Al-Azhar University Civil Engineering Research Magazine (CERM)
Vol. (45) No. (1) January 2023

Strut-and-Tie Modeling of Short and Deep Coupling Beams

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ملخص البحث :

يقدم هذا البحث نموذجا بسيطا لعوارض الإقتران (لحوائط القص) القصيرة والعميقة بتطبيق أحكام الضاغط والشداد الواردة بالفصل الثالث والعشرون بكود معهد الخرسانه الأمريكي رقم 318 لسنة 2019. ومن أجل التصميم المباشر لعوارض الإقتران القصيرة والعميقة المعرضة لإجهادات قص عالية ذات التسليح التقليدي، تم تقديم نموذج ضاغط وشداد أحادي عام واشتقاق معادلات تصميم بسيطة. ويتألف هذا النموذج من ضاغط خرساني مائل مباشر عبر العارضة جنبا إلي جنب مع روابط الشد الأفقية المقابلة وشبكة حديد التسليح الجذعي التي تتطلبها أكواد البناء للتحكم في الشروخ. علما بأن هذا النوع من عوارض الإقتران يستخدم حيثما تكون زاوية ميل الضاغط أكبر من الحد الأدني لشرط كود معهد الخرسانة الأمريكي البالغ 25 درجة. ولإثبات سهولة تطبيق نموذج الضاغط والشداد الأحادي المقترح البسيط والمباشر، تم عرض عينة لأمثلة تصميم لعوارض إقتران ومقارنتها مع الحسابات الأخرى وأظهرت المقارنة توافقا جيدا.

Abstract

This paper presents simple modeling of short and deep coupling beams by application of the strut-and-tie provisions of ACI 318-19 Ch. 23. For the direct design of short and deep coupling beams subjected to high shear demand with conventional reinforcement, a general one-panel strut-and-tie model is introduced and simple design equations have been derived. It consists of a direct inclined compression concrete strut through the beam along with corresponding tension ties and the crack-control web reinforcement required by building codes. This type of coupling beam is used wherever the strut angle is greater than the ACI minimum of 25 degrees. To demonstrate the ease of implementation of the proposed simple and direct one-panel strut-and-tie model, sample design examples are presented and compared with other calculations and the comparison shows a good agreement.

Keywords

Abaqus program; Conventional reinforcement; Coupling beam; Finite-element; High shear force demand; One-panel strut-and-tie model; Sectional model; Short and deep.

INTRODUCTION

Sometimes, short and deep coupling beams are used to couple shear walls, creating a system known as coupled shear walls, **Figure 1**. Coupling beams often have a small span-depth ratio and are heavily loaded. Conventional reinforcement using longitudinal and transverse bars or diagonal reinforcement are the two most often used types in coupling beams. Experimentally, diagonal reinforcement in coupling beams allows them to withstand higher loads and dissipate more energy than conventionally reinforced beams (Paulay, 1969). Only short and deep coupling beams with conventional reinforcement are considered in this work, which occasionally refers to coupling beams throughout.

The application of sectional method in building codes can result in a substantial variation in the maximum shear stress limit. This is due to the high shear force which significantly influences the behavior of coupling beams as they do not obey the plane-sections-remain-plane. Instead, according to (Mihaylov and Franssen, 2017), coupling beams are commonly designed using strut-and-tie systems. Coupling beams of the Burj Khalifa in Dubai were designed using the strut-and-tie model (Lee et al., 2008). The strut-and-tie method described in Appendix A of (ACI 318-02, 2002) and (ACI 318-02, 2005) allows reinforced concrete coupling beams to be designed for significantly higher loads than would be allowed if the ACI sectional design method were used, it was determined. Additionally, reinforced concrete coupling beams have much higher shear capacities than those predicted by the strut-and-tie model of (ACI 318-05, 2005), according to the results of the nonlinear finite element studies. In addition, (Zhao et al., 2018) focused on creating a strut-and-tie model to predict the shear capacity in steel fiber reinforced concrete coupling beams with small span-to-depth ratios. The diagonal mechanism, vertical mechanism, and horizontal mechanism compose the strut-and-tie model that is being proposed. The one-panel strut-and-tie model is comparable to the diagonal mechanism.

