ASCE-31 and ASCE-41: What Good Are They?

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ABSTRACT

Two relatively new standards, ASCE-31, *Seismic Evaluation of Existing Buildings*, and ASCE-41, *Seismic Rehabilitation of Existing Buildings*, are being touted as great developments in earthquake engineering -- the next wave of performance-based engineering. However, belying the polished appearance of these standards, is the reality that standards like these will constrict the application of engineering first-principles in the existing buildings evaluations -- even by engineers who have significant performance-based experience in earthquake engineering -- and will result in condemnation or strengthening of elements of structures or structures in their entirety that ought not be condemned or upgraded. In parallel, use of these standards by engineers who are inexperienced in earthquake engineering may result both in a false sense of security and in buildings that will not perform as expected. In our opinion, ASCE-31 and ASCE-41 represent significant dangers to our profession, simultaneously squelching appropriately creative performance-based methodologies, promoting the expenditures of large sums of money on inappropriate engineering analysis and strengthening, and encouraging inexperienced practitioners to leap unprepared into performance-based seismic evaluation and design.

INTRODUCTION

The American Society of Civil Engineers has recently published two new standards, ASCE-31, *Seismic Evaluation of Existing Buildings* [1], and ASCE-41, *Seismic Rehabilitation of Existing Buildings* [2], as well as a very recently released *Supplement No. 1* [3]. Both of these standards purportedly represent the consensus knowledge of a large number of engineers and are touted as major advances in earthquake engineering; both are evidence of the ongoing “standardization” and “cookbook-ization” of earthquake engineering, with all the negative implications of those phrases intended. To be fair, both documents represent a tremendous quantity of effort and contain a great deal of valuable material for reference purposes, but performance-based earthquake engineering for existing buildings is not a subject the authors believe to be suitable to either “standardization” and “cookbook-ization”. Earthquake engineering -- in particular, dealing with the assessment and strengthening of existing non-conforming buildings -- used to be the purview of a relatively small number of engineers who had gained experience through years of observing building response during actual earthquakes and performing earthquake engineering under the watchful eye and guidance of evermore experienced engineers. These engineers had the knowledge to justify doing engineering outside the margins of the prescriptive code. However, due to the relatively high fees that can be charged for performance-based earthquake engineering, more and more inexperienced engineers are jumping into the fray, resulting in many designs that are either significantly over- or under-conservative. The push to standardize the industry was initiated in part, rightly, to encourage the use of performance-based earthquake engineering --- in the right hands a "better tool" than prescriptive engineering --- and in part to control, rightly or wrongly, its application.

ASCE-31, *Seismic Evaluation of Existing Buildings*, and ASCE-41, *Seismic Rehabilitation of Existing Buildings*, were developed from government publications FEMA 310 [4] and FEMA 356 [5], respectively, with the goal of providing a set of rules, regulations, and procedures dealing with assessment and strengthening of existing structures, with each set of rules, regulations, and procedures organized by basic building type, and written like a cooking recipe: do this, do this, do this, and poof! -- you have yourself a perfectly assessed or perfectly strengthened building. Apparently however, with the development of these now-prescriptive standards also came the attempt to assure a "conservative" outcome for all structures of a given type, with a one-size-fits-all mentality, thus severely curtailing if not eliminating the use of judgment, which was arguably the greatest tool of experienced earthquake engineers.

While the goal of documents such as ASCE-31 and ASCE-41 was performance-based engineering, these standards quickly devolved back into code-like documents, requiring use of "consensus-based" m-factors for all elements of a certain type, stringent and overconservative k-factors representing some fictional quantification of across-the-board uncertainty, and fairly irrational factors like C_1, C_2, J, and R_{OT}.

COMPARISON OF ASCE-31 AND ASCE-41 WITH THE BUILDING CODE

Traditional building codes specify a demand, usually in the form of a base shear equation:

$$V = \frac{2.5 * Ca * I}{R} * W \tag{1}$$

where the Ca is the effective peak ground acceleration, I is an importance factor that ranges from 1.0 to 1.5, R is a fairly arbitrary response reduction factor that ranges from 2.2 to 8.5 (and is intended to account for the facts that materials are generally stronger than we give them credit for; that the earthquake demands are damped out and reduced by inelastic behavior; and that for whatever reason, well-detailed structures tend to survive earthquakes); and W is the seismic weight of the structure. Typical codes also specify a capacity, based on accepted standards such as AISI and ACI. If the capacity is greater than the reduced demand, the building and its structural components are judged acceptable.

