NON-RECTANGULAR REINFORCED CONCRETE SHEAR WALLS: DESIGN ISSUES AND PERFORMANCE

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ABSTRACT

Non-rectangular reinforced concrete shear walls are often used in building systems as a means of resisting lateral forces. Much of what is assumed about the behavior of these walls has been extrapolated from tests of rectangular walls or tests of non-rectangular walls subjected to unidirectional loading. A collaborative research effort is underway incorporating the National Science Foundation George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Multi-Axial Subassemblage Testing (MAST) System at the University of Minnesota to investigate the behavior of non-rectangular wall systems subjected to multi-directional loading. This paper discusses issues encountered in the design of a prototype T-shaped shear wall from a six-story office building assigned to Seismic Design Category D. Issues include investigation of the critical biaxial loading combination and direction, distribution of forces among individual walls in a floor level, and detailing of the wall to comply with the intent of the ACI 318-02 Building Code Requirements for Structural Concrete.

Introduction

Engineers often use structural walls in buildings to serve as the primary lateral load resisting elements because of their large in-plane stiffness and strength which enables them to carry large lateral loads due to wind and earthquakes while also minimizing drifts. Architects often prefer the walls to be concentrated around hallways or elevator cores to minimize their impact on the floor plan and to maximize available window space. As a consequence, linear rectangular walls are often combined to form I-, C-, T- and L-shapes. These configurations lead to complicated interactions between the wall segments; assuming that the linear segments of the walls resist lateral loads in each direction independently is no longer reasonable. Because the walls often become non-symmetric (e.g., C-, T- and L-shaped configurations), the uniaxial behavior of the wall is also affected.

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Previous laboratory testing of T-shaped walls loaded uniaxially along the direction of the stem of the T has indicated that these walls can be detailed using methods similar to those used for rectangular walls. However, multidirectional loading of these walls has not been carried out at large scales. This more complicated but more realistic loading scheme will provide additional understanding of the behavior of these members under real-world loading histories. The Multi-Axial Subassemblage Testing (MAST) System at the University of Minnesota, part of the National Science Foundation (NSF) George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES), is the first facility of its kind capable of applying full 6-DOF mixed-mode loading to large-scale specimens. One-half and three-quarter-scale T-shaped wall specimens are to be tested to failure. Loads will be applied in orthogonal and skew directions to facilitate comparison to previous tests on T-shaped walls subjected to unidirectional loading. To maximize the applicability of this research to current practice, the specimens are being designed according to the appropriate design specifications (ACI 318-02 and IBC 2003) and accepted best practices.

Design Philosophy

The use of a displacement-based design approach to guide detailing of reinforced concrete shear walls was introduced into the model building codes with the 1994 edition of the UBC and was adopted into ACI 318 in 1999 (Thomsen and Wallace, 2004). Field experience (Wallace, 1996) and testing (Thomsen and Wallace, 2004) have shown that while these provisions may relax the strength-based requirements that they have replaced, they provide a more rational design basis that leads to walls with good seismic behavior. Because previous tests addressed only unidirectional loading, some questions remain regarding the effectiveness of the detailing requirements of non-rectangular shear walls and the effects of skew loading on them.

Prototype Building

The prototype building used to determine the design forces on the T-shaped walls considered in this study was a six-story office building assigned to Seismic Design Category D. The building had a 22,500 square foot floor plan with story heights of 12 feet for all levels. The floor system consisted of a 7 inch thick cast-in-place concrete slab spanning between precast or cast-in-place concrete beams. The beams were supported by columns and, in some locations, the web tip of the T-wall, on a 20 foot x 45 foot grid. The total seismic weight of the building was 180 psf, or 4,050 kips per floor.