This study analyzes the one-panel strut-and-tie model for designing conventionally reinforced concrete short and deep coupling beams based on Ch. 23 of (ACI 318-19, 2019). First, using Abaqus' linear elastic finite element analysis, the directions of the struts and ties are constructed. Next, the one-panel strut-and-tie model is introduced and reviewed. From this, simple equations have been derived to directly design coupling beams with small span-to-dept ratios and high shear force demand. Additionally, design examples are provided to show how simple and straightforward the proposed approach is applicable.

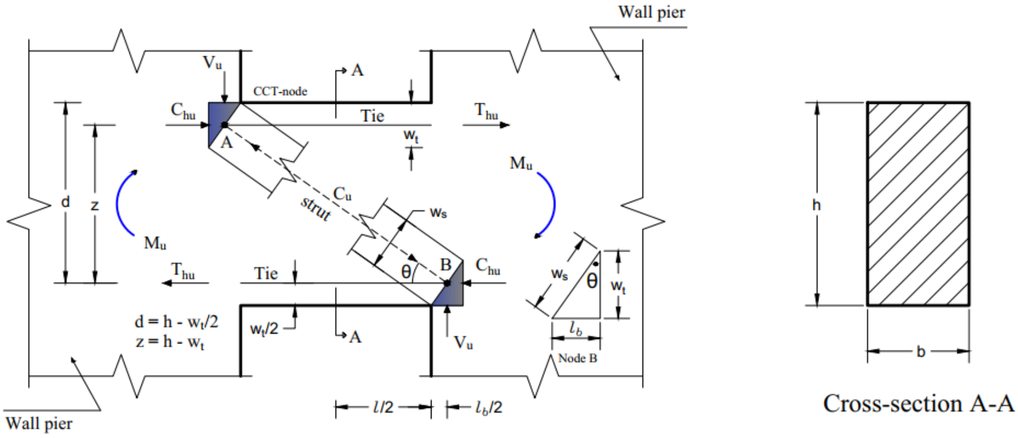


Figure 1: Geometry and load demand of one-panel strut-and-tie model.

PROPOSED ONE-PANEL STRUT-AND-TIE MODEL

To design conventionally short and deep reinforced concrete coupling beams, a general one-panel strut-and-tie model, or as often named, the arch-action mechanism, was proposed, **Figure 1**. The geometry and basic assumptions are next presented and discussed. Besides, simple and direct design equations have been derived and worked design examples are presented and compared with other calculations.

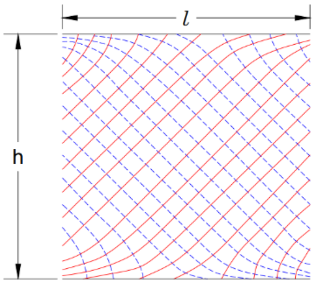
For the one-panel model shown in **Figure 1**, a direct inclined compression concrete strut *AB* is formed between compressive end zones to resist shear demand V_u (Zhao et al., 2018). The inclined strut, *AB*, along the beam with its corresponding horizontal tension ties, T_h, transfers the shear force from one wall pier to the other by *arch-action* mechanism (Mihaylov and Franssen, 2017). Considering coupling beams with span-to-depth ratios less than 2, the results obtained from linear elastic finite-element analysis, based on Abaqus program, showed that the inclined compressive principal stress trajectories mainly between two compression end zones were along the shear span, as shown in **Figure 2a**. This was also verified by (Zhao et al., 2018). Therefore, wherever the strut angle is greater than 25 degrees as specified by (ACI 318-19, 2019), the one-panel strut-and-tie model can be considered the most efficient one. This complies with (AASHTO LRFD, 2004) provisions which require minimizing the number of vertical ties between a load and support while still satisfying the 25-degree minimum. The one-panel model was used, in 2012, in the design of the coupling beams of Kingdom Tower in Jeddah, Saudi Arabia. Following the Z-shaped load path approach proposed by (El-Zoughiby, 2021), the one-panel strut-and-tie model can simply and conceptually be constructed by combining the two opposite similar simple cantilevers where each one transfers half of the shear force demand, $0.5V_u$, as shown in **Figures 2b** and **2c**.

