Title no. 106-S43

Shear Strength of Lightly Reinforced Wall Piers and Spandrels

by Kutay Orakcal, Leonardo M. Massone, and John W. Wallace

Between the 1950s and 1970s, a significant number of buildings were constructed using lightly reinforced perimeter walls with openings. Evaluation and rehabilitation of such buildings requires accurate assessment of the expected shear strength, stiffness, and ductility of the wall segments (wall piers and spandrels) that comprise the primary lateral load-resisting elements. Assessing wall shear strength is complicated by factors such as use of a single curtain of distributed reinforcement, lack of hooks, and use of weakened plane joints, which are all common in older construction. To address these issues, a database of existing test results was assembled and reviewed; and tests were conducted on lightly reinforced wall piers and spandrels to address significant gaps in the available test data. Observations indicate that the amount of boundary reinforcement provided, presence of axial load, and the location of a weakened plane joint on the wall are the most important factors in the assessment of nominal shear strength.

Keywords: friction; pier; reinforced concrete; shear; spandrel; strength; wall.

INTRODUCTION

Between the 1950s and 1970s, the use of lightly reinforced perimeter walls with openings was fairly common. For example, according to the California Office of Statewide Health Planning and Development, 1012 of the 2585 California hospitals (39%) are rated as SPC-1, that is, they pose a significant risk of collapse.[1] Of the 821 SPC-1 buildings that were classified by building type, 271 (33%) are reinforced concrete wall buildings, which account for 39% of the total square footage for SPC-1 buildings. A majority of these 271 buildings were constructed between 1950 and 1970 and include perimeter walls with lightly reinforced wall piers (vertical wall segments between window openings) and wall spandrels (horizontal wall segments between window openings) (Fig. 1). In contrast, concrete moment frames make up only 2% of the inventory (by number or square footage). Therefore, accurate assessment of the as-built strength, stiffness, and deformation characteristics of lightly reinforced wall piers and spandrels could have a substantial impact on the evaluation and rehabilitation process, as well as the cost associated with rehabilitation.

A representative, lightly reinforced wall segment is typically 125 to 250 mm (5 to 10 in.) thick, with a single curtain of distributed reinforcement in two directions that often is close to the ACI 318-05[2] minimum reinforcement ratio of 0.25% in each direction. Additional boundary, or jam, bars are common at the wall edges. A weakened plane joint, where part of the longitudinal web reinforcement is discontinued and the wall thickness is reduced, may exist on some wall segments, typically spandrels, to initiate and control cracking. In some cases, no hooks are provided on the transverse (web) reinforcement.

The guidelines for structural walls in the FEMA 356[3] report on seismic evaluation and rehabilitation of existing buildings are outlined in Section 6.8.2 and, in general, focus more on applications for walls controlled by flexure (slender walls) versus shear-controlled cases (squat walls, for example, wall piers, and spandrels). Shear strength provisions of FEMA 356[3] generally follow ACI 318-05[2] requirements, which were developed for new buildings. Therefore, the impacts on the shear strength calculation of using one curtain (versus two) of distributed web reinforcement, discontinuity of reinforcement at the weakened plane joint, and the lack of hooks on transverse reinforcement are not explicitly considered for evaluation purposes. Furthermore, limited information is provided in FEMA 356[3] to derive load versus deformation backbone relationships of shear-controlled wall segments to be used in the seismic evaluation (for example, pushover analysis) of existing buildings. For example, in Table 6-19,[3] only one row of information is provided for shear-controlled segments; otherwise, wall segments must be treated as force-controlled components.

Based on the preceding discussion, an experimental program was conducted on selected lightly reinforced wall pier and spandrel configurations to investigate various response attributes including shear strength, stiffness, and deformation capacity, as well as the effect of outdated construction practices on the shear strength and lateral load behavior of wall segments in existing buildings. Test results were compared with the shear strength equations defined in ACI 318-05[2] and FEMA 356[3] and the lateral load versus deformation backbone relationships described in FEMA 356[3] to assess the reliability of these documents or the conservatism embedded therein, pertaining to seismic evaluation and

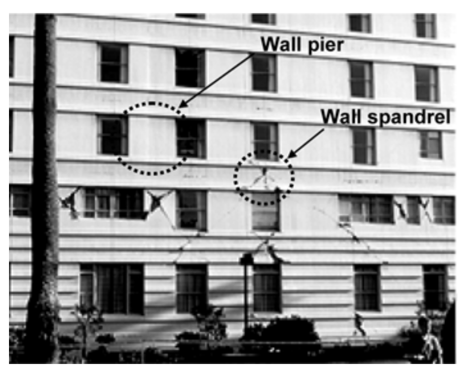


Fig. 1—Wall piers and spandrels in a perimeter-wall building (Santa Monica, CA, 1994).

ACI Structural Journal, V. 106, No. 4, July-August 2009.
MS No. S-2007-382.R1 received December 12, 2007, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published in the May-June 2010 *ACI Structural Journal* if the discussion is received by January 1, 2010.

Kutay Orakcal *is an Assistant Professor at the Department of Civil Engineering at Boğaziçi University, Istanbul, Turkey. He received his BS from Middle East Technical University, Ankara, Turkey, in 1998, and his MS and PhD from the University of California Los Angeles (UCLA), Los Angeles, CA, in 2001 and 2004, respectively. His research interests include structural and earthquake engineering, with an emphasis on response assessment for structural elements and systems subjected to earthquake actions through laboratory testing and multi-scale analytical modeling.*

Leonardo M. Massone *is an Assistant Professor in the Department of Civil Engineering at the University of Chile, Santiago, Chile. He received his BS from the University of Chile in 1999 and his MS and PhD from UCLA in 2003 and 2006, respectively. His research interests include analytical and experimental studies of reinforced concrete systems, with an emphasis on seismic response.*

John W. Wallace, *FACI, is a Professor of civil engineering at UCLA. He is a member of ACI Committees 318-H, Seismic Provisions, and 374, Performance-Based Seismic Design of Concrete Buildings, and Joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures. His research interests include seismic design of buildings and bridges, laboratory and field testing of structural components and systems, and seismic monitoring using sensor networks.*

rehabilitation of existing buildings. Also, a database of relevant test results available in the literature[4-6] was assembled and studied to assess shear strength requirements for lightly reinforced wall segments with both single and double curtains of distributed reinforcement.

This paper investigates only the lateral load capacity (ultimate shear strength) attributes of lightly reinforced wall piers and spandrels that fail in shear or sliding shear. Lateral load versus deformation response characteristics (for example, stiffness, deformation capacity, strength degradation, and axial load collapse) of shear-controlled wall piers and spandrels will be presented in a follow-up paper.

RESEARCH SIGNIFICANCE

Seismic evaluation and rehabilitation of existing buildings requires an assessment of the lateral load behavior (shear strength, stiffness, and ductility) of structural elements be characterized. The FEMA 356[3] guidelines for evaluation of buildings with wall piers and spandrels are, for the most part, based on the ACI 318-05[2] provisions, which were developed for new buildings. For existing buildings that feature outdated construction details, uncertainties arise in the application of FEMA 356[3] guidelines and ACI 318-05[2] code provisions, which may lead to excessive conservatism and thus costly and intrusive rehabilitation or inadequate safety.

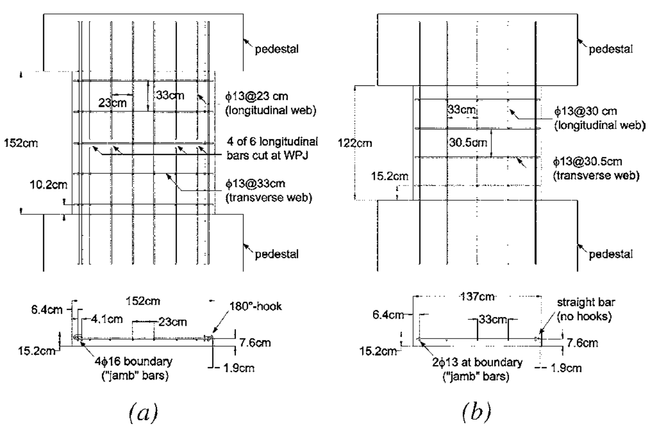


Fig. 2—Sample wall specimen geometry and reinforcement: (a) Type 1 wall spandrel; and (b) Type 5 wall pier.

