Use of Reinforced Concrete Shear Walls in Steel Framed Buildings

The cost of construction in developed countries is primarily a function of construction speed and schedule. This is especially true for speculative commercial and residential real estate, where complex financing and preconstruction sales can impose severe financial penalties for delays in building completion. For this reason, tall office and apartment buildings have traditionally favored steel structural systems over concrete for their quick erection and lack of time-dependent strength properties.

However, as construction technology and urban real estate markets create demand for taller structures, the design of lateral force resisting systems is having an increasing impact on the cost and schedule of building construction. Taller steel moment resisting and braced frames require complex detailing during the design phase, a greater amount of machine time during fabrication, and the assembly of complex bolted or welded connections in the field during construction. In addition to longer lead times, stiffer frames require a larger material premium over basic strength demands in order to meet code-prescribed drift and acceleration limits.

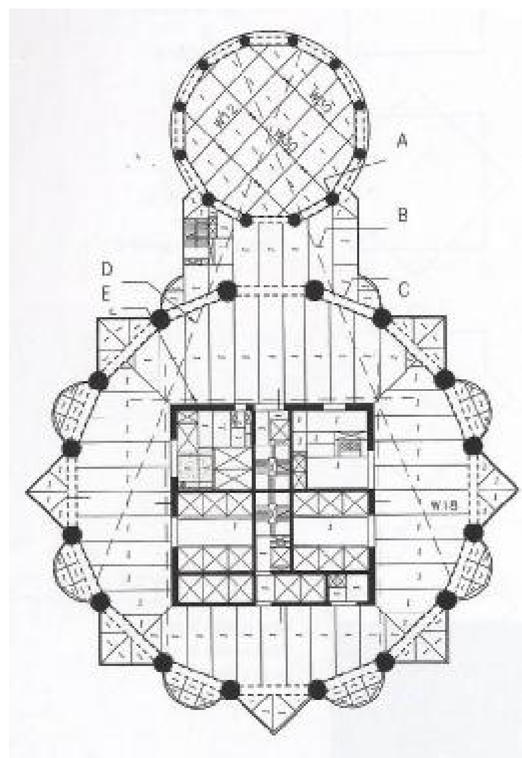

Fig. 1 – Cross section of one of the Petronas Towers with the concrete elements heavily shaded. Note the relatively light (W18) steel floor framing and efficient use of composite deck. [4]

The increase in material and labor costs associated with stiffer concrete walls are lower than those for steel frames. What's more, concrete lateral force resisting systems possess stiffness and damping characteristics that have made them popular in many of the world's tallest residential and commercial buildings. Many of these same buildings – including Taipei 101 in Taiwan, the Petronas Towers in Kuala Lampur (Fig. 1), and the Jin Mao Tower in Shanghai – have utilized composite steel and concrete floor systems to reduce the construction time associated with concrete floor forming and shoring [4, 6]. The use of these two systems in some of the most expensive and laterally demanding buildings in the world is testament to their economic and structurally efficiency.

This report serves as an overview of the construction issues arising from combining concrete walls with composite floor construction. In addition, the design of shear walls and composite floors are discussed both in general and in relation to the interaction of the two systems. Finally, a checklist of recommendations is presented for engineers involved in the design of steel framed buildings utilizing reinforced concrete shear walls.

1 – Construction Considerations

Fig. 2 – Example of a tower crane and jump forms used to construct a concrete shear wall in a steel framed building. Note that the wall is two stories ahead of the floor construction. (Photo courtesy of N. Sosin)

1.a – Formwork

As with all concrete construction, shear wall formwork costs can be reduced by standardizing wall dimensions and details. Maintaining consistent wall thickness, story heights, and the size and location of openings allows the reuse of forms and reduces the likelihood of dimensional error [7]. For walls in steel buildings, it is also recommended to repeat the size and location of embedded plates for the connection of steel elements.

Plates are typically nailed to the interior of forms prior to pouring; repetition in plate installation reduces the need for construction oversight and can help minimize costly errors in connection alignment (see the discussion on Tolerances below).

