-by-step explanation of the procedure used in the following design example is provided within the STM guidebook[2].

### DESCRIPTION OF DESIGN TASK

The precast bent cap considered in this design example is shown in Figure 1. The bent cap is supported by three columns spaced at 8 ft and, in turn, supports four lines of pretensioned girders also spaced at 8 ft. The two bearing pads (each 8 in. by 21 in.) that support each pair of pretensioned girders in a line combine to form an area of 16 in. by 21 in. as shown. The diameter of the circular columns is 30 in. To simplify the analysis of the nodes located at the column-cap connections, square bearing areas equal to that of the 30-in. circular columns are used. These equivalent square columns have dimensions of 26.6 in. by 26.6 in.

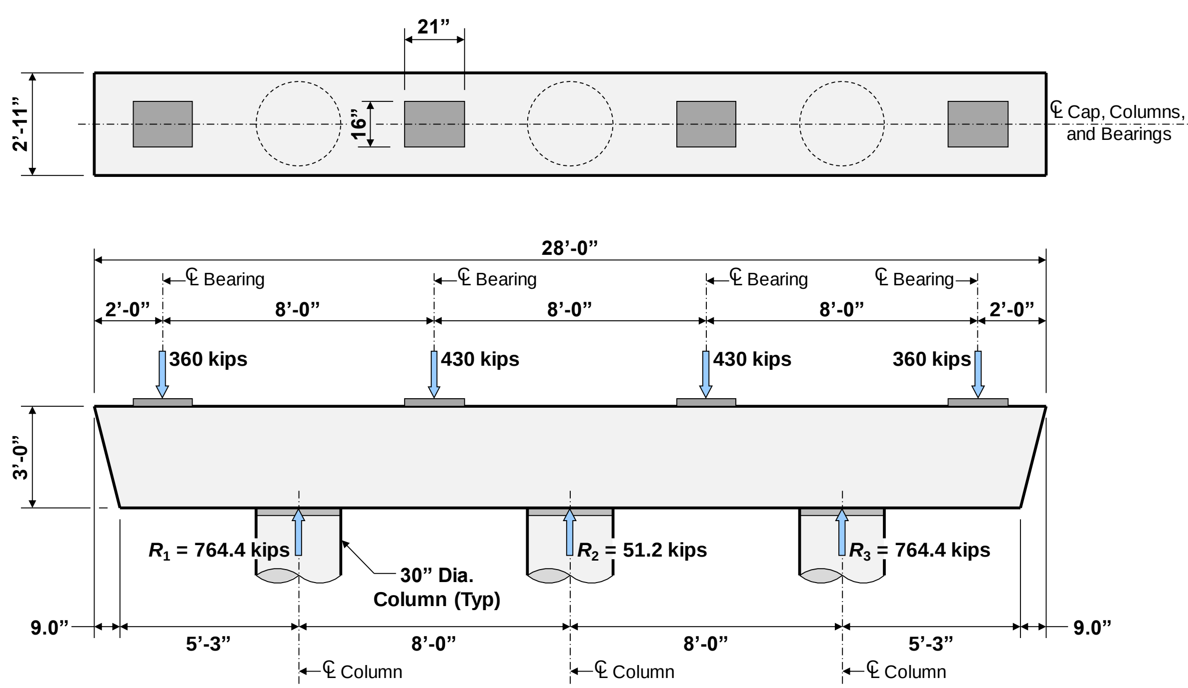


Fig. 1 Bent cap geometry and applied factored loads

The factored superstructure loads considered within the design example are also shown in Figure 1. Each pair of exterior girders applies a factored load of 360 kips, while each pair of interior girders applies a factored load of 430 kips. The factored self-weight of the bent cap is included within the loads shown in Figure 1.

The connection between each column and the precast bent cap is detailed in Figure 2. The detail is based upon the Texas Department of Transportation's precast bent cap design standards[3]. Six galvanized steel ducts with outer diameters of 4 in. are installed in the bent cap above each column. Six No. 9 bars from each column are inserted into the ducts and embedded within the cap as shown. After the bent cap is placed in the field, cementitious grout is pumped into the ducts as well as between the top of the columns and the cap. The finished bent cap is therefore supported on a layer of grout at each column-cap connection.

