A Summary of Significant Updates in ASCE 41-17

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Abstract

ASCE/SEI 41 is the standard for seismic retrofit and evaluation of existing buildings, required for all federal buildings, as well as several recently passed California ordinances. This presentation provides an overview of the recent updates to the standard in the upcoming ASCE/SEI 41-17. Significant revisions were included for the standard's Basic Performance Objectives, seismic hazard used in Tier 1 and Tier 2, treatment of force-controlled components, nonlinear analysis provisions, non-structural performance levels, demands on out-of-plane wall forces, modeling parameters and acceptance criteria of steel and concrete columns, and anchor testing. The updates will have significant impacts on the evaluation and retrofit approach for a variety of existing buildings. This paper provides a high-level summary of the changes most likely to impact practice.

Introduction

ASCE 41-13 was a major advancement in the practice of seismic evaluation and retrofit. It combined the evaluation and retrofit standard, ASCE 31-03 and ASCE 41-06, in to one standard to eliminate inconsistencies between evaluation and retrofit and made significant technical changes to both standards (Pekelnicky & Poland, 2012). An update to the ASCE 41 standard was just completed. The update includes significant changes to the Basic Performance Objective for Existing Buildings, the linear and nonlinear analysis procedures, and the material specific provisions. This paper discusses the most significant changes to the standard.

At the beginning of the standard's update cycle, the committee discussed a number of issues that could be potential updates. A very significant concern was raised by some committee members who practiced in the Midwest related to the seismic hazard level used for the Basic Performance for Existing Buildings (BPOE). These members expressed a primary concern around the change from ASCE 31-03, which used 2/3rds of the ASCE 7 Maximum Considered Earthquake as the hazard, to ASCE 41-13, which used an earthquake with a 20% probability of exceedance in 50 years. The committee members felt the change reduced the seismic hazard intensity used for Tier 1 and Tier 2 too much. In the most extreme case, the forces used in evaluation were one-seventh (1/7) that of ASCE 31-03. ASCE 31-03 did have more generous m-factors than ASCE 41-13, because the "break" for existing buildings was accomplished by increasing the commensurate m-factors in ASCE 41-06 by about 1.3 (the reciprocal of the historic 0.75 factor applied to the base shear for assessing existing buildings). However, the resulting evaluation using ASCE 41-13 resulted in such a discrepancy from ASCE 31-03 that committee members were concerned that hazardous buildings might be given a pass. The committee spent considerable time on this issue and proposed several changes to the BPOE.

As the ASCE 41 update cycle was beginning, the ASCE 7-16 update cycle was underway. As part of that cycle, significant changes to the ground motion parameters, site factors, and nonlinear response history analysis procedure had been proposed and approved. The committee felt that the method of determining seismic hazard parameters and site factors should not be different between standards, so it chose to simply reference ASCE 7 for that material instead of reciting it.

Because the nonlinear response history analysis procedure is a significant component of the ASCE 41 standard, there was a lot of deliberation about incorporating the changes made to that procedure in ASCE 7 into ASCE 41.

As with past cycles, there were many items identified where the standard was conservative, and recent research indicated that provisions could be changed to reduce some of that conservatism. This was especially true with steel columns. There were instances where the standard was potentially unconservative or did not provide sufficient guidance, specifically with the provisions for unreinforced masonry and masonry infill buildings. A significant addition to the standard was the creation of separate provisions for cold-formed steel light frame construction. More updates were proposed to better align the unreinforced masonry and masonry infill provisions with recent research.

BPOE Changes

ASCE 41-13 introduced the Basic Performance Objective for Existing Buildings (BPOE). The intent of the BPOE was to represent the reduced performance level for an existing building compared to that of a new building – a concept which had historically been deemed acceptable. The performance objectives in the ASCE 31-03 standard had been based on this concept and is why ASCE 31-03 was generally less conservative than ASCE 41-06 for the same performance objective (i.e. Life Safety in the 2/3*MCE). The most significant change that the BPOE made from ASCE 31-03 was in the seismic hazard used for a Tier 1 screening and Tier 2 evaluation. Instead of using 2/3*MCE with higher m-factors, ASCE 41-13 chose to specify a lower seismic hazard intensity and use the same m-factors and analysis procedure as ASCE 41-06, with appropriate updates. The committee chose the 20% probability of exceedance in 50 years shaking intensity as the BSE-1E hazard intensity to use for Tier 1 and Tier 2. The update also provided a reduced hazard comparable to the MCE, the 5% probability of exceedance in 50 years shaking intensity, which would be checked as a second performance objective in a Tier 3 evaluation. Table 1 below identifies the consequential hazards and related performance objectives corresponding to BPOE from ASCE 41-13.