Similarly, ASCE-31 and ASCE-41 specify a demand -- typically in the form of a base shear equation, such as:

$$V = C_1 * C_2 * C_m * S_a * W \tag{2}$$

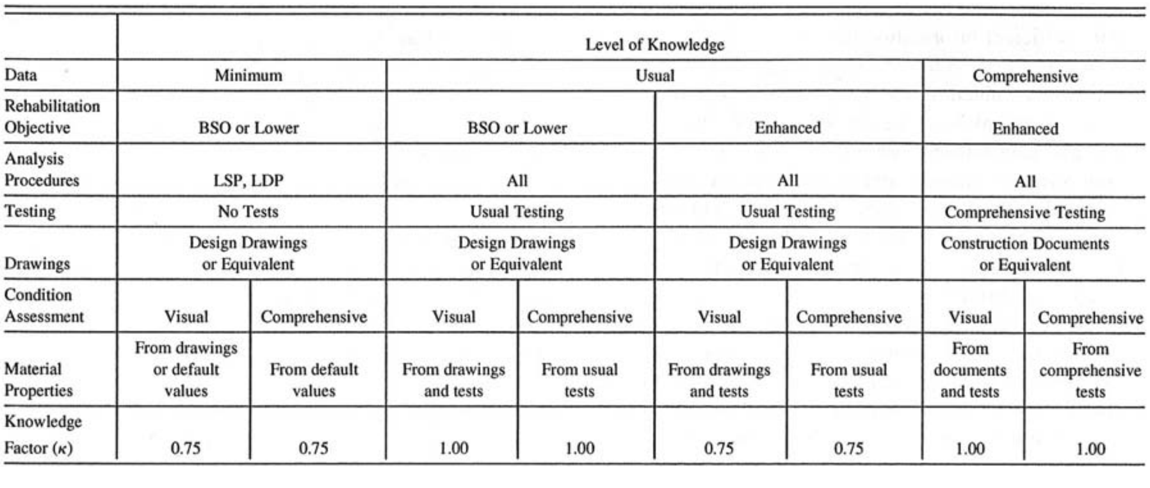
The demand on any given element is represented by a variable, such as Q_D, Q_G, Q_L, Q_{UD} or Q_{UF}. And ASCE-31 and ASCE-41 also specify a capacity, Q_{CE}, based on accepted standards such as AISI and ACI and require that the capacity be greater than the demand, such as:

$$m * k * Q_{CE} \geq Q_{UD} \tag{3}$$

If the factored capacities of the elements are greater than the demands, the structure is considered acceptable. Note that the arbitrary R-factor from the building code has been removed from the demand side (where the demand was reduced by dividing by the R-factor) and instead, a newly minted and pretty much arbitrary m-factor has been added to the capacity side. The R-factor in modern codes typically ranges from 2.2 to 8.5 and the m-factor has a value of anywhere from 1.0 to 12 for primary elements of the lateral force resisting system. Put another way, while modern codes have an R-factor that *reduces* the demands, ASCE-31 and ASCE-41 have an m-factor that *increases* the capacities. Thus while at first blush the two approaches appear fairly different, there is actually very little difference between the two. Both have coefficients and variables that are essentially made-up, with very little justification or logic (e.g. R, I, C_1, C_2, m, and k). Apparently, the more things change, the more they stay the same.

Neither approach allows a significant amount of judgment; though codes typically allow "alternative means and methods", ASCE-41 typically does not provide similar latitude. Admittedly, the m-factor may appear to represent some modest incremental improvement over "R" since R was system dependent and "m" is element dependent; however, the improvement is largely theoretical since buildings pass or fail the tests of the ASCE-31 and ASCE-41 largely on the basis of the "failure" of only one or few elements; thus the element level response is hopelessly-as-ever intertwined with the system's response.

The knowledge factor, k, has a value of 1.0 or less and penalizes the structure if the material strengths are not known in "sufficient" detail. In order to qualify for a k of 1.0, some amount of testing is required, as shown in Table 1 below. If material testing data from construction is not available (and what owner of an existing building has that type of data if the building is of any significant age), destructive testing is required even if the minimum required material strengths are shown on the drawings -- a requirement more stringent at least for some materials than even required for brand new buildings. And if one is assessing an existing building for an enhanced objective, "comprehensive" testing must be performed in order to qualify for a k of 1.0 -- again a requirement more stringent than required for brand new buildings. Note that the k factor disproportionately penalizes historic structures, since testing of historic structures is even more difficult in these structures because the locations where testing can be performed are often limited by the need/desire to protect the historic integrity of the structure.