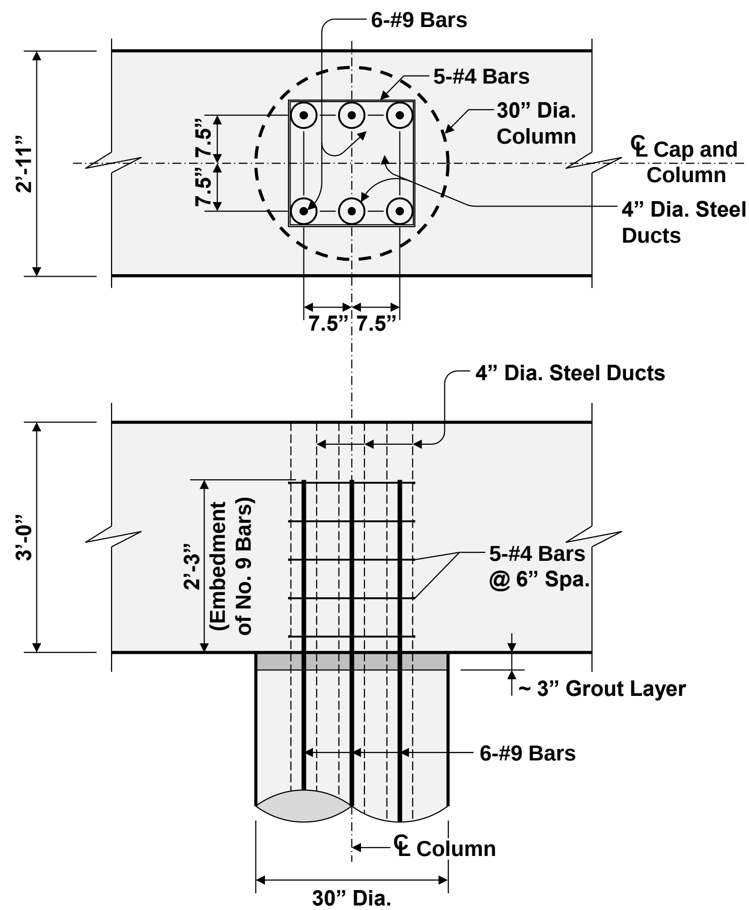


Fig. 2 Details of the connection between the bent cap and each column

The specified compressive strength of concrete, $f'_c$, used for the design of the bent cap is 3600 psi, and the specified 28-day compressive strength of the grout, $f'_g$, is 5800 psi. The specified yield strength of reinforcement, $f_y$, is 60,000 psi. Please note that the resistance factors, $\phi$, used within the design example are consistent with the AASHTO LRFD (2013)[4] specifications.

Considering the relatively close spacing of the superstructure loads and column reactions, the entire bent cap is a D-region and is expected to exhibit deep beam behavior. Application of the strut-and-tie modeling procedure is therefore appropriate for the design of the bent cap.

ANALYZE STRUCTURAL COMPONENT AND DEVELOP STRUT-AND-TIE MODEL

Prior to developing a strut-and-tie model, an elastic analysis of the bent cap is first performed to calculate the support reactions at the columns. The columns are assumed to behave as pin supports, as is common practice for multi-column bents in the State of Texas. The bent cap is treated as a continuous member supported by the columns for the purpose of determining the reactions. The reaction at each of the exterior columns is thereby found to be 764.4 kips, and the reaction at the interior column is determined to be 51.2 kips, as shown in Figure 1.

To define the nodal geometries and to simplify the nodal strength check calculations that are performed below, the interior loads and resulting reactions should each be divided into two separate forces as shown in Figure 3. Dividing the forces in this manner essentially separates the nodes associated with each bearing/column area into two parts. The forces and reactions acting at Nodes B, C, and D in Figure 3 are each split based on the value of the load that travels in each direction from the nodes. For example, considering the loads acting at Node C, 404.4 kips flows from the applied girder load to the column reaction at Node B, and 25.6 kips flows from Node C to the column reaction at Node D. The line of action, or position, of each component of the girder load is determined by maintaining uniform pressure over the bearing area.

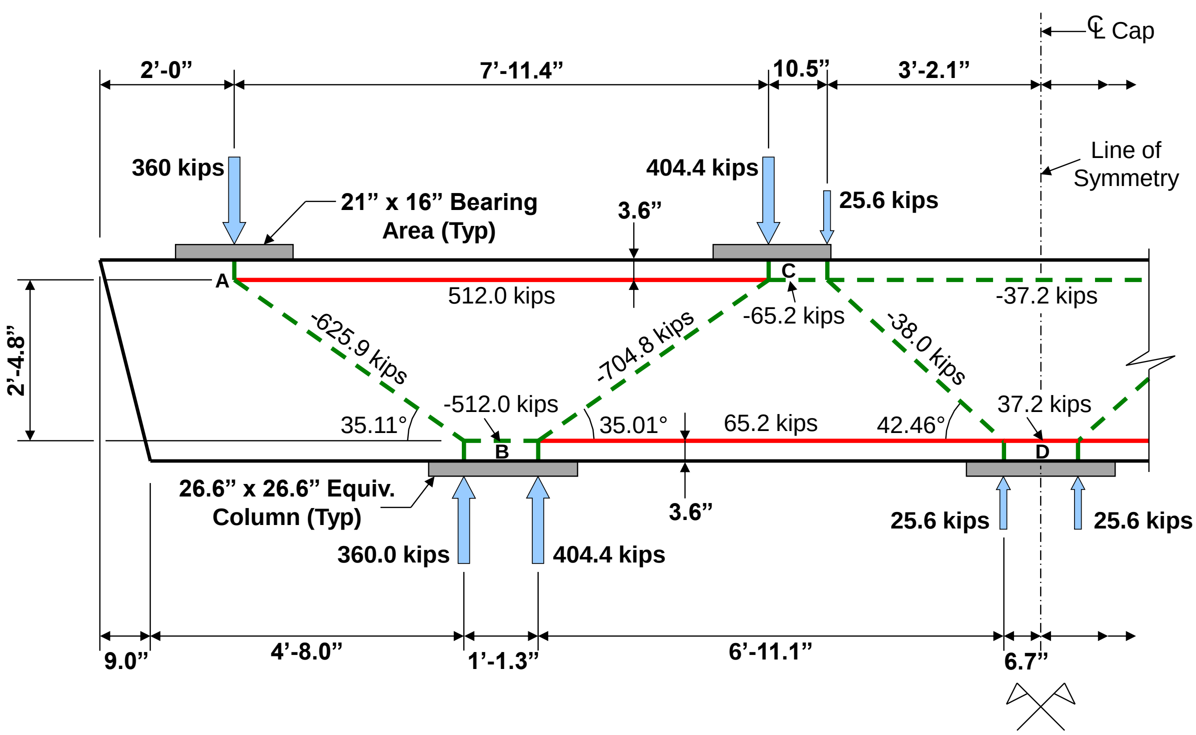


Fig. 3 Strut-and-tie model of the bent cap

Once the loads and reaction forces are divided appropriately, a strut-and-tie model representing the flow of forces from the loads to the column supports is developed. The STM for the bent cap is provided in Figure 3. In this figure, compressive members (i.e., struts) are represented by dashed lines, and tensile members (i.e., ties) are represented by solid lines. Due to the symmetry of the system, only half of the member is shown.