A combination of four pairs of T-shaped walls and two rectangular cast-in-place concrete walls was used to resist all of the lateral loads. The rectangular walls were added to increase the lateral force capacity of the building in the E-W direction in order to improve the efficiency of the building design, as discussed below. The number and gross dimensions of the T-shaped walls was determined by the building core elevator and mechanical layout. Each T-shaped wall had a 15 foot long web and 12 foot wide flange. The thickness of each wall was 12 inches. The T-shaped walls were assumed to resist all of the lateral forces in the N-S direction of the building. The flanges of the T-shaped walls and the two additional rectangular walls were assumed to resist all of the lateral forces in the E-W direction of the building. Fig. 1 shows the building framing plan with the T-shaped and rectangular shear walls highlighted.

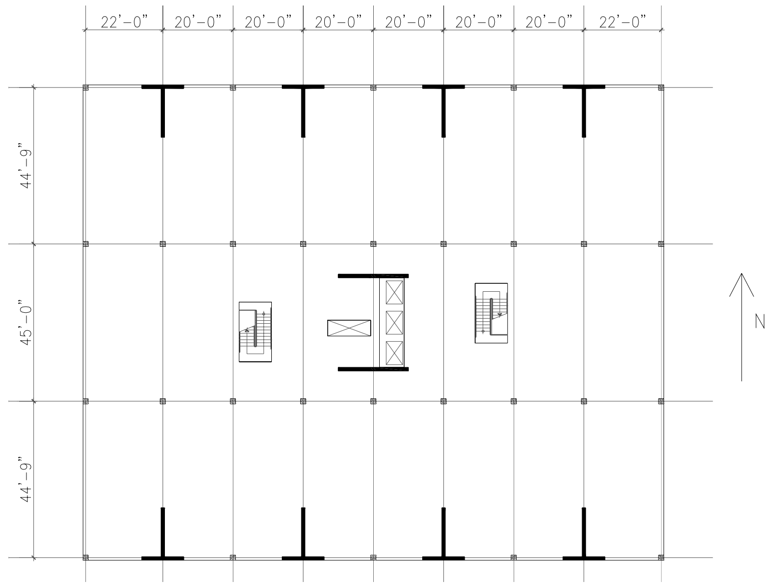

Figure 1. Typical floor level framing plan.

Design lateral loads for the building were determined using IBC 2003. The seismic coefficients for this building were Site Class: C S_S: 1.50 g S_1: 0.60 g I_E: 1.0 F_a: 1.0 F_v: 1.3. The lateral system was considered to be a Building Frame System with Special Reinforced Concrete Shear Walls. This system has an R value equal to 6.0, Ω_0 equal to 2, and C_d equal to 5. The resulting design forces were a base shear of 3510 kips and a base overturning moment of 183,887 kip-feet in each orthogonal direction.

Distribution of Forces among Walls

The first step in the design of the walls was to distribute the lateral forces among the walls based on relative stiffness. When loading along the direction of the flange, i.e. in the E-W direction, although the nonrectangular walls were not individually symmetric about the direction of loading, pairs of walls maintained symmetry assuming that the floor diaphragm was rigid and prevented twisting of the individual walls. Consequently, the lateral forces assigned to the T-walls in the E-W direction were assumed to be divided equally among the walls. For loading along the axis of the web of the T-shaped wall, i.e. the N-S direction, the direction of loading had a significant influence on the stiffness of the wall. For the flange in tension case, the large amount of reinforcement within the tension flange caused a large depth of the web to be in compression for equilibrium. However, in the flange in compression case, the large width of the flange in compression and smaller amount of reinforcement in tension resulted in a very shallow neutral axis depth.

Proportioning load between pairs of walls loaded in the web direction requires assumed relative wall stiffnesses. Section 10.11.1 of ACI 318-02 recommends assuming an effective cracked flexural stiffness of $0.35EI_g$ for all for wall sections. Because the total story shear is defined in IBC 2003 independently of member stiffness, the relative stiffnesses of the wall segments resisting the load are important rather than the actual stiffnesses. Assuming constant wall stiffness does not account for the asymmetry of a T-shaped wall loaded in the web direction.