Table 1 – ASCE 41-13 Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1 & 2	Tier 3	
Risk Category	BSE-1E	BSE-1E	BSE-2E
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

In California, this change generally resulted in similar demands on components. For example, take the case of a three story reinforced concrete shear wall building governed by shear in the walls located in Los Angeles. 2/3*MCE = 1.44 and the 20%/50-yr is 0.84. The ASCE 31-03 m-factor for shear controlled walls is 2.5 and C = 1.0. Therefore the wall would be evaluated for an equivalent base shear in ASCE 31-03 of:

$V/m = 1.0*1.44W/2.5 = 0.58W$

The ASCE 41-13 m-factor is 2.0 and $C_mC_1C_2 = 0.8*1.4 = 1.1$. In ASCE 41-13, the wall would be evaluated for an equivalent base shear of:

$V/m = 1.1*0.84W/2.0 = 0.46W$.

In this case the ASCE 41-13 demand is about 80% of the ASCE 31-03 demand.

However, consider the same building in Memphis, TN, where 2/3*MCE = 0.93 and the 20%/50-yr = 0.13. In that case, the ASCE 31-03 equivalent base shear is:

$V/m = 1.0*0.93W/2.5 = 0.37W$

And the ASCE 41-13 equivalent base shear is:

$V/m = 1.1*0.13W/2.0 = 0.07W$

In this case, the ASCE 41-13 shear demand in the wall is one-fifth of what was used in ASCE 31-03. As engineers in that region and other regions outside of California began to use the ASCE 41-13 standard, a growing consensus arose which considered the reduction with respect to ASCE 31-03 to be too extreme.

The concern about the low reduction was not simply based on the discrepancy between ASCE 31-03 and ASCE 41-13. If a Tier 3 evaluation were required, the same building would also have to be evaluated for Collapse Prevention in the BSE-2E. If one were to take the LA building and Memphis buildings and calculate the equivalent base shear to evaluate the walls it would be:

Los Angeles

$V/m = 1.1*1.76W/3 = 0.65W$

Memphis

$V/m = 1.1*0.71W/3.0 = 0.26W$

In the case of Los Angeles, Collapse Prevention in the BSE-2E yields a slightly more conservative demand on the walls than ASCE 31-03. In Memphis, the demand is still less than ASCE 31-03, but the discrepancy is a 30% reduction as opposed to an 80% reduction. When comparing the demand from Life Safety in the BSE-1E to Collapse Prevention in the BSE-2E, it is clear that in Memphis there is a very significant difference. Which brings up the question of whether ASCE 41-13's Tier 1 and Tier 2 approach of deeming the performance in the BSE-2E to be met by demonstrating performance in the BSE-1E.

The reason for these discrepancies come from the nature of the seismic hazard at the different sites. In the case of Los Angeles, the hazard is characterized by the possibility of extreme earthquakes and also other major earthquakes. While Memphis' hazard is based on an extreme event, but no other appreciable seismic sources. Therefore, both sites will have large hazard intensity parameters for the MCE and the BSE-2E, but the hazard intensity parameters for the BSE-1E (20%/50 yr) are quite different. The lack of any moderate seismic sources around Memphis leads to a very small BSE-1E intensity relative to the BSE-2E and MCE.

After considerable deliberation, the committee felt that the best way to solve this issue was to change the BPOE for Tier 1 and Tier 2 so it required explicit evaluation of the performance objective based on the BSE-2E hazard and allowed the performance objective in the BSE-1E hazard to be deemed compliant for Risk Category I, II, and III buildings.

There is a reason Risk Category IV buildings are not in the list above to only consider performance at the BSE-2E for Tier 1 and Tier 2. That is because the committee felt that the difference between the Immediate Occupancy and Life Safety performance levels was large enough that one could not be assured that meeting Life Safety in the BSE-2E would demonstrate meeting Immediate Occupancy in the BSE-1E. As such, in ASCE 41-17, a Tier 1 and Tier 2 evaluation of a Risk Category IV building for the BPOE requires looking at two performance objectives. Table 2 summarizes the updated BPOE per ASCE 41-17.

Hazards Reduced Nonstructural

The discussion of structural performance in the BSE-1E versus the BSE-2E, moved to a discussion about nonstructural performance. Nonstructural hazards have not been evaluated at shaking intensities greater than the design earthquake, but for new design, the design earthquake is coupled with the MCE as opposed to being a separately defined hazard. Therefore, there is likely some margin of safety in the anchorage of nonstructural components if they experience a greater–than-design-level earthquake shaking. Concerns were raised that major nonstructural hazards could be ignored if the BSE-1E was very low, but the BSE-2E was significant.