The first step in the development of the STM is to position the top and bottom chords of the truss model. Tensile forces, or tie forces, develop along both the top and bottom chords, as shown in Figure 3. These longitudinal ties represent the forces carried by the flexural tension reinforcement within the beam. The chords are therefore positioned to correspond with the centroid of the longitudinal reinforcement located along the top and bottom of the member. One layer of No. 11 bars is likely to be necessary for each chord given the applied girder loads. No. 5 stirrups with 2.25 in. of clear cover are used. Therefore, the centroid of the longitudinal reinforcement is located at about 3.6 in. from both the top and bottom surfaces of the member (refer to the final reinforcement layout in Figure 9).

Once the placement of the longitudinal chords is determined, the diagonal struts are positioned to model the transfer of forces through the depth of the member. A diagonal strut is extended between the applied girder load at Node A and the support reaction at Node B to model the deep beam behavior in that region of the beam. Additional diagonal struts are added to model similar behavior between the loads and supports within the remainder of the bent cap. It should be noted that the proposed STM provisions state that the angle between a strut and a tie entering the same node shall not be taken as less than 25 degrees. The strut angles within the STM of Figure 3 all satisfy this requirement.

After the geometry of the STM is determined, the member forces of the struts and ties are found by enforcing equilibrium. The factored girder loads and the column reactions found earlier through the elastic analysis of the bent cap are applied to the truss model. Since the model is internally statically determinate, all member forces can be calculated by satisfying equilibrium at the joints of the truss (i.e., by using the method of joints). The resulting forces are shown in Figure 3.

## PROPORTION LONGITUDINAL TIES

The next step in the design process is to proportion the longitudinal reinforcement to satisfy the tie requirements of the STM. The strength of the reinforcement should be sufficient to carry the calculated tie forces. For the top chord, the force in Tie AC (512.0 kips) must be considered. The area of steel, $A_{st}$, necessary to carry the 512.0-kip factored tie force is calculated as follows:

Factored load: $F_u = 512.0 \text{ kips}$

Nominal tie strength: $F_n = f_y A_{st}$

Required area of reinforcement:

$$A_{st,req} = \frac{F_u}{\phi f_y} = \frac{512.0 \text{ kips}}{(0.9)(60 \text{ ksi})} = 9.48 \text{ in.}^2$$

Using No. 11 bars, each with a nominal area of 1.56 in.$^2$, 7 bars are required and are provided in a single layer along the top of the bent cap. For the bottom chord of the STM, sufficient reinforcement must be provided to carry the governing force of 65.2 kips in Tie BD. The area of steel, $A_{st}$, necessary to carry this factored tie force is calculated as follows:

Factored load: $F_u = 65.2$ kips
Nominal tie strength: $F_n = f_y A_{st}$
Required area of reinforcement:

$$A_{st,req} = \frac{F_u}{\phi f_y} = \frac{65.2 \text{ kips}}{(0.9)(60 \text{ ksi})} = 1.21 \text{ in.}^2$$

Only 1 No. 11 bar is required to carry the tensile force along the bottom of the bent cap. The nodal strength calculations performed later reveal, however, that additional bars are needed to provide sufficient strength to Node B.

## DETERMINE NODAL GEOMETRIES AND PERFORM NODAL STRENGTH CHECKS

For this step of the design procedure, the geometries of the nodal regions are defined. Then, the nodes are checked to ensure that they have adequate strength to resist the imposed forces determined from the analysis of the STM. Nodes are the most highly stressed regions of a structural component because stresses from multiple struts and/or ties must be equilibrated within a small volume of concrete.

By inspection, Node B is identified as the most critical node within the strut-and-tie model. In order to demonstrate the application of the proposed STM provisions, strength checks are included for both Nodes A and B. The geometries of these nodes are illustrated in Figure 4. Please note that Strut AB is a bottle-shaped strut and that crack control reinforcement is later proportioned to restrain the cracks that tend to form along the strut.