A parametric study was conducted comparing the secant wall stiffness at the yielding moment in each direction for varying amounts of steel. The overall wall dimensions were held constant through this study. It was found that when the area of steel in each of the three boundary elements was increased from 6.24 in^2 to 18.72 in^2, the ratio of secant stiffness of flange-in-tension to flange-in-compression increased from 0.87 to 1.52. When the section is lightly reinforced, the wall is stiffer when the flange is in compression; when the section is more heavily reinforced, the wall is significantly stiffer when the flange is in tension than when the flange is in compression. This experience indicates that proportioning of web direction loads between T-shaped walls may require an iterative approach.

For the final T-shaped wall section considered in this study, the effective stiffness was $0.27EI_g$ for the flange in compression case and $0.42EI_g$ for the flange in tension case, or a stiffness ratio of 1.56. Wallace (2002) reported cracked stiffness values of $0.27EI_g$ and $0.30EI_g$ for a particular T-shaped wall geometry loaded with the flange in compression and flange in tension, respectively, or a ratio of 1.11. Assuming that a T-shaped wall has the same cracked stiffness when the flange is in tension and when the flange is in compression may lead to an inefficient design or premature degradation. It may overestimate the load carried by a wall in one direction, resulting in an increase in the amount of steel provided, and underestimate the load carried by a wall in the other direction, resulting in inadequate capacity.

A second design challenge associated with the distribution of loads between T-shaped walls is creating a section with appropriate strength in both orthogonal directions. For the prototype building, designing the wall flanges to resist 100% of the lateral load in the E-W direction would have required a longer wall flange length or substantially more flange reinforcement. A larger flange length was considered not architecturally acceptable, and initial analysis indicated that the chosen wall geometry was adequate for resisting the lateral load in the N-S direction. It was decided that the most effective design option was to add two rectangular walls to the core of the building in the E-W direction. The two additional walls were designed to resist 45% of the load in the E-W direction with the remaining demand on the wall flanges being 55% of the total load, or 1930 kips base shear. The reinforcement chosen for the flanges of the T-wall for the E-W loading case resulted in a 125% flexural overstrength for the flange-in-tension case for loading in the N-S direction. Avoiding the use of the rectangular walls would have led to a significantly further increased moment capacity in the flange in tension direction (N-S loading) and consequently increased the shear demand associated with designing a flexurally-controlled wall. The total required length of shear walls is substantially larger in the E-W direction than in the N-S direction. This is due to the increased strength (and thus efficiency) provided by a T-shaped wall in web direction loading.

Critical Biaxial Bending Combinations

When rectangular walls are used in structural systems, there is not a need to investigate critical biaxial bending combinations for each wall. The walls are assumed to <u>only</u> carry in-plane loads, and each wall must be able to carry 100% of the load that is assigned to it according to its stiffness. When non-rectangular walls are used, however, biaxial bending effects must be considered. The choice of the appropriate loading direction and combination can be interpreted in different ways in many documents (i.e. IBC 2003, FEMA 450, ASCE 7). These documents state that for all buildings with T-shaped walls in seismic category C (IBC 2003) or D (FEMA 450) and higher, the design orthogonal loading combination is equal to "100% of the forces for one direction plus 30% of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used," (IBC 2003 1620.3.2; FEMA 450 4.4.2.2). Several practicing engineers were polled regarding their interpretation of this requirement, and it was found that it is generally interpreted in two very different ways. The first interpretation is that members should be designed for uniaxial bending moments of $M_x^L + 0.3M_x^T$ and $M_y^L + 0.3M_y^T$ independently, where L represents longitudinal direction loading, T represents transverse direction loading, and x and y coincide with the L and T directions, respectively. With this interpretation, biaxial loading effects need only be considered for asymmetrical buildings in which loading in the direction transverse to an element causes resistance in the element in the orthogonal direction. For symmetrical buildings, this interpretation is identical to the uniaxial bending requirement, and it does not lead to a required increase in capacity. The second interpretation is that members should be designed for the simultaneous occurrence of biaxial bending in the combinations of $M_x^L + 0.3M_y^T$ and $M_y^L + 0.3M_x^T$. For symmetrical buildings, this interpretation represents a significantly larger demand on the members than the first, and it affects the design of symmetrical and asymmetrical members alike. The design points for both of these interpretations are superimposed onto the biaxial bending moment diagram from the T-shaped walls in the prototype building in Fig. 2. For the prototype building considered in this study, the four pairs of T-shaped walls and two rectangular walls would not be adequate if the second interpretation were followed. In this case, designing for biaxial bending significantly amplifies the demands on the wall, and it leads to a requirement for an "overdesigned" wall relative to the uniaxial bending requirements. Design of asymmetrical buildings requires more careful handling of biaxial bending requirements, as well as torsion on the building.