Basic assumptions

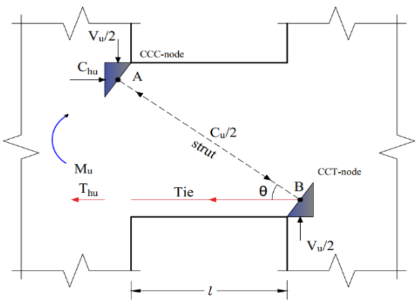
The considered basic assumptions of the one-panel model are as follows:

1. In strut-and-tie models, dotted lines represent struts, solid lines represent ties, and the intersection of struts and ties defines the nodes.
2. Factored loads are equal for both ends of coupling beams.
3. All nodes are assumed hydrostatic.

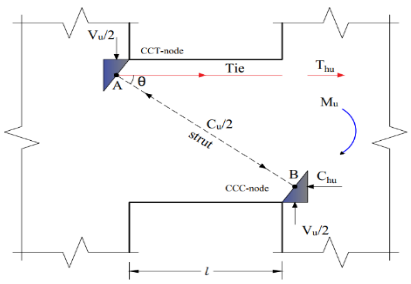
4. Strut factor β_s is 0.75 for interior struts crossed by an orthogonal grid of reinforcement with a minimum ratio of 0.0025 in each direction. For boundary struts, β_s is taken 1.0.
5. Node factor β_n is 1.0 for *CCC* nodes, 0.80 for *CCT* nodes, and 0.60 for *CTT* nodes. However, as all nodes are assumed hydrostatic, the node which has a common interface with the inclined strut, the beta factor of the inclined strut or 0.75 is the one that will control. Therefore, the design beta factor will be 0.75.
6. Confinement factor β_c, for struts and nodes, which permits an increase in the effective compressive strength f_{ce} of the end of a strut (node) if the strut is confined by surrounding concrete, is taken 1.0; for simplicity matters and just to be in the safe side. For this, β_c will be excluded from the equations.
7. The target demand-to-capacity ratio (*DC*) is set to 0.95, when calculating the width of the inclined strut.
8. The geometry of the beam is defined by its span *l*, its depth *h*, and its breadth *b*.
9. Due to the expected moment reversal, the longitudinal reinforcement remains the same at top and bottom and anchored with its full development length within the wall piers and, also, the distributed horizontal reinforcement.



a- Principal stress trajectories - Abaqus



b- Simple cantilever with bottom steel



c- Simple cantilever with top steel

Figure 2: One-panel strut-and-tie model

Proposed design equations

Considering **Figure 1**, the one-panel model consists of one direct inclined compression strut, *AB*, two horizontal top and bottom tension ties, T_h, and two *CCT* nodes; *A* and *B*.

Span-depth ratio

For the one-panel strut-and-tie model, the current span-to-dept ratio should be less than the required one which is next presented in Eq. (2). For the nodal zones *A* and *B*, **Figure 1**, the horizontal width of the bearing area is l_b, the height of the vertical side is w_t, and the width of the strut is w_s. Considering the two different definitions of the angle θ of the inclined strut (where θ should not be less than the ACI minimum of 25 degrees); or simply, for ease, one vertical-to-two horizontal) where $l_b/w_t = 1/2 = (h - w_t)/(l + l_b)$, we obtain:

$$(\frac{l}{h}) = 2(1 - 2.5\frac{l_b}{h}) \tag{1}$$

introducing the value of l_b from the next presented Eq. (16) gives:

$$(\frac{l}{h}) = 2(1 - \frac{V_u}{0.18f_c'bh}) \tag{2}$$

where f_c' is the cylinder strength and bh is the cross-section area of the coupling beam. Equation (2) indicates that $\frac{l}{h}$ for the one-panel model should be less than two.