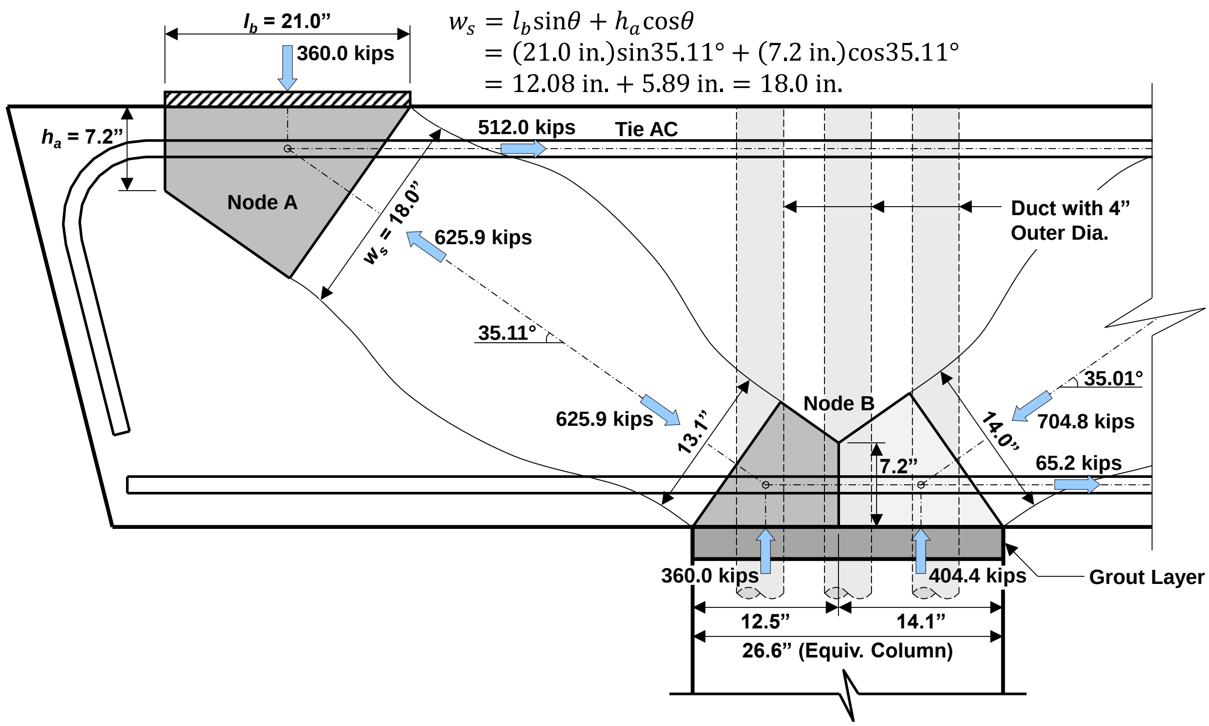


Fig. 4 Nodes A and B

The strength of Node A is evaluated first. The length of the bearing face, $l_b$, is taken as the width of the bearing area, or 21.0 in. (refer to Figures 1 and 4). The height of the back face, $h_a$, is taken as twice the distance from the top surface of the bent cap to the longitudinal tension reinforcement, or Tie AC. The back face height is therefore 2(3.6 in.) = 7.2 in. The calculation for the length of the strut-to-node interface, $w_s$, for Node A is provided in Figure 4 ($w_s$ = 18.0 in.). The width of the node into the page is taken as the length of the bearing area, or 16 in. (refer to the strength calculations below). Only one tensile force acts on Node A, and it is therefore classified as a CCT node (i.e., the concrete efficiency factors for CCT nodes are used).

According to the proposed STM provisions, a node that abuts a bearing area with a width that is smaller than the width of the structural member has an increased concrete strength due to triaxial confinement. This increase in strength can be assumed for all the faces of that node. The precast bent cap of this design example is wider than the girder bearing areas, and the triaxial confinement at Node A is therefore taken into account. The triaxial confinement modification factor, $m$, is determined as illustrated in Figure 5 and outlined in the calculation below (please note that $m$ cannot be greater than 2):

$$m = \sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{(40.0 \text{ in.})(35.0 \text{ in.})}{(21.0 \text{ in.})(16.0 \text{ in.})}} = 2.04 > 2 \quad \therefore \text{Use } m = 2$$

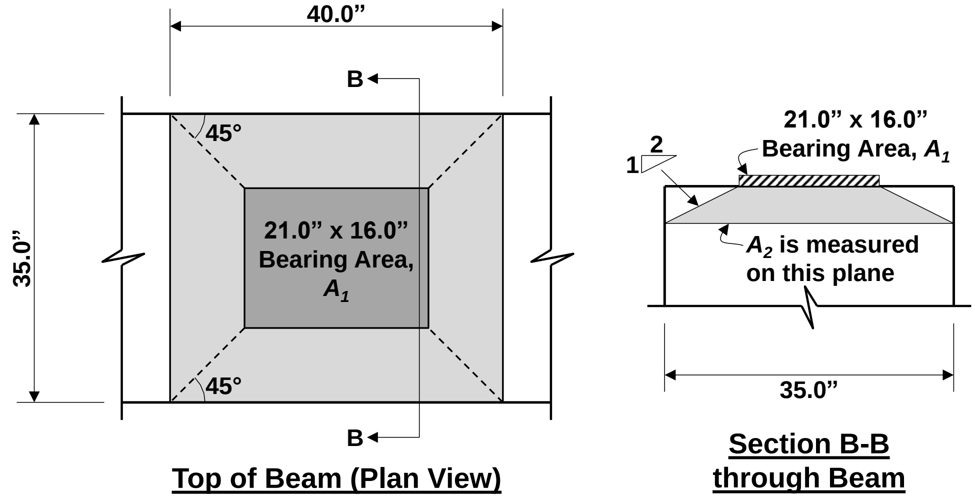


Fig. 5 Determination of the triaxial confinement modification factor, $m$, at Node A

The bearing face of Node A is checked first. The strength of the bearing face is calculated and compared to the applied load as demonstrated below (refer to Figure 4). Please note that the concrete efficiency factor, $v$, for the bearing face of a CCT node is 0.70 according to the proposed STM provisions.

