Proposed Update to Masonry Provisions of ASCE/SEI 41: Seismic Evaluation and Retrofit of Existing Buildings

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SUMMARY

This paper summarizes significant updates to the Masonry Provisions now undergoing public balloting for the 2013 revision of the standard ASCE/SEI 41: Seismic Evaluation and Retrofit of Existing Buildings. These updates are informed by performance observations of unreinforced masonry (URM) buildings in the recent Christchurch (New Zealand) earthquakes and recent research, including component testing. New evaluation provisions include the classification of bed joint sliding and stair-stepped bed joint sliding as deformation-controlled modes of behavior. Provisions to evaluate diagonal tension strength are reintroduced from precursors to the standard. The provisions for rocking walls and piers are also substantially revised, to explicitly consider strength and stiffness degradation, and deformation capacities based on testing and other rational limit states. Guidance is also provided via commentary for the effects that flanges and spandrels have on in-plane wall or pier response. The wall anchorage provisions are proposed to be expanded to address chemical adhesive anchors in conjunction with installation quality assurance requirements. The performance criteria for out-of-plane wall actions are updated based on a review of currently available research. Improved modelling guidance is also provided via commentary to address the unique modeling and evaluation limitations that apply to URM buildings, including their resilience relative to other types of construction, as supported by performance observations from recent major earthquakes.

Keywords: ASCE 41, URM, masonry, evaluation, retrofit

1. INTRODUCTION: ASCE 41-13 UPDATE PROCESS

The American Society of Civil Engineers (ASCE) established a Committee with 96 members in 2009 to update two national standards: ASCE 31-03 "Seismic Evaluation of Existing Buildings" and ASCE 41-06 "Seismic Rehabilitation of Existing Buildings." The Committee agreed to merge the two standards and re-title it for the 2013 version. Issue Teams that function as ad hoc advisory subcommittees were established including a 22 member Masonry Issue Team that focused primarily on unreinforced masonry, but also developed minor changes to reinforced masonry provisions. Experts from both engineering practice and academia in the U.S., New Zealand, Canada, and Turkey collaborated by teleconference and email exchanges on the Masonry Issue Team. Six of the Masonry Issue Team were also voting members of the main ASCE Committee and the rest were Associate Members who were encouraged to participate in voting and their input was addressed with the same weight as voting members.

The Masonry Issue Team generated 10 ballot proposals by internally voting and refining the ballots. Those were, in turn, submitted to the full committee for its balloting and later merged into a public

ballot that will be under consideration and refinement over the summer of 2012. The balloting process relies on individual members of the Issue Team and Committee reviewing the work of others on the committee, voting "Yes", "Yes with Reservations", or "No." Then the Issue Team and Committee worked to reconsider and address those reservations and No votes that were accompanied by persuasive suggestions on how to improve the drafts.

The public ballot for ASCE 41-13 includes a significant rewrite of the masonry provisions compared to the two prior standards. It includes the work of the Issue Team that reviewed much of the masonry research generated prior to the earlier standards as well as more recent applicable research. In addition, the team considered the experiences from Issue Team members who evaluated and retrofitted masonry buildings and observed their performance in recent earthquakes.

As in the past, the Committee and Issue Team encountered many gaps in their understanding about the performance of masonry buildings, and in some cases promising research and performance observations that are beginning to fill those gaps. In addition the standards developers were aware of several problematic aspects within the earlier standards that were an outgrowth of prior balloting and inability to resolve such issues previously.

The ASCE 41 Committee and the Masonry Issue Team were also aware of the engineering practice that is still transitioning from prescriptive approaches to evaluations and retrofits toward more performance-based approaches. However, users will still find a mix of both approaches in ASCE 41-13 that still somewhat reflects the current practical limitations of performance-based engineering as well as continuing gaps in research, testing and awareness of the limitations of reliability in performance. Therefore, ASCE 41-13 will continue to draw from both an experimental basis and a range of approaches from simpler linear procedures to more complex nonlinear procedures.

