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Review and Analysis of Codes and Experimental Tests for Steel and Steel-Reinforced Coupling Beams: Advancing Canadian Design

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ABSTRACT

Steel and Steel-Reinforced Coupling Beams (SCB and SRCBs) are increasingly being adopted within reinforced coupled wall systems due to their superior ductility, energy-dissipation capacity, and practicality. Despite their growing popularity, specific design procedures must still be developed for SCBs and SRCBs in Canadian building codes. This paper will review existing experimental studies on SCB and SRCB and analyze how different design decisions can impact their response. Specifically, degradation of rotation, initial stiffness, and overstrength in both SCB and SRCBs were examined. Our findings reveal potential gaps in the stiffness calculation and question the appropriateness of these calculations. We also discuss adapting the current design philosophies used in the US to Canadian design philosophy, acknowledging that Canadian designs utilize a different approach. This study provides valuable considerations for Canadian practitioners looking to incorporate SCBs and SRCBs into the Canadian design philosophy.

Keywords: Composite coupling beam, energy dissipation, inelastic rotation, nonlinearity, seismic design.

INTRODUCTION

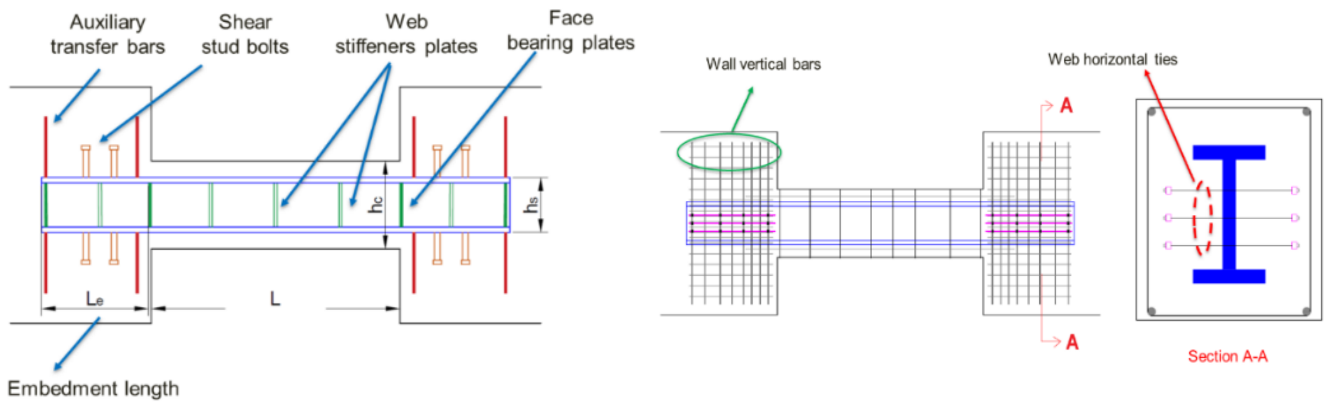
The core-wall system, a popular seismic force-resisting system in North America, efficiently resists the lateral forces caused by earthquakes on buildings. The core-wall system is versatile, allowing for elevator placement and maximizing constructible area. The core-wall system is composed of interconnected shear walls linked by coupling beams. These coupling beams play a crucial role in the structure's overall seismic response.

Coupling beam selection depends on the aspect ratio defined as the ratio of overall sections length over depth, and required inelastic rotation. Diagonally reinforced coupling beams (DRCBs) are used for low aspect ratio applications, while conventional coupling beams with longitudinal bars are preferred for higher aspect ratio values. However, conventional coupling beams may have limited rotational capacity and low strength. Steel coupling beams (SCBs), which consist of embedded steel sections (typically wide-flange shapes) within adjacent walls, provide enhanced rotational capacity and ductility. SCBs facilitate faster construction, minimize reinforcement congestion, and deliver comparable performance in a more compact form. However, due to the exposed steel's vulnerability to fire, steel-reinforced coupling beams (SRCBs) were developed, incorporating concrete encasement around the beam for added protection.

The size and shape of the steel section, the reinforcement configuration, and the strength of the concrete are the main design criteria for SRCBs. The behaviour of SRCBs highly depends on the composite action between the steel section and its surrounding concrete. In composite coupling beams, the concrete confines the steel part and avoids local buckling, while the steel section resists the tension and compression pressures.

Several factors must be considered when designing SCBs and SRCBs. Embedment of the steel section into the wall plays a crucial role in enabling the section to achieve its full strength before the connection fails. Engineers must design a suitable embedment length to ensure proper composite action in the boundary region (L_e). They may also opt to use auxiliary transfer bars to facilitate the load-transferring mechanism from the wall to the embedded steel section. Moreover, some designers may incorporate shear stud bolts, enabling more composite action between the steel section and surrounding concrete. The web of stiffener plates can be used to increase the bonding between the steel section and surrounding concrete, and to prevent the web buckling of the steel section under shear loads. These plates are commonly used in SCBs, rather than SRCBs, and have been found to enhance the performance of face bearing plates. Figure 1(a) illustrates the typical detailing used in SRCBs.

In terms of reinforcing steel within the concrete, the encased beam features ties around the beam and vertical bars throughout the section. One challenge with both SRCBs and SCBs is securing ties around the boundary elements of the shear wall. Putting vertical bars in the boundary region facilitates the transfer of the loads from the adjacent walls to the embedded steel section. In addition to vertical bars, transverse reinforcement is required in the boundary region to improve the ductility of the SCB and SRCBs at the beam-wall interface, and prevent the wall vertical reinforcement from buckling under applied axial forces. Figure 1(b) depicts the reinforcing layout at the boundary region for both SCB and SRCBs.



(a) *(b)*

Figure 1. Sketch of an SRCB (a)without and (b)with reinforcements.

In the present study, we review the design philosophy of SCB and SRCB, focusing on the USA's provisions, specifically the AISC 341-16 [1] and ASCE 41-17 [2] guidelines. We then examined various experimental and numerical studies conducted to understand better the behaviour and function of SCB and SRCB under seismic loads [5-8] [11-19]. These investigations reveal that SRCBs demonstrate greater ductility, strength, and energy dissipation when compared to traditional RC coupling beams. Although most experimental programs have focused on strength requirements in the embedment region, fewer studies have examined these beams' initial stiffness and the impact of design decisions on rotational capacity. In this study, we analyze the results from previously tested SCBs and SRCBs and scrutinize the seismic design parameters for each research program. Based on these findings, we provide a discussion of the studies and present some future avenues of work. Ultimately, this paper proposes Canadian design recommendations for SCB and SRCBs to enhance their performance in seismic conditions.