The committee identified a small subset on nonstructural components whose failure represented as much a risk to the building occupants as a partial or total collapse of a building would. It was felt that such hazards should have a significant margin of safety beyond the BSE-1E hazard. The committee did not feel that such margin was warranted for falling hazards that pose a limited risk of death or injury to an isolated individual or would simply relate to property damage. With that philosophy, the committee chose to create a new nonstructural performance level, Hazards Reduced, which would encompass mitigating only the most egregious nonstructural hazards. ASCE 41-06 and its predecessor FEMA documents had a Hazards Reduced nonstructural performance level that attempted to accomplish a similar objective.

Items which were incorporated into the Hazards Reduced nonstructural performance level are:

- Release of hazardous materials
- Failure of heavy cladding over sidewalks where many people congregate
- Failure of heavy ceilings in assembly spaces
- Failure of large architectural appendages and marquees
- Failure of heavy interior partitions and veneers

There is a note that permits components identified above to be excluded from the Hazards Reduced nonstructural performance level if it can be demonstrated that the component does not pose a threat of serious injury to many people due to falling or failing under the Seismic Hazard Level being considered.

Recognizing that an explicit evaluation of nonstructural components at the BSE-2E hazard level could result in demands greater than required for a new building, there is a statement capping the evaluation and retrofit requirements for any nonstructural components to be no greater than what is required in Chapter 13 of ASCE 7-16.

Table 2 – ASCE 41-13 Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1 & 2		Tier 3	
Risk Category	BSE-1E	BSE-2E	BSE-1E	BSE-2E
I & II	Structural Performance Not Evaluated Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance (5-D)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance (5-D)
III	Structural Performance Not Evaluated Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance (4-D)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance (3-D)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance (3-D)

Tier 1 & Tier 2 Structural Provisions

The most significant changes to the Tier 1 Screening provisions and the Tier 2 Evaluation provisions were made in response to the change in the BPOE, requiring evaluation at the BSE-2E. In order to evaluate structural performance at the BSE-2E, the checklists and quick check procedures had to be revised to accommodate screening for the Collapse Prevention and Limited Safety structural performance levels, in addition to Life Safety. In researching the checklist development, the committee felt that all the items identified in the Life Safety structural checklists were there because they affect the collapse probability of the building. Therefore, the structural checklists could be retitled as Collapse Prevention with little change.

In updating the checklist from Life Safety to Collapse Prevention, the quick check equations were reviewed. Most of the quick check equations allow for a simplified and conservative assessment of structural element capacity against an estimated demand. That estimated demand is arrived at by dividing the unreduced Psuedo-lateral force, calculated as CS_aW, by a global response modification factor, M_s. The M_s factors were not changed between ASCE 31-03 and ASCE 41-13, even though the demand was reduced from 2/3*MCE to the BSE-1E. To maintain parity, the M_s factors should have been reduced to account for the reduced performance "break" being moved from the capacity side to the demand side. In adjusting the M_s factors for Collapse Prevention, the committee elected to fix this omission by reducing the m-factors by 75% – the historic "break" for existing buildings – and then increased the M_s factors by a factor of 1.5 to translate them from Life Safety to Collapse Prevention. The 1.5 factor was based on the judgement of the committee. Section 7.6 states that the ratio of Life Safety to Collapse Prevention component m-factors is 0.75. However, the committee felt that for system-based M_s factors, the ratio could be slightly larger. The ratio of BSE-2E to BSE-1E shaking intensity parameters for much of the western US showed a range of 1.5 to 2.5. That information coupled with the view that global system behavior may have a slightly larger spread between Life Safety and Collapse Prevention, led the committee to choose 1.5 as the scale factor. The Limited Safety M_s factors are to be interpolated between the Life Safety and Collapse Prevention M_s factors.

The only major change to the benchmark building table was the elimination of the URM special procedure, including the IEBC Appendix A1, UCBC, and GRSB, as a benchmark standard. This was done because the change in the BPOE indicates that a building designed or retrofit to a benchmark standard should meet Collapse Prevention in the BSE-2E hazard. The committee consensus was that the URM special procedure provided Collapse Prevention performance at the hazard used to apply the procedure. In most cases that would be a shaking intensity of 50% to 100% of the new building design hazard, the BSE-1N, which is lower than the BSE-2E.

There were some changes to specific statements in the Tier 1 structural checklists. Typically, the changes were to clarify the intent of the statement, eliminate conservatism, or further separate the requirements for Immediate Occupancy from Collapse Prevention. Checklist for CFS light frame buildings were created. Changes to the nonstructural checklists are discussed in the nonstructural section of this paper.