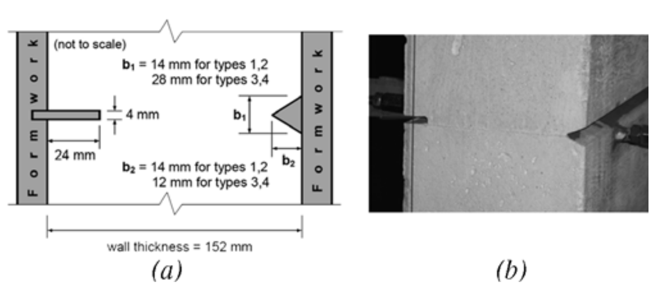


Fig. 3—WPJ on wall spandrels: (a) WPJ detail; and (b) finished WPJ on Type 1 specimen.

EXPERIMENTAL PROGRAM

The experimental program conducted at the University of California Los Angeles (UCLA) Structural/Earthquake Engineering Research Laboratory (SEERL) involved testing of six wall pier (WP) and eight wall spandrel (WS) specimens, with dimensions, reinforcement configuration, and material properties based on as-built conditions for two hospital buildings constructed in California[7,8] in the early 1960s using perimeter walls for lateral load resistance. The specimens were 3/4-scale and comprised specific construction features commonly used in construction at that time, including use of a single curtain of distributed reinforcement, lack of hooks on transverse (web) reinforcing bars, and existence of weakened plane joints (where the concrete cross-section is reduced and part of the longitudinal reinforcement is discontinued to initiate and control cracking). A detailed description of the complete experimental program can be found elsewhere.[7-9]

Specimen description

All wall specimens were tested in an upright position; therefore, spandrel specimens were rotated 90 degrees from the actual orientation in a building. The vertical direction for all specimens represents the longitudinal direction of an actual wall segment. The spandrel specimens were 152 cm (60 in.) tall, 152 cm (60 in.) long, and 15 cm (6 in.) thick, with aspect ratios (h/l) of 1.0, whereas the piers were 122 cm (48 in.) tall, 137 cm (54 in.) wide, and 15 cm (6 in.) thick, with h/l of approximately 0.9.

Figure 2 shows representative wall geometry and reinforcement layout for a spandrel and a pier specimen. Four different types (in terms of geometry and reinforcement) of WS specimens were tested, with two identical specimens of each type. Specimen Types 1 and 2 were differentiated primarily by the amount of the longitudinal reinforcement provided at wall boundaries to increase the flexural capacity (jamb bars), whereas for specimen Types 3 and 4, 180-degree hooks were not provided on the transverse reinforcement and a lower longitudinal web reinforcement ratio was used. In addition, the location of the weakened plane joint (WPJ) for specimen Types 3 and 4 was varied. The WPJs were created by attaching prefabricated wood strips on the interior surface of the formwork over the full width (transverse direction) of the spandrels (Fig. 3), as well as cutting (during construction) a portion of the longitudinal web bars at the location of the WPJ (Fig. 2(a)). For testing, because the spandrels were rotated by 90 degrees, the WPJ was located at wall midheight for specimen Types 1, 2, and 3 (Fig. 2(a)), whereas it was located at a distance of 25 mm (1 in.) from the bottom wall-pedestal interface for Type 4 specimens. Longitudinal reinforcing bars provided at wall boundaries (jamb bars) were continuous over the height of the specimens (not cut at the WPJ), and they were not confined by boundary transverse reinforcement.

All six of the WP specimens were identical in geometry and reinforcement detail (Type 5). Two of the pier specimens were subjected to zero axial load during testing, whereas each two of the remaining four were tested under axial load levels of 5 and 10% of their axial load capacity (5%$A_g f_c'$, 10%$A_g f_c'$). There was no WPJ on the pier specimens; both the longitudinal web bars and the boundary bars were continuous over the height of the specimen. No hooks were provided on the transverse reinforcement (Fig. 2(b)).

The distributed reinforcing steel ratios of the specimens in longitudinal and transverse directions (ρ_l and ρ_t), the quantity of jamb bars provided at wall boundaries and the corresponding boundary reinforcement ratio ρ_b, the number of longitudinal web bars cut at the WPJ, the anchorage conditions on the transverse reinforcing bars (presence of 180-degree hooks), and the axial load levels applied on the specimens during the tests are presented in Table 1. The reinforcing steel ratios in the table were calculated based on the total area of boundary or distributed web reinforcement per the total tributary area of concrete (boundary or web) over which the reinforcement is located. The tributary area of the boundary zones was calculated by multiplying the wall thickness with twice the in-plane concrete cover (to the center of jammed bars), as specified in ACI 318-05[2] Fig. R21.7.6.5. Tributary web area was calculated by subtracting the tributary areas of the boundary zones from the entire cross-sectional area of the wall.

Materials

Uniaxial tension tests on coupon samples of the Grade 40 (f_y = 276 MPa [40 ksi]) reinforcing bars used in the construction of specimen Types 1, 2, and 5 revealed yield strengths f_y of 424 and 448 MPa (61.5 and 65 ksi) the ϕ13 and ϕ16 mm (No. 4 and No. 5) bars, respectively. Tests on a different type of Grade 40 reinforcing bar (supplied by a different manufacturer) revealed a yield strength of 352 MPa (51 ksi) for the ϕ13 mm (No. 4) bars used in the construction of specimen Types 3 and 4. The compressive strength of concrete at the day of testing of the wall specimens was obtained from uniaxial compressive tests on standard 15 x 30 cm (6 x 12 in.) concrete cylinder samples, which gave compressive strengths f_c' varying from 25.5 to 43.7 MPa (3.70 to 6.34 ksi), as listed in Table 1.

Test setup and control

The specimens were tested in an upright position, as shown in Fig. 4. Relatively low shear span-to-depth ratios (corresponding to one-half of the aspect ratio for each specimen) were achieved during testing of the specimens via fixing the base of the walls, restraining rotations at the top of the walls, and applying the lateral load at specimen midheight. This produced a linear bending moment distribution with moments equal in magnitude and opposite in direction applied at the top and bottom of the walls, representing the boundary conditions of an actual wall segment in a building. The test specimens incorporated reinforced concrete pedestal blocks, which were secured to an L-shaped steel loading frame at the top and the strong floor at the bottom with high-strength post-tensioning anchor bars. An out-of-plane support frame was provided to prevent twisting of the specimens under lateral loading. Two vertical actuators connected to the L-shaped loading frame were used to maintain

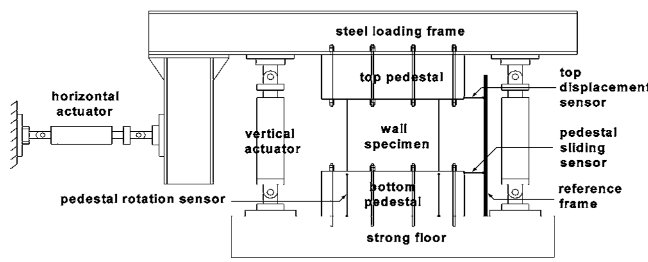


Fig. 4—Test setup.

Table 1—Properties of wall spandrel and wall pier specimens

Specimen								Transverse web reinforcement			Longitudinal web reinforcement			Boundary reinforcement			Axial load
ID No.	Type	Test no.	t_w, cm	l_w, cm	h_w, cm	$M/(Vl_w)$*	f_c', MPa	Reinforcing bar†	ρ_t, %	Hooks	Reinforcing bar†	ρ_t, %	Cut bars	Reinforcing bar†	A_{sb}, cm^2	ρ_b, %	$N/A_g f_c'$, %
WS-T1-S1	1	Test 1	15.2	152	152	0.50	25.5	ϕ13 at 33 cm	0.278	Yes	ϕ13 at 23 cm	0.428	4 of 6‡	4-ϕ16	8	3.12	0
WS-T1-S2		Test 4	15.2	152	152	0.50	43.7	ϕ13 at 33 cm	0.278	Yes	ϕ13 at 23 cm	0.428	4 of 6‡	4-ϕ16	8	3.12	0
WS-T2-S1	2	Test 2	15.2	152	152	0.50	31.4	ϕ13 at 33 cm	0.278	Yes	ϕ13 at 23 cm	0.400	4 of 6‡	1-ϕ13 + 1-ϕ16	3.29	1.70	0
WS-T2-S2		Test 3	15.2	152	152	0.50	31.0	ϕ13 at 33 cm	0.278	Yes	ϕ13 at 23 cm	0.400	4 of 6‡	1-ϕ13 + 1-ϕ16	3.29	1.70	0
WS-T3-S1	3	Test 11	15.2	152	152	0.50	31.7	ϕ13 at 28 cm	0.278	No	ϕ13 at 28 cm	0.256	2 of 4‡	2-ϕ13	2.58	1.33	0
WS-T3-S2		Test 14	15.2	152	152	0.50	33.6	ϕ13 at 28 cm	0.278	No	ϕ13 at 28 cm	0.256	2 of 4‡	2-ϕ13	2.58	1.33	0
WS-T4-S1	4	Test 12	15.2	152	152	0.50	31.9	ϕ13 at 28 cm	0.278	No	ϕ13 at 33 cm	0.256	2 of 4§	2-ϕ13	2.58	1.33	0
WS-T4-S2		Test 13	15.2	152	152	0.44	33.0	ϕ13 at 28 cm	0.278	No	ϕ13 at 28 cm	0.256	2 of 4§	2-ϕ13	2.58	1.33	0
WP-T5-N0-S1	5	Test 9	15.2	137	122	0.44	29.9	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	0
WP-T5-N0-S2		Test 10	15.2	137	122	0.44	31.0	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	0
WP-T5-N5-S1		Test 7	15.2	137	122	0.44	31.9	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	5
WP-T5-N5-S2		Test 8	15.2	137	122	0.44	32.0	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	5
WP-T5-N10-S1		Test 5	15.2	137	122	0.44	28.3	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	10
WP-T5-N10-S2		Test 6	15.2	137	122	0.44	31.4	ϕ13 at 30.5 cm	0.278	No	ϕ13 at 33 cm	0.227	—	2-ϕ13	2.58	1.33	10