OVERVIEW OF THE CURRENT STATE OF DESIGN PROVISIONS FOR STEEL AND STEEL-REINFORCED COUPLING BEAMS

At present, Canadian design codes, such as the National Building Code 2020 (NBC) [3] and CSA A23.3-19 [4], do not provide guidance for Steel Coupling Beams (SCBs) and Steel-Reinforced Coupling Beams (SRCBs). In contrast, the US has provisions for these in their codes. ASCE 41-17 [2] categorizes coupled shear walls incorporating SCB or SRCB as composite walls.

Composite Ordinary Shear Walls (C-OSW) and Composite Special Shear Walls (C-SSW) are two types of composite walls. C-OSW is designed for limited inelastic deformations, while C-SSW is intended to dissipate substantial energy through inelastic deformations. Both C-OSW and C-SSW share the same seismic force reduction factors as walls as the equivalate Reinforced Concrete (RC) coupling beams.

According to ASCE 41-17 [2], the C-OSW and C-SSW must adhere to the American Institute of Steel Construction (AISC) 341-16 [1] standard. The AISC 341-16 [1] standard recommends design provisions for SCB and SRCB specific to whether the coupling beam will be within a C-OSW or a C-SSW. For both C-OSW and C-SSW systems, the shear strength of the SCB is selected as the minimum of $\frac{2M_p}{L}$ and V_p, where M_p is the plastic flexural moment, and V_p is the nominal shear strength of the steel section calculated per Eq. (1) and (2), respectively.

$$M_p = f_y Z \tag{1}$$

$$V_p = 0.6 f_y A_{tw} \tag{2}$$

Where f_y is the specified yield strength of the steel section, Z is the plastic section modulus about the axis of bending, A_{tw} is the area of the steel beam web, and L is the length of the clear span of the coupling beam.

For both C-OSW and C-SSW, the shear strength of the SRCB should be taken as the minimum of $\frac{2M_p}{L}$ and $\varphi V_{n,comp}$ (with $\varphi = 0.9$), which M_p is calculated by sectional analysis software considering the effects of the concrete encasement and existing reinforcement in the beams' cross section. $V_{n,comp}$ is the available shear strength of the composite coupling beam by considering the concrete encasement, longitudinal and transverse reinforcement in the reinforced concrete beam, and are calculated per Eq. (3) and (4) for C-OSW and C-SSW systems, respectively.

$$V_{n,comp} = V_p + \left(0.166\sqrt{f_c'}b_{wc}d_c + \frac{A_{sr}F_{ysr}d_c}{s}\right), C - OSW \tag{3}$$

$$V_{n,comp} = 1.1R_yV_p + 0.21\sqrt{R_cf_c'}b_{wc}d_c + \frac{1.33R_{yr}A_{sr}F_{ysr}d_c}{s}, C - SSW \tag{4}$$

Where f'_c is the concrete compressive strength (Mpa), b_{wc} is the width of concrete encasement mm), d_c is the effective depth of concrete encasement (mm), A_{sr} is the area of transverse reinforcement (mm2), F_{ysr} is the nominal yielding strength of transverse reinforcement (Mpa), s is the spacing of transverse reinforcement (mm), R_y is the ratio of the expected yield stress of the steel profile material to the specified minimum yield stress, R_c is the factor to account for expected strength of concrete which is considered as 1.5, and R_{yr} is the ratio of the expected yield stress of the transverse reinforcement material to the specified minimum yield stress.

The reason for the difference between the equations proposed by AISC 341-16 [1] for calculating the shear strength of SCB and SRCBs relies on the fact that each of the C-OSW and C-SSW systems provides different levels of nonlinearity to the whole structure. Therefore, in the C-SSW system in which higher plastic deformations are expected, the shear strength of the SRCBs is increased by R_y, R_c, and R_{yr}, in addition to some coefficients to consider the strengths of the expected materials.

To ensure proper anchorage and transfer of forces between the beam and the surrounding concrete, steel sections in both SCB and SRCBs are required to be embedded into the adjacent walls (L_e). The embedment length is determined per Eq. (5) for SCB and SRCBs in both C-OSW and C-SSW systems. Eq. (5) was developed based on the test results of the conducted research programs by Mattock and Gaafar, 1982 [5], Shahrooz et al., 1993 [6], Harries et al., 1993 [7] and 1997 [8]. The term $\frac{b_w}{b_f}$ in Eq. (5) accounts for the spreading of the compressive stress beneath the beam flange. These developed models will be discussed in the following sections.

$$V_{n,connection} = 4.04\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58 - 0.22\beta_1}{0.88 + \frac{g}{2L_e}}\right) \tag{5}$$

$$V_n = \frac{2(1.1R_y)M_p}{g} \le (1.1R_y)V_p \tag{6}$$

Where $\varphi V_{n,connection}$ (with $\varphi = 0.9$) is the minimum required shear strength at the wall-beam interface (N), which in the C-OSW system it should be taken as the smallest of Eq. (1) and 2 or SCB and Eq. (3) for SRCB, and in C-SSW system it should be taken as Eq. (6) for SCB and Eq. (4) for SRCB, bw is the thickness of the wall (mm), b_f is the width of beam flange (mm), β_1 is the factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in the American Concrete Institute (ACI) 318-19 [9], and g is the clear span of the coupling beam plus the wall concrete cover (c) at each end of the beam (mm).

AISC 341-16 [1] offers information on the detailing of steel beams and refers to ACI 318-19 [9] for detailing reinforced concrete walls, except for clause H4.5b.(c), which necessitates additional vertical reinforcing. The vertical reinforcement bars in the adjacent walls assist in transferring the stresses resulting from applied lateral forces due to earthquakes from the coupling beam to the walls, spreading them more uniformly and reducing the possibility of localized damage or failure. Additionally, wall vertical bars in the embedment region control the crack propagation in the wall-beam interface, particularly at the steel section flange location, avoiding the creation of gaps in that region. According to AISC 341-16 [1], the provided force by wall vertical bars in the embedment region should be at least equal to the required shear strength at the beam-wall interface ($V_{n,connection}$). Thus, the area of required wall vertical bars (A_{sl}) is obtained per Eq. (7).

$$A_{sl} \geq \frac{V_{n,connection}}{f_{ysl}} \tag{7}$$

Where f_{ysl} is the nominal yielding strength of the wall vertical bars (Mpa).

For the C-SSW system, AISC 341-16 [1] mandated to use of auxiliary transfer bars, in which the first layer is placed at a distance no less than $d_c/2$ from the embedment length end, and the second one be located to coincide with the location of wall vertical bars closest to the face of the wall. Auxiliary transfer bars are placed in the boundary region since they facilitate the transfer of bearing stresses from the flanges to the surrounding concrete, and allow for a more symmetric distribution of strength and stiffness under load reversal due to the transfer of tension at the flange concrete interface, thereby limiting the separation of flange and concrete. Also, these bars will assist the load-transfer mechanism in the embedment region by transferring the axial forces resulting from overturning moments and gravity loads applied on the wall to the steel section in the boundary region.