The majority of the changes to the Tier 2 procedure were in clarifying the appropriate level of analysis required and what needs to be evaluated based on the checklist statement that is found noncompliant. The bigger change to the Tier 2 procedure comes from the BPOE change, which now requires explicit consideration of the BSE-2E hazard performance objective.

Linear Analysis

The only significant change to the linear analysis procedure related to the treatment of force-controlled actions. In ASCE 41-13 and previous editions, there was no difference in how force-controlled actions were evaluated between various performance levels. All force-controlled actions were evaluated through a capacity-based design approach or using the following equation:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J}$$

The J-factor was either the lowest demand-to-capacity ratio in the load path or a value between 1 and 2 based on the level of seismicity at the site.

Not adjusting the force-controlled evaluation for performance levels creates a situation that is in conflict with the definition of Life Safety performance:

> *"Structural Performance Level S-3, Life Safety, is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin against the onset of partial or total collapse. A structure in compliance with the acceptance criteria specified in this standard for this Structural Performance Level is expected to achieve this state.*

The issue relates to the concept of providing a margin against collapse. Consider the force-displacement curve shown in Figure 1. Here you have a force-controlled element which has a demand that is within 5% of its capacity. If the demand had been 10% greater, the element would have failed.

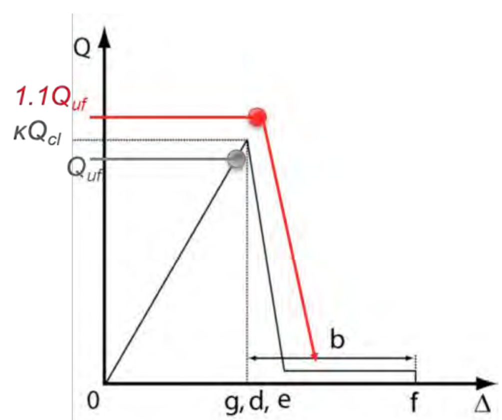

Figure 1. Force-controlled action example.

The committee decided that in order to provide a margin of safety against collapse which is called for in the definition of Life Safety, there should be some margin against failure of a force-controlled action built in to the provisions. To accomplish this, the equation to evaluate force-controlled actions was changes to be:

$$Q_{UF} = Q_G \pm \frac{Q_E \chi}{C_1 C_2 J}$$

The χ factor is 1.3 for Life Safety and high performance levels and 1.0 for the Collapse Prevention performance level. This provide the same margin between Life Safety and Collapse Prevention as Section 7.6 stipulates be provided for deformation controlled actions. The committee did not feel that any further increase beyond 1.3 for higher performance levels than Life Safety was justified.

It is important to note that the χ factor only applies when the demand is calculated using the pseudo-lateral force, Q_e, and not when the demand is calculated based on a capacity-based design. If the shear demand in a concrete column is based on the formation of a plastic moment at each end, then no χ factor amplification is required. However, if the demand is calculated based on the force reported from the analysis model divided by C_1C_2 and a J factor equal to the lesser DCR of the column bending moments or 2, then the χ factor would apply.

The other major update to the linear analysis provisions related to the design forces for walls subjected to out-of-plane forces and their anchorage to floor and roof diaphragms. An issue was identified which had the ASCE 41-13 provisions providing higher design forces for walls and their anchors when evaluating Collapse Prevention at the BSE-2N (ASCE 7 MCE$_R$) than would be required per ASCE 7. This was not intended. The goal of the ASCE 41 provisions is that they align with ASCE 7 provisions for the Basic Performance Objective for New Buildings (BPON). The reason for this misalignment was due to out-of-plane wall and anchorage equations being calibrated to performance objective the BSE-1N, but not the BSE-2N. This discrepancy was corrected in ASCE 41-17.

Nonlinear Analysis

The nonlinear dynamic procedure within ASCE 41 is a key component of the standard, and its provisions are frequently used to fill gaps in ASCE 7 for new design using nonlinear response history analysis. The 2015 NEHRP Provisions update included a complete re-write of the nonlinear response history analysis provisions found in ASCE 7. Those updates were then passed on to the ASCE 7 committee, which further refined them for incorporation into ASCE 7-16. It is common to have a nonlinear response history analysis show significant variation in building performance based on the ground motions records, Figure 2. The goal of the updates was to provide provisions targeted to the 10% probability of collapse in the MCE$_R$ objective of ASCE 7. A detailed discussion of the updates can be found in our papers in *Earthquake Spectra* Vol. 32, No. 2 (Haselton et. al., 2017a&b, Jarret et. al, 2017, and Zimmerman et. al., 2017).