2. DIAGONAL TENSION, FLANGE EFFECTS & SPANDRELS

2.1. Diagonal tension

Diagonal tension will be reintroduced to ASCE 41-13, and will be considered as a force-controlled mechanism. Turnšek & Sheppard (1980) developed an equation for determining the diagonal tension strength (V_{dt}) of URM wall panels subjected to in-plane shear forces, and this equation was incorporated into FEMA 306. FEMA 356 maintained this equation for determining V_{dt}, but with modified applicable L/h_{eff} limits. Turnšek & Sheppard (1980) and FEMA 306 capped L/h_{eff} at 1.0, but there was no upper limit on this ratio in FEMA 356, and consequently the expected V_{dt} could be increased according to the wall's aspect ratio. ASCE 41-06 does not include diagonal tension as a failure mechanism, but the subcommittee determined that diagonal tension is an observed failure mechanism in buildings when being subjected to earthquakes, and consequently this failure mechanism is proposed to be reintroduced into the 2013 provisions. The equation for determining V_{dt} in ASCE 41-13 (Equation 2.1, below) will be the same as the equation included in FEMA 306, and the upper limit of 1.0 on L/h_{eff} will also be maintained.

$$Q_{CL} = V_{dt} = f'_{dt}\, A_n \beta \sqrt{1 + \frac{fa}{f'dt}} \tag{2.1}$$

The subcommittee identified some difficulties associated with this equation, but determined that its inclusion with further guidance provided in the commentary section was a suitable approach. This determination was made on the basis that diagonal tension is a frequently observed failure mode, and users of ASCE 41 should be given guidance as to how to quantitatively account for this mode. The difficulties associated with the equation relate to the material properties to consider. The relevant part of the commentary to the provisions is repeated here:

"This equation was calibrated for the range of $0.67 \leq L/h_{eff} \leq 1.0$, and requires determination of

masonry diagonal tension strength, f'_{dt}. For walls with L/h_{eff} above or below the caps, using the capped values is recommended, however users should be aware that no substantiating research is available. In lieu of determining the diagonal tension strength, the lower-bound bed-joint shear strength, v_{mL}, as measured with the in-place shear test, may be substituted where it is assumed that the lower bound diagonal tension strength is equal to the lower-bound value of the bed-joint strength. However, this strength value only applies to the mortar, not the masonry units. Thus, there is considerable uncertainty in diagonal tension-strength estimates.

For conditions where axial stresses on walls or wall piers are relatively low and the mortar strengths are also low compared to the splitting strengths of the masonry units, diagonal tension actions may be judged not to occur prior to bed joint sliding. However, there is no available research to help determine a specific threshold of axial stress and relative brick and mortar strengths that differentiates whether cracking occurs through the units or through the mortar joints."

The diagonal compression test used to determine f'_{dt} is expensive, infrequently used in practice and the results may not be directly useful. Consequently the user is guided to substitute bed-joint shear strength for the diagonal tension strength, but this substitution assumes that cracking will occur through the mortar joints only. In buildings where the mortar strength is weaker than the brick strength, it is likely that cracking will propagate through the mortar joints in a stair-stepped pattern, but this is not always the case and the user of ASCE 41 is guided to use caution when making this substitution.

2.2. Flange effects and spandrels

The previous versions of ASCE 41 and its pre-standards (namely FEMA 307 and FEMA 356) did not include reference to flanges or spandrels, but recent research has indicated that flanges and spandrels can influence the performance of URM walls subjected to in-plane forces. Consequently, ASCE 41-13 will include reference to the effects of flanges and spandrels, although the currently available research is insufficiently 'mature' to provide definitive provisions to quantitatively consider their influence. It was identified through the ballot process for ASCE 41-13 that further research into quantitatively accounting for flange and spandrel effects will be beneficial.

The new provisions state that:

"The effects of wall flanges, spandrels and the vertical component of earthquake loading shall be considered when determining in-plane strength."