	Level of Knowledge							
Data	Minimum		Usual				Comprehensive	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design Drawings or Equivalent		Design Drawings or Equivalent		Design Drawings or Equivalent		Construction Documents or Equivalent	
Condition Assessment	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material Properties	From drawings or default values	From default values	From drawings and tests	From usual tests	From drawings and tests	From usual tests	From documents and tests	From comprehensive tests
Knowledge Factor (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

TABLE 1 - KNOWLEDGE FACTOR TABLE FROM ASCE-41

Even with highly complicated procedures such as nonlinear time-history analyses, ASCE-41 is not significantly better than a prescriptive approach, since the acceptance criteria for these analyses are laid out in tables that do not allow for judgment. For welded steel moment frame buildings, for example, connections are either ductile or brittle, and they either pass the acceptance criteria or they don't. No provision is given or allowed to evaluate whether or not welds are of better-than-average quality or whether the weld metal is ductile or not. A structure can be condemned as requiring strengthening based on these arbitrary and immutable acceptance criteria; this despite all analytical and historical evidence to the contrary that demonstrates that even if significant numbers of fractures do occur, life safety is typically not jeopardized.

In short, engineering first principals that once made performance-based engineering worthwhile and useful have been replaced by tables upon tables of coefficients and values that do not allow for engineering judgment -- the primary tool of the skilled earthquake engineer.

CASE STUDIES

The authors have significant experience using these two standards and their predecessors and have seen many instances where the use and mis-use of these documents have resulted in nonsensical and overconservative results. In the pages that follow, a number of case studies are presented that show some of the problems inherent in the two standards.

Case Study 1: A Historic Two-Story Wood-Framed Structure

As part of a peer review for a government agency, the authors reviewed the proposed seismic upgrade of a two-story, wood-framed, national historic landmark in an area of moderate seismicity. The structure had wood-lath-and-plaster walls, walls with vertical wainscoting, a few non-historic walls with gypsum board, an unreinforced stone masonry foundation, and at one time, two unreinforced stone masonry chimneys. The engineer retained to design the upgrade used FEMA 310 (the predecessor to ASCE-31) to evaluate the structure and FEMA 356 (the predecessor to ASCE-41) to design the strengthening. The engineer made four critical errors worth discussing. The first error was a failure to recognize that the structure had a lateral resisting system that had worked adequately for almost a century, albeit had not been subjected to a design level ground-shaking event; nonetheless, the engineer concluded that, “The building has no readily apparent lateral load resisting system.” The engineer assigned a value of zero to nearly all of the existing lateral force resisting elements and specified the construction of a large number of new plywood walls that necessitated destruction of the existing historic finishes. Apparently, the fundamental message of FEMA 310 -- that non-complying construction does have some inherent seismic capacity -- was not picked-up by the engineer. Their second mistake was a failure to identify that one of the two unreinforced stone masonry chimneys had been demolished during a previous remodel from the foundation all the way up into the attic. A small stub of the masonry chimney was left supported by wood framing in the attic, and the top of the chimney projected up above the roof ridge. The center of gravity of the chimney was actually above the roof ridge, making proper bracing of the chimney impossible; the engineer failed to even identify this condition as being problematic, apparently because there is no tabulated m-factor for discontinuous chimneys in FEMA 310. Their third mistake was failure to recognize that some of the existing walls were lath and plaster and that other walls had vertical wainscoting on the bottom half without lath and plaster underneath; consequently, the walls with wainscoting had a significantly lower capacity than the walls with lath-and-plaster, yet the engineer failed to distinguish between the two types of walls (possibly because the engineer had already discounted the capacity of the existing lateral force resisting system essentially completely). The fourth error was most egregious. Two prior geotechnical studies had been performed for the site; one predicted maximum liquefaction-induced settlements of approximately 0.5 inches (1.2 mm) and one predicted a maximum of 8 inches (20 mm) of settlement caused by liquefaction. Note that the settlements predicted were *maximum* settlements, and that *differential* settlements would be expected to be far smaller. Despite the fact that liquefaction settlement would be at most only a few inches over a length of more than 100 feet (30 meters) and would be extremely unlikely to result in distress significant enough to cause a life-safety hazard in any wood-framed structure, the engineer specified hand-excavation of the crawlspace and construction of a massive 24-inch-thick reinforced concrete mat under this two-story, light-framed wood structure