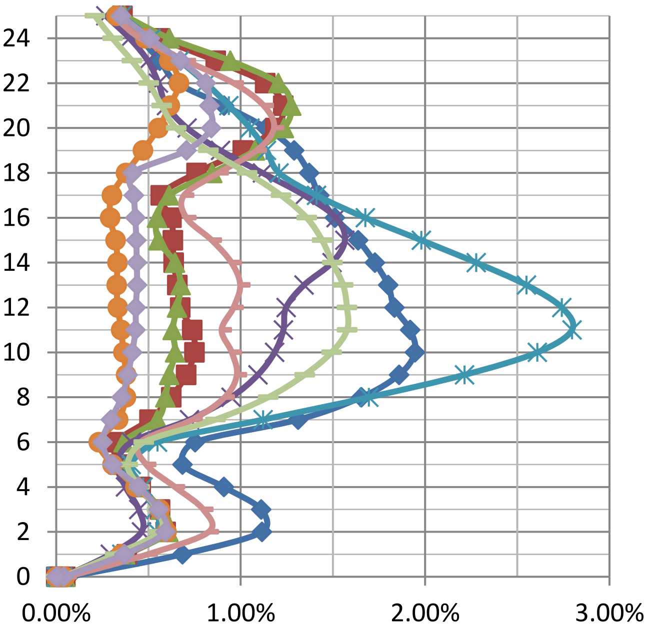

Figure 2: Example Nonlinear Dynamic Procedure Story Drift Plot

The ASCE 41 BPON targets Collapse Prevention in the BSE-2N (ASCE 7 MCE$_R$) for Risk Category II buildings. It can be inferred that providing a 10% probability of collapse means that the provisions have a 90% reliability of achieving collapse prevention. The committee adopted this level of reliability for the nonlinear dynamic procedure and incorporated many of the ASCE 7 updates. The three most significant updates relate to how ground motion acceleration histories are selected and scaled, what an unacceptable response is, and how force-controlled actions are treated. Some modifications to the ASCE 7-16 procedures were made.

The ground motion acceleration record selection and scaling updates to ASCE 7 are discussed in detail in Haselton et. al. (2017a). ASCE 41-17 now points directly to ASCE 7-16 for provisions to develop general and site-specific response spectra and how to select and scale ground motion acceleration records. The maximum component of the acceleration record is scaled instead of the square root sum of square of the record's two horizontal components to the target spectrum. The target spectrum can be the general response spectrum, a site-specific response spectrum, or multiple site-specific spectra, such as the Conditional Mean Spectrum approach per Baker (2011). 11 records are now required for each target spectrum, which can be randomly oriented unless the site is within 15 km of an active fault. In that case, the records should be applied based on the fault-normal and fault-parallel directions. The only major difference between ASCE 41-17 and ASCE 7-16 is the upper-bound period in the range used for scaling and matching need only be 1.5 times the largest first-mode period in the principal horizontal direction or 1 second, as opposed to 2 times the largest first-mode period. Both amplitude scaling and spectral matching are permitted, but there are penalties for using spectral matched ground motions.

In order to achieve the desired 90% reliability of the provisions, the way the standard addressed force-controlled actions was changed. The previous edition allowed one to compute the average of the maximum demand in a force-controlled element in each record and check it against the lower-bound capacity of the element. This approach had some potential issues. The first being that an analysis could show that the force-controlled action was overstressed in multiple ground motions, while the average was still less than the capacity. This would indicate that the element could fail in that record, and had the failure of that force-controlled action been modeled in the analysis, the building might show a collapse potential or the analysis not complete due to its failure. The provisions addressed this by creating a mechanism where force-controlled actions are placed in to one of three categories: Critical, Ordinary, or Noncritical. Critical force-controlled actions are those whose failure would lead to a collapse of multiple bays of the structure, such as the failure of a column. Ordinary are those whose failure would lead to collapse of a single bay, such as the failure of a beam's connection. All other actions are noncritical. The equation to evaluate force-controlled actions in the nonlinear procedure is:

$$\gamma\chi(Q_{uf} - Q_g) + Q_g \le Q_{cl}$$

Where χ is the same amplification factor for performance levels higher than Collapse Prevention, and γ is a factor to amplify forces on critical force-controlled actions by 1.3. In the ASCE 7 provisions, the factor on critical force-controlled actions is 1.5, but it is checked against mean or expected capacities. The 1.3 amplification factor on the mean response of the earthquake component of the demand was derived assuming the same lognormal probability distribution discussed in Haselton et al (2017b) with the same coefficients of 0.45 for demand prediction and 0.15 for capacity prediction. With those assumptions, the gamma factor only needs to be 1.3 with mean minus one standard deviation (ASCE 41's definition of lower-bound) material properties on the capacity to achieve a 90% reliability of not collapsing.