*Shear span-to-depth ratio.
†ϕ13 (13 mm diameter) = U.S. No. 4; ϕ16 (16 mm diameter) = U.S. No. 5.
‡Weakened plane joint at wall midheight.
§Weakened plane joint at wall-pedestal interface.
Note: 1 cm = 0.394 in.; 1 cm^2 = 0.155 in.^{2}; and 1 MPa = 0.145 ksi.

constant (or zero) axial load and zero rotation at the top of the wall specimens during the entire test period. Reversed cyclic lateral loads were applied at the midheight level of the specimens, through a horizontal actuator connected to the loading frame (Fig. 4).

The specimens were initially subjected to three complete (push and pull) load-controlled reversed lateral loading cycles (for measuring wall response at low drift and damage levels), followed by three complete drift-controlled reversed lateral loading cycles to drift levels of 0.2, 0.3, 0.4, 0.6, 0.8, 1.2, 1.6, 2.0, and 2.4%. For some specimen Types 3 and 4 (piers and spandrels), only two cycles were applied for drift levels larger than 0.6%. For spandrel specimen Type 4, additional drift-controlled loading cycles to drift levels of 3.2, 3.6, and 4.8% were applied.

Instrumentation and data acquisition

An extensive set of instrumentation was provided during the test program for measuring loads, displacements, average deformations, and strains at various locations on the wall specimens. DC-type linear variable displacement transducers (LVDTs) (DC-excited) were mounted horizontally between each specimen and an external reference frame to measure lateral displacements. The effects of possible pedestal movement (sliding and uplift) were excluded from wall lateral displacement measurements (Fig. 4). Additional LVDTs were mounted diagonally (in an "X" configuration), vertically, and horizontally and at specified locations on the specimens to measure the magnitude and distribution of shear deformations, flexural deformations, sliding shear deformations, and average normal strains in longitudinal and transverse directions. The instrumentation used during the test program is described in detail by Massone[9] and Wallace et al.[7,8] Details on instrumentation and assessment of flexural, shear, and shear sliding deformations will also be presented in a follow-up paper on lateral load-deformation response and ductility attributes of wall piers and spandrels.

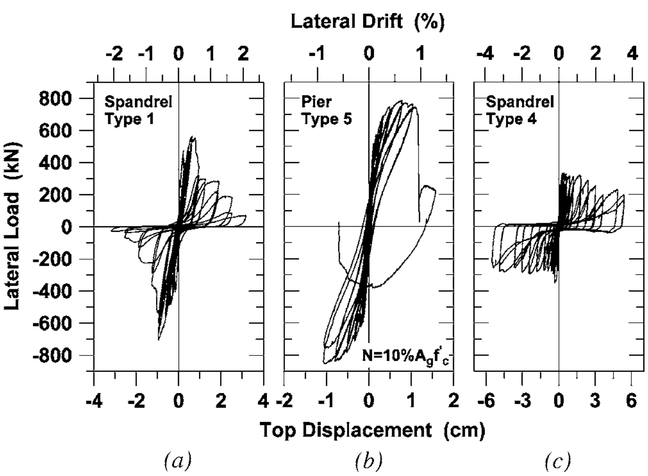


Fig. 5—Representative lateral load-displacement responses measured for wall specimens: (a) Type 1 spandrel (Test 1); (b) Type 5 pier (Test 6); and (c) Type 4 spandrel (Test 13).

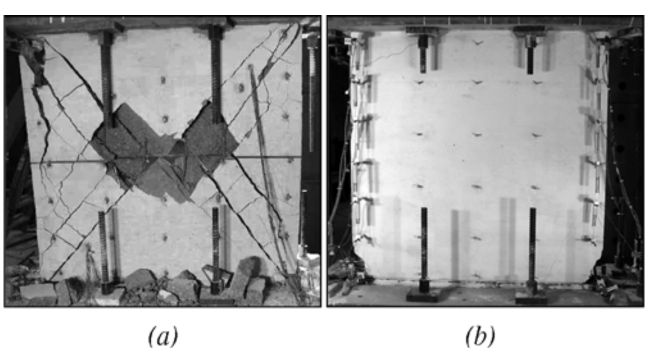


Fig. 6—Typical failure mode of wall specimens: (a) Type 1 wall spandrel at 2.0% lateral drift (typical for Types 1, 2, 3, and 5); and (b) Type 4 wall spandrel at 3.0% lateral drift.

Observed failure modes

Figure 5 presents typical lateral load versus top displacement responses measured for selected wall specimen types. Measurements from local instrumentation revealed that the lateral displacement of spandrel Types 1, 2, and 3, as well as Type 5 piers, was governed by shear deformations associated with diagonal cracking, followed by widening of and sliding along the diagonal cracks. For these specimen types, the contribution of flexural deformations (measured using multiple vertical LVDTs attached along the height of the specimens) and sliding along the WPJ (measured using LVDTs placed diagonally across the WPJ of the spandrels) were found to have minor influence on the overall wall displacement history, particularly in the nonlinear response range. For all of these specimens, lateral load failure (degradation of lateral load capacity) was associated with crushing of concrete close to the center of the wall (where the constraining effect of the top and bottom pedestals are minimized), followed by spalling of diamond-shaped wedges of concrete (Fig. 6(a)) on both sides.

The lateral load behavior and failure mode of Type 4 spandrel specimens (where the WPJ was located at the wall-pedestal interface) was unique. The lateral stiffness of these specimens was reduced significantly when a large visible crack formed (at 0.2% drift) across the entire length of the WPJ at the bottom wall-pedestal interface (Fig. 6(b)). Applying larger drift levels resulted in sliding along the WPJ, with no other form of significant damage observed at any other location on the wall. Measurements from local instrumentation (LVDTs for measuring flexural, shear, and sliding deformations) confirmed that the lateral displacement of these specimens was governed by sliding deformation along the WPJ.

ASSESSMENT OF WALL SHEAR STRENGTH

Code provisions

ACI 318 nominal shear strength—Except for minor changes in format, the ACI 318 equation for wall nominal shear strength has not changed since it was introduced in the ACI 318-83 as Eq. (A7). In ACI 318-05,[2] the corresponding equation (Eq. (21-7)) is in the form of

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y) \tag{1}$$

where the coefficient α_c is 3.0 for height-to-length ratios $(h_w/l_w) \leq 1.5$, is 2.0 for $h_w/l_w \geq 2.0$, and varies linearly between 3.0 and 2.0 for h_w/l_w between 1.5 and 2.0. In this equation, A_{cv} is the cross-sectional web area of a wall, ρ_t is transverse reinforcement ratio, f_y is the yield strength of transverse reinforcement, and f'_c is the compressive strength of concrete. The variation of α_c for h_w/l_w between 1.5 and 2.0 accounts for the observed increase contribution of concrete in low h/l walls. The nominal shear strength for wall piers and spandrels cannot be taken larger than $0.83A_{cw}\sqrt{f_c'}$ (MPa) $= 10A_{cw}\sqrt{f_c'}$ (psi), where A_{cw} is the cross-sectional area of the wall. The longitudinal reinforcement ratio ρ_l should not be less than transverse reinforcement ratio ρ_t for walls where the ratio of $h_w/l_w \leq 2.0$. A minimum reinforcement ratio of 0.0025 (in both transverse and longitudinal directions) is required if the shear force Vu exceeds $0.083A_{cv}\sqrt{f_c'}$ (MPa) $= A_{cv}\sqrt{f_c'}$ (psi), and it is stated that the reinforcement spacing in each direction should not exceed 46 cm (18 in.).