Table 1 – Characteristics of Wall Form Systems [7]

	Ganged	Jump	Slip	Self-Raising
Tolerance for Variation (Wall Size & Location)	High	Moderate	None	Low
Tolerance for Openings	Moderate	Moderate	Low	Moderate
Finish Quality	High	High	Low	High
Cycle Time (Days/Floor)	3-4	2-3	1	2-3
Stripping Cost	High	Low	None	Low
Requires Adjacent Floor	Yes	No	No	No
Crane Time per Floor	High	Low	None	None

Table 1 summarizes the characteristics of the four wall forming systems listed below:

- Ganged Forms – Basic tied and stiffened panel forms, fastened together in modular sets to simplify placement and stripping.
- Jump Forms – Similar to ganged forms, but built to enclose an entire wall pier. Jump forms also include work platforms and a support system that fastens to previously constructed portions of wall below; these can be used to place rebar and concrete without an adjacent floor structure (Fig. 2).
- Slip Forms – Utilize ultra-low slump concrete to continuously form walls as they ascend upward. Slip forms are self-raising and include a work platform.
- Self-Raising Forms – Similar to jump forms but incorporate a hydraulic lifting mechanism that raises the form without the use of a crane.

Fig. 3 – Shear connection with bolt holes enlarged to accommodate variation in wall dimension. The connection was designed for the moment resulting from the maximum possible reaction eccentricity. (Photo courtesy of N. Sosin)

1.b – Scheduling

The construction procedures for steel frames and concrete walls vary significantly in speed and dependency. With the exception of ganged forms, all systems in Table 1 permit the construction of walls entirely or several stories ahead of frame and floor construction. This can be used to avoid delays stemming from the interdependency of the two systems, such as beam-wall connections. In addition, structures with relatively simple foundation systems and long steel lead times can begin construction of walls and cores before any steel arrives on site. This, in turn, can hasten the scheduled placement of central stairwells, elevators, and some building utilities.

Table 2 – Dimensional Tolerances for Walls per ACI 117-90 [3]

Vertical Alignment	
Height < 100'	
Lines & Surfaces	1"
Exposed Edges, Joints & Grooves	1/2"
Height > 100'	
Lines & Surfaces	L/1000 < 6"
Exposed Edges, Joints & Grooves	L/2000 < 3"
Level Alignment	
Lintels, Sills, Exposed Lines	1/2"
Cross-Sectional Dimensions	
Wall Thickness	
Dim. < 12"	+3/8" -1/4"
12" < Dim. < 3'	+1/2" -3/8"
Dim. > 3'	+1" -3/4"
Openings	
Size	+1" -1/4"
Location C.L.	+1/2"

1.c – Tolerances

Table 2 [3] lists the relevant tolerances for concrete wall construction and Table 3 [2] lists steel beam and column tolerances. Note that the allowable variation in the length of steel members (±1/8" or less) is a third of the allowable variation in the plan dimension for walls (approximately ±3/8"). This implies that beams framing into shear walls should be shortened to accommodate any possible increase in wall size. Connections, too, need to be detailed for the maximum possible eccentricity occurring (theoretically) when a 1/8" short beam frames into a wall with 3/8" of additional thickness. The slop arising from the opposite case – a long beam and a thick wall – is 5/8", making the total variation in possible connection location equal one and one-eighths inch.

Table 3 – Dimensional Tolerances for Steel Beams per AISC 303-05 [2]

Beam Mill Length	
L < 30'	±1/16"
L > 30'	±1/8"
Column Mill Length	
All Columns	±1/32"

Keep in mind that these tolerances are merely specified values; many contractors may be used to more lenient dimensional restrictions for wall construction and may price their bids as such. The design of a bolted connection resulting from an assumed wall tolerance of ±2" can be seen in Fig. 3. To avoid designing embedded plates for the moment caused by this added eccentricity, it is important to clearly note the assumed tolerances on all relevant design drawings and to communicate these expectations to contractors or construction managers early in the design phase.