Another change to the ASCE 7 and 41 provisions is to allow one unacceptable response for Life Safety and lower performance levels. An unacceptable response is defined as an analysis run which failed to converge, the demands on the deformation-controlled actions exceed the valid range of modeling, demands on a critical force-controlled actions exceed the expected (not lower-bound) capacity of that action, or members not modeled exceed deformation limits where they are able to carry gravity loads. All of these situations could be indicators that the ground motion record being applied to the model is causing instability with the potential for collapse. Since a minimum of 11 ground motion records are applied, a predicted collapse under one record is still within the desired 90% reliability of not collapsing. See Haselton et al (2017b) for more discussion. For example, the one record with 3% story drift in Figure 2 may be indicative of a potential collapse. However, the other ten records are all within the limits. Since the provisions target 90% reliability, not absolute certainty, this is acceptable for Life Safety and Collapse Prevention performance assessments.

Steel Provisions Updates

The most significant update to the steel provisions were changes to the modeling and acceptance criteria for steel columns. A series of studies were conducted by NIST to benchmark ASCE 7 and ASCE 41 to eachother (NIST 2015a, 2015b, and 2015c). The studies looked at steel framed building of different heights designed to ASCE 7-10. Each building was subjected to all four analysis procedures contained within ASCE 41-06. The results showed the buildings which met the ASCE 7-10 and AISC 341-10 criteria did not meet the BPON. The main reason for the buildings failing to achieve the expected performance objective, as they were designed to the new building standard, pointed to a potential area of conservatism in how the standard treats steel columns. A number of committee members independently identified the same potential issue with steel column criteria.

The column provisions in ASCE 41-13 and previous editions require the ductility of a column be reduced from that of a beam once the axial force including both gravity loads and seismic forces exceeds 20% of the expected axial buckling capacity in the direction of bending ($P_{uf}/P_{cl,x} > 0.2$). Like beams, the ductility (m-factor and nonlinear modeling and acceptance criteria) is also reduced if the columns flanges or webs do not meet the seismic compactness requirements of AISC 341-10. The columns then become force-controlled when the axial force ratio increases to more than 50% of the expected axial buckling capacity in the direction of bending ($P_{uf}/P_{cl,x} > 0.5$). The transition to force-controlled whenthe axial force exceeds 50% of the expected axial capacity is what both the NIST reports and the committee member investigations independently identified as the main source of likely conservatism.

A subcommittee reviewed a number of different research reports on the performance of steel columns under combined axial load and bending. Those reports are listed in Bech et al (2017), which also outlines the methodology used to develop the new column modeling parameters and acceptance criteria. A review of the reports indicated that the ductility of steel columns were most affected by sustained axial force, as opposed to the maximum transient axial force spike it may see during an earthquake. That led to the change of the column axial load ratio from the maximum axial load divided by the expected capacity to the gravity load divided by the yield capacity. Additional research indicated that the axial buckling capacity could be replaced by the yield capacity in the denominator. A regression analysis of the data from the papers showed that the ductility of the column could be expressed as a function of the gravity axial load ratio, the web and flange compactness ratios (h/tw and b/2tf), and the length divided by the weak-axis radius of gyration (L/r_y). Both the m-factors and the nonlinear modeling and acceptance parameters were updated based on this research and will yield less conservative assessments of columns in steel buildings.

The other significant update to the steel provisions was the introduction of rotation limits to the evaluation of moment frames when panel zones yield before the beams. The provisions of earlier editions were found to be potentially unconservative because they did not distinguish if the connection of the beam to column was made with notch tough weld metal or older weld metal with a high fracture potential. Further, the panel zone criteria did not account for the flexibility of the flange or the axial load on the panel zone. The new provisions account for both in the nonlinear procedure and axial force in the linear procedure.

Concrete Provisions Updates

In the current operating structure of ASCE 41, the technical changes for concrete provisions are provided by ACI Committee 369, Seismic Repair and Rehabilitation, prior to the ASCE 41 main committee voting to approve. The primary changes from the committee in this cycle involved the testing of existing anchors, updated modeling parameters and acceptance criteria for concrete columns, updated wall stiffness provisions, and clarifications regarding the evaluation of concrete elements with net tension.