The relevant portion of the commentary referring to flange effects is repeated here:

"URM walls responding in-plane in an earthquake are often of non-rectangular section. Walls connected to, and oriented perpendicular to in-plane walls are termed "flanges", "return walls" or "transverse walls". Costley & Abrams (1996), Paquette & Bruneau (2003), Moon et al. (2006), Yi et al (2008) and Russell (2010) recognized through experimental research that flanges have the potential to influence the response of walls that resist lateral forces in-plane. Flanges can influence in-plane wall failure modes, maximum strengths and displacement capacities. Flanges can significantly increase sliding and rocking strength, but may only contribute to minor increases in diagonal tension strength. Flanges were found to increase the limiting drift of walls failing in diagonal tension.

Flanges are defined by Moon et al (2006) as the portions of the walls oriented out-of-plane that participate with the walls oriented in the plane of lateral loading. Yi et al. (2008) noted that previous experimental research on URM building systems (Costley, 1996; Paquette & Bruneau, 2003; Moon et al., 2006; Yi et al., 2006b) highlighted the beneficial effects of flanges on the response of in-plane loaded walls and indicated the potential for flanges to influence maximum strength and pier failure modes. Paquette found that wall flanges increase overall wall stiffness for low intensity ground motions compared to un-flanged walls, but the influence of flanges on stiffness becomes significantly

reduced after cracking in response to high intensity ground motions.

Russell et al. (2010) conducted further experimental analysis and also concluded that the effect of flanges on in-plane wall response can be significant. It was found that for URM walls with flanges, flexure is less likely as a controlling behaviour mode and shear is more likely to limit the lateral strength. It was also found that URM walls with flanges are able to sustain larger lateral forces than walls without flanges. Flanges were found to increase the displacement capacity of in-plane loaded walls when the flange is in compression, compared to similar walls without flanges. Moreover, a flange acting in tension increases the lateral strength of in-plane loaded walls."

The relevant part of the commentary referring to spandrels is repeated here:

"Wall spandrels that are stronger than piers can couple multiple piers and transmit overturning to adjacent piers, increasing axial forces in end piers and potentially changing their sequence of actions. Component tests on URM spandrels are available for reinforced concrete floor beams acting compositely with URM spandrels (Beyer et al 2011) as well as arched URM spandrels (Knox 2012). Several full-scale and partial-scale tests of walls with openings also provide insight into the influence of spandrels (Paquette 2003, Yi & Moon 2006). Estimates of spandrel strengths, while not confirmed by component tests, can be used to determine if spandrels are likely to be weaker or stronger compared to piers (FEMA 306, 1998).

The effects of global and component overturning and rocking of entire perforated walls depend on how effectively spandrels can transmit vertical shears and bending. Conversely, wall spandrels that are weak relative to adjacent piers may not provide fixity at the tops and bottoms of piers and may result in piers acting as cantilevers."

3. BED JOINT SLIDING

When assessing the in-plane performance of URM elements bed joint sliding (BJS) will be re-classified as a deformation controlled action in ASCE 41-13. Research undertaken by Abrams (1992), Bruneau & Paquette (2004), Magnenes & Calvi (1992), Franklin et. al (2001), Anthoine et. al (1995), Moon et. al (2006) and Russell & Ingham (2010) have confirmed that unreinforced masonry elements exhibiting bed-joint sliding behavior have significant deformation capacity past initial cracking. This is supported by FEMA 306 (1998), FEMA 307 (1998), and observations made by the Authors during the recent 2011 Christchurch Earthquakes.

Referring to Figure 3.1 below, Section 2.4.4.3 of ASCE 41-06 Supplement 1 requires that for components to be classified as deformation controlled the plastic range 'd' (i.e. from points 1 to 2) shall be greater than 2 x 'g' (i.e. from points 0 to 1). The results of this investigation are summarised in Table 3.1 below.