1.d – Union Interaction

In regions with strong labor unions, the construction of steel framed buildings with concrete shear walls may be hampered by regulations discouraging or preventing the interaction of steel and concrete tradesmen. A contractor or construction manager familiar with local union policies should be consulted before any substantial design is initiated.

2 – Steel Framing

2.a – Beam Design

The design of steel members has become increasingly automated. A number of popular software packages streamline the process, from input of building layout (typically taken from converted CAD drawings), load assignment (including choice of applicable building code or codes), strength and serviceability checks, and member optimization. These programs also handle the complete design of composite floor systems, either for full composite action or for a user-defined number of shear studs [8, 9]. Composite design is carried out in accordance with the AISC Specification for Structural Steel Buildings Section I. Note that proper identification of the span direction of the steel deck is important: Section I allows for concrete below the top of the deck in compression to contribute to the moment capacity if the deck flutes are running parallel to the beam.

2.b – Column Design

Columns for gravity loads are often selected by the same programs designing floor systems. Many programs, however, require the user selection of stability parameters such as K values and end conditions. In the presence of concrete shear walls and floor diaphragms that limit inter-story drift, AISC Specification Section C permits the use of a stability factor K = 1.0. Structural analysis can be used to justify a smaller value but "second-order effects, flexural, shear, and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses" must be considered in accordance with AISC Appendix 7 [10]. Such analyses are considered unconservative and are rarely performed.

2.c – Fabrication & Erection

The fabrication of steel structures is as optimized as their design. Computer output from CAD and structural models can be sent directly to fabricators and easily converted into shop drawings and CNC machine processes. Electronic communication regarding mill orders and steel deliveries is often electronically monitored by construction managers, who in turn coordinate the erection sequence and minimize downtime.

Structural steel mill orders, particularly of unusual or custom shapes, do require a certain amount of lead time. Automating the steel design process and limiting it to gravity systems can lead to quicker design finalization and subsequent placement of mill orders.

Finally, regarding the international movement toward "green" or environmentally responsible construction, newly milled structural steel boasts over 99% recycled content.

Fig. 4 – Underside of a composite steel gravity system framing into a concrete shear wall. Note the variation of steel elements – w-shapes, joists, deck, etc. (Photo courtesy of N. Sosin)

3 – Composite Floor Systems

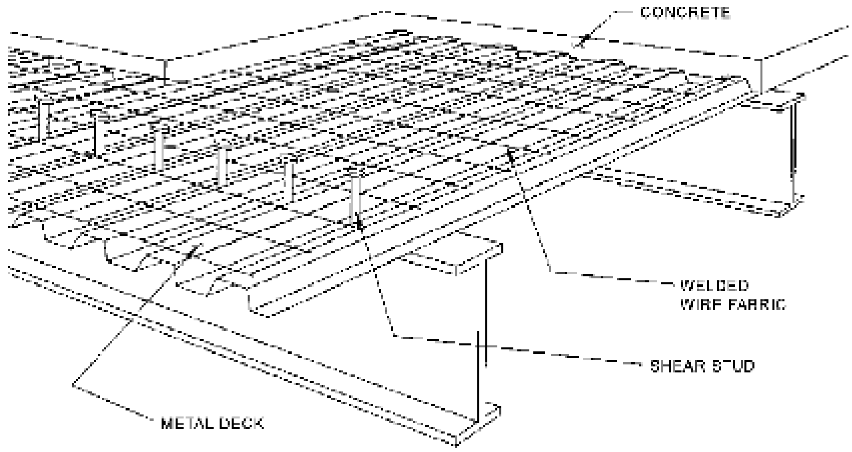

Fig. 5 - Typical Composite Floor System [5]