Inclined strut

With reference to **Figure 1**, considering the vertical equilibrium at nodes *A* and *B*, the internal force C_u in the inclined strut *AB* due to the factored shear demand V_u is given by:

$$C_u = \frac{V_u}{sin\,\theta} \tag{3}$$

and the inclined strut is, then, proportioned using:

$$\phi C_n \geq C_u \tag{4}$$

where *n* represents nominal strength and, when design is based on the strut-and-tie method, the resistance factor ϕ is taken 0.75 (ACI 318-19, 2019). The strut strength C_n is given in ACI Code Section 23.4.1 as:

$$C_n = f_{ce}^s A_{cs} = f_{ce}^s(w_s b) \tag{5}$$

where f_{ce}^s is the effective compressive strength of the concrete in the strut, A_{cs} is the cross-section area of the end of the strut where C_n is being evaluated and w_s and b are the width and breadth of the inclined strut, respectively. The term f_{ce}^s includes two beta factors. One factor, β_s, accounts for potential longitudinal cracking along the strut due to transverse tension. A second factor, β_c, accounts for confinement provided by transverse reinforcement or surrounding concrete, or both, at the end of a strut and the node it is connected to. Excluding the factor β_c, the f_{ce}^s ACI Code Section 23.4.3 is:

$$f_{ce}^s = 0.85\beta_s f_c' \tag{6}$$

where the strength of concrete in struts tends to be less than the cylinder strength, f_c', by 0.85. The value for the strut factor, β_s, is taken 0.75, as previously assumed.

For the inclined strut, at its interface with nodes *A* and *B*, the demand-to-capacity ratio DC_{strut} is:

$$DC_{strut} = \frac{C_u}{\emptyset C_n} = \frac{V_u}{sin\,\theta} \times \frac{1}{\emptyset(0.85\beta_s f_c')(w_s b)} \tag{7}$$

observing that:

$$sin\,\theta = \frac{l_b}{w_s} \tag{8}$$

$$DC_{strut} = \frac{1}{l_b} \times \frac{V_u}{\emptyset(0.85\beta_s f_c')(b)} \tag{9}$$

Horizontal ties

The horizontal tie force T_{hu} in **Figure 1** (where $T_{hu} = C_{hu}$ and at either top or bottom chord $T_{hu} + C_{hu}$ or $2T_{hu}$ equals $C_u\,cos\theta$) is determined as follows:

$$T_{hu} = \frac{C_u}{2}cos\,\theta \tag{10}$$

and, thus, the longitudinal reinforcement A_{sl} required to resist the tie force, T_{hu}, is:

$$A_{sl} = \frac{T_{hu}}{\emptyset f_y} \tag{11}$$

Nodes A and B

As an example, considering the horizontal face of nodes *A* and *B*, the nominal strength F_{nn} is given in ACI Code Section 23.9.1 as:

$$F_{nn} = f_{ce}^{n} A_{cn} = f_{ce}^{n}(l_b b) \tag{12}$$

where f_{ce}^{n} is the effective compressive strength of the concrete in the node, A_{cn} is the area of the face of the node that the vertical strut acts on, taken perpendicular to the axis of the strut, and l_b and b are the width and breadth of that face, respectively. Again, upon excluding the factor β_c, the effective compressive node strength is

$$f_{ce}^{n} = 0.85\beta_n f_c' \tag{13}$$

value for the node factor, β_n, is 0.80, as previously assumed. The demand-to-capacity ratio DC_{node} is thus:

$$DC_{node} = \frac{V_u}{\emptyset F_{nn}} = \frac{V_u}{\emptyset(0.85\beta_n f_c')(l_b b)} \tag{14}$$

obviously, putting $\beta_s = \beta_n = 0.75$, since the beta factor of the inclined strut or 0.75 is the one that will control, and can be named the <u>design beta factor</u>, as all nodes are assumed hydrostatic nodal zones, the strut-to-node *DC* ratio is 1.0.