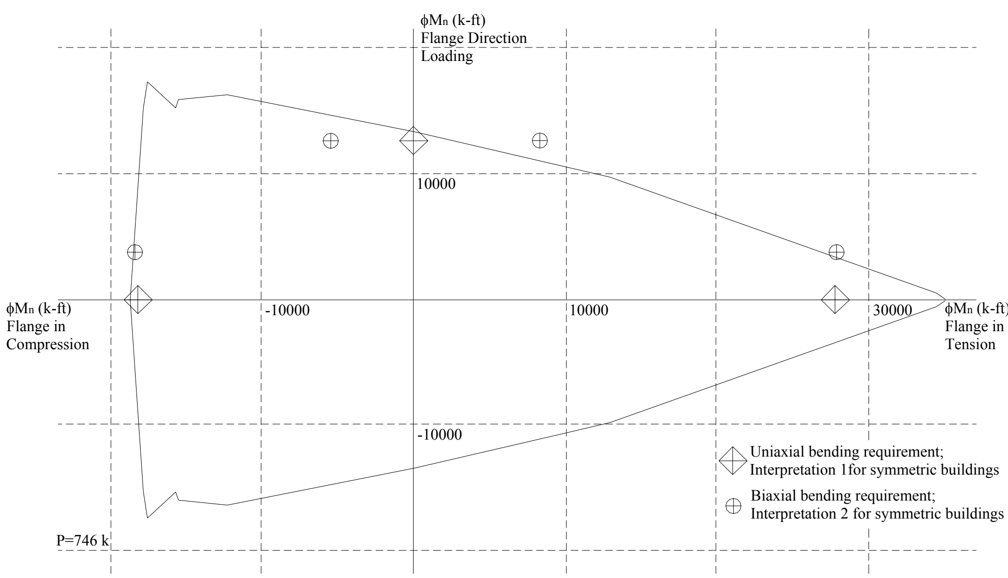

Figure 2. Interaction diagram for walls showing design moments for two interpretations of requirement.

Detailing Issues

Some issues regarding detailing of flexural walls are vague or not explicitly covered in ACI 318-02. In these cases, engineers should ensure that their design process meets the intent of the code and use their own judgment to produce a wall that performs well in a seismic event.

Confined Regions

The ACI 318-02 requirements for confined boundary elements in special reinforced concrete walls are taken directly from the requirements for confinement of columns. Special confinement is required when the extreme compression fiber of the wall exceeds a critical strain when the wall undergoes a design-level displacement. This critical strain level corresponds to spalling of the cover concrete and strength degradation of an unconfined section. The required horizontal and vertical dimensions of the confined boundary element are calculated so that the confinement will extend over all portions of the wall where spalling of cover concrete is expected. The horizontal dimension of the boundary element is based on the neutral axis depth and horizontal wall dimension, and the vertical or longitudinal dimension is based on a conservative estimate of the plastic hinge length. In the T-shaped walls large strains are expected in the web tip, as a result of loading with the flange in tension, and in the flange tips, as a result of flange direction loading. Confinement is not required for loading with the flange in compression, because the compression block is extremely shallow, resulting in very small compression strains.