3.a – Description

Most modern steel framed buildings utilize concrete-filled steel deck, or composite floor systems (Fig. 5). These systems serve as permanent formwork for the placement of concrete floors and are specified to control deflections both during concrete placement and throughout their service life. Composite steel deck earns its name through deformations in the deck side walls that transfer shear between the deck and hardened concrete. This composite action reduces the need flexural reinforcement and helps increase the overall stiffness of the floor system. Like steel framing, steel floor deck is made from large percentages of recycled material.[11, 12]

3.b – Design

The flexural strength and stiffness of composite slab systems are typically specified in manufacturers' product literature in accordance with the Steel Deck Institute. The Steel Deck Institute publishes a number of deck design guides, including a manual for deck diaphragm design. However, composite floor calculations are often trivial and can typically be carried out under the guidance of ACI 318.

Composite floor systems can be very thin due to their inherent structural efficiency: 1½" deep deck typically produces a total slab depth of 3½", 2" deck produces a 4" slab, and

3" deck produces a 5" slab. Note that while properly specified composite deck will eliminate the need for flexural reinforcement in the concrete topping, the minimum slab reinforcement ratio of 0.18 percent still applies [13]. Thin decks are also very light, which can sometimes lead to vibration issues. AISC Design Guide 11, "Floor Vibrations due to Human Activity" provides guidelines for checking the perceived vibration of slab systems.

With as little as 2" of continuous concrete above the metal deck, the flexural and shear strengths of the floor as a diaphragm must be checked against predicted demand. Flexural strength will likely not be a concern, given the flexural depth of a flat slab and the presence of temperature and shrinkage steel. However, because composite slabs are typically cracked due to shrinkage and service loads, it is recommended that provisions of ACI 318 Chapter 21 be applied for all severe cyclic loading conditions (i.e. both wind and seismic loading for tall buildings). These provisions require flexural reinforcement in sections of slab where uncracked analysis reveals flexural stresses greater than $0.2f_c$; this requirement can be satisfied by temperature and shrinking reinforcement [14].

The shear strength of composite slab diaphragms neglects the shear resistance of the concrete (due to cracking) and is given as:

$$V_n = A_{cv} r_n f_y \tag{4.1}$$

where V_n is the nominal resistance of the slab, A_{cv} is the cross sectional area of slab resisting shear, ρ_n is the transverse reinforcement ratio of the slab (often $\rho_{min} = 0.18\%$), and f_y is the yield strength of the reinforcing steel [14]. Note that welded wire fabrics often used as minimum reinforcement have yield strengths fy ≥65 ksi [15].

4 – Concrete Walls & Wall Systems

4.a – Location

Architectural considerations typically locate shear wall systems toward the interior of buildings. Here they can be effectively used to house elevator shafts, stairwells, and mechanical and electrical risers without interrupting valuable window space. From a structural standpoint, relegating walls to the center of structures tends to limit their flexural depth and therefore their stiffness. Engineers have overcome this problem by coupling walls to steel fascia columns, other walls, or large concrete "super columns" (Fig. 6). Coupling of walls is discussed in Section 4.c.

Centrally locating walls has the hidden benefit of limiting the eccentricity between the building's center of mass and the center of resistance of the wall system. While the above architectural limitations generally minimize the torsional demand on shear walls, even a small eccentricity may contribute an additional shear force substantial to smaller walls. A guide from [16] showing the calculation of wall eccentricity, the resulting torsion, and the distribution of shear is given in Appendix A.

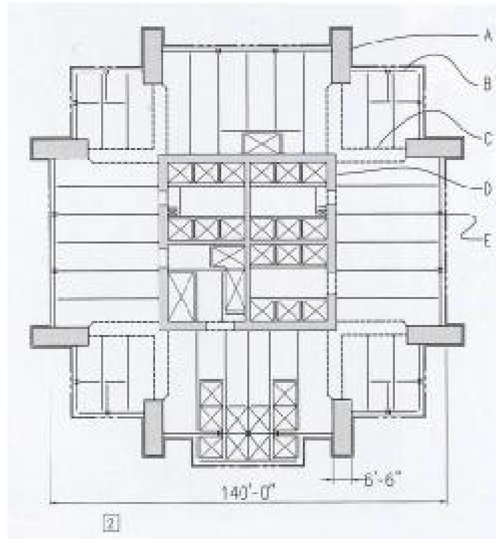

Fig. 6 – Proposed structural layout for a 2000 foot tower with concrete elements shaded in grey. Note the large exterior "super columns" coupled to the interior core wall [5].