Auxiliary transfer bars should be welded to the top and bottom of the steel profile's flanges by mechanical couplers. The area of auxiliary transfer bars at (each layer) at each side of the steel section flanges should be at least $0.03 f_c' \frac{L_e b_f}{f_{ya}}$, and the total area of these bars in the boundary region should not exceed $0.08 L_e b_w - A_{sl}$, where f_{ya} is the specified yield strength of the auxiliary transfer bars (Mpa),

Based on AISC 341-16 [1], face bearing plates should be utilized at certain locations throughout the clear span of the steel coupling beams in the C-SSW system. Face bearing plates are important in the design procedure since they allow the formation of a diagonal strut between the two plates. Also, these plates could be considered web stiffeners since they will resist the concentrated loads applied on the coupling beams and prevent the crippling of the steel section web. The spacing of the face bearing plates is a function of the coupling beams' clear span length (L) and expected total chord rotation.

For modelling purposes, AISC 341-16 [1] recommended considering cracked effective stiffness for elastic analysis according to ACI 318-19 [9] for all concerned concrete structural elements. In this regard, ACI 318-19 [9] clause 6.6.3.1.1. recommended that the cracked effective moment of inertia for walls and beam would be considered $0.35 I_g$, where I_g is the gross moment of inertia of the structural element. For calculating the effective stiffness of SRCBs, the transformed concrete section using elastic material properties is suggested. According to AISC 341-16 [1] commentary, the effective moment of inertia of the steel coupling beam (I_{eff}) can be calculated per Eq. (8).

$$I_{eff} = 0.6 I_s \left(1 + \frac{\lambda 12 E_s I_s}{g^2 G A_{tw}}\right) \tag{8}$$

Where I_s is the moment of inertia of the steel coupling beam (mm^2), λ is the cross-section shape factor for shear equals to 1.5 for W-shape steel sections, E_s is the modulus of elasticity of steel (Mpa), and G is the shear modulus of steel (Mpa).

Aside from AISC 341-16 [1] and ACI 318-19 [9] recommendations for calculating the effective stiffness for SCB and SRCBs, the Los Angeles Tall Buildings Structural Design Council (LATBSDC) 2020 [10] recommends modifying the moment of inertia and area of steel coupling beams per Eq. (9) and (10), respectively.

$$EI_{eff} = 0.07 \left(\frac{L}{h_c}\right) EI_{tr} \tag{9}$$

$$EA_{eff} = 0.4 E_s A_{tw} \tag{10}$$

Where EI_{tr} is the transformed moment of inertia for an SRCB, calculated per Eq. (11).

$$(EI)_{tr} = \left(\frac{E_c I_g}{5}\right) + E_s I_s \tag{11}$$

Where E_c is the elastic modulus of concrete (Mpa).

Overall, the AISC 341-16 [1] adopts a conservative approach to the design of SRCB and SCB. This approach includes several requirements for face-bearing plates and auxiliary transfer bars, leading to excessive congestion and making the implementation cost-prohibitive. Consequently, numerous ongoing research programs aim to improve constructability while still achieving comparable performance. Further discussions on how design parameters impact the response of SRCB and SCB are detailed in subsequent sections, offering insights on potential modifications to enhance these systems' overall effectiveness and practicality.

REVIEW OF EXPERIMENTAL PROGRAMS OF SRC AND SRCBS

The experimental data from 33 SRC and SRCBs were collected, and two main seismic design parameters, namely $\theta_{80\%}$ and overstrength, were studied. The experimental programs were conducted from 1993 to 2021. Some experimental programs included the loading on the wall to simulate the real earthquake forces. In this study, we examine the hysteretic response of each tested coupling beam. Specifically, we look at the backbone parameters, shown in Figure 2. According to Figure 2, K_i is the initial stiffness, V_{max} is the maximum shear strength, and the degradation rotation ($\theta_{80\%}$) is the rotation corresponds to 80% of the maximum shear force.

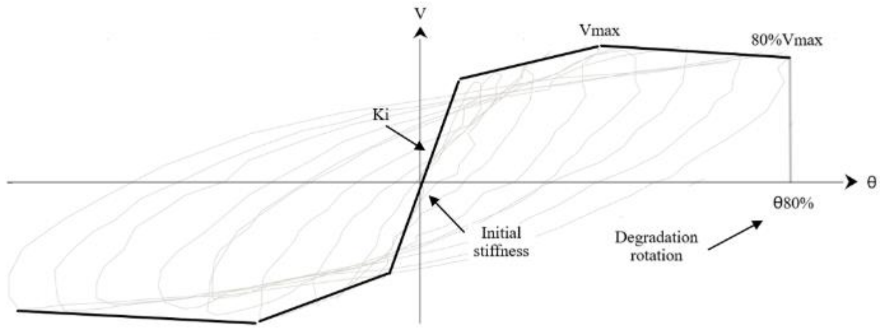


Figure 2. Backbone parameters for coupling beams [20]

In our review of experimental studies, we extracted information that we believed would impact the hysteretic response of the coupling beams. Researchers have explored many different characteristics of coupling beams, making it challenging to identify some correlations. Table 1 presents some of the characteristics of the completed studies.

In Table 1, we recorded whether the beam had concrete encasement (CE), or in other words if the specimen was an SRCB. Out of the 33 specimens, 48% had concrete encasement. Table 1 also shows the aspect ratio, which is the clear span length(L) over the depth of the steel beam (h_s), and the composite beam's aspect ratio, defined as the clear span length (L)over the depth of the concrete-encased beam (h_c). Only 36% of the specimens had face bearing plates (FBP). Some tested specimens had auxiliary transfer bars (ATR), and shear stud bolts (SD). According to Table 1, almost half of the specimens had auxiliary transfer bars, and shear stud bolts. Some specimens (30%) had web horizontal ties (WHT), meaning that horizontal ties were passed through the steel section's webs in the boundary region. Out of 33 specimens, 11 specimens had web stiffeners (WS) placed inside or outside the boundary region to stop the web crippling of the steel sections. Only two specimens had axial restraint (AR). The type of axial restraints used in these experimental programs were post-tensioned rods (PT rods) or floor slabs. The embedment length (L_e) was determined for each specimen based on different models developed for calculating the minimum required embedment length. These models are described in detail in the next section.

Table 1. General information about studied specimens.