The addition of testing requirements for existing concrete anchors was one of the most critical changes for the concrete chapter. In many existing concrete buildings, there are existing cast-in-place and post-installed connections of structural and nonstructural components necessary for transferring seismic forces or anchoring falling hazards (e.g., out-of-plane wall anchorage, anchorage of heavy equipment in evacuation route). Until more recent building codes, these anchors were not designed and installed per well-defined design procedures and quality control requirements, and there were typically no testing requirements, especially for post-installed mechanical and adhesive anchors. ASCE 41-13 contained requirements to test concrete cores and steel reinforcement, however there were no requirements for critical connection elements to establish design strength. The strength of concrete anchors is exceptionally sensitive to the installation method used, and thus the expected performance of existing anchors in a seismic event is difficult to predict in the absence of testing. The committee thus decided to add minimum testing requirements for usual and comprehensive data collections for existing cast-in-place and post-installed anchors. The testing frequency was selected to mimic the number required for reinforcement testing, and the testing loads were tied to the necessary design strength.

Another major revision in the concrete provisions involved concrete columns. Ghannoum and Matamoros (2014) summarize much of the work that led to column modeling changes, resulting in column parameters in the form of equations rather than the past table form. The new equation format makes it easier to calculate modeling parameters (MP's) for different conditions and removes the need for triple interpolation required in previous editions of ASCE 41.

The ASCE 41-13 column parameters had conservative reductions embedded in the MP table values, however the committee felt any conservatism should be placed on the acceptance criteria instead so as to avoid skewing analytical results. As such, the modeling parameter "a" and "b" values are meant to represent the "best estimate" based on extensive test data, consistent with Section 7.6 of ASCE 41. Since the column test data showed superior performance for circular columns as compared to rectangular columns, the MP's were also broken up into separate equations for circular and rectangular columns in ASCE 41-17.

In addition to the anchor testing and column provisions, other technical concrete changes improved the evaluation of structural walls and elements with net axial tension, also lead to more consistency between linear and nonlinear procedures. Test data and fiber analysis of walls demonstrate more flexibility for structural walls than the previously employed effective stiffness modifier of 0.5. The increased flexibility in wall buildings could be critical when increased deformation demands on the non-ductile concrete gravity-force-resisting elements could lead to collapse. The stiffness modifier was thus reduced to 0.35 for walls, and commentary was added for an alternate "constant yield curvature" approach. The proposed alternate approach results in far more parity with nonlinear fiber analysis and also captures the effects of wall reinforcing and axial demands.

The change for elements in net tension was another which had notable consequences for linear procedures. Linear analyses frequently result in high tension demands on some wall and column elements. Strict adherence to ASCE 41-13 required axial demands to be evaluated as force-controlled actions, which is appropriate for concrete elements in compression. However the committee felt that the requirements led to unnecessary conservativism for elements in net tension, which were expected to behave in a more ductile manner than their compression-loaded counterparts. Axial demands in tension were thus clarified to be analyzed as deformation-controlled actions. This change, along with the changes for wall stiffness, are expected to provide more consistency between ASCE 41 linear and nonlinear procedures.

Masonry Provisions Updates

The masonry provisions underwent a number of significant updates. There were updates to the assessment of out-of-plane actions in unreinforced masonry walls based on research. The collapse prevention evaluation can still be carried out using the table that provides maximum h/t ratios. For life safety, an assessment of the wall for dynamic stability based on Penner and Elwood (2016) has been added. The assessment is for walls with $h/t > 8$ and compares the 1.0s acceleration parameter against a series of coefficients multiplied together. The coefficients account for the wall aspect ratio, the diaphragm flexibility, the height of the walls in the building, and the axial force on the walls. For Immediate Occupancy, the walls must not experience any overstress in the flexural tension strength of the mortar under out-of-plane loading.

ASCE 41-13 commentary discussed the potential impact on spandrel beams on the performance of unreinforced masonry walls, but there was not sufficient research available at the time to develop provisions. There were however provisions for unreinforced masonry spandrel beams added. Equations are provided to determine the shear and flexural capacity of a spandrel beam, which are based whether there is a lintel or a masonry arch supporting the spandrel beam. Shear and flexure can be considered deformation-controlled actions.

The provisions for steel and concrete frames with masonry infill were completely rewritten and provide an easier method to model masonry as a compression strut within the frame. The panels are classified as strong or weak and flexible or stiff with respect to the frame. There are different modeling parameters and capacities if the frame is non-ductile concrete or either ductile concrete or steel framing encased in concrete. The acceptance criteria for the infill panels is based on the ratio of the strength of the frame without the panels to the strength of the infill panels and the aspect ratio of the panels. The criteria for out-of-plane actions when considering arching action has also been updated.