In general, boundary elements of reinforced concrete walls behave similarly to columns, so modeling a boundary element as a column is a reasonable approach for estimating the special confinement requirements. The primary differences between boundary elements in walls and columns are aspect ratio and minimum dimension. The boundary elements in the prototype walls have aspect ratios of approximately 5. According to section 1908.1.1 of IBC 2003, the maximum aspect ratio of a column section is 2.5; an aspect ratio of 5 can never arise in column confinement design. Additionally, concrete walls are generally much narrower than columns. Section 21.4.4.2 a) of ACI318-02 requires that the minimum transverse reinforcement spacing in a confined element be less than one-quarter of the minimum member dimension. Because walls tend to be narrower than columns, this provision leads to very tight spacing of hoops and a high level of congestion. Part c) of this same requirement states a minimum spacing of s_x, which need not be less than 4 inches. For this study, it was decided to investigate the behavior of the walls ignoring the requirement of part a) because it was felt that it led to too tight of spacing for the transverse reinforcement due to the small thickness of the wall.

Shear Transfer and Shear Lag

No specific guidance is given in ACI 318-02 regarding designing for sufficient vertical shear transfer capacity across the joint between the web and flange. Previous research (Thomsen and Wallace, 2004) has included a small confined region at the joint between the web and flange of a T-shaped wall; this confined region is not required by ACI 318-02, and it is not included in typical practice. It may, however, have facilitated the vertical shear transfer across the interface. In the current study, force transfer through the joint was ensured by hooking the web transverse reinforcement bars into the flange and anchoring the corners of these hooks around vertical bars in the flange.

Section 21.7.5.1 of ACI 318-02 defines the maximum effective flange width of a non-rectangular wall as the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height. In the prototype wall, as with most multi-story walls, the actual flange width was considered effective in the N-S loading direction. This indicates that all of the longitudinal reinforcement in the flanges is effective for this loading direction. However, because large amounts of reinforcement are often concentrated in the flange tips of T-shaped walls, there may be an increased possibility of shear lag problems which lead to a lower bending capacity than expected when the flange is in tension. The vertical reinforcement in the wall flange was instrumented to investigate potential shear lag in the T-wall under investigation.

Shear Reinforcement

There are a number of reasons in T-wall design, including two described above (i.e., additional reinforcement in flange to carry loading in E-W direction and design for biaxial loading effects) that result in increased flexural capacity in the web direction. As a consequence, the amount of shear reinforcement required to carry the design shear force may be insufficient to ensure a flexural failure prior to shear failure in the wall. Section 9.3.4(a) of ACI 318-02 states that a shear-controlled failure is allowed in special reinforced concrete walls as long as a reduced ϕ of 0.6 is used. The commentary to this section states that this provision is included for "members such as low-rise walls, portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength about the nominal flexural strength for the pertinent loading conditions." The reduced ϕ is used to help ensure that the wall will remain in the elastic range, with minimal damage, during a design-level seismic event. In the case of a special reinforced concrete structural wall strengthened flexurally as described above, use of this more severe ϕ factor of 0.6 may lead to lower shear reinforcement demands than those required to ensure a flexural failure.

To ensure ductile behavior and good energy dissipating characteristics of flexurally-controlled special reinforced concrete structural walls, the shear reinforcement should be designed to achieve the maximum probable flexural strength M_{pr} of the wall before the nominal shear capacity V_n is exceeded. ACI 318-02 does not specifically require the designer to ensure development of the flexural capacity of the walls; it only requires consideration of the design for the factored shear forces. It is noted in the commentary to ACI 318-02 Section 21.7.3 “However, the designer should consider the possibility of yielding in components of such structures, as in the portion of a wall between two window openings, in which case the actual shear may be in excess of the shear indicated by lateral load analysis based on factored design forces.”