4.b – Design Requirements

The strength design of conventional concrete shear walls is covered in ACI 318, Chapters 10, 11, and 14. Walls defined as "Special Reinforced Concrete Shear Walls" according to ASCE 07-05 must adhere to the strength and detailing provisions of ACI 318 Chapter 21 (in addition to ASCE 07, Section 14.2). ASCE 07 also places limits on the deformation of individual members.

Wind loading requirements are given in ASCE 07-05 Chapter 6. These are to be used when referenced by the local building code or International Building Code (IBC). Note that some regions, such as the City of Chicago, have building codes containing their own wind loading provisions. Limitations on total building and interstory drift, due to wind or seismic forces, are also given in the IBC and some local building codes.

4.c – Stiffness Parameters

The stiffness of individual wall piers is governed by two main parameters: the moment of inertia of the section and the modulus of elasticity of the material. Most stiffness analyses can begin by using the gross moment of inertia for wall piers; concrete coupling beams should be assumed cracked initially with an effective moment of inertia:

$$I_e = 0.35 I_g \qquad (4.1)\ [13]$$

where I_g is the gross moment of inertia of the section. An iterative analysis is recommended when tension in the extreme flexural fiber of any member exceeds the modulus of rupture of the concrete. Cracked wall piers should be analyzed with an effective moment of inertia given by Eq. 4.1.

Determination of the elastic modulus of concrete shear walls has a large impact on the overall stiffness of walls and wall systems. ACI 318 provides the following equations for concrete moduli:

$$E_c = 33 w_c^{1.5} \sqrt{f_c^{'}} \qquad (4.2)\ [13]$$

$$E_c = 57,000 \sqrt{f_c^{'}} \qquad (4.3)\ [13]$$

The Code allows the universal use of these expressions despite their having been derived for compressive strengths less than 6000 psi [1]. For high-strength concrete (greater than 6000 psi) often used in high-rise wall construction, the following equation has been developed:

$$E_c = \left[40,000 \sqrt{f_c^{'}} + 10^6 \right] \times \left(\frac{w_c}{145}\right)^{1.5} \qquad (4.4)\ [17]$$

Table 4a and Table 4b compare the moduli computed using Eqs. 4.2, 4.3, and 4.4 for normal and lightweight concrete (w_c = 145 pcf and 110 pcf, respectively) of varying strengths. Note that the differences can be in excess of ten percent.

Table 4a -Elastic Moduli for Normal Weight Concrete

f_c' (psi)	Eq. 4.2	Eq. 4.3		Eq. 4.4	
	E_{C1} (ksi)	E_{C2} (ksi)	$\%E_{C1}$	E_{C3} (ksi)	$\%E_{C1}$
4000	3644	3605	99%	3530	97%
5000	4074	4031	99%	3828	94%
6000	4463	4415	99%	4098	92%
7000	4821	4769	99%	4347	90%
8000	5154	5098	99%	4578	89%
9000	5466	5407	99%	4795	88%
10000	5762	5700	99%	5000	87%

Table 4b - Elastic Moduli for Light Weight Concrete

f_c' (psi)	Eq. 4.2	Eq. 4.3		Eq. 4.4	
	E_{C1} (ksi)	E_{C2} (ksi)	$\%E_{C1}$	E_{C3} (ksi)	$\%E_{C1}$
4000	2408	3605	150%	2332	97%
5000	2692	4031	150%	2530	94%
6000	2949	4415	150%	2708	92%
7000	3185	4769	150%	2872	90%
8000	3405	5098	150%	3025	89%
9000	3612	5407	150%	3168	88%
10000	3807	5700	150%	3304	87%

4.d – Coupled Walls

Coupled wall systems, including wall-to-column and wall-to-wall coupling, are powerful tools for limiting the drift of tall buildings. Coupling can be achieved through concrete beams, composite concrete and steel beams, or steel beams and trusses. There is a large body of research on different coupling systems and their associated stiffness and deformation capacity. Careful consideration should be given to the forces and drift demands placed on coupling beams for any given wall system; a three-dimensional model and dynamic analysis is recommend for systems relying heavily on coupling.