Additional Updates

There were a number of other updates made to the standard. Below is a summary of some:

- Revisions to the soil structure interaction provisions related to their applicability and limitations on response parameter reductions.
- Updates to the seismic isolation and energy dissipation provisions, now in separate chapters, to make them consistent with updates made to those provisions in ASCE 7-16.
- Requirements to test existing anchors securing nonstructural components.

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References

AISC 341-10, 2010, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL.

ASCE 31-03, 2003, *Seismic Evaluation of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia.

ASCE 41-06, 2006, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia.

ASCE 41-13, 2013 *Seismic Evaluation and Upgrade of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia. Public Comment Edition available through the American Society of Civil Engineers.

ASCE 41-17, 2017, *Seismic Evaluation and Upgrade of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia.

ASCE 7-10, 2010, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, Virginia.

ASCE 7-16, 2016, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, American Society of Civil Engineers, Reston, Virginia.

Baker, J. W., 2011, "Conditional Mean Spectrum: Tool for Ground-Motion Selection." *Journal of Structural Engineering,* Vol. 137, No. 3, March 1, 2011. American Society of Civil Engineers. Reston, VA.

Bech, D., Houston, J., Tremayne, B. 2017. "ASCE41-17 Steel Column Modeling & Acceptance Criteria." *Proceedings of the 2017 Structures Congress.* American Society of Civil Engineers, Reston, VA.

Ghannoum, W.M., Matamoros, A.B., "Nonlinear Modeling Parameters and Acceptance Criteria for Concrete Columns," ACI Special Publication, SP-297, No. 1, pp. 1.1-1.24, March 2014.

Harris, J.L. and Speicher, M.S. (2015a). *Assessment of First Generation Performance-Based Design Methods for New Steel Buildings, Volume 1: Special Moment Frame Braced Frames*, NIST TN 1863-1. National Institute of Standards and Technology, Gaithersburg, MD. http://dx.doi.org/10.6028/NIST.TN.1863-1.

Harris, J.L. and Speicher, M.S. (2015b). *Assessment of First Generation Performance-Based Design Methods for New Steel Buildings, Volume 2*: Special Concentrically Braced Frames – NIST TN 1863-2. National Institute of Standards and Technology, Gaithersburg, MD. http://dx.doi.org/10.6028/NIST.TN.1863-2.

Harris, J.L. and Speicher, M.S. (2015c). *Assessment of First Generation Performance-Based Design Methods for New Steel Buildings, Volume 3*: Eccentrically Braced Frames – NIST TN 1863-3. National Institute of Standards and Technology, Gaithersburg, MD. http://dx.doi.org/10.6028/NIST.TN.1863-3.

Haselton, Curt B., Jack W. Baker, Jonathan P. Stewart, Andrew S. Whittaker, Nicolas Luco, Andy Fry, Ronald O. Hamburger, Reid B. Zimmerman, John D. Hooper, Finley A. Charney, and Robert G. Pekelnicky, 2017a "Response History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part I - Overview and Specification of Ground Motions." *Earthquake Spectra*: May 2017, Vol. 33, No. 2, pp. 373-395.

Haselton, Curt B., Andy Fry, Ronald O. Hamburger, Jack W. Baker, Reid B. Zimmerman, Nicolas Luco, Kenneth J. Elwood, John D. Hooper, Finley A. Charney, Robert G. Pekelnicky and Andrew S. Whittaker., 2017b "Response History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part II - Structural Analysis Procedures and Acceptance Criteria." *Earthquake Spectra* 33:2, 397-417.

Jarrett, J. A Reid B. Zimmerman, Finley A. Charney and Afshar Jalalian. (2017) "Response-History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part IV - A Study of Assumptions." *Earthquake Spectra* 33:2, 449-468.

Pekelnicky, R. G., and Poland, C. D., "ASCE 41-13: Seismic Evaluation and Retrofit of Existing Buildings," *SEAOC 2012 Convention Proceedings*, 2012, Structural Engineers Associate of California.

Penner, O. and Elwood, K. J. (2016) Out-of-Plane Dynamic Stability of Unreinforced Masonry Walls in One-Way Bending: Parametric Study and Assessment Guidelines. *Earthquake Spectra*: August 2016, Vol. 32, No. 3, pp. 1699-1723.

Zimmerman R. B, Jack W. Baker, John D. Hooper, Stephen Bono, Curt B. Haselton, Albert Engel, Ron O. Hamburger, Ayse Celikbas and Afshar Jalalian. (2017) "Response History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part III - Example Applications Illustrating the Recommended Methodology." *Earthquake Spectra* 33:2, 419-447.