Author	Name	CE	L/h_s	L/h_c	FBP	ATR	SSB	WHT	WS	AR	Le (mm)	L (mm)	f'c (Mpa)
Shahrooz et al.1993	Wall 1	-	2.33	-	-	-	-	Yes	-	-	864	267	35
	Wall 2	-	2.33	-	-	-	-	Yes	-	-	864	267	35
	Wall 3	-	2.33	-	-	Yes	-	Yes	-	-	864	267	35
Harries et al., (1993) & (1997)	S1	-	3.43	-	Yes	-	-	-	outside	-	600	1200	26
	S2	-	3.46	-	Yes	-	-	-	inside & outside	-	600	1200	43
	S3	-	1.29	-	Yes	-	-	-	inside & outside	-	500	450	33
	S4	-	3.44	-	Yes	-	-	-	-	-	600	1200	35
Shahrooz et al.2001(a)	CB1	-	5.34	-	-	Yes	-	-	outside	-	216	406	14
	CB2	Yes	5.34	4.00	-	Yes	-	-	outside	-	216	406	12
	CB3	Yes	5.34	4.00	-	Yes	-	-	inside & outside	-	216	406	16
	CB4	Yes	5.34	4.00	-	Yes	-	-	-	-	216	406	14
Shahrooz et al.2001(b)	WB1	Yes	5.34	4.00	-	Yes	-	-	-	-	343	406	58
	WB2	Yes	5.34	4.00	Yes	Yes	-	-	-	-	343	406	58
	WB3	Yes	5.34	4.00	Yes	Yes	-	-	-	Slab	343	406	52

Park et al., (2005)	SBVRF	-	2.29	-	Yes	Yes	Yes	Yes	-	-	300	400	30
	SCF	-	2.46	-	Yes	Yes	Yes	Yes	-	-	300	300	30
	FCF	-	4.92	-	Yes	Yes	Yes	Yes	-	-	300	600	30
Park et al., (2006)	PSF	-	3.43	-	-	Yes	Yes	-	outside	-	300	300	30
	PSFF	-	3.43	-	Yes	Yes	Yes	-	outside	-	250	300	30
	PSFFT	-	3.43	-	Yes	Yes	Yes	Yes	outside	-	250	300	30
Fortney et al., (2007)	SCB	-	2.57	-	Yes	Yes	-	-	-	-	775	914	35
Motter et al., (2014)	SRC1	Yes	4.72	3.33	-	-	-	Yes	-	-	813	762	51
	SRC2	Yes	4.72	3.33	-	-	-	Yes	-	-	610	762	51
	SRC3	Yes	3.40	2.40	-	-	-	Yes	-	-	660	549	35
	SRC4	Yes	4.72	3.33	-	-	-	-	-	PT rods	610	762	32
Li et al., (2019)	Beam 1	Yes	3.45	2.00	-	-	Yes	-	-	-	700	500	58
	Beam 2	Yes	5.17	3.00	-	-	Yes	-	-	-	700	750	58
	Beam 3	Yes	8.62	5.00	-	-	Yes	-	-	-	700	1250	58
	Beam 4	Yes	3.42	2.00	-	-	Yes	-	-	-	700	500	58
	Beam 6	Yes	5.00	3.00	-	-	Yes	-	-	-	700	750	58
	Beam 7	Yes	4.84	3.00	-	-	Yes	-	-	-	700	750	58
Nahvinia et al., (2021)	SCBC1	-	2.78	-	-	-	Yes	-	outside	-	290	375	40
	SCBC2	-	2.78	-	-	-	Yes	-	outside	-	290	375	41

Table 2 shows some of the response parameters in addition to the wall loading. In the experimental programs, some researchers tried to capture the stress within the boundary zone in the wall by loading the wall as well as the coupling beam. A notable example of this is Motter Et al., 2014 [11-12], where the experimental setup (Shown in Figure 3) includes actuators, allowing for a cyclic moment on the wall, cyclic shear force on the top of the wall, and cyclic shear force on the coupling beams. Out of the different experimental programs, some studies included a constant load using PT or actuators, or no loading on the wall.

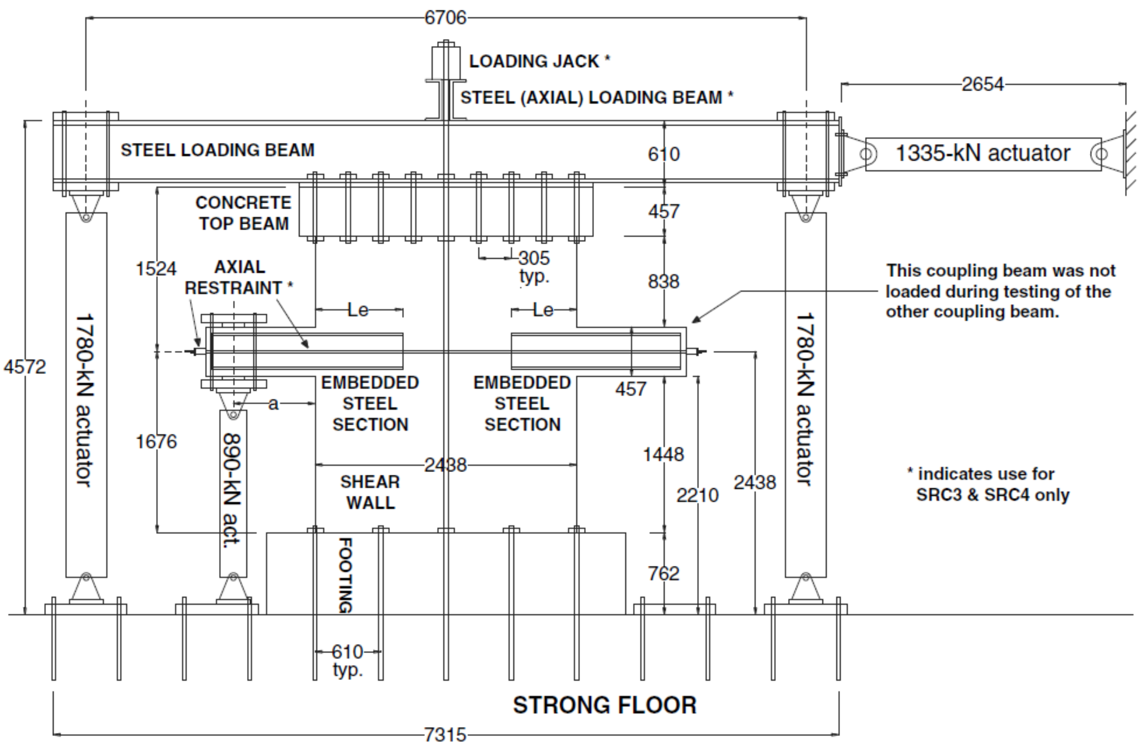


Figure 3. Test setup of the Motter et al., 2014 [11-12] experimental program (Dimensions are in mm).