*Bearing Face (Node A – CCT)*

Factored load: $F_u = 360.0\text{ kips}$

Concrete efficiency factor: $\nu = 0.70$

Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (2)(0.70)(3.6\text{ ksi}) = 5.04\text{ ksi}$

Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(5.04\text{ ksi})(21.0\text{ in.})(16.0\text{ in.})$

$\phi F_n = 1185.4\text{ kips} > 360.0\text{ kips } \boldsymbol{OK}$

Therefore, the bearing face at Node A has adequate strength.

The tie force at Node A results from the anchorage of the longitudinal reinforcing bars (i.e., bonding stress) and does not concentrate at the back face of the node. Thus, the back face does not resist a direct force and is not critical provided the reinforcement is properly anchored. A check of the back face is therefore unnecessary according to the proposed STM procedure.

The strength of the strut-to-node interface of Node A is considered next. This check is performed as shown below. According to the proposed STM provisions, the concrete efficiency factor, $v$, for the strut-to-node interface for all node types (i.e., CCC, CCT, and CTT nodes) is defined by the expression $v = 0.85 - f'_c/20$ ksi, where $f'_c$ has units of ksi. Please note that $v$ cannot exceed 0.65 or be less than 0.45.

*Strut-to-Node Interface (Node A – CCT)*

Factored load: $F_u = 625.9$ kips

Concrete efficiency factor: $\nu = 0.85 - {}^{3.6\text{ ksi}}/_{20\text{ ksi}} = 0.67 > 0.65$

$\therefore$ Use $\nu = 0.65$

Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (2)(0.65)(3.6\text{ ksi}) = 4.68\text{ ksi}$

Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(4.68\text{ ksi})(18.0\text{ in.})(16.0\text{ in.})$

$\phi F_n = 943.5\text{ kips} > 625.9\text{ kips } \boldsymbol{OK}$

Thus, the strength of Node A is sufficient to resist the applied factored forces.

Now, the geometry of the most critical node, Node B, is defined and the strength is checked. The geometry of the left and right portions of Node B are illustrated in Figures 6 and 7, respectively.

$$w_s = l_b \sin\theta + h_a \cos\theta$$
$$= (12.5\text{ in.})\sin 35.11° + (7.2\text{ in.})\cos 35.11°$$
$$= 7.19\text{ in.} + 5.89\text{ in.} = 13.1\text{ in.}$$

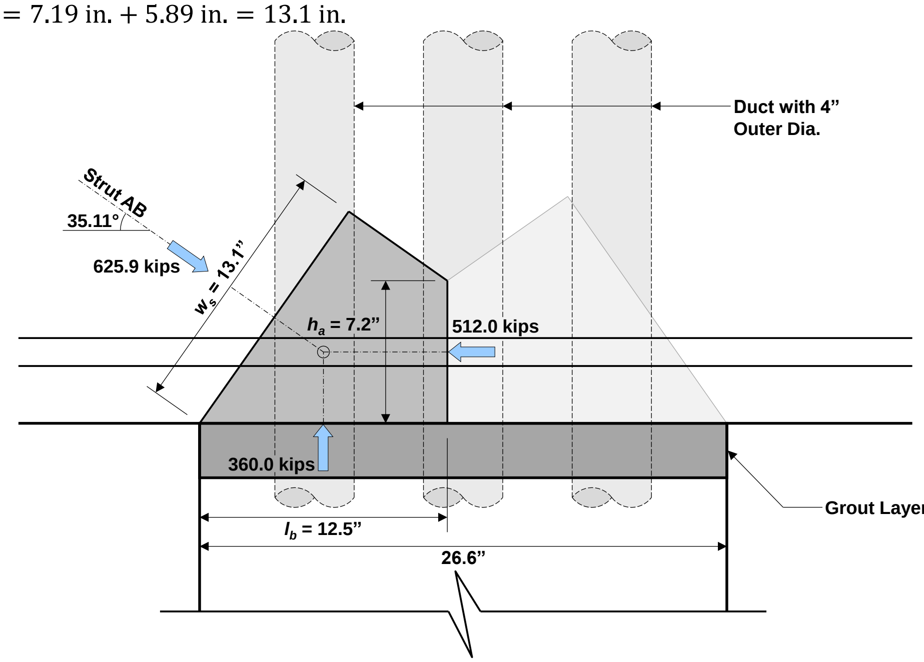


Fig. 6 Node B – left portion

$$\begin{aligned} w_s &= l_b \sin\theta + h_a \cos\theta \\ &= (14.1\text{ in.})\sin 35.01° + (7.2\text{ in.})\cos 35.01° \\ &= 8.09\text{ in.} + 5.90\text{ in.} = 14.0\text{ in.} \end{aligned}$$