**Table 3.1.** Evaluation of available test results for BJS

Reference	Specimen	Loading	Observed Failure Mode	Check against ASCE 41-06 Supplement 1 Criteria				Comment
				g	d	d/g	Force or Deformation Controlled?	
Magenes & Calvi (1992)	MI1	Reversed Quasistatic	Diagonal tension and then stepped bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed
Magenes & Calvi (1992)	MI2	Reversed Quasistatic	Bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed
Magenes & Calvi (1992)	MI4	Reversed Quasistatic	Stair-stepped bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed
Abrams & Shah (1992)	W1	Reversed Quasistatic	Bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed
Magenes & Calvi (1995)	5	Shake Table	Flexural cracking then horizontal and stepped bed joint sliding.					Stable response but no drift data given
Magenes & Calvi (1992)	MI2	Reversed Quasistatic	Horizontal bed joint sliding at top course then stair stepped bed joint sliding	0.1	0.7	7.0	Deformation-controlled	0.1% drift at "g" assumed

Reference	Specimen	Loading	Observed Failure Mode	Check against ASCE 41-06 Supplement 1 Criteria				Comment
				g	d	d/g	Force or Deformation Controlled?	
Russell (2010)	A6	Reversed Quasistatic	Diagonal tension and then stepped bed joint sliding	0.1	0.7	7.0	Deformation-controlled	Drift at "g" conservatively rounded-up to nearest 0.1%
Russell (2010)	A7	Reversed Quasistatic	Diagonal tension and then stepped bed joint sliding	0.1	0.4	4.0	Deformation-controlled	Drift at "g" conservatively rounded-up to nearest 0.1%
Russell (2010)	A8	Reversed Quasistatic	Diagonal tension and then stepped bed joint sliding	0.1	0.4	4.0	Deformation-controlled	Drift at "g" conservatively rounded-up to nearest 0.1%
Manzouri et al (1995)	W1	Reversed Quasistatic	Flexural cracking, toe crushing and then bed joint sliding	0.1	0.6	6.0	Deformation-controlled	0.1% drift at "g" assumed
Manzouri et al (1995)	W2	Reversed Quasistatic	Flexural cracking, toe crushing, diagonal cracking and then bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed
Manzouri et al (1995)	W3	Reversed Quasistatic	Flexural cracking, toe crushing and then bed joint sliding	0.1	0.4	4.0	Deformation-controlled	0.1% drift at "g" assumed

Referring to Table 3.1 it can be observed that the URM walls tested to date satisfy the requirements of Section 2.4.4.3 ASCE 41-06 Supplement 1 for deformation controlled elements i.e. $d \geq 2g$.

ASCE 41-13 expected bed joint sliding strengths are calculated in accordance with the procedure detailed in FEMA 306 (1998). Equations for both the initial (un-cracked) and final (frictional) capacity will be included in the 2013 revision (these will be termed V_{bjs1} and V_{bjs2} respectively).

For linear analysis procedures the m-factors previously used in FEMA 356 will be reinstated. A review of research undertaken since the publication of FEMA 356 (Bruneau & Paquette, 2004, Moon et. al. 2006, Russell & Ingham, 2010) did not support changing the m-factors previously specified. The original FEMA 356 acceptance criteria have been modified such that the final capacity (V_{bjs2}) is used when assessing component behavior at the LS and CP performance levels. V_{bjs1} is retained for the IO performance levels. The use of V_{bjs1} at the IO performance level and V_{bjs2} at the LS and CP performance levels reflects the level of structural deterioration that is expected to occur at each performance level.
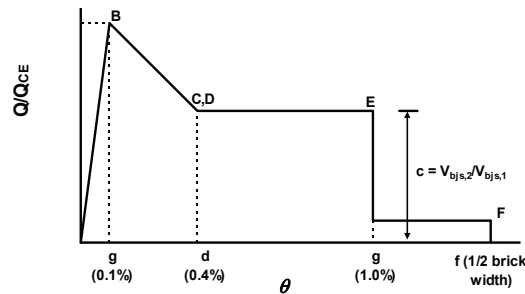
Figure 3.1. Non-linear analysis modelling parameters for URM in-plane walls and wall piers – BJS