In SCB and SRCBs, overstrength is defined as the maximum tolerated applied lateral force (V_{max}) over the minimum required shear strength at the beam-wall interface ($V_{n,connection}$), using Eq. (5) without resistance factors. Figure 4(a) illustrates the overstrength of all the SCB and SRCB specimens. As depicted in Figure 4(a), the median response is approximately 1.2. However, some outliers have overstrength values as high as 1.8 or above. Interestingly, some specimens exhibit an overstrength of less than one, suggesting that the calculations or determination of overstrength may not accurately represent the beam's actual behaviour. These discrepancies warrant further investigation and refinement in the design approach to ensure that the overstrength values adequately capture the performance of SCBs and SRCBs under various loading conditions.

Table 2 also introduces $\theta_{80\%}$, a parameter used to assess rotational ductility. Figure 4(b) displays the distribution of $\theta_{80\%}$ for all the specimens. Among all the specimens, the median value of $\theta_{80\%}$ is approximately 7%. However, $\theta_{80\%}$ ranges from 4% to 14%, indicating a significant variation in rotational ductility. The reasons for the variations in overstrength and $\theta_{80\%}$ depend on the design provisions and are explored in subsequent sections. By understanding these variations, structural designers can make more informed decisions when using SCB and SRCBs in seismic force resisting systems and potentially improve the overall performance and ductility of these components.

Notably, as shown in Figure 4(a), some specimens had an overstrength greater than 1.5, which is related to the application of floor span, in which the overstrength will be increased significantly. Furthermore, according to Figure 4(b), some specimens had $\theta_{80\%}$ less than 5%. According to the experimental programs through which these specimens were tested, the researchers decided to stop the testing process, or continue the testing by changing the loading protocol from cyclic to monotonic due to the out-of-plane instability of specimens. Therefore, these specimens experienced a relatively small $\theta_{80\%}$ compared to the others.

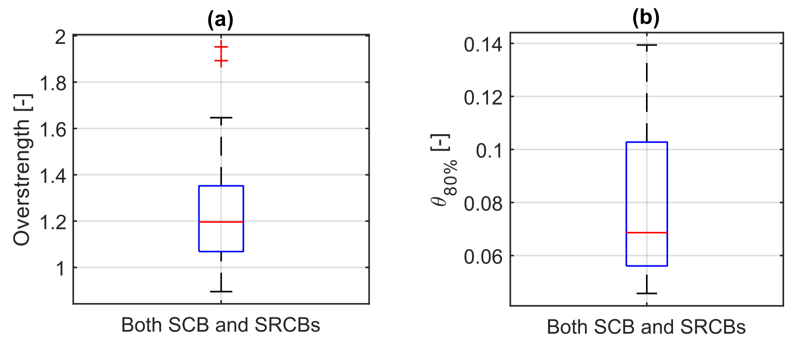


Figure 4. Range of (a) overstrength and (b) $\theta_{80\%}$ for previously tested SCB and SRCBs.

Table 2 introduces the failure modes of each previously tested SCB and SRCBs. According to Table 2, four main types of failure modes could be seen among the tested specimens. Most specimens failed due to the lack of shear or flexural resistance at the beam-wall connection. Furthermore, some of the specimens failed due to the web buckling of the steel section through the clear span. Out of 33 tested specimens, only two experienced flexural failure through the clear span, indicating that more experimental programs are required to study this type of failure mode in SCB and SRCBs.

Table 2. Test results of studied specimens.

Author	Name	Wall's loading protocol	overstrength	$\theta_{80\%}$	Failure mode
Shahrooz et al.1993	Wall 1	reversed-cyclic moment, and constant axial load	1.13	4.6%	Flexural failure at connection
	Wall 2	reversed-cyclic moment, and constant axial load	1.20	4.6%	Flexural failure at connection
	Wall 3	reversed-cyclic moment, and constant axial load	1.29	6.8%	Flexural failure at connection
Harries et al., (1993) & (1997)	S1	Constant axial load (compression)	1.20	8.7%	Shear failure at connection
	S2	Constant axial load (compression)	1.42	7.5%*	Shear failure through clear span
	S3	Constant axial load (compression)	1.35	6.9%*	Shear failure through clear span
	S4	Constant axial load (compression)	1.23	3.3%*	Flexural failure at connection
Shahrooz et al.2001(a)	CB1	Constant axial load (compression)	1.13	5.8%	Shear failure at connection
	CB2	Constant axial load (compression)	0.99	4.6%	Shear failure at connection
	CB3	Constant axial load (compression)	1.16	4.9%	Shear failure at connection
	CB4	Constant axial load (compression)	1.06	5.2%	Shear failure at connection
Shahrooz et al.2001(b)	WB1	reversed-cyclic moment, and constant axial load	1.37	9.4%	Shear failure at connection
	WB2	reversed-cyclic moment, and constant axial load	1.46	13.9%	Shear failure at connection
	WB3	reversed-cyclic moment, and constant axial load	1.89	9.9%	Shear failure at connection
Park et al., (2005)	SBVRF	Constant axial load (compression)	1.07	5.2%	Shear failure at connection
	SCF	Constant axial load (compression)	1.26	12.9%	Shear failure through clear span
	FCF	Constant axial load (compression)	1.06	4.6%	Flexural failure through clear span
Park et al., (2006)	PSF	Constant axial load (compression)	1.06	11.4%	Shear failure at connection
	PSFF	Constant axial load (compression)	1.65	12.7%	Shear failure at connection
	PSFFT	Constant axial load (compression)	1.95	12.3%	Shear failure at connection
Fortney et al., (2007)	SCB	Constant axial load (compression)	1.43	4.1%*	Shear failure at connection
Motter et al., (2014)	SRC1	reversed-cyclic lateral loading and overturning moment	0.91	13.3%	Flexural failure at connection
	SRC2	reversed-cyclic lateral loading and overturning moment	0.90	9.6%	Flexural failure at connection
	SRC3	reversed-cyclic lateral loading and overturning moment, and constant axial load	1.00	11.5%	Flexural failure at connection
	SRC4	reversed-cyclic lateral loading and overturning moment, and constant axial load	0.90	9.5%	Flexural failure at connection
	Beam 1	None	1.23	7.0%	Flexural failure at connection

Li et al., (2019)	Beam 2	None	1.18	6.4%	Flexural failure at connection
	Beam 3	None	1.29	6.0%	Flexural failure at connection
	Beam 4	None	1.23	6.1%	Flexural failure at connection
	Beam 6	None	1.18	6.7%	Flexural failure at connection
	Beam 7	None	1.17	6.9%	Flexural failure at connection
Nahvinia et al., (2021)	SCBC1	Constant axial load (compression)	1.16	5.7%	Shear failure at connection
	SCBC2	Constant axial load (compression)	1.41	9.3%	Flexural failure through clear span

* The $\theta_{80\%}$ was determined at the stop of cyclic loading; the specimen was then tested monotonically [7-8] [19].