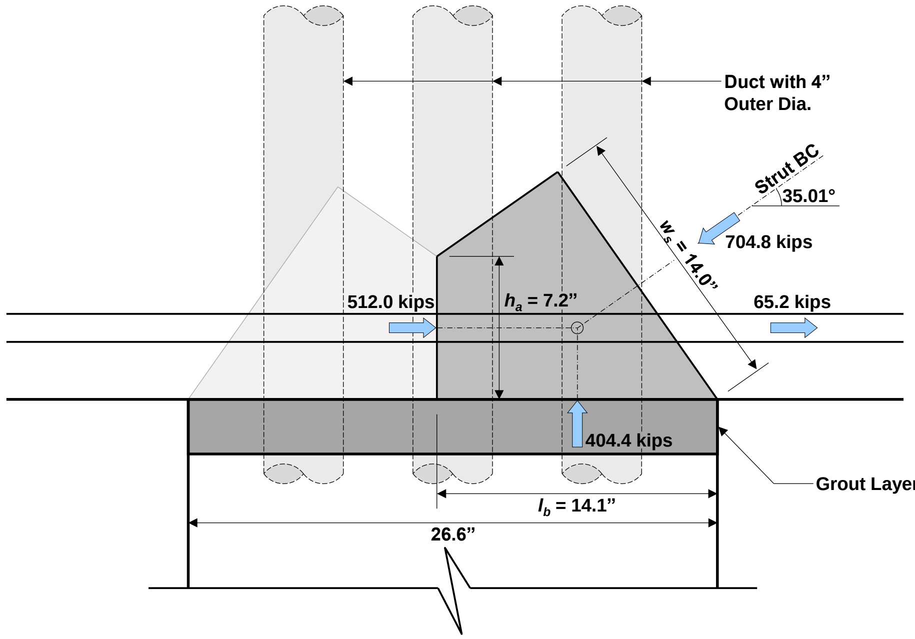


Fig. 7 Node B – right portion

The length of the bearing face for each portion of the node is based on the division of the column reaction that was performed prior to the development of the strut-and-tie model (refer to Figure 3). The height of the back face, $h_a$, is taken as twice the distance from the surface of the bent cap to the bottom chord of the STM. The length of the strut-to-node interface for each portion of the node is determined from the calculations provided in Figures 6 and 7. The width of the node into the page is taken as the dimension of the equivalent square column, or 26.6 in. Since no tensile forces act on the left portion of the node, it is treated as a CCC node (i.e., the concrete efficiency factors for CCC nodes are applied). The right portion of the node is treated as a CCT node since one tie force is present.

As with Node A, the triaxial confinement of Node B can be considered and applied to the strength of each face of the node. The triaxial confinement modification factor, $m$, is calculated using the dimensions of the equivalent square column as follows:

$$m = \sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{(35.0\text{ in.})^2}{(26.6\text{ in.})^2}} = 1.32 < 2 \quad \therefore \text{Use } m = 1.32$$

The bearing strength at Node B is checked first. The entire column reaction acting over the area of the column is considered. Please recall that the bent cap is supported on a layer of grout. The strength of the concrete, however, controls the design. The area of the grouted vertical ducts are conservatively subtracted from the total bearing area. The concrete efficiency factor for the bearing face of a CCT node ($\nu$ = 0.70) governs the strength check shown below.

*Bearing (Node B - CCT)*
Factored load: $F_u = 764.4$ kips
Concrete efficiency factor: $\nu = 0.70$
Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (1.32)(0.70)(3.6 \text{ ksi}) = 3.33 \text{ ksi}$
Effective bearing area: $A_{cn} = (26.6 \text{ in.})^2 - 6 \cdot \pi (2 \text{ in.})^2 = 632.2 \text{ in.}^2$
Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(3.33 \text{ ksi})(632.2 \text{ in.}^2)$
$\phi F_n = 1473.7 \text{ kips} > 764.4 \text{ kips } \boldsymbol{OK}$

Therefore, the bearing strength at Node B is adequate.

The strut-to-node interfaces of both the left and right portions of Node B are evaluated next. The checks are performed in the same manner as the check of the strut-to-node interface of Node A. Two grouted ducts are subtracted from each interface area. The effect of the ducts on the strengths of the nodal faces is uncertain, and further research is needed to determine any potential reduction in strength due to the presence of the ducts.

*Strut-to-Node Interface (Left Portion of Node B – CCC)*
Factored load: $F_u = 625.9$ kips
Concrete efficiency factor: $\nu = 0.85 - {}^{3.6 \text{ ksi}}/_{20 \text{ ksi}} = 0.67 > 0.65$
$\therefore$ Use $\nu = 0.65$
Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (1.32)(0.65)(3.6 \text{ ksi}) = 3.09 \text{ ksi}$
Effective bearing area: $A_{cn} = (13.1 \text{ in.})(26.6 \text{ in.}) - \frac{2}{\cos(90° - 35.11°)} \cdot \pi (2 \text{ in.})^2 = 304.8 \text{ in.}^2$
Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(3.09 \text{ ksi})(304.8 \text{ in.}^2)$
$\phi F_n = 659.3 \text{ kips} > 625.9 \text{ kips } \boldsymbol{OK}$

*Strut-to-Node Interface (Right Portion of Node B – CCT)*
Factored load: $F_u = 704.8$ kips
Concrete efficiency factor: $\nu = 0.85 - {}^{3.6 \text{ ksi}}/_{20 \text{ ksi}} = 0.67 > 0.65$
$\therefore$ Use $\nu = 0.65$
Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (1.32)(0.65)(3.6 \text{ ksi}) = 3.09 \text{ ksi}$
Effective bearing area: $A_{cn} = (14.0 \text{ in.})(26.6 \text{ in.}) - \frac{2}{\cos(90° - 35.01°)} \cdot \pi (2 \text{ in.})^2 = 328.6 \text{ in.}^2$
Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(3.09 \text{ ksi})(328.6 \text{ in.}^2)$
$\phi F_n = 710.8 \text{ kips} > 704.8 \text{ kips } \boldsymbol{OK}$

Therefore, the strut-to-node interface of each portion of Node B has adequate strength.