FEMA 356 provisions—FEMA 356[3] requirements for reinforced concrete wall construction, located in Section 6.8, are summarized in the following sentences. Section 6.8.1.1 states: "Walls with horizontal and vertical reinforcement ratios less than 0.0025, but with reinforcement spacing less than 46 cm (18 in.), shall be permitted where the shear force demand does not exceed the reduced nominal shear strength of the wall in accordance with 6.8.2.3." Section 6.8.2.3 covers requirements for strength, which are based on the provisions of ACI 318-05[2]; however, because shear strength is often reached prior to flexural yielding, the provisions for shear strength have the greatest impact on evaluation and rehabilitation process. Specific requirements for shear strength state that the ACI 318-05[2] equations can be used to assess wall nominal shear strength if the transverse reinforcement ratio (ρ_n in FEMA 356,[3] replacing ρ_t in ACI 318-05[2]) falls between 0.0025 and 0.0015; however, if ρ_n is less than 0.0015, the contribution of reinforcement to wall shear strength should be held constant at the value obtained for ρ_n of 0.0015. These modifications to the ACI provisions are based on the work of Wood,[10] who found that wall shear strength was relatively insensitive to changes in ρ_n, particularly for low ratios of ρ_n.

ACI 318 requirements—Specific requirements of the ACI 318-05[2] building code also impact the evaluation process. For example, ACI 318-05[2] states: "At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $(1/6)A_{cv}\sqrt{f_c'}$ (MPa) $= 2A_{cv}\sqrt{f_c'}$ (psi)," where A_{cv} is the gross area of the wall section and f_c' is the compressive strength of concrete. If strictly adhered to, this section implies that the wall shear strength cannot be taken greater the concrete shear strength for wall segments with a single curtain of reinforcement. The intent of this provision appears to be to ensure that shear reinforcement is distributed across the shear plane; however, when applied to existing construction, it has the unintended impact of limiting the wall nominal shear capacity by neglecting the contribution of reinforcement to shear strength.

ACI 318-05[2] also requires: "Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane." In some wall segments in existing buildings, similar to the wall spandrel specimens tested as part of this experimental program, WPJs are provided; and technically, reinforcement provided for shear strength is not continuous, that is, reinforcement is cut at the WPJ to allow for crack initiation and control. Requirements for spacing (that is, a maximum spacing of 46 cm [18 in.]) at WPJs may also be called into question, given that some of the distributed reinforcement is cut. Furthermore, the presence of a WPJ may induce a crack, which, in a wall with a relatively low amount of longitudinal reinforcement across the shear plane, can widen enough to promote sliding between the crack faces, producing a shear-friction mode of failure.

ACI 318 shear-friction capacity—The nominal shear-friction capacity across a shear transfer (sliding) plane perpendicular to the shear-friction reinforcement is specified in ACI 318-05[2] as

$$V_n = A_{vf} f_y \mu \tag{2}$$

where μ is the coefficient of friction ($\mu = 1.4\lambda$ for concrete placed monolithically, where $\lambda = 1.0$ for normalweight concrete), A_{vf} is the total area of reinforcement crossing the shear plane, and f_y is the yield strength of reinforcement. It is stated in Section 6.4.4 of FEMA 356[3] that "shear-friction strength shall be calculated according to ACI 318-05,[2] taking into consideration the expected axial load due to gravity and earthquake effects," and ACI 318-05[2] Section 11.7.7 permits a permanent net axial compression force across a shear plane to be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$. According to Section 11.7.5, the nominal shear-friction capacity calculation should be capped by the upper limits of $0.2f_c'A_c$ and $5.516A_c$ (mm^2) (for V_n in Newtons) ($800A_c$ [in.^{2}] for V_n in lb).

Shear strength database from prior wall tests

Based on the preceding discussion, prior to evaluation of current test results, a systematic review of available research information was conducted to assess shear strength requirements for lightly reinforced wall segments with both single and double curtains of web reinforcement. A database of relevant test results was assembled by reviewing available research including work summarized by Hirosawa[4] and the papers by Hwang et al.,[5] Hidalgo et al.,[6] and Wood.[10] The tests were screened to exclude walls with thickness was less than or equal to 75 mm (3 in.), because of concerns that post-crack behavior could be substantially influenced by the narrow wall web geometry (for example, instability after concrete spalling). Some of the tests in the database were conducted on wall piers subjected to typically low levels of axial load ($N/A_g f_c' \leq 0.23$). The longitudinal and transverse web reinforcement ratios for the test specimens in the database were generally between 0.07 and 0.77%; however, for nine tests, no longitudinal web reinforcement was provided. The specimens typically had relatively high amounts of boundary reinforcement; and for some specimens, the wall thickness was increased at the wall edges to accommodate the larger quantity of boundary reinforcement (that is, so-called barbell-shaped wall cross-sections with boundary columns). Specimens tested by Sugano (reported by Hirosawa[4] and Hwang et al.[5]) and Cardenas (reported by Hwang et al.[5]) were tested under monotonic loading, whereas others were tested under reversed cyclic loading.

The geometric and material properties of the wall specimens included in the database, as well as levels of axial load applied ($N/A_g f_c'$) and the maximum lateral loads measured during the tests (V_{TEST}), are listed in Table 2. The maximum lateral load measured during each test (V_{TEST}) was compared with the FEMA nominal shear strength ($V_{n,FEMA}$) computed using the FEMA 356[3] provisions described in the preceding section. The measured-to-calculated shear strength ratios ($V_{TEST}/V_{n,FEMA}$) are presented in the table. For cases where the longitudinal and transverse web reinforcement ratios are different, the shear strength computed using Eq. (1) is based on the minimum value of the web reinforcement ratio