For non-linear analysis procedures the proposed modeling parameters and primary component acceptance criteria are summarized in Figure 3.1 above. The modeling parameters and non-linear acceptance criteria illustrated in the figure are the same as those detailed previously in FEMA 356 except that:

1. Secondary LS and CP limits have been extended to 0.75% and 1.0% respectively.
2. A negative slope has been introduced between points B and C reflecting the degradation of strengths observed in some tests.
3. Residual strength ratio c is calculated explicitly from V_{bjs1} and V_{bjs2}.

The increase in the secondary limits reflects the observed good performance of URM walls failing in the BJS mode with drifts in excess of 1% observed in multiple tests (refer Table 3.1). A review of research undertaken since the publication of FEMA 356 did not support changing the existing FEMA 356 IO or primary LS and CP limits.
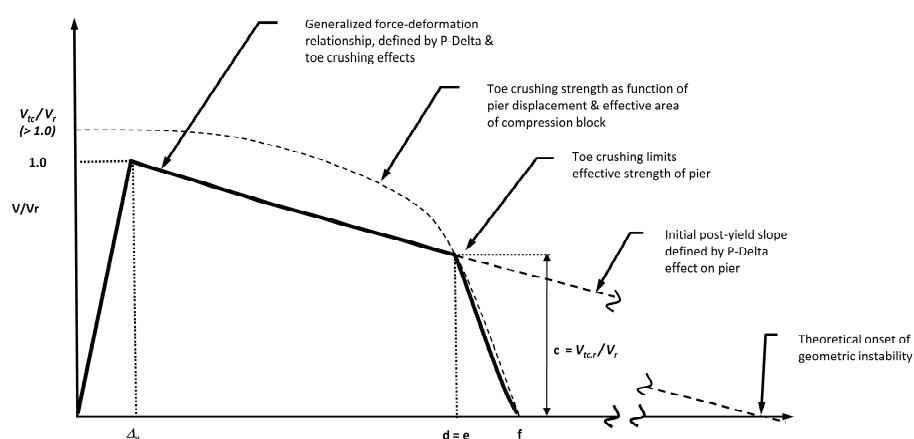
4. ROCKING

Rocking of unreinforced masonry in-plane walls and piers is classified as a deformation-controlled response by chapter 7 of ASCE 41-06. It is proposed to retain this classification in the 2013 edition of ASCE 41; however, significant revisions to the modeling and acceptance criteria are also proposed based on a review of previously undertaken testing and other rational limit states.

Moment-curvature analysis can be used to estimate the drift or rotation at which latent crushing of the compression toe will occur as a function of the superimposed gravity load, pier geometry and masonry or mortar compressive strength. This characteristic is also documented by the component testing undertaken by Moon et al (2006). Moment-curvature analyses were undertaken for the limited tests with sufficient published details to verify that this approach could conservatively estimate the deformations associated with the onset of toe crushing as summarized in Table 4.3, below.

The proposed Chapter 11 of ASCE 41-13 introduces latent onset toe crushing as an integral part of the modeling and acceptance criteria, in addition to P-delta stability, effective stiffness and elastic unloading characteristics for rocking walls and piers.

Figure 4.1 summarizes the generalized force-deformation curve which incorporates the modeling considerations outlined above.

Figure 4.1. Generalized force-deformation curve for rocking in-plane walls and wall piers