From Eq. (9), if the target demand-to-capacity ratio DC_{strut} is set to 0.95, as previously assumed, the width l_b can be easily calculated from:

$$DC_{strut} = 0.95 = \frac{1}{l_b} \times \frac{V_u}{\emptyset(0.85\beta_s f_c')(b)} = \frac{1}{l_b} \times \frac{V_u}{0.75(0.85 \times 0.75 \times f_c')(b)} \tag{15}$$

or simply, l_b is obtained from

$$l_b = \frac{V_u}{0.45 f_c' b} \tag{16}$$

Re-defining the strut angle θ, as shown in **Figure 1**, in its two different ways as:

$$tan\,\theta = \frac{h - w_t}{l + l_b} = \frac{l_b}{w_t} \tag{17}$$

or, in a simple form, as:

$$w_t^2 - h w_t + l_b(l + l_b) = 0 \tag{18}$$

or, simply as:

$$w_t = \frac{h - [(h)^2 - 4 l_b (l + l_b)]^{0.5}}{2} \tag{19}$$

knowing the length and depth of the coupling beam, *l* and *h*, and the width of the vertical strut or bearing length, l_b, the width of the horizontal strut or tie, w_t, can be calculated using Eq. (19). The strut angle θ can, then, be re-calculated utilizing the values of l_b and w_t as:

$$\theta = tan^{-1}\frac{l_b}{w_t} \tag{20}$$

if θ were greater than 25 degrees (or simply 1-to-2), calculate the width w_s from:

$$w_s = \frac{l_b}{sin\,\theta} \tag{21}$$

otherwise, if θ is less than 25 degrees, the one-panel strut-and-tie model option fails and a truss or multi-panel strut-and-tie model is required.

Design example

In the following, the one-panel model will be used for the design of conventionally reinforced concrete coupling beams subjected to high shear demand.

<u>Reference:</u>

Kingdom Tower and Retail Mall in Jeddah, Saudi Arabia, 2012

<u>Dimensions:</u>

h = 1600m, l = 1500mm, and b = 800mm

<u>Materials:</u>

$f_c' = 85$MPa, $f_y = 420$MPa, and $\phi = 0.75$

<u>Factored shear and moment:</u>

The factored shear and corresponding factored moment diagrams are shown in **Figure 3b**:

V_u = 8131kN and $M_u = 0.5V_u(l + l_b) = T_{hu}(h - w_t) = C_{hu}(h - w_t)$

Factored moment and horizontal forces can be calculated once l_b and w_t are determined.