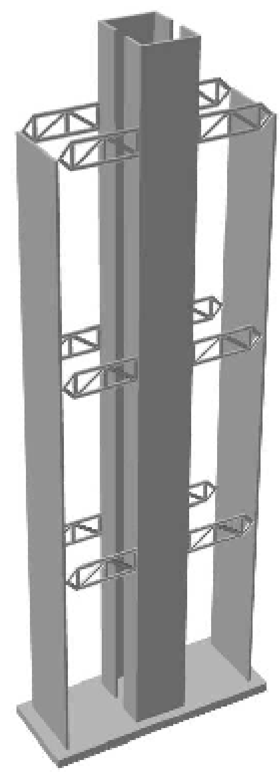

Fig. 7 – Schematic of a shear wall group coupled by steel trusses. Pin connections at the exterior walls prevent the introduction of moments about the weak axes of the walls. (Courtesy N. Sosin)

Before modeling begins, it may be useful to get a rough idea of how effective a coupled system is compared to the total stiffness of individual piers. A number of interesting models have been developed toward this end, two of which are presented in this paper.

Hoenderkamp [18] illustrated a method for determining the optimal story height for coupling core walls with fascia columns. This method applies to columns of both concrete and steel and is based on deformation compatibility. It presents an expression for horizontal deflection at the top of a single wall system consisting of a truss outrigger pinned to a fascia column on each side of the shear wall. This easy-to-use expression and its derivation are given in Appendix B.

Though complicated in form, the Hoenderkamp model is derived from simple assumptions and input readily available early in the design phase. What's more, the method is useful for coupling a wall to any axially stiff element, including walls that would otherwise be bending about their weak axis (Fig. 7). It should be noted that when coupling to elements slender with respect to the considered direction of flexure, it is appropriate to design the connection between the outrigger and axial element as pinned. This prevents the transfer of moment into slender axially loaded elements and subsequent reduction in their axial efficiency.

Harries, Moulton, and Clemson [19] adapted the Continuous Medium Method for their study of the deformation demand on coupling beams. They defined a wall system's Degree of Coupling as:

$$doc = \frac{NL_w}{\sum M_w + \sum NL_w} \qquad (4.5)\ [19]$$

where N is the axial load in the wall system due to coupling action, L_w is the distance between the centroids of the coupled walls, and M_w is the overturning moment in each pier. They also discuss the dimensionless parameter *kαH* given as:

$$k\alpha H = \sqrt{\left(1 + \frac{AI}{A_1 A_2 L_w^2}\right)\frac{12 I_c L_w^2}{L_b^3 h I} H^2} \qquad (4.6)\ [19]$$

with A_1 and A_2 as the area of the two wall piers and A as their sum; I as the sum of the moments of inertia of the two piers; h as the story height; H as the total height of the wall; and L_w as given above. I_c is the moment of inertia of the coupling beam accounting for shear deformations:

$$I_c = \frac{I_b}{1 + \left(\frac{24 I_b}{L_b^2}\right)} \qquad (4.7)\ [19]$$

with I_b as the flexural moment of inertia of the coupling beam; L_b as the beam clear span; and where one-half the concrete modulus E_c has been substituted for the concrete shear modulus G_c.

