SHEAR STRENGTH OF LIGHTLY REINFORCED WALL PIERS AND SPANDRELS

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ABSTRACT :
Between 1950's and 1970's, 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), which 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, 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 assessment of nominal shear strength.

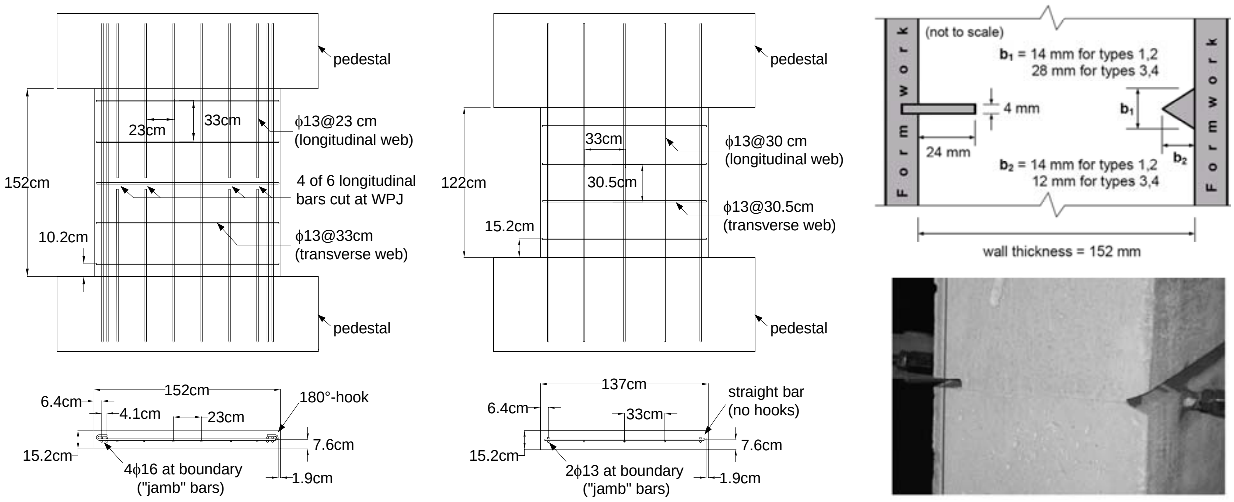
KEYWORDS:
shear; friction; strength; wall; spandrel; pier; reinforced concrete; experiment

1. INTRODUCTION

Between 1950's and 1970's, use of lightly reinforced, perimeter walls with openings was fairly common. For example, according to California Office of Statewide Health Planning and Development (2001), 1012 of the 2585 California hospitals (39%) are rated as SPC-1, that is, they pose a significant risk of collapse. Of the 821 SPC-1 buildings that were classified by building type, 271 or 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). In contrast, reinforced 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 the rehabilitation. However, the guidelines for structural walls in the FEMA 356 (2000) report on seismic rehabilitation of existing buildings focus more on applications for walls controlled by flexure (slender walls) versus shear-controlled cases (squat walls; e.g., wall piers and spandrels). Shear strength provisions of FEMA 356 generally follow ACI 318-99 (1999) requirements, which were developed for new buildings. Therefore, the impacts, on the shear strength calculation, of typical outdated construction details such as using one curtain (vs. two) of web reinforcement, discontinuity of reinforcement at a weakened plane joint, and the lack of hooks on transverse reinforcement, are not explicitly considered for evaluation purposes. Based on these shortcomings, an experimental program was conducted on selected lightly reinforced wall pier and spandrel configurations to investigate the effect of such outdated construction practices on the shear strength and lateral load behavior of wall segments in existing buildings. As well, a database of relevant test results available in the literature was assembled and studied to assess shear strength requirements for lightly reinforced wall segments with both single and double curtains of web reinforcement.

2. EXPERIMENTAL PROGRAM

The experimental program conducted at the 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 in the early 1960's utilizing 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 in order to initiate and control cracking). The specimens were tested in an upright position, and relatively low shear-span-to-depth (*M/(Vl)*) ratios (corresponding to one-half of the aspect ratio) were achieved during testing via fixing the base of the walls, restraining rotations at the top of the walls, and applying the lateral load at specimen mid-height level, via an L-shaped steel loading frame. 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. A detailed description of the complete experimental program can be found elsewhere (Massone (2006), Wallace et al. (2006, 2007)).