If the designer chooses to promote flexural failure of the wall, ACI 318-02 does not provide guidance. As an example, there is no information about calculating M_{pr} for special reinforced concrete walls. More extensive guidance is provided for special moment frames, and this design philosophy was adopted for the design of the wall specimen in this study. Section 21.5.4 states that the design shear force V_e should be used in the design of special moment frames. V_e is the shear force required to generate the maximum probable moment strength M_{pr}

of the member. M_{pr} is defined in 21.0 as the "probable flexural strength of members… determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor ϕ of 1.0." In this calculation, f_y is the specified minimum yield strength of the steel. Additionally, Section 9.3.4(c) allows an increased ϕ of 0.85 to be used for shear in joints and diagonally-reinforced coupling beams. There is no guidance provided in ACI 318-02 regarding the appropriate choice of ϕ factors to use for determination of the shear reinforcement in walls to promote a flexural failure.

In this study, M_{pr} of the wall in each direction was calculated using this code-based approach and a more refined analysis. This additional analysis was conducted using BIAX (Wallace 1992). BIAX was used for a sectional analysis with a strain compatibility approach. Table 1 summarizes the material properties and assumptions used in the determination of M_{pr} with each of these methods, as well as with the code-specified calculations for M_{pr} using $1.25f_y$ and M_n using $1.0f_y$. Table 2 presents the ultimate moment and associated base shear calculated for the prototype specimen using each of these methods. The centroid of the applied loads on the specimen was taken as 52 feet above the foundation from the initial lateral load analysis, leading to the calculation of V_e from M_{pr}. The design shear, design moment, and nominal moment (V_u, M_u, and M_n) are also included in this table for reference and comparison.

Table 1. Assumptions used in analysis methods.

Model	BIAX	Whitney Stress Block	
Moment capacity calculated	M_{pr}	M_{pr}	M_n
f_y (ksi)	68	75	60
f_y source	coupon tests	1.25*specified f_y	specified f_y
Strain hardening included	yes	no	
f'_c (psi)	5000	5000	5000
f'_c source	specified minimum		
Concrete model	Modified Kent-Park	ε_c=0.003	
Confinement included	yes	no	

Because T-shaped walls often must be significantly overdesigned for bending about one of the two orthogonal axes, the difference between the factored shear load V_u and the shear demand to reach flexural capacity V_e may be very large. For the six-story walls considered in this study, the required reinforcement for bending about the axis of the web controlled the amount of reinforcement in each tip of the flange. When loading in the direction of the flange, the moment capacity ϕM_n was only 2% greater than the factored moment M_u for the flange-in-compression case, but it was 25% greater for the flange-in-tension case. Using the biaxial bending combinations would have led to even larger values.

There is no industry-wide consensus on whether a smaller area of ductile, flexural walls is preferable over a larger area of shear-controlled walls. IBC 2003 does not distinguish between these two systems, so both use the same R value in determining the design loads. As a result, shear-controlled walls will generally require less reinforcement in these situations and are thus less expensive and potentially less congested than flexurally-controlled walls. Because flexural walls are capable of dissipating much more energy than shear-controlled walls, allowing the use of a larger R value may be warranted. This change would provide an economic benefit to the use of flexural walls.

Table 2. Shear requirements for prototype walls calculated with various analysis methods.