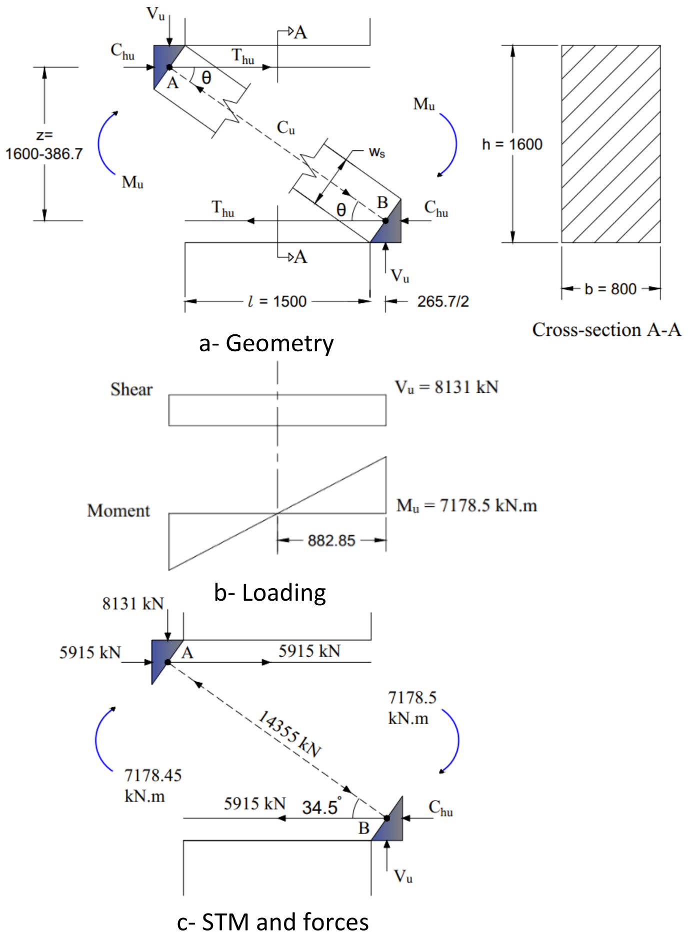


Figure 3: Geometry, loading, and STM and forces - Example 1

<u>Span-depth ratio:</u>

Check if the current ratio $\frac{l}{h}$ were less than the required one given by Eq. 2:

Using Eq. (2), we obtain $\frac{l}{h} = 2\left(1 - \frac{V_u}{0.18 f_c' bh}\right) = 2\left(1 - \frac{8131000}{0.18 \times 85 \times 800 \times 1600}\right) = 1.17$

The current $\frac{l}{h}$ ratio is $\frac{1500}{1600} = 0.9375$ which is less than the required ratio or 1.17, which means that the one-panel model is okay and proceed with **Figures 1** and **3**.

Geometry, strut-and-tie model, and strut-tie forces:

Width l_b of vertical strut at its interface with nodes *A* and *B* as given by Eq. 16:

$$l_b = \frac{V_u}{0.45 f_c{'} b} = \frac{8131 \times 1000}{0.45 \times 85 \times 800} = 265.7\text{mm}$$

Width w_t of horizontal strut or tie at its interface with nodes *A* and *B* as given by Eq. 19:

$$w_t = \frac{nh - \left[(nh)^2 - 4nl_b(l + l_b)\right]^{0.5}}{2n} = \frac{1600 - \left[1600^2 - 4 \times 265.7(1500 + 265.7)\right]^{0.5}}{2} = 386.7\text{mm}$$

Strut-tie angle θ as given by Eq. 20

$\theta = tan^{-1}\frac{l_b}{w_t} = tan^{-1}\frac{265.7}{386.7} = 34.5°$ which is greater than 25 degrees.

Width w_s of inclined strut at its interface with nodes *A* and *B* as given by Eq. 21:

$$w_s = \frac{l_b}{sin\theta} = \frac{265.7}{\sin 34.5} = 469\text{mm}$$

The one-panel strut-and-tie model is shown in **Figure 3c** and the model forces are:

$V_u = 8131\text{kN}$ and, thus, $C_u = \frac{V_u}{sin\,\theta} = \frac{8131}{sin\,34.5} = 14355\text{kN}$ which gives

$T_{hu} = C_{hu} = \frac{C_u}{2} cos\,\theta = \frac{14355}{2}\cos 34.5 = 5915\text{kN}$ and

$M_u = 0.5V_u(l + l_b) = 0.5 \times 8131(1.5 + 0.2657) = 7178.5\text{kN.m}$

Adequacy of struts and nodes:

No need to check the adequacy of struts and nodes, as the geometry is defined based on a target *DC* ratio of 0.95 and a design beta factor of 0.75. However, for illustration purposes;

Inclined strut

$DC_{strut} = \frac{14355 \times 1000}{0.75(0.85 \times 0.75 \times 85)(469.0 \times 800)} = 0.94$ which is okay.

Nodes *A* and *B* – vertical strut

$DC_{node} = \frac{8131 \times 1000}{0.75(0.85 \times 0.75 \times 85)(265.7 \times 800)} = 0.94$ which is, also, okay.