(a) Wall spandrel (type 1) (b) Wall pier (type 5) (c) WPJ detail

Figure 1 Sample wall specimen geometry and reinforcement.

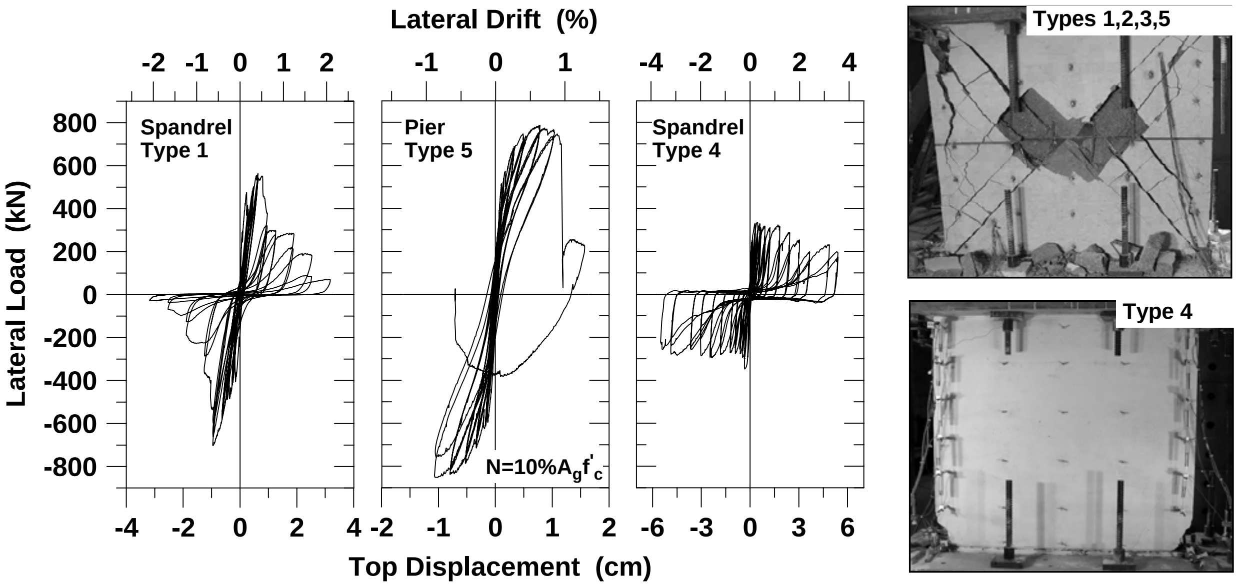
Figures 1(a) and 1(b) show representative wall geometry and reinforcement layouts for selected wall spandrel wall pier specimen configurations included in the experimental program. Four different types of wall spandrel (WS) specimens were tested, with two identical specimens of each type. Type 1 and 2 specimens were differentiated primarily by the amount of the longitudinal reinforcement provided at wall boundaries ("jamb" bars), whereas for Type 3 and 4 specimens, 180° hooks were not provided on the transverse reinforcement and a lower longitudinal web reinforcement ratio was used. The WPJ was located at wall mid-height for specimen Types 1, 2, and 3, whereas it was located at a distance of 25 mm from the bottom wall-pedestal interface for Type 4 specimens. The WPJs were created by attaching prefabricated wood strips on the interior surface of the formwork over the full width of the spandrels (Fig. 1(c)), as well as cutting a portion of the longitudinal web bars at the location of the WPJ (Fig. 1(a)). All six of the wall pier (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 weakened plane joint on the pier specimens; however, no hooks were provided on the transverse reinforcement (Fig. 1(b)). Dimensions, reinforcement, and material properties of the test specimens are presented in Table 2.1.