Upper-bound limits on drift have been added based on test results of individual URM piers that had rocking as primary modes of response (Moon et al 2006, Yi et al 2004, Franklin et al 2001, Anthione 1995, Magnes & Calvi 1995, Costley & Abrams 1996, Xu & Abrams 1992, Paquette & Bruneau 2003). The absolute drift limits are in addition to the limits derived as a function of the wall or pier geometry (height to thickness ratio), per ASCE 41-06. These test results indicate that for URM walls governed by an initial rocking response, drifts of at least 1.5% are achievable for certain configurations of aspect ratio and axial load with nominal strength degradation provided that toe crushing is not found to control at lower drifts. For drifts greater than 1.5%, out-of-plane effects (e.g. twisting of piers at their base) can also influence wall performance. This drift was used as the delineation between Primary and Secondary acceptance criteria. Users of the Standard are cautioned as to the increased fragility of rocking walls and piers subjected to the Secondary drift criteria, which are consequently only recommended for use with piers with a minimum thickness of 12 inches (300 mm) to minimize the risk of bearing loss due to out-of-plane effects. Based on the limited dataset of relevant rocking wall tests, an absolute upper-bound secondary CP drift limit of 2.5% is proposed.

4.1 Linear procedures

The drift limitations described above are directly applied to the nonlinear procedures but need to be

converted to m-factors for use with linear analysis procedures. In accordance with the Analysis Provisions of the Standard, the resultant m-factor limits are based on 0.75 times the ratios of maximum tested drift to yield drift and also as a function of pier aspect ratio and axial stress. As drifts and deformations are not explicitly evaluated for ASCE 41 linear procedures, the m-factors are a proxy for limiting allowable drifts of rocking piers. The proposed m-factors have been reduced from the 2006 values and capped to correlate with test results.

The proposed modeling and acceptance criteria for rocking walls and piers are summarized in Tables 4.1 and 4.2. Table 4.3 summarizes the test results used to develop the proposed rocking revisions.

Table 4.1. Proposed acceptance criteria for rocking walls and wall piers, linear procedures

Performance Level				
Primary			Secondary	
IO	LS	CP	LS	CP
$1 \leq 1.5 h_{eff}/L \leq 1.5$	$1.5 \leq 3 h_{eff}/L \leq 3.75$	$2 \leq 4 h_{eff}/L \leq 5$	$2 \leq 4 h_{eff}/L \leq 5$	$3 \leq 6 h_{eff}/L \leq 8$

Table 4.2. Proposed modelling and acceptance criteria for rocking walls and wall piers, nonlinear procedures

				Acceptance Criteria				
				Performance Level				
				Primary			Secondary	
c	d (%)	e (%)	f (%)	IO (%)	LS (%)	CP (%)	LS (%)	CP (%)
$V_{tc.r}/V_r$	$100\Delta_{tc.r}/h_{eff}$	$100\Delta_{tc.r}/h_{eff}$	$100(\Delta_{tc.r}+\Delta_y)/h_{eff}$	0.1	$0.3 h_{eff}/l \leq 1.12\%$	$0.4 h_{eff}/l \leq 1.50\%$	$0.6 h_{eff}/l \leq 2.25\%$	$100\Delta_{tc.c}/h_{eff} \leq 2.5\%$

Table 4.3. Evaluation of available test results for rocking piers

| Test	Specimen	Predicted 1st yield per ASCE 41 Rocking (R), Toe Crushing (TC), BJS (SL)	Max test drift (%)	Yield drift (%)	Tested yield sequence 1	Tested yield sequence 2	Test notes	Masonry Issue Team Comment
Yi et al, 2004	W-2.7-L1-a	R	1.93%	0.10%	R			
	W-2.7-L2-a	R	1.63%	0.20%	R	TC		Moment-curvature analysis conservatively predicts toe crushing at 1.3% drift.
	W-1.2-L2-a	R	2.97%	0.20%	R			
	W-1.8-L2-a	R	2.00%	0.20%	R	TC		Moment-curvature analysis predicts toe crushing at 2.0% drift
Franklin, Lynch, Abrams, 2001	1F	R	2.00%	0.20%	R		At 2% drift pier began to rotate about vertical axis	
	2F	R	2.50%	0.10%	R			
	6F	TC	2.50%	0.30%	R			Per ASCE 41, pier would conservatively be assumed as force-controlled due to prediction of toe crushing mechanism
Anthione, 1995	High Wall 1st run	TC	0.60%	0.30%	R			See comment above
	2nd run	TC	0.75%	0.38%	R	SL		See comment above
Magenes, Calvi, 1995	3 Runs 7-12	TC	1.3%	-	R			See comment above
Costley & Abrams, 1996	S1 Door Wall Outer	R	1.3%	-	R			
	S1 Inner	R	1.3%	-	R			
	S2 Outer	R	1.1%	-	R			
	S2 Inner	R	1.1%	-	R			
Xu, Abrams, 1992	Fig 3.2	R	2.0%	0.30%	R			
Paquette, Bruneau, 2003	Door	R	0.9%	-			Spandrel uplifted, resulting in loss of stability & test terminated	Spandrel uplift must be checked as part of proposed revisions to in-plane rocking provisions (see C11.3.2.2.1). See comment for S2 Outer & S2 Inner (Costley & Abrams, 1996)
	Window	R	1.8%	-	R			
	Central	R	1.8%	-	R			