The two portions of Node B share a back face. The concrete efficiency factor for the back face of a CCT node ($\nu$ = 0.70) therefore governs the strength check. The projected area of the vertical ducts are conservatively subtracted from the total area of the back face, as demonstrated below:

*Back Face (Node B – CCT)*
Factored load: $F_u = 512.0$ kips
Concrete efficiency factor: $\nu = 0.70$
Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = (1.32)(0.70)(3.6 \text{ ksi}) = 3.33 \text{ ksi}$
Effective bearing area: $A_{cn} = (7.2 \text{ in.})(26.6 \text{ in.}) - 2 \cdot (4 \text{ in.})(7.2 \text{ in.}) = 133.9 \text{ in.}^2$
Factored nominal resistance: $\phi F_n = \phi \cdot f_{cu} \cdot A_{cn} = (0.7)(3.33 \text{ ksi})(133.9 \text{ in.}^2)$
$\phi F_n = 312.1 \text{ kips} < 512.0 \text{ kips}$ ***NO GOOD***

The strength check reveals that the back face of Node B does not have adequate strength to resist the factored force. Please recall that longitudinal reinforcement is provided along the bottom chord of the STM. If this longitudinal reinforcement is detailed to develop its yield stress in compression, the bars will contribute to the strength of the back face of the node. If 4 No. 11 bars are provided along the bottom of the bent cap, the back face of Node B has sufficient strength, as indicated by the following calculations:

*Back Face with Longitudinal Reinforcement (Node B – CCT)*
Factored load: $F_u = 512.0$ kips
Concrete efficiency factor: $\nu = 0.70$
Limiting compressive stress: $f_{cu} = m \cdot \nu \cdot f_c' = 3.33 \text{ ksi}$
Effective bearing area: $A_{cn} = 133.9 \text{ in.}^2$
Factored nominal resistance:
$\phi F_n = \phi \cdot \left(f_{cu} \cdot A_{cn} + f_y \cdot A_{sn}\right) =$
$(0.7)[(3.33 \text{ ksi})(133.9 \text{ in.}^2) + (60 \text{ ksi})(4 \cdot 1.56 \text{ in.}^2)]$
$\phi F_n = 574.2 \text{ kips} > 512.0 \text{ kips}$ ***OK***

Thus, the critical node of the strut-and-tie model has adequate strength to resist the factored loads.

PROPORTION CRACK CONTROL REINFORCEMENT

The proposed STM provisions require that crack control reinforcement be provided in each orthogonal direction within the deep beam regions of a member to provide adequate strength and serviceability performance. This distributed reinforcement restrains cracks in the concrete caused by the transverse tension that crosses diagonal bottle-shaped struts. The reinforcement should be spaced evenly within the effective strut area and must satisfy the following expressions:

$$\rho_v = \frac{A_v}{b_w s_v} \geq 0.003 \quad (1)$$

$$\rho_h = \frac{A_h}{b_w s_h} \geq 0.003 \quad (2)$$

where:

$A_h$ = total area of horizontal crack control reinforcement within spacing $s_h$ (in.$^2$)
$A_v$ = total area of vertical crack control reinforcement within spacing $s_v$ (in.$^2$)
$b_w$ = width of member's web (in.)
$s_v$, $s_h$ = spacing of vertical and horizontal crack control reinforcement, respectively (in.)

Using two-legged No. 5 stirrups, the required spacing of the vertical crack control reinforcement is:

$$A_v = 0.003 b_w s_v \quad \rightarrow \quad 2(0.31 \text{ in.}^2) = 0.003(35 \text{ in.}) s_v$$
$$s_v = 5.90 \text{ in.}$$

Using No. 5 bars as skin reinforcement, the required spacing of the horizontal crack control reinforcement is:

$$A_h = 0.003 b_w s_h \quad \rightarrow \quad 2(0.31 \text{ in.}^2) = 0.003(35 \text{ in.}) s_h$$
$$s_h = 5.90 \text{ in.}$$

To satisfy these requirements, two legs of No. 5 stirrups spaced at 5.5 in. are provided, and No. 5 bars spaced at about 5.75 in. are used as skin reinforcement.

PROVIDE NECESSARY ANCHORAGE FOR TIES

According to the proposed STM provisions, the top and bottom chord (i.e., longitudinal) reinforcement must be properly anchored at the ends of the bent cap (i.e., the bars should be developed at Nodes A and B). The available length for development of the tie bars is measured from the point where the centroid of the reinforcement exits the extended nodal zone, as shown at Node A in Figure 8.