Table 2.1 Properties of wall spandrel (WS) and wall pier (WP) specimens

Specimen		Test	t_w	l_w	h_w	$M/(Vl_w)$	Transverse Web Reinf.			Longitudinal Web Reinf.			Boundary Reinf.		Axial Load	Material properties (MPa)		
ID No.	Type	No.	(cm)	(cm)	(cm)		Rebar(1)	ρ_t (%)	Hooks	Rebar(1)	ρ_l (%)	Cut Bars	Rebar(1)	ρ_b (%)	$N/A_g f'_c$ (%)	f'_c	f_y, ϕ13	f_y, ϕ16
WS-T1-S1	1	test1	15.2	152	152	0.50	ϕ13@33cm	0.278	Yes	ϕ13@23cm	0.428	4 of 6(2)	4-ϕ16	3.12	0	25.5	424.0	448.2
WS-T1-S2		test4	15.2	152	152	0.50	ϕ13@33cm	0.278	Yes	ϕ13@23cm	0.428	4 of 6(2)	4-ϕ16	3.12	0	43.7	424.0	448.2
WS-T2-S1	2	test2	15.2	152	152	0.50	ϕ13@33cm	0.278	Yes	ϕ13@23cm	0.400	4 of 6(2)	1-ϕ13 + 1-ϕ16	1.70	0	31.4	424.0	448.2
WS-T2-S2		test3	15.2	152	152	0.50	ϕ13@33cm	0.278	Yes	ϕ13@23cm	0.400	4 of 6(2)	1-ϕ13 + 1-ϕ16	1.70	0	31.0	424.0	448.2
WS-T3-S1	3	test11	15.2	152	152	0.50	ϕ13@28cm	0.278	No	ϕ13@28cm	0.256	2 of 4(2)	2-ϕ13	1.33	0	31.7	351.6	-
WS-T3-S2		test14	15.2	152	152	0.50	ϕ13@28cm	0.278	No	ϕ13@28cm	0.256	2 of 4(2)	2-ϕ13	1.33	0	33.6	351.6	-
WS-T4-S1	4	test12	15.2	152	152	0.50	ϕ13@28cm	0.278	No	ϕ13@28cm	0.256	2 of 4(3)	2-ϕ13	1.33	0	31.9	351.6	-
WS-T4-S2		test13	15.2	152	152	0.50	ϕ13@28cm	0.278	No	ϕ13@28cm	0.256	2 of 4(3)	2-ϕ13	1.33	0	33.0	351.6	-
WP-T5-N0-S1	5	test9	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	0	29.9	424.0	-
WP-T5-N0-S2		test10	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	0	31.0	424.0	-
WP-T5-N5-S1		test7	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	5	31.9	424.0	-
WP-T5-N5-S2		test8	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	5	32.0	424.0	-
WP-T5-N10-S1		test5	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	10	28.3	424.0	-
WP-T5-N10-S2		test6	15.2	137	122	0.44	ϕ13@30.5cm	0.278	No	ϕ13@33cm	0.227	-	2-ϕ13	1.33	10	31.4	424.0	-

(1) ϕ13 (13 mm diameter) = US #4; ϕ16 (16 mm diameter) = US #5; (2) Weakened plane joint at wall midheight; (3) Weakened plane joint at wall-pedestal interface

Figures 2(a), 2(b), and 2(c) present typical lateral load vs. top displacement responses measured for selected wall specimen types. Measurements from local instrumentation revealed that the lateral displacement of the spandrels of Types 1, 2, 3, as well as the piers (Type 5), 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 and sliding along the WPJ were found to have minor influence on the overall wall displacement history, and lateral load failure (degradation of lateral load capacity) was associated with crushing of concrete close to the center of the wall, followed by spalling of diamond-shaped wedges of concrete (Fig. 2(d)). The lateral load behavior and failure mode of Type-4 spandrel specimens (where the WPJ was located at the wall-pedestal interface), however, 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 weakened plane joint at the bottom wall-pedestal interface (Fig. 2(d)). 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.



(a) Test 1 (b) Test 6 (c) Test 13 (d) Typical failure modes

Figure 2 Representative lateral load-displacement responses and failure modes for the wall specimens.

3. ASSESSMENT OF WALL SHEAR STRENGTH

3.1. Code Provisions

Except for minor changes in format, the ACI 318 equation for wall nominal shear strength has not changed since it was introduced into the 1983 ACI 318 code. In the ACI 318-05 code, the equation is in the form of:

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_t f_y\right) \tag{3.1}$$

where the coefficient α_c varies linearly between 3.0 and 2.0 for h_w/l_w between 1.5 and 2.0. In this equation, A_{cv} represents 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 (height-to-length) ratios between 1.5 and 2.0 accounts for the observed increase contribution of concrete in low-aspect ratio walls. The nominal shear strength for wall piers and spandrels cannot be taken larger than $0.83A_{cw}\sqrt{f'_c}(MPa)$, where A_{cw} represents 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 V_u exceeds $0.083A_{cv}\sqrt{f'_c}(MPa)$, and it is stated that the reinforcement spacing in each direction should not exceed 45 cm.