This parameter can be interpreted as a measure of the relative stiffness of the coupling beams and thereby the degree of coupling of the wall system. The authors then make the following useful observations:

1. Values of $k\alpha H < 1$ are associated with degrees of coupling less than 20%. In such cases the piers are more likely to behave as linked, not coupled.
2. Values of $k\alpha H > 8$ are associated with degrees of coupling greater than 75% for most practical wall systems.
3. Because of limitations on coupling beam deformation, large degrees of coupling typically indicate flexible wall piers. Code-prescribed drift limitations – as opposed to strength or deformation capacities – are likely to govern the design of flexible wall systems.

Observations 1 and 2 are useful for sizing coupling beams in wall systems with repeated openings, or for determining the feasibility of coupling across openings of a particular size. Graphs from the study (included in Appendix C) can be used to approximate the degree of coupling for values of $k\alpha H$ between 1 and 8.

Observation 3 is of particular interest for structures with steel gravity systems. It implies that large degrees of coupling either increase the likelihood of localized cracking at coupling beams or are the result of wall groups behaving more like frames. Both scenarios can raise the second-order effects on slender steel columns to unacceptable levels. It is therefore important to understand that $k\alpha H$ is only a measure of a wall group's relative efficiency and does not preclude a more thorough – possibly non-linear – analysis of building stiffness.

This observation should not be a deterrent to using the Continuous Medium Method to determine the degree of coupling between any two wall piers. It and the Hoenderkamp model are both easy to script or program in Excel; require simple, readily available input that can be quickly modified for parametric study; and are based on assumptions broad enough to be verified by more thorough modeling later on in the design phase.

4.e – Differential Shortening

Steel columns and concrete walls behave differently under axial loads over time. To prevent uneven floors, especially in tall buildings, the effects of differential shortening should be investigated. These effects include but are not limited to elastic shortening of both walls and columns and creep and shrinkage of the concrete walls.

Wall elements are typically designed for ultimate axial loads between five and ten percent of their nominal capacity. Therefore they undergo very little elastic shortening under gravity service loads. Steel columns, on the other hand, are typically assumed to take no moment and are designed for service stresses very close to their axial limit. A comparison is given as follows: a typical critical compressive stress of F_{cr} = 40 ksi in a steel column equates to an axial strain of 0.0014, or approximately one inch in 60 feet. An axial stress equal to 8% of *nominal* (not ultimate) in a 6 ksi normal weight concrete wall with $\rho_{l,min}$ = 0.0012 and E_c from Eq. 4.4 yields an axial strain approximately 20 times smaller (one inch in 1200 feet).

The time dependent effects of shrinkage and creep have a larger impact on the axial deformation of tall concrete walls. Recommended values for the ultimate shrinkage strain, assumed to occur at the service life of the structure, are approximately $\varepsilon_{su} \approx 6\times10^{-4}$ or one inch in 140 feet [1]. Of this strain, roughly half will occur within a month of wall construction and nearly ninety percent will occur after a year [1].

Table 5 – Approximate Ultimate Creep Coefficients based on Concrete Strength [1]

f_c' (psi)	C_{cu}
3000	3.1
4000	2.9
5000	2.7
6000	2.4
7000	2.2
8000	2.0
9000	1.6
10000	1.4

Strains from creep are harder to approximate as they are affected by the initial concrete strain, the age of the material at time of loading, the volume-to-surface area ratio of the member (typically quite small for walls), and the concrete strength, among other factors.

Table 5 [1] gives estimates of ultimate creep coefficients C_{cu} – the ratio of ultimate creep strain occurring at a structure's service life to the initial axial strain – as a function of concrete strength. A more detailed discussion of creep is given in reference [1].

5 – Detailing

5.a – Embedded Plates

Steel plates embedded in concrete elements are used to connect steel deck, floor beams, and steel outriggers or coupling beams (Fig. 8). The design of these connections is governed by several different codes. The stress distribution along the height of the plate is a function of the plate thickness, as given in the AISC Connections Design Manual, Volume 3. For plates located far from any boundary of the concrete element, simplified design of embedded stud groups can be carried out according to stud manufacturer recommendations or the PCA Design Handbook Section 6.5, both of which are typically conservative. For critical connections, those near wall edges, or those in thin wall elements, the provisions of ACI 318-05 Appendix D should be used.