5. URM WALL ANCHORS

Committee and Issue Team members have observed the performance of retrofit anchors in URM in recent earthquakes and some have conducted testing programs to gain a better understanding of their performance. In particular, the Committee noted the critical importance of testing and quality

assurance for anchors as means to achieve reliable performance. As a result, the Committee has proposed to expand provisions in regards to adhesive anchors as well as require testing and quality assurance plans for anchors. Minimum bed joint shear strengths for the masonry as well as a minimum percentage of anchors to be tested are now included in the provisions. While the Standard will now include somewhat prescriptive provisions, users are also permitted to develop their own project-specific quality assurance and testing program that is not necessarily required to follow each suggested prescription in order to meet the evaluation and retrofit performance objectives.


6. URM OUT-OF-PLANE ACTIONS

The Masonry Issue Team also explored available research regarding acceptable height to thickness slenderness ratios for brick unreinforced masonry walls.

Sharif (2007) used principles of rocking mechanics and concluded that ABK-based h/t ratios in ASCE 41 may eventually be found conservative but it expressed caution that this was a preliminary conclusion since "not all the significant variables influencing the performance of out-of-plane walls were captured". In contrast, Derakhshan (2011) used ground motion time-history analyses and suggested that the ABK-based h/t ratios in ASCE 41 are likely to be un-conservative particularly for walls of one-storey buildings and upper walls in two-storey buildings, for sites with higher SD1 values, and for sites nearest active earthquake sources that can cause long pulse ground motions. Derakhshan (2011) also suggested that wall thickness should be considered in addition to wall slenderness ratio in order to assess out-of-plane wall dynamic stability. Ingham and Derakhshan (2012) utilized a spectral procedure based on the research by Griffith et al. (2003) and proposed a generic method for URM walls out-of-plane assessment. The results suggested that the ASCE 41 provisions are un-conservative. However, research is not yet available to account for the scatter of performance. Additional research on the response of URM walls to out-of-plane displacements considering vertical load eccentricities and in-plane damage is underway at the University of British Columbia (Vancouver, Canada).

(a) Shallow soil　　　　　　　　　　(b) Deep soil
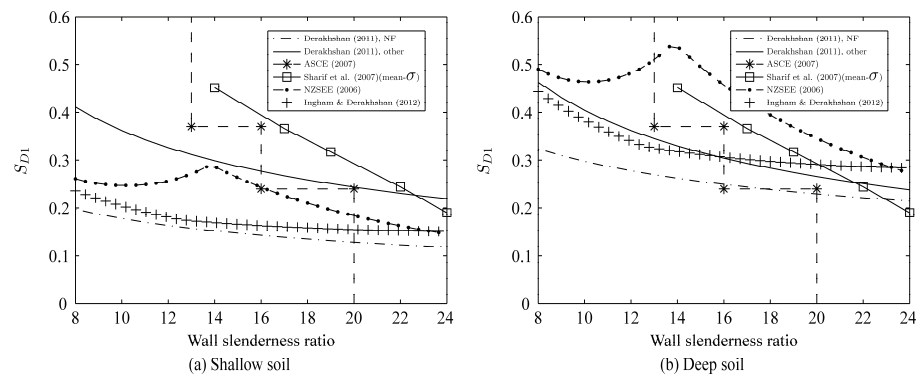
Figure 6.1. One-story URM wall out-of-plane acceptable h/t ratios suggested by different guides/studies