FEMA 356 requirements for wall shear strength state that the ACI 318 equations can be used to assess wall nominal shear strength if the transverse reinforcement ratio (ρ_n in FEMA 356, replacing ρ_t in ACI 318) 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 (1990), who found that wall shear strength was relatively insensitive to changes in ρ_n, particularly for low ratios of ρ_n.

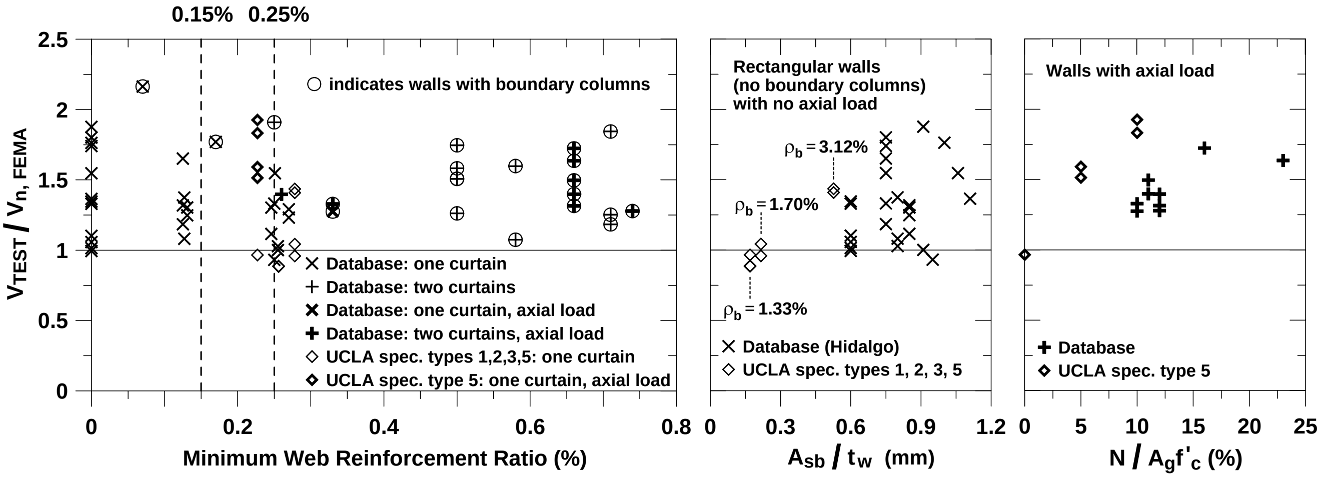
Specific requirements of ACI 318 also impact the evaluation process. For example, ACI 318 states: "*At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds* $(1/6)A_{cv}\sqrt{f'_c}(MPa)$". If strictly adhered to, this section implies that the wall shear strength cannot be taken greater the concrete shear strength for wall segments with a single curtain of reinforcement, and it has the unintended impact of limiting the wall nominal shear capacity by neglecting the contribution of reinforcement to shear strength. ACI 318 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, weakened plane joints are provided; and technically, reinforcement provided for shear strength is not continuous, that is, reinforcement is cut at the weakened plane joint to allow for crack initiation and control. The presence of a weakened plane joint may therefore induce a crack, which can widen enough to promote sliding between the crack faces, producing a shear-friction mode of failure. The nominal shear-friction capacity across a shear transfer (sliding) plane is specified in ACI 318-05 as $V_n = A_{vf} f_y \mu$ where μ is the coefficient of friction, and A_{vf} is the total area of reinforcement crossing the shear plane. ACI 318 permits a permanent net axial compression force across a shear plane to be taken as additive to the force in the shear-friction reinforcement, $A_{vf}f_y$. Also, the nominal shear-friction capacity calculation should be capped by the upper limits of $0.2f'_c A_c$ and $5.516\,A_c(\text{mm}^2)$.