Longitudinal reinforcement:

Longitudinal reinforcement A_{sl} required to resist tension tie force T_{hu} as given by Eq. 11:

The tie force at the bottom and the top of the beam (T_{hu}) due to the applied factored moment is equal to 5915kN. The required reinforcement for each tie is:

$A_{sl} = \frac{T_{hu}}{\Phi f_y} = \frac{5915 \times 1000}{0.75 \times 420} = 18778\text{mm}^2$

It can be provided with the use of 24T32 bars (19310mm^2).

Anchorage of longitudinal reinforcement:

The tie reinforcement, due to load reversal, must be anchored for the full yield strength at the beam-wall pier junction. The code-required length for a straight T32 bar is equal to:

$$L_d = \frac{f_y}{1.1\lambda\sqrt{f_c{'}}} \frac{\psi_t\psi_e\psi_s\psi_g}{\frac{c_b + k_{tr}}{d_p}} d_p = \frac{420}{1.1 \times 1.0 \times \sqrt{85}} \frac{1.3 \times 1.0 \times 1.0 \times 1.0}{2.50} 32 = 689\text{mm}$$

<u>Required confinement distributed vertical and horizontal reinforcement for inclined strut:</u>
ACI Code Sections 9.9.3.1(a) and (b) require vertical and horizontal reinforcement, respectively, not less than 0.0025*bs* at a maximum spacing of 0.20*d*; or 0.20(1600–0.5×386.7) = 281.3mm, but not more than 300mm.

Vertical reinforcement

The vertical reinforcement selected is 4 legs-T16 at 250mm on centers. This reinforcement layout satisfies the minimum requirement as follows:

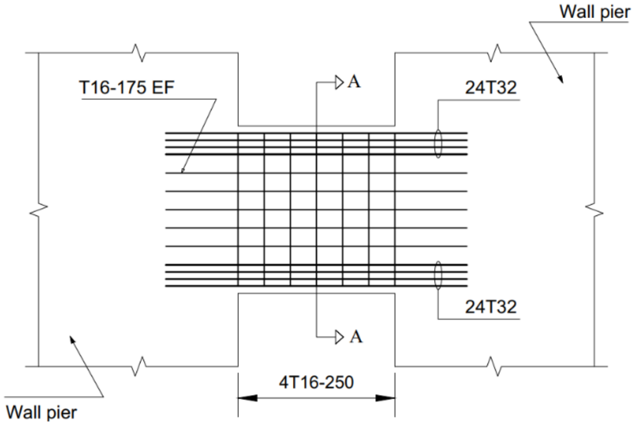
$\rho_v = \frac{A_v}{bs_v} = \frac{4\times201.1}{800\times250} = 0.004022$ which is greater than 0.0025.

Horizontal reinforcement

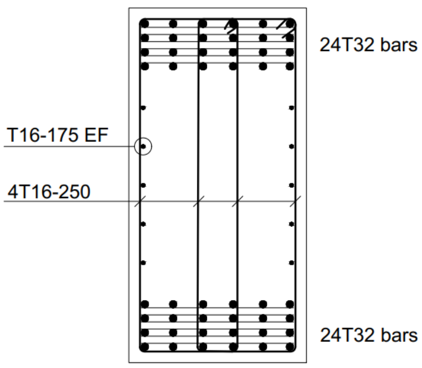
The horizontal reinforcement selected is T16 at 175mm on centers in each face. This reinforcement layout satisfies the minimum requirement as follows:

$\rho_h = \frac{A_h}{bs_h} = \frac{2\times201.1}{800\times175} = 0.00287$ which is greater than 0.0025.

<u>Design summary and detailing:</u>
As in **Figure 4**, the design resulted in 24T32 rebars with straight ends at top and bottom, 4T16 closed vertical stirrups at 250mm on center, and 2T16 face horizontal bars at 175mm on center. Due to load reversal, the reinforcement provided was symmetric.



a- Elevation view



b- Cross-section A-A

Figure 4: Reinforcement detailing.