Table 2—Assembed wall test database

Reference	Specimen ID no.	h_w, cm	l_w, cm	α_c*	t_w, cm	f_c', MPa	ρ_t, %	f_{yl}, MPa	ρ_t, %	f_{yt}, MPa	A_{sb}, cm^2	Boundary columns	Axial load $N/A_g f_c'$	V_{TEST}, kN	No. of curtains	$V_{TEST}/V_{n,FEMA}$	$V_{TEST}^{\dagger}/V_{n,FEMA}$	$V_{TEST}^{\ddagger}/V_{n,FEMA}$	$V_{TEST}^{\dagger}/(1/6)A_{cv}\sqrt{f_c'}$
Sugano (1973)§,‖	140-1	180	396	0.25	12	20.6	0.66	572	0.66	572	23	Yes	0.12	2354	2	1.31	—	1.31	—
	141-2	180	396	0.25	12	20.8	0.66	572	0.66	572	23	Yes	0.23	2942	2	1.64	—	1.64	—
	142-3	180	396	0.25	12	21.3	0.66	572	0.66	572	23	Yes	0.16	3138	2	1.72	—	1.72	—
	143-4	180	396	0.25	12	19.6	0.33	572	0.33	572	23	Yes	0.10	1814	1	1.27	1.27	—	5.17
	144-5	180	396	0.25	12	20.8	0.33	572	0.33	572	23	Yes	0.10	1912	2	1.33	—	1.33	—
	145-6	180	396	0.25	12	20.5	0.69	284	0.66	284	23	Yes	0.11	2138	2	1.50	—	1.50	—
	146-7	180	396	0.25	12	19.6	0.69	284	0.66	284	23	Yes	0.11	1981	2	1.40	—	1.40	—
	147-8	180	396	0.25	12	20.9	0.77	397	0.74	397	23	Yes	0.12	2305	2	1.28	—	1.28	—
Hirosawa§	72	170	170	0.25	16	17.2	0.50	407	0.26	419	15	No	0.12	809	2	1.40	—	1.40	—
Barda et al. (1977)‖	B1-1	95	191	0.25	10	29.0	0.50	543	0.50	496	11	Yes	0	1276	2	1.75	—	1.75	—
	B2-1	95	191	0.25	10	16.3	0.50	552	0.50	499	40	Yes	0	965	2	1.51	—	1.51	—
	B3-2	95	191	0.25	10	27.0	0.50	545	0.50	513	26	Yes	0	1112	2	1.51	—	1.51	—
	B6-4	95	191	0.25	10	21.2	0.25	496	0.50	496	26	Yes	0	872	2	1.91	—	1.91	—
	B7-5	48	191	0.25	10	25.7	0.50	531	0.50	501	26	Yes	0	1140	2	1.58	—	1.58	—
	B8-5	191	191	0.25	10	23.4	0.50	527	0.50	496	26	Yes	0	889	2	1.26	—	1.26	—
Cardenas et al. (1980)‖	SW-7	206	191	0.25	8	43.0	0.94	448	0.27	414	12	No	0	519	1	1.23	1.23	—	3.11
	SW-8	206	191	0.25	8	42.5	2.93	448	0.27	465	0	No	0	569	1	1.29	1.29	—	3.43
Hidalgo (2002)#	1	200	100	0.17	12	19.4	0.25	392	0.13	392	10	No	0	198	1	1.25	1.25	—	2.25
	2	200	100	0.17	12	19.6	0.25	402	0.25	402	10	No	0	270	1	1.30	1.30	—	3.05
	4	200	100	0.17	12	19.5	0.25	402	0.38	402	13	No	0	324	1	1.55	1.55	—	3.67
	6	180	130	0.25	12	17.6	0.26	314	0.13	314	10	No	0	309	1	1.30	1.30	—	2.83
	7	180	130	0.25	12	18.1	0.13	471	0.25	471	10	No	0	364	1	1.32	1.32	—	3.29
	8	180	130	0.25	12	15.7	0.26	471	0.25	471	10	No	0	374	1	1.12	1.12	—	3.63
	9	180	130	0.25	10	17.6	0.26	366	0.26	366	9	No	0	258	1	1.00	1.00	—	2.84
	10	180	130	0.25	8	16.4	0.25	367	0.25	367	8	No	0	187	1	0.93	0.93	—	2.66
	11	140	140	0.25	10	16.3	0.26	362	0.13	362	8	No	0	235	1	1.08	1.08	—	2.49
	12	140	140	0.25	10	17.0	0.13	366	0.26	366	8	No	0	304	1	1.37	1.37	—	3.16
	13	140	140	0.25	10	18.1	0.26	370	0.26	370	8	No	0	289	1	1.03	1.03	—	2.91
	14	120	170	0.25	8	17.1	0.25	366	0.13	366	6	No	0	255	1	1.18	1.18	—	2.72
	15	120	170	0.25	8	19.0	0.13	366	0.25	366	6	No	0	368	1	1.65	1.65	—	3.72
	16	120	170	0.25	8	18.8	0.25	366	0.25	366	6	No	0	362	1	1.33	1.33	—	3.68
	21	180	130	0.25	10	24.2	0	366	0	366	6	No	0	258	1	1.06	1.06	—	2.42
	22	180	130	0.25	10	17.2	0	366	0	366	6	No	0	222	1	1.01	1.01	—	2.47
	23	180	130	0.25	10	24.2	0	431	0.25	431	11	No	0	333	1	1.37	1.37	—	3.12
	24	180	130	0.25	10	23.9	0.25	431	0	431	6	No	0	323	1	1.33	1.33	—	3.05
	25	140	140	0.25	10	23.9	0	431	0	431	6	No	0	352	1	1.35	1.35	—	3.09
	26	140	140	0.25	10	17.7	0	431	0	431	6	No	0	262	1	1.10	1.10	—	2.67
	27	140	140	0.25	10	23.9	0	431	0.25	431	9	No	0	491	1	1.88	1.88	—	4.30
	28	140	140	0.25	10	23.3	0.25	431	0	431	6	No	0	258	1	0.99	0.99	—	2.29
	29	105	150	0.25	8	23.2	0	431	0	431	6	No	0	400	1	1.80	1.80	—	4.15
	30	105	150	0.25	8	17.9	0	431	0	431	6	No	0	356	1	1.74	1.74	—	4.21
	31	105	150	0.25	8	23.1	0	431	0.25	431	8	No	0	391	1	1.76	1.76	—	4.07
	32	105	150	0.25	8	23.3	0.25	431	0	431	6	No	0	344	1	1.55	1.55	—	3.56
Ryo§	31	145	230	0.25	8	17.3	0.17	485	0.18	485	16	Yes	0	608	1	1.77	1.77	—	4.76
Sugano§	71	145	230	0.25	8.3	25.2	0.07	461	0.07	461	16	Yes	0	804	1	2.16	2.16	—	5.03
Aoyagi§	150	152	272	0.25	16	29.4	0.58	339	0.62	339	18	Yes	0	1553	2	1.07	—	1.07	—
	152	152	272	0.25	16	29.2	0.58	339	0.62	339	66	Yes	0	2308	2	1.60	—	1.60	—
	148	152	272	0.25	8	19.7	0.71	353	0.76	353	18	Yes	0	931	2	1.18	—	1.18	—
	149	152	272	0.25	8	25.9	0.71	353	0.76	353	18	Yes	0	1029	2	1.25	—	1.25	—
	151	152	272	0.25	8	23.8	0.71	353	0.76	353	66	Yes	0	1495	2	1.84	—	1.84	—
Average		153.4	204.9	0.25	10.3	21.9	0.39	429	0.32	425	16	—	0.02	871	1.38	1.40	1.36	1.48	3.35
Standard deviation		36.3	95.6	0.02	2.1	5.6	0.45	79	0.24	75	13	—	0.05	787	0.49	0.28	0.30	0.23	0.79

*For nominal shear strength ($V_{n,FEMA}$) calculation in SI units.
†One curtain of distributed web reinforcement.
‡Two curtains of distributed web reinforcement.
§Reported by Hirosawa.[4]
‖Reported by Hwang et al.[5]
#Reported by Hidalgo et al.[6]
Note: 1 cm = 0.394 in.; 1 cm^2 = 0.155 in.^{2}; 1 MPa = 0.145 ksi; 1 kN = 0.225 kips.

multiplied by the corresponding value for the reinforcement yield stress, that is, $\rho_t f_y$ in Eq. (1) was taken as minimum of $\rho_t f_{yt}$ and $\rho_l f_{yl}$ for each test. This approach is consistent with common interpretations (for example, Sozen and Moehle[11] and Wood[10]) of the aforementioned ACI 318-05[2] requirement that the longitudinal reinforcement ratio cannot be less than the transverse reinforcement ratio for walls with h/l not exceeding 2.0. The ACI 318-05[2] equation (Eq. (1)) was used to assess wall nominal shear strength ($V_{n,FEMA}$) when the transverse reinforcement ratio (ρ_n in FEMA 356,[3] replacing ρ_t in ACI 318-05[2]) falls between 0.0025 and 0.0015; however, when ρ_n is less than 0.0015, the contribution of reinforcement to wall shear strength was held constant at the value obtained for ρ_n of 0.0015. Further, the nominal shear strength calculations in the table were capped by the ACI 318-05[2] upper limit of $0.83A_{cw}\sqrt{f_c'}\text{ (MPa)} = 10A_{cw}\sqrt{f_c'}\text{ (psi)}$.

For the wall tests in the database, the ratio of the wall thickness (typically 80 to 125 mm [3.15 to 5 in.]) to the diameter of the reinforcing bars used for web reinforcement (typically ϕ9.5 mm [U.S. No. 3]), was between 8.5 to 13.2. For actual construction, where wall thickness is likely to be in the range of 125 to 250 mm (4.9 to 9.8 in.) and the diameter of the web reinforcement is likely to be ϕ13 to ϕ19 mm (U.S. No. 4 to 6) for a single curtain, ratios of wall thickness to web bar diameter of (125 mm [4.9 in.])/(13 mm [0.51 in.]) = 9.6 to (250 mm [10 in.])/(19 mm [0.75 in.]) = 13.1 should be expected. Therefore, the ratios of wall thickness to bar sizes for the tests included in the database are reasonably representative of walls with thickness of up to approximately 250 mm (10 in.) or possibly 300 mm (12 in.) in actual building construction. Extrapolation of the test results reviewed herein to thicker walls (>250 mm [10 in.]) with single curtains of web reinforcement is not appropriate unless additional data are located.

Among the measured-to-calculated shear strength ratios ($V_{TEST}/V_{n,FEMA}$) listed in Table 2, ratios of $V_{TEST}/V_{n,FEMA} > 1.0$ imply that FEMA 356[3] specifications produce a conservative estimate of the wall shear strength. As indicated in Table 2, average measured-to-calculated shear strength ratios ($V_{TEST}/V_{n,FEMA}$) for the tests are 1.36 and 1.48 for walls with one and two curtains of web reinforcement, respectively. A ratio of less than 1.0 is obtained for only two tests (Hidalgo et al.,[6] Specimens 10 and 28). The standard deviation of the ratios for walls with one curtain of web reinforcement (0.30) is only slightly higher than that for walls with two curtains (0.23), despite the substantial variation in the web reinforcement provided.