EXPERIMENTAL STUDIES ON SCB AND SRCBS

In this section, the impact of different design parameters, such as embedment length (L_e), auxiliary transfer bars (ATBs), etc., on the seismic behaviour of SCB and SRCBs is investigated. Particularly, $\theta_{80\%}$ and overstrength as the seismic design parameters aligned with Canadian design philosophy are studied.

Embedment length (Le)

Embedment length (L_e) is defined as the length of the steel beam extended into the RC wall. The impact of Le on ductility and overall strength of SRC and SRCBs has been studied widely through previously conducted research programs. In 1993, Shahrooz et al. [6] tested three SRCs with embedment lengths calculated based on Mattock and Gaafar (1982) [5] model (Figure 5(a)).

According to Figure 5(a), the calculation of embedment length according to Mattock and Gaafar's model in 1982 [5], is based on satisfying the force and moment equilibrium of the embedded member. In this model, the effect of auxiliary transfer bars, transverse reinforcement in the boundary region, shear studs, face bearing plates, concrete spalling, etc., are not considered.

In 1993, Harries et al. [7] tested two SCBs with adequate embedment length according to Mattock and Gaafar's (1982) [5] model. The experimented specimens had different configurations of web stiffeners affecting the crack propagation at the beam-wall interface and the steel web section.

According to Shahrooz et al., 1993 [6] test results, the concrete near the beam-wall interface was spelled after the numbers of cyclic loading applied on the coupling beams. This means that the connection region does not remain intact during earthquakes, and the spalling of concrete should be taken into consideration for calculating the initial stiffness, which is related to the determination of the fixing point. Therefore, Harries et al., 1993 [7] recommended modifying the embedment length calculation model suggested by Mattock and Gaafar (1982) [5] by considering the effect of concrete spalling (Figure 5(b)).

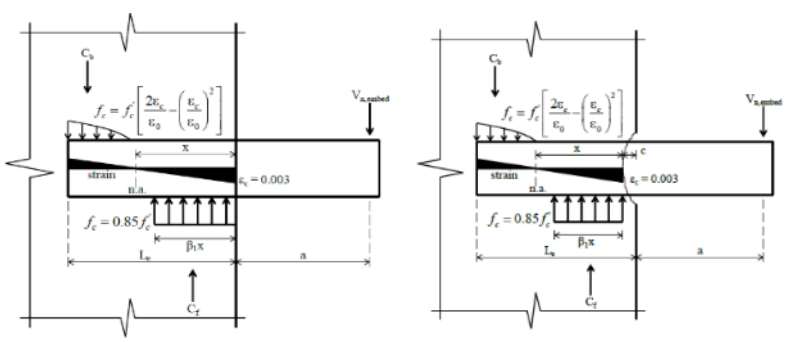


Figure 5. (a) Marcakis and Mitchell (1980) [13] and Mattock and Gaafar (1980) [5], and (b) Harries et al., 1993 [7] model for calculating the minimum embedment length (L_e)

In 2001, Shahrooz et al. [14-15] tested seven SRC and SRCBs during two experimental programs. The embedment length for all seven specimens was determined according to Mattock and Gaafar's (1982) [5] model; therefore, the effect of spalling concrete was not considered. Out of seven tested specimens, the last three had embedment lengths longer than the first four ones by about 59%. This increment in embedment length was considered since according to the test results of the Shahrooz et al., 2001(a) [14] research program, it was seen that some specimens had a premature failure due to the lack of embedment length.

Based on the reported values of $\theta_{80\%}$ for the tested specimens of Shahrooz et al., 2001 (a) [14] and (b) [15], it can be concluded that by increasing the embedment length, $\theta_{80\%}$ and overstrength would be increased by 115% and 45%, respectively.

Since no researcher studied the effect of auxiliary transfer bars and web horizontal ties on the embedment length, Park et al., 2005 [16] recommended a new model of embedment length calculation accounting for the above parameters (Figure 6). Based on the test results of Park et al., 2005 [16] and 2006 [17], the tested specimen with the embedment length proposed by Park et al., 2005 [16] had $\theta_{80\%}$ between 5% to 13%. This variation is highly related to the effect of other design parameters, such as web horizontal ties and face bearing plates.

In 2014, Motter et al. [11-12] tested four SRCBs with different embedment lengths. For SRC1, they calculated the embedment length according to the model suggested by Marcakis and Mitchell (1980) [13] by considering the effect of concrete spalling (Figure 5(a)). Motter et al., 2014 [11-12] claimed that this model provides the upper-bound amount of embedment length. Therefore, they determined the embedment length of specimens SRC2 and SRC4 based on the model suggested by Mattock and Gaafar (1982) [5] by considering the effect of concrete spalling (Figure 5(b)). They claimed that the latter model represents the lower-bound value of embedment length.

They also tested another specimen (SRC3) by calculating the embedment length equal to the average of the required one by Mattock and Gaafar (1982) [5] and Marcakis and Mitchell (1980) [13] models. Based on the test results, it has been seen that SRC1 with upper-bound embedment length experienced the highest $\theta_{80\%}$ (13.3%). SRC 3 recorded lower $\theta_{80\%}$ compared to SRC1 (about 15%) but higher than SRC 2 and SRC4 (almost 16%), indicating that the longer embedment length would result in higher $\theta_{80\%}$.

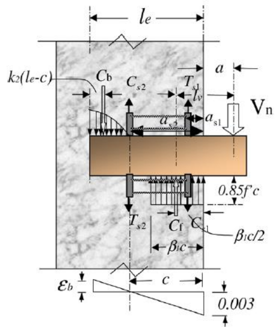


Figure 6. Park et al. 2005 [16] model for calculating the minimum embedment length (L_e)

Overall, many research programs have been conducted to study the minimum required embedment length of steel sections in SCB and SRCBs to provide adequate shear strength at the beam-wall interface and $\theta_{80\%}$. However, none of the recommended models for calculating the embedment length considered the effect of all required design provisions according to AISC 341-16 [1], namely, auxiliary transfer bars, face bearing plates, web horizontal ties, web stiffeners inside and outside of the embedded region, spalling of concrete, and shear studs.

Auxiliary transfer bars (ATBs)

Among the conducted research programs about the performance of SCB and SRCBs, only Shahrooz et al., 1993 [14] particularly investigated the impact of auxiliary transfer bars on $\theta_{80\%}$ and overstrength. According to the test results, it can be concluded that by using auxiliary transfer bars $\theta_{80\%}$ and overstrength would be increased by 32% and 7%, respectively.

Although using auxiliary transfer bars is required according to AISC 341-16 [1] for C-SSW systems, conducted research programs, i.e., Harries et al., 1997 [8] and Motter et al., 2014 [11-12] show that by providing adequate embedment length, sufficient values of inelastic rotation and dissipated energy can be reached. One probable reason for omitting auxiliary transfer bars is the extremely hard construction process for implementing these bars through the highly congested shear walls with different reinforcements.