Loading Direction: Flange in Tension					
	BIAX (M_{pr})	Code M_{pr}	M_n	M_u	V_u
M (k-ft)	48582	45192	38430	27769	---
M/M_u	1.76	1.64	1.39	---	---
V_e (k)=M/52 ft.	934	869	739	534	530
ϕ	0.85	0.85	0.85	0.75	0.6
$V_{n\text{-}req}$ (k)=V_e/ϕ	1099	1022	869	712	883
Flange in Compression					
	BIAX (M_{pr})	Code M_{pr}	M_n	M_u	V_u
M (k-ft)	34617	24438	20419	18203	---
M/M_u	1.92	1.35	1.13	---	---
V_e (k)=M/52 ft.	666	470	393	350	347
ϕ	0.85	0.85	0.85	0.75	0.6
$V_{n\text{-}req}$ (k)=V_e/ϕ	783	553	462	467	578
Flange Direction					
	BIAX (M_{pr})	Code M_{pr}	M_n	M_u	V_u
M (k-ft)	20607	17225	14476	12642	---
M/M_u	1.64	1.37	1.16	---	---
V_e (k)=M/52 ft.	396	331	278	243	241
ϕ	0.85	0.85	0.85	0.75	0.6
$V_{n\text{-}req}$ (k)=V_e/ϕ	466	390	328	324	402

Conclusions

An engineer must take particular care when designing non-rectangular shear walls. There are several design issues that are not handled in detail by the governing code documents, IBC 2003 and ACI 318-02. Also, the body of research for non-rectangular shear walls is much smaller than that available for rectangular walls particularly under the effects of multi-directional loading, so guidance from additional sources is limited. The engineer must have a good understanding of the behavior of non-rectangular shear walls and their performance during a seismic event.

Issues of particular concern identified in the design of non-rectangular reinforced concrete shear walls include

- Estimating relative stiffnesses of T-shaped walls for appropriate distribution of design loads among walls
- Determining correct critical loading combination (i.e. whether or not to consider biaxial loading effects)
- Appropriate detailing of wall boundary elements without undue congestion

- Appropriate calculation of M_{pr} and ϕ for shear associated with development of M_{pr}
- Distinguishing between design philosophies of shear- and flexural-controlled walls, including revisiting the determination of appropriate values of R for each case
- Detailing of joint between web and flange to address potential vertical shear transfer problems associated with engaging reinforcement in flange tips.

The test specimens to be investigated as part of this study are intended to address the issues identified above to facilitate the development of new code provisions and design guidelines that will promote the design of economical walls with good seismic performance capabilities. While the prototype building for this study was symmetric, the understanding gained from the T-wall testing will also facilitate the understanding of buildings with non-rectangular walls arranged asymmetrically. In this situation, torsion must be considered in addition to uniaxial and biaxial bending requirements.

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Notation

ϕ = strength reduction factor
Ω_0 = system overstrength factor
C_d = deflection amplification factor
F_a = site coefficient
F_v = site coefficient
f'_c = specified minimum concrete strength
f_y = specified minimum steel yield strength
I_E = occupancy importance factor
M_n = nominal moment strength at section
M_{pr} = probable flexural strength of member
M_u = factored moment at section
R = response modification coefficient
S_1 = spectral accelerations for a 1 second period
S_S = spectral accelerations for short periods
V_e = design shear force to reach moment capacity
V_n = nominal shear strength at section
V_u = factored shear force at section

References

ACI Committee 318, 2002. *Building Code Requirements for Structural Concrete (ACI 318-02)*, American Concrete Institute, Farmington Hills, MI,

Federal Emergency Management Agency, 2003. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450)*, Federal Emergency Management Agency, Washington, DC.

International Code Council, 2002. *2003 International Building Code*, Country Club Hills, IL.

Thomsen IV, J.H., and J.W. Wallace, 2004. Displacement-based design of slender reinforced concrete structural walls – experimental verification, *Journal of Structural Engineering* 130 (4), 618-630.

Wallace, J.W., 1992. BIAX: Revision 1 – Computer program for the analysis of reinforced concrete and reinforced masonry sections, *Report CU/CEE-92/4*, Structural Engineering, Mechanics, and Materials, Clarkson University, Potsdam, NY.

Wallace, J.W., 1996. Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls, *Earthquake Spectra* 12 (2), 327-348.