COMPARISON OF PREDICTIONS

The comparison between only six, just for illustration purposes, sample calculations published in literature and the main results from predictions of the proposed one-panel strut-and-tie model is next presented. All samples are conventionally reinforced concrete coupling beams. To begin with, the geometry, factored loads, materials, and design methods of six Kingdom Tower and Retail Mall coupling beams are shown in **Table 1**. The reinforcing details of all coupling beams are shown in **Table 2**. The proposed model design results are shown in **Table 3**. The comparison shows a good agreement. The small variation is due to considering a demand-to capacity ratio *DC* of 0.95 and not 1.0.

Table 1 - Geometry, factored loads, materials, and design method of considered beams

Ref.	Beam ID	Geometry			Factored shear demand, kN	Materials			Design method
		b mm	*h* mm	*l* mm		f'_c MPa	$f_y{}^{a}$ MPa	$f_y{}^{b}$ MPa	
Kingdom Tower, Jeddah, Saudi Arabia, 2012	04-1	800	1600	1500	8131	85	420	420	Strut-and-tie: ACI 318-02, Appendix A
	06-4	600	1600	1200	4769	85	420	420	
	07-4	600	2100	1500	6702	85	420	420	
	13-4	600	1800	1500	5301	75	420	420	
	15-3	600	1800	1200	5884	75	420	420	
	16-3	600	1800	1200	6324	85	420	420	

[a]flexural reinforcement and [b]web horizontal and vertical reinforcement.

Table 2 - Reinforcing details

Ref.	Beam ID	Reinforcement			Stirrups		
		Top bars	Bott. bars	Side bars, each face	Size	Spacing, mm	Type
Kingdom Tower, Jeddah, Saudi Arabia, 2012	04-1	28T32	28T32	T16-175	T16	250	4-legs
	06-4	11T32	11T32	T16-200	T16	275	
	07-4	15T32	15T32	T16-200	T16	275	
	13-4	14T32	14T32	T16-200	T16	275	
	15-3	13T32	13T32	T16-200	T16	275	
	16-3	14T32	14T32	T16-200	T16	275	

Table 3 - Predictions of current study

Ref.	Beam ID	Reinforcement			Stirrups		
		Top bars	Bott. bars	Side bars, each face	Size	Spacing, mm	Type
Kingdom Tower. Jeddah, Saudi Arabia, 2012	04-1	24T32	24T32	T16-175	T16	250	4-legs
	06-4	10T32	10T32	T16-200	T16	275	
	07-4	14T32	14T32	T16-200	T16	275	
	13-4	13T32	13T32	T16-200	T16	275	
	15-3	12T32	12T32	T16-200	T16	275	
	16-3	13T32	13T32	T16-200	T16	275	

CONCLUSIONS

As coupling beams are usually cast with often small span-to-depth ratio, this paper presents a general one-panel strut-and-tie model with simple design equations derived to directly design conventionally reinforced concrete short and deep coupling beams subjected to high shear demand. The following is a summary of conclusions from this study:

1. A general one-panel strut-and-tie model has been proposed and, based on some basic assumptions, simple design equations have been derived to directly design conventionally reinforced coupling beams having small span-to-depth ratios and subjected to high shear demand. The model complies with AASHTO LRFD provisions which require minimizing number of vertical ties. It consists of a direct inclined strut with horizontal tension ties. It is effectively used wherever the strut angle is greater than the ACI minimum of 25 degrees.
2. The proposed design simple and direct process begins with checking if the current span-to-depth ratio is less than the required one, and, then, the geometry of nodes is determined. The design equations are derived based on a 0.95 *DC* ratio and, therefore, no need to check adequacey of struts and nodes. Lastly, the longitudinal and web vertical and horizontal reinfocement is calculated and detailed.
3. The design results of the the proposed model are compared with sample calculations puplished in literature and the comparison shows a good agreement.

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