Face bearing plates (FBPs)

Shahrooz et al., 2001(b) [15] conducted a research program in which they studied the effect of face bearing plates and floor slabs. According to the test results, the specimen (WB2) with face bearing plates experienced almost 5% higher $\theta_{80\%}$ compared to the one (WB1) without any face bearing plates. In addition, it has been seen that using face bearing plates would increase the overstrength of the coupling beams significantly.

To investigate the effect of face bearing plates, Park et al., 2006 [17] did a research program in which two specimens were equipped with face bearing plates (PSFF and PSFFT) and another without face bearing plates (PSF). It has been seen that although the specimens PSFF and PSFFT had shorter embedment lengths compared to the specimen PSF, both specimens with face bearing plates experienced higher $\theta_{80\%}$ and overstrength, about 10% and 36%, respectively.

Concrete encasement (CE)

The first research program to assess the performance of concrete encasement for steel coupling beams was conducted in 2001 by Shahrooz et al. [14-15] through two phases. According to the test results of the Shahrooz et al., 2001(a) [14] research program, the specimen without concrete encasement experienced higher $\theta_{80\%}$ compared to the other tested SRCBs. One probable reason is that the wall connected to the specimens through Shahrooz et al., 2001(a) [14] was loaded axially, meaning no overturning moments were simulated on the walls. However, when the loading protocol was changed during Shahrooz et al., 2001(b) [15] research program, tested SRCBs recorded significantly higher $\theta_{80\%}$ compared to the one tested during Shahrooz et al., 2001(a) [14] without any concrete encasement (about 115%). Several years after 2001, some other research programs were done, including testing SRCBs. However, it is quite challenging to make a conclusion on the effect of the concrete encasement on the $\theta_{80\%}$ and overstrength. One reason is that researchers include other design parameters, such as face bearing plates and auxiliary transfer bars, resulting in different trends for $\theta_{80\%}$ and overstrength.

Web horizontal ties (WHTs)

Web horizontal ties are referred to the ties passed through the web of the steel section in the embedment region. In terms of the construction process, predrilled holes are created in the factory where steel sections are produced, and then the horizontal ties are implemented. Although implementing web horizontal ties seems to be complicated, it is necessary for special shear walls to be constructed in high seismicity regions, according to ACI 318-19 [9]. According to the test results of Park et al., 2006 [17] and Motter et al., 2014, specimens equipped with web horizontal ties experienced higher $\theta_{80\%}$ and overstrength. This increment was more sensible for $\theta_{80\%}$ as the purpose of using web horizontal ties is related to increasing the ductility rather than the strength of coupling beams. Nahvinia et al., 2021 [18] tested two SCBs; one had tighter tie spacing in the embedded region. Based on the test results of the tested specimens, Nahvinia et al., 2021 [18] concluded that the $\theta_{80\%}$ could be increased by almost 40% by having tighter web horizontal ties.

Shear stud bolts (SSBs)

Shear stud bolts are generally used in SCB and SRCBs to ensure that the composite action between concrete and embedded steel section is mobilized. Through the conducted research programs, they have yet to particularly investigate the effect of shear stud bolts on the seismic behaviour of SCB and SRCBs. In terms of studying the minimum required shear strength at the beam-wall interface, Nahvinia et al., 2021 [18] developed a model including the effect of shear stud bolts. According to the test results, they concluded that the presence of shear stud bolts in the embedment region could increase the shear and flexural resistance of the embedded steel section at the beam-wall interface.

Web stiffeners inside/outside of the boundary region (WS)

Web stiffeners are used in coupling beams to prevent the web crippling of the steel section inside or outside the embedded region. Based on the expected failure modes, different specimens with different configurations of web stiffeners were tested throughout conducted research programs.

In 1993 and 1997, Harries et al. [8] tested four SCBs to assess the effect of web stiffeners and embedment length on the seismic performance of steel coupling beams. According to the Harries et al., 1993 [7] and 1997 [8] research program, specimen S1 had adequate embedment length and web stiffeners throughout the clear span. Compared to specimen S1, specimen S2 had the same embedment length but had web stiffeners inside and outside the embedded region. Test results show that specimen S1 experienced shear failure at the connection since no stiffeners were provided in the embedment region. Therefore, web crippling happened in that region. However, specimen S2 experienced shear failure through a clear span by experiencing web bucking of the steel section outside the embedment region. By comparing the recorded test results, it has been seen that specimen S1 tolerated higher $\theta_{80\%}$ compared to specimen S2, but the recorded overstrength for S2 was higher than S1. By comparing the test results of specimen S2 with specimen S3 which had the same features as S2 unless shorter embedment length, it can be concluded that specimen S2 had higher $\theta_{80\%}$ and overstrength. This conclusion means that although specimens S2 and S3 had inside and outside web stiffeners, the specimen with longer embedment length experienced higher $\theta_{80\%}$ and overstrength, by 7% and 5%, respectively.

Comparing the test results of Park et al., 2006 [17] and Park et al., 2005 [16], it can be concluded that for specimens without any concrete encasement, adding web stiffeners throughout the clear span can increase the $\theta_{80\%}$ by 38%.

In 2001, Shahrooz et al. [15] tested four specimens, three of which had concrete encasement. According to test results, it can be concluded that using web stiffeners in specimens with concrete encasement will not change the seismic behaviour of SRCBs since the encased concrete would provide some level of confinement for the web steel section forbidding the occurrence of web crippling.

Axial restraint (AR)

In 2001, Shahrooz et al. [14] tested three SRCBs, one of which had no face bearing plates, one had face bearing plates, and the third had face bearing plates and a floor slab acting as an axial restraint. According to the test results, the specimen with floor span had a lower $\theta_{80\%}$ compared to other specimens but significantly higher overstrength (increased by 30%).

Based on the research program by Motter et al., 2014 [11-12], one of the specimens out of four was axially restrained by post-tensioned (PT) rods. Other test variables for this specimen were also minimum embedment length and the least required transverse reinforcement ratio at the boundary region according to ACI 318-19 [9]. Since multiple variables were considered for this specimen (SRC4), it is difficult to identify the effect of the axial restraint on the overall seismic behaviour of this specimen, particularly overstrength and $\theta_{80\%}$.

Initial stiffness (Ki)

Most of the conducted research programs to study the seismic behaviour of SCB and SRCBs mainly focused on the strength aspects of the design procedure of these beams. However, some of these research studies investigated the initial stiffness of SCB and SRCBs.

Based on the test results of Shahrooz et al., 1993 [6], they suggested the effective fixity point at one-third of the embedment length ($L_e/3$) from the beam-wall interface for modelling purposes. This recommendation for steel coupling beams was changed to $L_e/4$ based on the test results of the Shahrooz et al., 2001(a) [14] research program. In 2001, Shahrooz et al. [14-15] also recommended considering the effective fixity point for SRCBs at one-third of the embedment length.