(a) Shallow soil　　　　　　　　　　(b) Deep soil
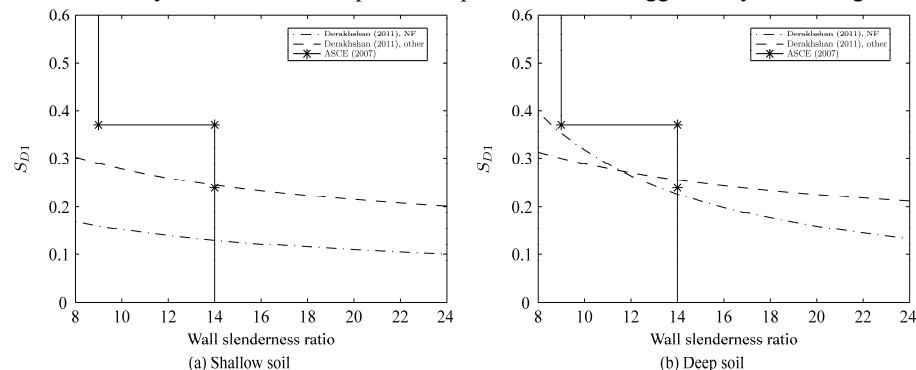
Figure 6.2. Top-story URM wall out-of-plane acceptable h/t ratios suggested by different guides/studies

Performance of retrofitted URM buildings in recent earthquakes (1989 Loma Prieta, 1994 Northridge, 2003 San Simeon, 2010 Baja, 2011 Christchurch) continue to demonstrate trends of out-of-plane failures in retrofitted buildings for ground motions with relatively short duration and moderate intensity. However, performance in long duration earthquakes and ground motions in excess of those anticipated when selecting performance objectives remain a matter of considerable conjecture. The margin against collapse is also a matter of debate, so advice on how users of ASCE 41-13 might make decisions given our collective limitation of knowledge is included in new commentary.

At this juncture, there is not a clear basis to warrant alterations to the height to thickness ratios prescribed in ASCE 41-13. However, they are proposed to be characterized for the time being as Collapse Prevention performance rather than Life Safety, as the margin against collapse is not yet rigorously determined for a wide range of wall configurations and strengths.

7. CONCLUSIONS

The proposed 2013 revision to the Masonry chapter of ASCE 41 includes a number of significant updates and changes to the provisions and guiding commentary. These revisions will assist users of the standard with making better informed decisions and recommendations with regard to the evaluation and retrofit of unreinforced masonry buildings. In particular, users of the standard should acknowledge the relative vulnerability of unreinforced masonry buildings compared to other construction types and the current limitations of the provisions and research into various aspects of their seismic performance.

A number of gaps exist in the current understanding of URM performance, some of which have been highlighted by the Masonry Issue Team and it is hoped that this will help spur further research which can inform future versions of ASCE 41 and other retrofit standards. In particular, the following issues warrant further research:

Quantitative differentiation between diagonal tension cracking though masonry units and through the masonry joints;

- Improved modeling and acceptance criteria guidance for spandrels and wall flanges.
- Effect of combined in-plane and out-of-plane actions on the performance of rocking walls and wall piers.
- Performance of both retrofitted and un-retrofitted unreinforced masonry buildings under repeated strong ground motion events; and in particular, the performance of wall anchors.
- A more comprehensive and quantitative understanding of the margins against collapse for out-of-plane wall actions, particularly with respect to the Life Safety and Collapse Prevention performance criteria of ASCE 41.

ASCE encourages others who might have an interest or specific expertise in masonry to plan to participate in this process in the future.

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