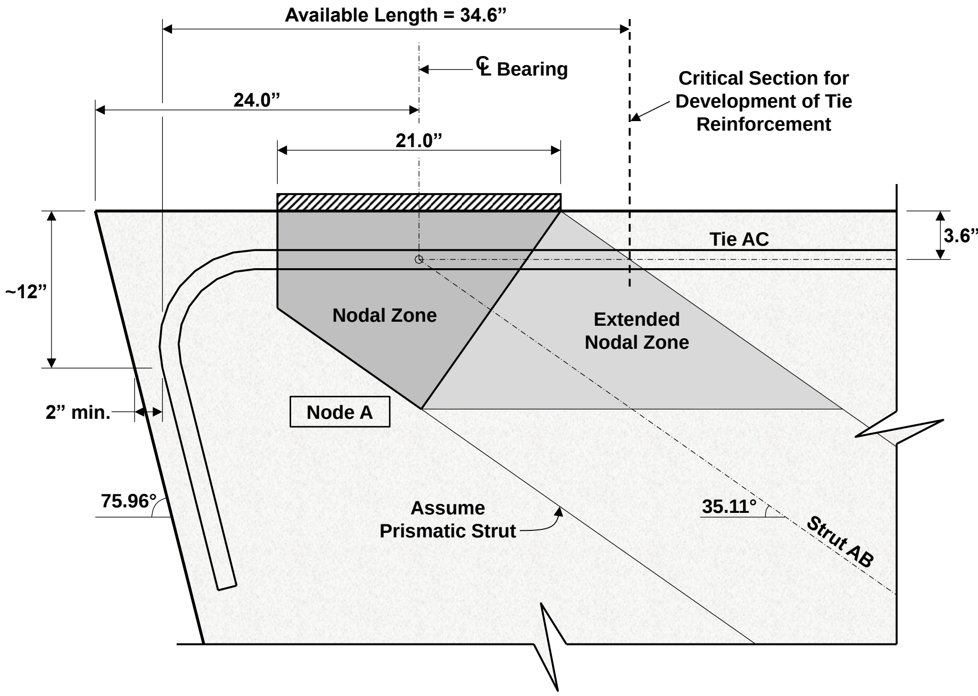


Fig. 8 Anchorage of top chord reinforcement at Node A

The available length for the development of Tie AC at Node A is 34.6 in., as shown in Figure 8 and calculated as follows:

$$\begin{aligned}\text{Avail. length} &= 24.0\text{ in.} + {}^{21.0\text{ in.}}\!/_{2} + {}^{3.6\text{ in.}}\!/_{\tan(35.11°)} - {}^{12\text{ in.}}\!/_{\tan(75.96°)} - 2\text{ in.}\\ &= 34.6\text{ in.}\end{aligned}$$

The use of 90-degree hooks provide adequate anchorage for the tie bars. The longitudinal bars along the bottom chord of the bent cap are properly anchored at Node B if they are extended to the end of the member, leaving the required clear cover.

## REINFORCEMENT LAYOUT

The final reinforcement layout for the bent cap subjected to the given loads is shown in Figure 9. The details in this figure are continued throughout the length of the member. Please note that additional steps are needed to properly design and detail the connections between the columns and the precast bent cap to ensure sufficient strength and serviceability. The complete connection details are not shown in Figure 9 for clarity of the bent cap reinforcement.

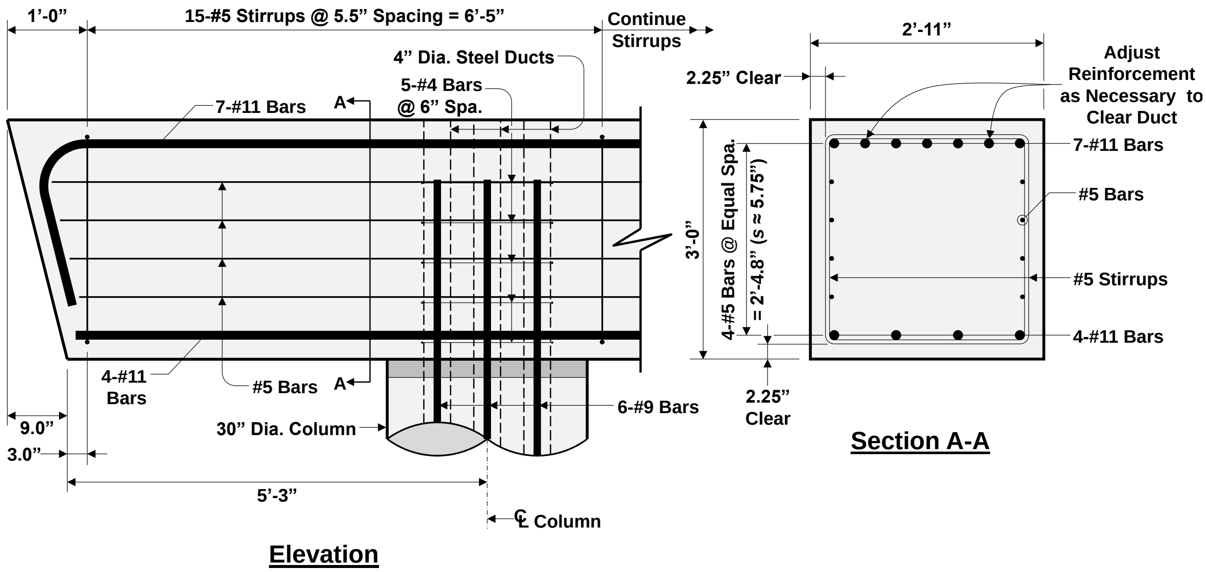


Fig. 9 Reinforcement details

## SUMMARY AND CONCLUSIONS

The STM design provisions presented in Birrcher et al.[1] and Williams et al.[2] provide a standard procedure that can be applied to virtually any structural component that exhibits deep beam, or D-region, behavior. The design of a precast concrete bent cap representative of those encountered in accelerated bridge construction has been demonstrated for a particular load case. The simplified STM procedure results in a more efficient design than that which may be achieved with existing AASHTO LRFD strut-and-tie modeling provisions. Given the bearing geometries and final reinforcement layout presented above, application of AASHTO LRFD (2013) would necessitate significant increases in the bent cap cross-section and/or specified concrete strength.

Due to the uncertain effect of the grouted vertical ducts on the strength of the nodal regions, assumptions were made for the node near the column-bent cap connection. Additional research is needed to refine the strut-and-tie modeling procedure at such connections. The STM recommendations nevertheless result in a procedure that can be routinely implemented for the design of such bridge components.

## REFERENCES

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