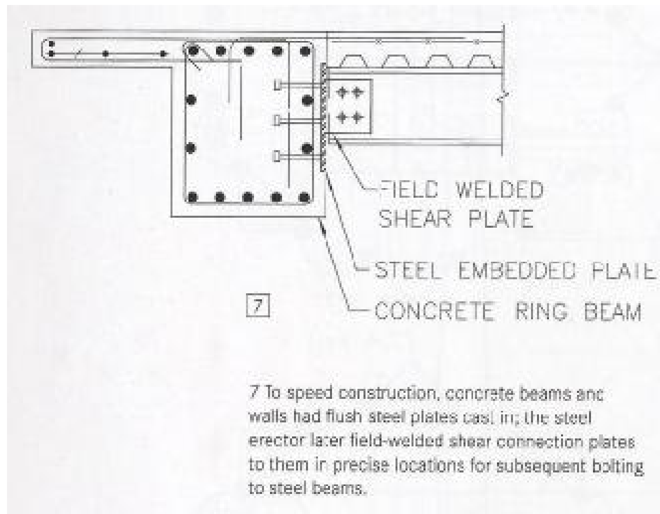

Fig. 8 – Section of plate embedded into a concrete beam for the attachment of a steel floor beam [4].

Fig. 9 – Diagonal reinforcement without confinement in a coupling beam. This beam was designed in a region with low seismic and moderate wind demand (Photo courtesy N. Sosin)

5.b – Coupling Beams

As previously mentioned, the design of concrete coupling beams is a matter of strength and deformation capacity, both of which have a large impact on the lateral performance of a structure. These demands are exaggerated in the presence of load reversals such as wind oscillation or seismic events. Therefore it is recommended that the provisions of ACI 318 Section 21.7.7 be considered for cast-in-place coupling beams in tall buildings with degrees of coupling in excess of fifty percent. For wind-controlled design, confinement of a coupling beam's diagonal bars may be omitted if deformations are low enough to prevent concrete spalling (see Fig. 9).

6 – Pre-Design Recommendations

Reinforced concrete shear walls and wall groups are an effective means of resisting lateral forces in tall structures. With the addition of steel and composite gravity systems, they are highly competitive with all-steel structures in terms of overall construction economy. The following is a list of recommendations for parties considering the design or construction of steel framed buildings with reinforced concrete shear walls:

- Enlist the expertise of contractors or construction managers early on.
  - Explore different formwork options for their impact on project cost, construction schedule, and shear wall layout.
  - Ensure expectations for dimensional tolerances are well-understood.
  - Manage design schedules and construction phases to get work started early, finished on time, and prepared for the next phase.
  - Be mindful of disruptive union interaction, site congestion, and delays caused by the interaction of the steel and concrete systems.
- Keep the steel design as simple as possible.
  - Utilize automation in the design, analysis, fabrication, and erection or steel structures wherever possible.
  - Simplify floor design and construction with composite metal deck.
  - Beware issues arising from vibration and diaphragm strength and stiffness.
- Utilize concrete shear wall systems to resist lateral forces and displacements.
  - Be sure to effectively identify member stiffness parameters such as moment of inertia and elastic modulus.
  - Use programmable parametric models to estimate the effects of wall coupling efficiency on building stiffness.
  - For walls or "super columns" with large axial loads, verify that the effects of differential shortening are within acceptable limits.
- Be sure to detail critical structural elements to ensure predictable performance.
  - Clearly communicate critical elements items to contractors or construction managers as soon as possible.

Appendix A – Torsion and Shear Distribution for an Eccentric Wall Group [16]

The following has been taken from the coursepack for CEE 515 – Advanced Design of Reinforced Concrete taught at the University of Michigan by Prof. James K. Wight.

Appendix B – Estimate in Drift Reduction due to Coupling of Fascia Columns [18]

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Appendix C – Graphs Correlating Degree of Coupling to $k\alpha H$ [19]

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