3.2. Shear Strength Database from Prior Wall Tests

Prior to evaluation of current test results, a preliminary review of available research information was conducted to assess shear strength requirements for lightly reinforced wall segments with single and double curtains of web reinforcement. A database of relevant test results was assembled by reviewing available research including work summarized by Hirosawa (1975), and the papers by Hwang et al. (2001) , Hidalgo et al. (2002), and Wood (1990). Details of the database are presented elsewhere (Orakcal et al. (2008)). The maximum lateral load measured during each test in the database (V_{TEST}) was compared with the FEMA nominal shear strength ($V_{n,FEMA}$)

computed using the FEMA 356 provisions described in the preceding section. For cases where the longitudinal and transverse web reinforcement ratios are different, the shear strength computed using Eq. 3.1 is based on the minimum value of the web reinforcement ratio multiplied by the corresponding value for the reinforcement yield stress. This approach is consistent with common interpretations (e.g., Wood (1990)) of the aforementioned ACI 318-05 requirement that the longitudinal reinforcement ratio cannot be less than the transverse.

The $V_{TEST}/V_{n,FEMA}$ ratios for the walls in the database are plotted against the minimum web reinforcement ratios in Fig. 3(a). Ratios obtained for the UCLA specimens that failed under diagonal tension (specimen types 1, 2, 3, and 5), are also included. 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. Therefore, the results indicate that the FEMA 356 procedure provides a lower-bound estimate to the shear strength measurements achieved during these tests, regardless 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 for wall segments with a single curtain of reinforcement. For the tests in the database, ratios of measured lateral load capacity of the wall specimens with a single curtain of web reinforcement to the concrete nominal shear strength ($V_{TEST}/(1/6)A_{cv}\sqrt{f'_c}$) are between 2.2 and 5.2, with an average of 3.35 and a standard deviation of 0.79, 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.



(a) walls with varying minimum web reinforcement ratios

(b) walls with varying boundary reinforcement

(c) walls with varying axial load

Figure 3 Comparison of wall test data with FEMA 356 nominal shear strength calculation.

3.3. Current Test Results

The wall spandrel and pier specimens tested 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 Type-3 and Type-4 spandrels, and 0.23% for Type-5 piers. However, part of the longitudinal web reinforcement (4 out of 6 bars for Types 1 and 2, and 2 out of 4 bars for Types 3 and 4) are cut at the weakened plane joints. 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 Type-4 spandrels to 0.13%. Based on common interpretations of the ACI code, the shear strength computed using Eq. 3.1 should be based on the minimum value of the web reinforcement ratios, and considering that FEMA 356 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%. A reinforcement ratio of 0.15% was also used for the FEMA shear strength calculation of the pier specimens, since no hooks were provided on the transverse web reinforcement of the piers.

The average maximum lateral load measurement (average of positive and negative loading directions) for each test (V_{TEST}) were 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 (Types 1, 2, 3, and 5) are presented in Table 3.1. The measured-to-calculated shear-friction capacity comparisons ($V_{TEST}/V_{n,ACI\text{-}SF}$) for Type-4 specimens, which experienced shear-friction failure along the weakened plane joint located at the wall-pedestal interface, are also presented in the table. Overall average of the results presented in Table 3.1 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. For all of these specimens, the measured lateral load capacity significantly exceeds the nominal shear strength of concrete alone, which contradicts the implication that the nominal shear strength of wall segments with a single curtain of web reinforcement cannot be taken greater the concrete shear strength. This is also consistent with the results obtained for the wall test database.

Table 3.1 Comparison of test results with nominal shear strength calculations

Specimen ID No.	Type	Test No.	V_{TEST} (1) (kN)	$V_{n,FEMA}$ (2) (kN)	$V_{n,ACI\text{-}SF}$ (3) (kN)	$V_{TEST}/V_{n,FEMA}$	V_{TEST} (4) / $(1/6)A_{cv}\sqrt{f'_c}$	$V_{TEST}/V_{n,ACI\text{-}SF}$
WS-T1-S1	1	test1	633	441	887	**1.44**	**3.25**	**-**
WS-T1-S2		test4	749	531	959	**1.41**	**2.94**	**-**
WS-T2-S1	2	test2	453	473	556	**0.96**	**2.10**	**-**
WS-T2-S2		test3	491	471	556	**1.04**	**2.29**	**-**
WS-T3-S1	3	test11	398	449	381	**0.89**	**1.84**	**-**
WS-T3-S2		test14	406	459	381	**0.88**	**1.82**	**-**
WS-T4-S1	4	test12	330	450	381	**-**	**-**	**0.87**
WS-T4-S2		test13	341	456	381	**-**	**-**	**0.89**
WP-T5-N0-S1	5	test9	404	419	536	**0.97**	**2.13**	**-**
WP-T5-N5-S1	5	test7	648	428	1003	**1.51**	**3.31**	**-**
WP-T5-N5-S2		test8	682	428	1003	**1.59**	**3.47**	**-**
WP-T5-N10-S1	5	test5	753	411	1153	**1.83**	**4.08**	**-**
WP-T5-N10-S2		test6	819	425	1153	**1.93**	**4.21**	**-**
					Average	**1.31**	**2.86**	**0.88**
					Std. Dev	**0.38**	**0.87**	**0.02**