The $V_{TEST}/V_{n,FEMA}$ ratios for the walls in the database (Table 2) are plotted against the minimum (of longitudinal and transverse) web reinforcement ratios in Fig. 7. Ratios obtained for the UCLA specimens that failed under diagonal tension (specimen Types 1, 2, 3, and 5), which will be discussed in detail in the following section, are also included in the figure. For the walls in the database that satisfy the minimum web reinforcement ratio of at least 0.25% in both directions, average $V_{TEST}/V_{n,FEMA}$ ratios obtained are 1.21 and 1.48 for walls with one and two curtains of web reinforcement, respectively, with standard deviations of 0.19 and 0.23. For the walls that do not satisfy the minimum web reinforcement ratio in both directions (all with one curtain of web reinforcement), the average $V_{TEST}/V_{n,FEMA}$ ratio obtained is 1.43, with a standard deviation of 0.33.

Results obtained for the wall test database indicate that the FEMA 356[3] procedure for calculating wall nominal shear strength essentially provides a lower-bound estimate to the shear strength measurements achieved during these tests, regardless of whether the walls satisfy the minimum web reinforcement ratio of at least 0.25% in both directions or whether the walls have one or two curtains of distributed web reinforcement. Furthermore, the experimental evidence does not support the implication that wall shear strength cannot be taken greater the concrete nominal shear strength $(1/6)A_{cv}\sqrt{f_c'}\text{ (MPa)} = 2A_{cv}\sqrt{f_c'}\text{ (psi)}$ for wall segments with a single curtain of reinforcement. Ratios of measured lateral load capacity of the wall specimens with a single curtain of web reinforcement to the concrete nominal shear strength ($V_{TEST}/(1/6)A_{cv}\sqrt{f_c'}\text{ (MPa)}$) are between 2.2 and 5.2, with an average of 3.35 and a standard deviation of 0.79 (refer to Table 2), indicating that the FEMA shear strength calculation provides a much better lower-bound estimate of the lateral load capacity of walls with a single curtain of reinforcement, compared to taking the nominal shear strength of concrete alone.

Current test results

The wall spandrel and pier specimens tested during the aforementioned experimental program at UCLA have transverse web reinforcement ratios of 0.28%. The longitudinal web reinforcement ratios are 0.43% for Type 1 spandrels, 0.4% for Type 2 spandrels, 0.26% for Types 3 and 4 spandrels, and 0.23% for Type 5 piers. Part of the longitudinal web reinforcement (four out of six bars for Types 1 and 2, and two out of four bars for Types 3 and 4), however, is not continuous over the entire height of the spandrels; that is, it is cut at the WPJ to provide a crack initiation plane. This can be interpreted as a reduction in the effective area of the longitudinal web reinforcement, which reduces the longitudinal reinforcement ratio of Type 1 spandrels to 0.14%, Type 2 spandrels to 0.13%, and Type 3 and 4 spandrels to 0.13%. Based on common interpretations of ACI 318-05,[2] the shear strength computed using Eq. (1) should be based on the minimum value of the web reinforcement ratios (provided the yield strength of the transverse and longitudinal reinforcement is the same); and considering FEMA 356[3] recommends using a minimum reinforcement ratio of 0.15% for the shear strength calculation, the expected shear strength of the spandrel specimens ($V_{n,FEMA}$) was calculated using a reinforcement ratio of 0.15%.

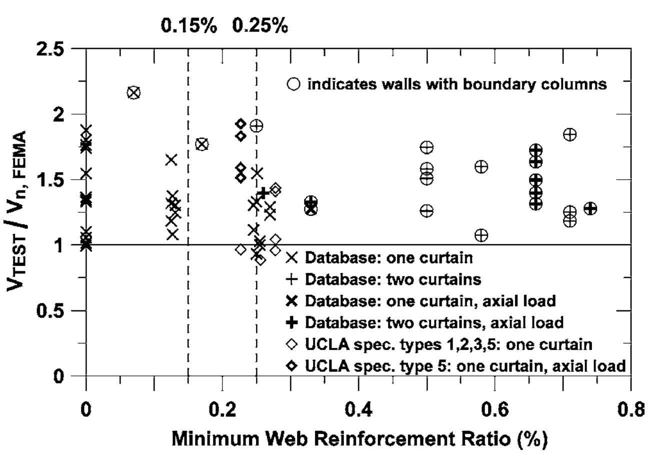


Fig. 7—Comparison of wall test data with FEMA 356[3] nominal shear strength calculation for walls with different minimum web reinforcement ratios.

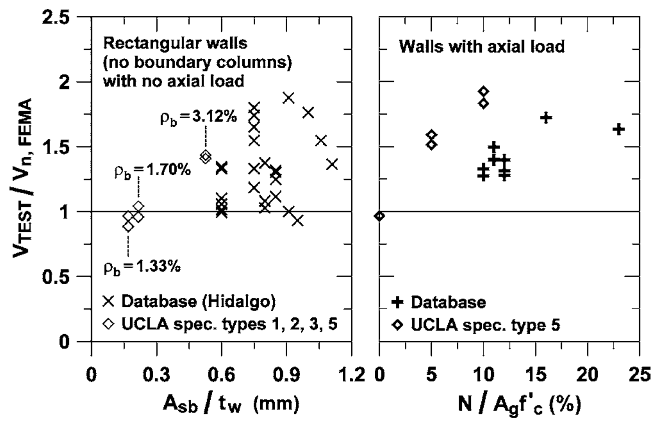
A reinforcement ratio of 0.15% was also used for the FEMA shear strength calculation of the pier specimens, because no hooks were provided on the transverse web reinforcement of the piers (as well as Type 3 and 4 spandrels); thus, the reinforcement might not be capable of reaching the yield stress at potential diagonal crack locations.

The average maximum lateral load measurement (average of positive and negative loading directions) for each test (V_{TEST}) was compared with the FEMA nominal shear strength ($V_{n,FEMA}$) calculations. The measured-to-calculated shear strength ratios ($V_{TEST}/V_{n,FEMA}$) for the wall specimens that failed under diagonal tension mechanism (Types 1, 2, 3, and 5) are presented in Table 3. It must be noted that the results of Test 10 are not included in the table for strength comparisons, as wall pier Specimen WP-T5-N0-S2 was substantially damaged (cracked) under accidentally-applied axial tensile load prior to lateral-load testing.

Comparisons of the maximum lateral load measured for each test with the nominal shear strength of concrete are also presented in Table 3. The measured-to-calculated shear-friction capacity comparisons ($V_{TEST}/V_{n,ACI-SF}$) for Type 4 specimens, which experienced shear-friction failure along the WPJ located at the wall-pedestal interface, are also included in the table.

Nominal shear strength—Overall average of the results presented in Table 3 indicate that the FEMA nominal shear strength calculation ($V_{n,FEMA}$) provides a lower-bound estimate of the measured lateral load capacity of the spandrel and pier specimens that failed in shear (diagonal tension). For all of these specimens, the measured lateral load capacity significantly exceeds the nominal shear strength contribution of concrete. This result in inconsistent with the ACI 318 requirement that the nominal shear strength of walls with a single curtain of web reinforcement cannot be taken greater than the concrete shear strength alone. This is also consistent with the results obtained for the wall test database (Table 2).

Effect of boundary reinforcement—A closer look at the results reveals that the FEMA nominal shear strength calculation seems to have underestimated the shear strength of Type 2 and 3 spandrel specimen and the Type 5 pier specimen with zero axial load. For Type 1 spandrel specimens, the nominal shear strength calculation provided a conservative estimate. The reason for this might be that Type 1 spandrels, similar to the specimens in the assembled wall test database, had relatively higher amounts of boundary reinforcement compared to Type 2, 3, and 5 specimens. In fact, Type 1 and 2 spandrel specimens were differentiated primarily by the amount of boundary reinforcement provided (ρ_b = 3.12% for Type 1, ρ_b = 1.70% for Type 2). This trend is apparent in Fig. 8(a), where the measured-to-calculated shear strength ratios ($V_{TEST}/V_{n,FEMA}$) are plotted against the amount of boundary reinforcement (boundary steel area per wall thickness), for Type 1, 2, 3, and 5 specimens, as well as for rectangular wall specimens in the test database with no axial load and with amounts of



(a) (b)

Fig. 8—Comparison of wall test data with FEMA 356[3] nominal shear strength calculation for walls with: (a) different amounts of boundary reinforcement; and (b) different levels of axial load.