The effective fixity point recommendations of Shahrooz et al., 1993 [6], 2001 (a) [14] and (b) [15] are based on not considering the effect of the auxiliary transfer bars. To consider the effect of auxiliary bars on the effective fixity point in steel coupling beams, Park et al., 2005 [16] suggested that the effective fixity point can be considered at $L_e/5$ or $L_e/6$ for SCBs.

Motter et al., 2014 [11-12] suggested different models to calculate the effective moment of inertia for SRCBs. In this regard, they suggested that the EI_{eff} can be calculated by considering the aspect ratio of the concrete section (L/hc), and the flexural rigidity of the transformed moment of inertia (E_sI_{trans}), where I_{trans} is determined based on transforming concrete to steel based on the modular ratio and neglecting cracked concrete.

Eq. (12) shows the effective stiffness equation for DRCB according to LATBSDC 2020 [10]. As shown, the factor which reduces the gross flexural rigidity (E_cI_g) is the same for SRCB (i.e., Eq. (9)). As a result, comparing the transformed flexural rigidity (EI_{trans}) to the gross flexural rigidity (E_cI_g) gives insight to the relative stiffness of an SRCB to DRCB. Figure 7 shows the ratio of the flexural rigidity of an SRCB calculated per Eq. (11) over the flexural rigidity of a gross concrete section with the same geometry versus the ratio steel section area ($A_{s,steel}$) over the concrete gross area (A_g), using the gathered specimen geometries. The concrete modulus (E_c) used in calculating flexural rigidity (EI_{tr} and E_cI_g) was assumed as 30000Mpa for all specimens to facilitate a clear comparison.

$$EI_{eff} = 0.07\left(\frac{L}{h_c}\right)E_CI_g \leq 0.3E_cI_g \tag{12}$$

According to Figure 7, increasing the steel section area in the SRCB increases the flexural rigidity of these beams. However, even though most SRCBs had a relatively small steel section area compared to the concrete area (i.e., $A_{S,steel}/A_g<0.1$), the flexural rigidity is significantly small. The implication is that a composite beam has lower stiffness than a DRCB. This low stiffness in design is a severe limitation for SRCBs, as it reduces the stiffness of the coupled wall system, which results in high displacements. Therefore, one potential area of study in SRCBs can be investigating an improved equation for calculating the flexural rigidity.

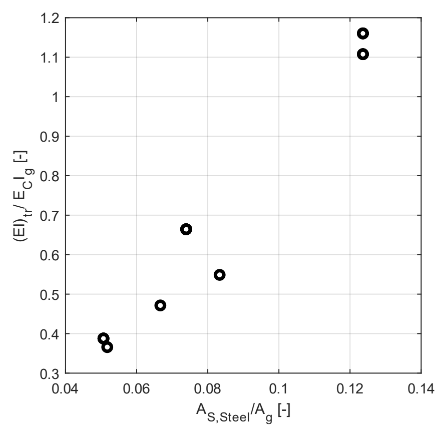


Figure 7. The ratio of the flexural rigidity of an SRCB calculated according to LATBSDC 2020 [10] over the flexural rigidity of the equivalent DRCB versus $A_{S,Steel}$ over A_g

THE RECOMMENDED DESIGN APPROACH FOR SC/SRC BEAMS ACCORDING TO CANADIAN DESIGN PHILOSOPHY

CSA A23.3-19 [4] could adopt several aspects currently used in US design, specifically by utilizing the equations for calculating the embedment length and the recommendations for determining the capacity of the beam. However, Canadian design differs from US design, particularly in the capacity design approach, where all wall panels must be designed for the overstrength of the coupling beams.

Based on Figure 4(a), an overstrength factor of 1.2 would be suitable for Canadian practice design. This value would be 1.25 for conventionally and diagonally reinforced coupling beams, according to CSA A23.3-19 [4]. Another way the Canadian code differs is in the rotational capacity check for ductility capacity. Currently, the diagonally reinforced coupling beam has an inelastic rotational capacity of 4%. However, based on the majority of experimental tests, especially well-detailed ones, the rotational capacity ranges from 5% to 13%.

According to Figure 4(b), proposing a 7% rotational capacity for the Canadian code seems reasonable, as this would better align with experimental findings while maintaining an appropriate safety margin. By incorporating these changes, CSA A23.3-19 [4] could better address the design and performance of coupled shear walls with SCB and SRCB, ultimately improving seismic performance in buildings.

FUTURE WORK

According to the studied previously conducted research programs, the following research gaps can be suggested:

- Proposing a new model for calculating the embedment length by considering auxiliary transfer bars, face bearing plates, shear stud bolts in the embedment region, wall vertical bars, full transverse reinforcement confinement per ACI 318-19 [9], and spalling of concrete cover at the beam-wall interface.
- A new model for calculating the initial stiffness for both SCB and SRCBs by considering the effect of flexural and shear deformations.
- Conducting experimental programs in which the effect of embedment length on the ductility of SCB and SRCBs would be investigated.
- An Improved model determining stiffness and composite action between the concrete and steel sections.
- Conduct a detailed experimental and numerical research program to test and model SCB and SRCB by considering Canadian design philosophy, such as considering inelastic rotation and overstrength.
- Investigate the seismic behaviour of SCB and SRCBs while the adjacent walls are loaded by cyclically overturning moments and various ratios of steel section area over the concrete gross area are being tested.
- Study feasible construction methods to implement anti-buckling ties in the boundary region, as creating holes in the steel section’s web is complicated.
- Study the effect of axial restrain (AR) on the seismic behaviour of SCB and SRCBs, particularly on $\theta_{80\%}$ and overstrength.
- Through the tested specimens, only two experienced flexural failure through the clear span. Further experimental programs can be performed to investigate this type of failure mode in SCB and SRCBs.

CONCLUSIONS

In this study, 33 previously tested SCB and SRCBs were investigated to provide guidance for Canadian design for steel and composite coupling beams placed in the coupled shear wall system. The US design procedure for these beams was investigated deeply. Since in Canadian design the coupled shear walls are required to be designed for the overstrength in the coupling beams, one seismic design parameter investigated in this study was overstrength. According to available experimental data, the overstrength value equal to 1.2 was suggested to be considered for designing the SCB and SRCBs based on Canadian design philosophy. In addition to overstrength, ductility is a critical design parameter in Canada, accounting for the capacity of the structural member to dissipate energy through inelastic deformations and rotations. In this regard, a suitable value of $\theta_{80\%}$ equal to 7% was proposed based on the investigation of the previously tested SCB and SRCBs. Furthermore, the possible fields of research work were recommended for future studies.

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