(1)

A closer look at the results reveals that the FEMA nominal shear strength calculation seems to have underestimated the shear strength of spandrel specimen Types 2 and 3, and of the pier (Type 5) 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 specimen Types 2, 3, and 5. This trend is apparent in Fig. 3(b), 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 specimen Types 1, 2, 3, and 5, as well as for rectangular wall specimens in the test database with no axial load and with amounts of 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 rectangular wall segments with boundary reinforcement ratios larger than 3%. For walls with boundary reinforcement ratios smaller than 3%, the FEMA nominal shear strength calculation may provide a slightly unconservative estimate of wall shear strength.

Type 2 and 3 spandrels 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° hooks are not provided on the transverse web reinforcement of Type-3 spandrels. However, average $V_{TEST}/V_{n,FEMA}$ values obtained for Type-2 and Type-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 (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. Hooks were also not provided on the transverse web reinforcement of the pier (Type 5) specimens, and longitudinal web reinforcement (ρ_l = 0.23%) was continuous over the specimen height, since 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 2 out of 4 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. 3(a) (for both the current specimens and the walls in the 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.

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 3.1). This is expected since 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, definitive conclusions can not be reached. However, the $V_{TEST}/V_{n,FEMA}$ ratios plotted in Fig. 3(c) against applied axial load levels for the Type-5 pier specimens, as well as for the walls in the test database with axial load, reveal that the FEMA 356 calculation tends to be conservative in estimating the nominal shear strength of walls piers subjected to even relatively low levels of axial load.

For the Type-4 spandrel specimens that failed in shear-friction across the weakened plane joint at the wall-pedestal interface, the ACI nominal shear-friction capacity calculation ($V_{n,ACI-SF}$) slightly overestimates the measured lateral load capacities. Weakened plane joints were also provided along the mid-height (when oriented vertically) of spandrel specimens of Types 1, 2, and 3. Types 3 and 4 were identical except for the location of the weakened plane joint. Type-3 specimens failed in diagonal shear (with diagonal cracks propagating across the WPJ with no significant deviation in crack path and direction), 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) was 362 kN. The ACI nominal shear-friction capacity of these specimens were calculated to be 381 kN, and their measured lateral load capacity was 335 kN 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 at the WPJ where the moment demand was maximum, and the initiation of flexural yielding 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 those expected to produce yielding of the boundary reinforcement.

4. SUMMARY AND CONCLUSIONS

An experimental program was conducted to assess shear strength requirements for lightly reinforced wall pier and spandrels commonly used in mid-1900's 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 and FEMA 356 provisions on wall nominal shear strength to evaluate the reliability or conservatism of these documents, pertaining to seismic evaluation and rehabilitation of existing buildings. The effect of outdated construction practices, including using a single curtain of web reinforcement, presence of weakened plane joints, and lack of hooks on transverse reinforcement on the shear strength of wall piers and spandrels also was investigated.

The findings of this study indicates that use of the FEMA 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, 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 (e.g., a boundary reinforcement ratio larger than 3% for wall rectangular wall segments). For rectangular wall spandrels with boundary reinforcement ratios (ρ_b) smaller than 3%, the FEMA nominal shear strength calculation 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%). Discontinuity of a portion of the longitudinal web reinforcement at a possible weakened plane joint at wall mid-height 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%).

The FEMA 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, since the influence of axial load on the shear strength of concrete is not considered in the FEMA nominal shear strength calculation; however, 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 retrofits that are not necessary.

Particular attention must be paid to the evaluation of the shear strength of wall segments with weakened plane joints (with part of the longitudinal web reinforcement discontinued), especially 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 weakened plane joints at the wall center where bending moments are low, since wall strength is limited by diagonal cracking versus sliding along the weakened plane joint.

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