Table 3—Comparison of test results with nominal strength calculations

Specimen ID no.	Type	Test no.	V_{TEST}, kN[*]	$V_{n,FEMA}$,[†] kN	$V_{n,ACI-SF}$,[‡] kN	$V_{n,ACI-FLEX}$,[§] kN	$V_{TEST}/V_{n,FEMA}$	$V_{TEST}/(1/6)A_{cv}\sqrt{f'_c}$ [‖]	$V_{TEST}/V_{n,ACI-SF}$
WS-T1-S1	1	Test 1	633	441	887	949	1.44	3.25	—
WS-T1-S2		Test 4	749	531	959	967	1.41	2.94	—
WS-T2-S1	2	Test 2	453	473	556	579	0.96	2.10	—
WS-T2-S2		Test 3	491	471	556	579	1.04	2.29	—
WS-T3-S1	3	Test 11	398	449	381	358	0.89	1.84	—
WS-T3-S2		Test 14	406	459	381	358	0.88	1.82	—
WS-T4-S1	4	Test 12	330	450	381	313	—	—	0.87
WS-T4-S2		Test 13	341	456	381	313	—	—	0.89
WP-T5-N0-S1	5	Test 9	404	419	536	415	0.97	2.13	—
WP-T5-N5-S1	5	Test 7	648	428	1003	741	1.51	3.31	—
WP-T5-N5-S2		Test 8	682	428	1003	741	1.59	3.47	—
WP-T5-N10-S1	5	Test 5	753	411	1153	948	1.83	4.08	—
WP-T5-N10-S2		Test 6	819	425	1153	1011	1.93	4.21	—
						Average	1.31	2.86	0.88
						Standard deviation	0.38	0.87	0.02

[*]Average of maximum lateral load measured in positive and negative directions.
[†]Nominal shear strength per FEMA 356.[3]
[‡]Nominal shear friction capacity per ACI 318-05.[2]
[§]Nominal flexural capacity per ACI 318-05.[2]
[‖]Nominal shear strength of concrete alone is $(1/6)A_{cv}\sqrt{f'_c}$ (MPa) = $2A_{cv}\sqrt{f'_c}$ (psi).
Note: 1 kN = 0.225 kips.

boundary and longitudinal web reinforcement comparable to those of the current specimens.

The results plotted indicate that the FEMA nominal shear strength calculation may provide a more reasonable lower-bound estimate of the shear strength of wall segments with boundary reinforcement ratios larger than 3% (assuming there are no boundary columns, that is, the wall cross section is rectangular). For rectangular walls with boundary reinforcement ratios smaller than 3%, the FEMA nominal shear strength calculation may provide a slightly unconservative estimate of wall shear strength.

Effect of lack of hooks on web reinforcement—Type 2 and 3 spandrel specimens have longitudinal web reinforcement ratios of 0.4% and 0.26%, respectively, when the effective reduction in the amount of longitudinal reinforcement due to discontinuity of longitudinal bars at the WPJ is ignored. When the reduction is considered, the reinforcement ratios are reduced to 0.13% for both specimen types. Unlike Type 2 spandrels, 180-degree hooks are not provided on the transverse web reinforcement of Type 3 spandrels. Average $V_{TEST}/V_{n,FEMA}$ values obtained for Type 2 and 3 spandrels are 1.00 and 0.88, respectively. Considering that the boundary reinforcement ratio of Type 2 specimens (1.70%) is slightly larger than that of Type 3 specimens (1.33%), it appears that the lack of 180-degree looks on the transverse web reinforcement of Type 3 spandrels does not have a significant influence on their measured shear strength.

Effect of discontinuity of web reinforcement—Hooks were also not provided on the transverse web reinforcement of the Type 5 pier specimens, and longitudinal web reinforcement (ρ_l = 0.23%) was continuous over the specimen height, because a WPJ was not provided. Comparing results of the Type 5 pier specimen with zero axial load ($V_{TEST}/V_{n,FEMA}$ = 0.97) with average results of Type 3 spandrels ($V_{TEST}/V_{n,FEMA}$ = 0.89) with the WPJs (ρ_l = 0.26% with two out of four longitudinal web bars discontinued), it is apparent that discontinuity of the longitudinal web bars at the WPJ has some negative influence on the expected shear strength of the walls, but the influence is not as pronounced as the effect of the amount of boundary reinforcement on the expected shear strength. This is consistent with the results plotted in Fig. 7 (for both the current specimens and the walls in the aforementioned test database), where it is apparent that the web reinforcement ratio does not significantly or consistently influence the $V_{TEST}/V_{n,FEMA}$ ratio obtained.

Effect of axial load—The FEMA nominal shear strength calculation significantly underestimates the lateral load capacity of the pier specimens with axial load levels of 5 and 10% $A_g f_c'$ (Table 3). This is expected because the influence of axial load on the shear strength of concrete is not considered in the nominal shear strength calculation. Unfortunately, few tests of wall piers with axial load exist; therefore, more definitive conclusions cannot be reached. The $V_{TEST}/V_{n,FEMA}$ ratios plotted in Fig. 8(b) against applied axial load levels for the Type 5 pier specimens (including the specimen with zero axial load for comparison), as well as for the walls in the test database with axial load, however, reveal that the FEMA 356[3] calculation tends to be conservative in estimating the nominal shear strength of walls piers subjected to even relatively low levels of axial load. For evaluation of existing buildings, underestimation of pier shear strength might imply pier failures (soft-story), when, in fact, strong piers exist and produce costly, unnecessary retrofits.

Nominal flexural capacity—Although none of the wall specimens failed in flexure, the calculated nominal flexural capacities ($V_{n,ACI\text{-}FLEX}$) of the specimens are also listed in Table 3 for comparison. Nominal moment capacities at wall cross sections (M_n) were calculated per Sections 10.2 and 10.3 of ACI 318-05,[2] assuming a rectangular stress block for concrete in compression, a compressive strain of 0.003 at extreme fiber of concrete and no strain hardening in reinforcing steel. Actual (measured) material strengths were used in the calculations. Reduction of the cross section and discontinuity of longitudinal web bars at the WPJ was considered only for the Type 4 specimens, where the WPJ was located at the cross section subjected to maximum bending moment. Axial load on the pier specimens was also considered in the nominal moment capacity calculations. The nominal flexural lateral load capacity values ($V_{n,ACI\text{-}FLEX}$) in Table 3 were determined based on the calculated nominal moment capacities M_n, considering the double-curvature loading condition imposed during the tests, that is

$$V_{n,ACI\text{-}FLEX} = (M_{n,TOP} + M_{n,BOTTOM})/(\text{wall height}) \quad (3)$$

The results indicate that the ACI nominal flexural capacity ($V_{n,ACI\text{-}FLEX}$) is less than the FEMA nominal shear strength ($V_{n,FEMA}$) and the ACI shear-friction capacity ($V_{n,ACI\text{-}SF}$) for Type 3 and 4 specimens and the Type 5 specimen with zero axial load (WP-T5-N0-S1). None of these specimens experienced a flexural mode of failure or significant nonlinear flexural deformations, however. For a more accurate estimation of the nominal flexural capacity (to better represents the test conditions), the nominal moment capacity calculations were revised to consider the weight (53 kN [12 kips]) of the steel loading frame (Fig. 4) as axial load on the wall cross section, which resulted in nominal flexural capacity estimates of 406, 362, and 467 kN (91, 81, and 105 kips) for Type 3 specimens, Type 4 specimens, and Specimen WP-T5-N0-S1 (Test 9), respectively. Further considering strain hardening in longitudinal reinforcement (calibrated using measured stress-strain test results[7-9] on reinforcing bar coupon samples) resulted in nominal flexural capacities of 465, 448, and 510 kN (104, 101, and 115 kips), respectively, all of which exceed the measured lateral load capacities (V_{TEST}) of these specimens. The more accurate flexural capacity estimations, which better represent the test conditions and the material properties, are consistent with test observations that indicated these specimens did not experience a flexural mode of failure. It is not possible, however, to reach definite conclusions on the conservatism (or lack thereof) of code-based nominal flexural capacity estimations because none of the specimens in this experimental program failed in flexure.

Nominal shear-friction capacity—For Type 4 spandrel specimens that failed in shear-friction across the WPJ at the wall-pedestal interface, the ACI nominal shear-friction capacity calculation ($V_{n,ACI\text{-}SF}$) slightly overestimates the lateral load capacities measured during testing. WPJs were also provided along the midheight (when oriented vertically) of Type 1, 2, and 3 spandrel specimens. Types 3 and 4 were identical except for the location of the WPJ. Type 3 specimens failed in shear (with diagonal cracks propagating across the WPJ with no significant deviation in crack path and direction and crushing of concrete at wall midheight at ultimate) and exhibited lateral load capacities larger than their calculated ACI nominal shear-friction capacities. Type 4 specimens, on the other hand, failed to reach their calculated shear friction capacities.

One possible reason for this is that the nominal flexural capacity of the Type 4 specimens (calculated considering the reduced cross-sectional area and the discontinuity of the reinforcing bars at the WPJ and the weight of the steel loading frame, but assuming no strain hardening reinforcing steel) was 362 kN (81 kips). The ACI nominal shear-friction capacity of these specimens was calculated to be 381 kN (86 kips) and its measured lateral load capacity was 335 kN (75 kips) on average. Therefore, it is very likely that these specimens experienced flexural yielding at or slightly below a lateral load level of approximately 335 kN (75 kips) at the WPJ where the moment demand was maximum, and the initiation of flexural yielding immediately triggered a sliding shear mechanism prior to crushing of concrete in the compression zone. This was confirmed by measurements obtained from vertical LVDTs straddling the WPJ, which indicated the magnitude of the flexural deformations approached that expected to produce yielding of the boundary reinforcement. Nevertheless, the failure mode of these specimens was clearly sliding shear failure and not flexural failure, as the sliding deformations measured along the WPJ dwarfed flexural deformations once the sliding mechanism was initiated.

Another reason why the ACI 318-05[2] nominal shear friction capacity calculation might have overestimated the capacity of the Type 4 specimens is that the ACI nominal capacity calculation is based on results of monotonic tests, whereas all of the Type 4 specimens that failed in sliding shear were tested under reversed cyclic loading. Unfortunately, definite conclusions cannot be reached on this issue given the limited test data provided in the present test program.

Overall, the results indicate that the shear-friction capacity of a wall segment may be significantly influenced by the location of the WPJ on the wall, and possibly, the load history. The specimens that had the WPJ at the wall-pedestal interface (where the bending moment was maximum) experienced significant flexural cracking and possible flexural yielding at the WPJ, which led to a progressive failure mode associated with sliding shear across the WPJ. Particular attention must therefore be paid to the assessment of the flexural yield capacity of the walls with WPJs at locations where the moment demand is significant, as flexural yielding at these sections may trigger a premature sliding shear failure across the WPJ.

SUMMARY AND CONCLUSIONS

An experimental program was conducted to assess shear strength requirements for lightly reinforced wall pier and spandrels commonly used in mid-1900s building construction. As well, a database of relevant test results available in the literature was assembled and studied. Test results were compared with ACI 318-05[2] provisions and FEMA 356[3] recommendations on wall nominal shear strength to evaluate the reliability of these documents or the conservatism embedded therein pertaining to seismic evaluation and rehabilitation of existing buildings. The effect of outdated construction practices was also investigated, including using a single curtain of distributed reinforcement, presence of WPJ, and lack of hooks on transverse reinforcement on the shear strength of wall piers and spandrels. The findings of this study are summarized in the following paragraphs:

1. Use of the FEMA[3] nominal shear strength calculation for walls with a single curtain of web reinforcement is appropriate, provided the wall thickness does not exceed approximately 300 mm (12 in.), the longitudinal reinforcement is continuous, the transverse web reinforcement is sufficiently anchored with 180-degree hooks, and a moderate amount of boundary reinforcement is provided at the wall boundaries (for example, a boundary reinforcement ratio larger than 3% for wall segments with rectangular cross sections). The experimental results do not support the implication of the ACI 318-05[2] provisions that the shear strength of existing wall segments with one curtain of web reinforcement cannot be taken larger than the nominal shear strength of concrete alone. In addition, based on very limited data, measured-to-calculated shear strength ($V_{TEST}/V_{n,FEMA}$) ratios obtained for specimens tested under reversed cyclic loading are generally not less than those obtained for specimens tested under monotonic loading[4,5];

2. Discontinuity of a portion of the longitudinal web reinforcement at a possible WPJ and the lack of hooks on transverse reinforcement may have some negative influence on the expected shear strength of wall segments expected to fail in diagonal tension; but the influence is rather modest (in the range of 10% for the specimens tested), and the impact is not as pronounced as that of the amount of boundary reinforcement provided. For wall spandrels with rectangular cross sections and boundary reinforcement ratios smaller than 3%, the FEMA[3] nominal shear strength calculation (assuming a web reinforcement ratio of 0.15%) provides a slightly unconservative estimate of wall shear strength ($V_{TEST}/V_{n,FEMA}$ = 0.88 for ρ_b = 1.33%; $V_{TEST}/V_{n,FEMA}$ = 0.96 for ρ_b = 1.70%);

3. The FEMA[3] provisions for calculating nominal shear strength substantially underestimates the shear strength of the wall piers subjected to even relatively low axial load levels of 5% ($V_{TEST}/V_{n,FEMA}$ = 1.55) and 10% $A_g f_c'$ ($V_{TEST}/V_{n,FEMA}$ = 1.88), regardless of the amount of boundary reinforcement provided and the anchorage conditions of transverse reinforcement. This finding is not unexpected, because the influence of axial load on the shear strength of concrete is not considered in the FEMA[3] nominal shear strength calculation; however, the level of conservatism is cause for concern for evaluation of existing buildings, as it may lead to erroneous prediction of soft-story failures and produce costly, unnecessary retrofits; and

4. Particular attention must be paid to the evaluation of the shear strength of wall segments with WPJs (with part of the longitudinal web reinforcement discontinued), particularly at locations where moment demands are critical. Under these conditions, the wall segments are prone to an early sliding shear type of failure following flexural yielding, and the ACI nominal shear-friction capacity equation may give an unconservative estimate of their shear strength. On the other hand, shear-friction failure seems to be less critical for wall segments with WPJ at the wall center where bending moments are low, because wall strength is limited by diagonal cracking versus sliding along the WPJ.

ACKNOWLEDGEMENTS

The work presented in this paper was supported by funds from KPFF Los Angeles Consulting Engineers and conducted in collaboration with KPFF engineers J. Gavan, A. Reynolds,. A. Kulahci, and Dr. L. Toranzo. The assistance of UCLA researchers A. Salamanca and T. Kang, as well as UCLA graduate students D. Naish and A. Lemnitzer are greatly appreciated. Undergraduate laboratory assistants Y. Majidi, B. Bozorgnia, PEER summer intern N. N. Lam, and Senior Development Engineer S. Keowen also helped conduct the experimental program. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect those of the supporting organization or other people acknowledged herein.

REFERENCES

1. Office of Statewide Health Planning and Development, "Summary of Hospital Seismic Performance Ratings, April 2001," Sacramento, CA, 2001, 27 pp.

2. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)," American Concrete Institute, Farmington Hills, MI, 2005, 430 pp.

3. Federal Emergency Management Agency, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," *Report* No. FEMA-356/Nov. 2000, Washington, DC, 2000, 518 pp.

4. Hirosawa, M., "Past Experimental Results on Reinforced Concrete Shear Walls and Their Analysis," Kenchiku Kenkyu Shiryo, No. 6, Building Research Institute, Ministry of Construction, Tokyo, Japan, 1975, 277 pp. (in Japanese)

5. Hwang, S. J.; Fang, W. H.; Lee, H. J.; and Yu H. W., "Analytical Model for Predicting Shear Strength of Squat Walls," *ASCE Journal of Structural Engineering*, V. 127, No. 1, 2001, pp. 43-50.

6. Hidalgo, P. A.; Ledezma, C. A.; and Jordan, R. M., "Seismic Behavior of Squat Reinforced Concrete Shear Walls," *Earthquake Spectra*, V. 18, 2002, pp. 287-308.

7. Wallace, J. W.; Massone, L. M.; and Orakcal, K., "St. Joseph's Healthcare Orange, California, SPC-2 Upgrade: E/W Wing Component Test Program—Final Report," *Report* No. UCLA SEERL 2006/1, University of California Los Angeles, Los Angeles, CA, 2006, 66 pp.

8. Wallace, J. W.; Orakcal, K.; Massone, L. M.; and Kang, T. H.-.K., "St. Jude Medical Center, Fullerton, California, Horizontal Wall Segment Component Test Program—Final Report," *Report* No. UCLA SEERL 2007/1, University of California Los Angeles, Los Angeles, CA, 2007, 45 pp.

9. Massone, L. M., "RC Wall Shear-Flexure Interaction: Analytical and Experimental Responses," PhD dissertation, University of California Los Angeles, Los Angeles, CA, 2006, 398 pp.

10. Wood, S. L., "Shear Strength of Low-Rise Reinforced Concrete Walls," *ACI Structural Journal*, V. 87, No. 1, Jan.-Feb. 1990, pp. 99-107.

11. Sozen, M. A., and Moehle, J. P., "Stiffness of Reinforced Concrete Walls Resisting In-Plane Shear," Electric Power Research Institute, 1993, 178 pp.