Assuming for the moment that seismic evaluation and strengthening projects are everyday being completed with similar results using these highly touted performance-based engineering documents, is this a consequence that we as a profession are ready to support as acceptable and necessary in order to make PBE accessible to the profession and to society? The engineer's astonishing difficulty in applying FEMA 310 led to a mammoth upgrade according to their interpretation of FEMA 356, with predictable consequent destruction of historic fabric. The proposed upgrade was so expensive that it exceeded available funding. In reality, the structure was very light, typically had well-distributed walls in both directions, and was relatively weak in only a few areas. Fortunately in this instance, the owner was technically savvy, recognized the unsound results and requested the authors to try an approach different than that suggested by the first engineer. The authors proposed using judgment in lieu of the rote standards of FEMA -- despite the absence of language permitting its use -- and developed a strengthening scheme that cost approximately one third of that proposed by the initial engineer; a scheme accepted by the client.

Case Study 2: A Historic Multi-Story Wood-Framed Structure

As part of a peer review for a government agency, the authors reviewed the proposed seismic upgrade of a multi-story, wood-framed structure in an area of moderate seismicity. Unlike the first structure, this structure had already been upgraded with numerous plywood shearwalls. Using FEMA-356 (the predecessor to ASCE-41), an engineer who studied the structure concluded that the structure was unsafe under seismic loads and needed to be upgraded again. Like the engineer in the first example, this engineer made a number of errors -- the first of which was to discount most of the capacity of the existing plywood shear walls. In ample demonstration of a wholesale misunderstanding of the concept of performance-based engineering, in attempting to follow FEMA-356 the engineer compared the demands from the full, unreduced earthquake in FEMA-356 with *allowable* shear capacities presented in the 1997 Uniform Building Code. Note that proper use of FEMA-356 would dictate that the strength-level capacities of the plywood walls be increased by an m-factor of approximately 4; consequently, given that the engineer had underestimated the "capacity" of the walls by a factor of approximately 5, the engineer concluded that the structure had almost no significant capacity (despite relatively well-designed and detailed existing plywood walls located throughout the structure). The engineer also neglected to consider the fact that the unbalanced snow loads on this structure are so large that they dwarf the seismic loads; since these snow loads occur on a frequent basis, the very fact that the structure is able to resist these loads indicates that the structure was substantially more competent than recognized by the engineer. Perhaps the absence of a chapter on the behavior of structures in snow country in FEMA-356 led to this oversight. With improper use of FEMA-356, the engineer specified the addition of plywood to nearly all of the walls within the structure, adding plywood to most of the diagonally sheathed floor diaphragms, and, somewhat unbelievably, adding reinforced *concrete* walls in a number of areas to provide support for the *wood-framed* structure. In this case, the client was unable to force a change in direction on the part of the engineer, and the unneeded and hideously expensive upgrade proceeded unhindered.

Case Study 3: A Two-Story Steel Braced Frame Structure

This example elegantly demonstrates why documents such as ASCE-31 and ASCE-41 will never achieve the goal of improving the seismic evaluation and performance of existing buildings on a large scale. Working for an owner, an engineer evaluated a two-story, steel braced-frame structure. The engineer concluded that the existing detailing of the braced frame was inadequate, completely ignored the capacity of the existing braces (to the point where the engineer even specified that the braces be cut so that they would take no forces) and designed new buckling-restrained braces to replace the old braces using FEMA 356 provisions (the basis for ASCE-41). While there may indeed have been detailing issues with the existing braces, a primary goal of ASCE-31 and ASCE-41 was to allow proper evaluation of the existing capacities of elements that may not conform to the detailing required by current code. Rather than work with the existing system, the engineer did what many engineers typically (and unfortunately) do when they encounter an existing building -- design a brand new lateral force resisting system. By completely discounting the existing lateral force resisting system, the engineer subverted a primary goal of ASCE-31 and ASCE-41. Furthermore, while the engineer opted to require performance more stringent than required by FEMA 356, he could not have allowed any relaxation of FEMA 356, since FEMA 356 has little or no provision for permitting the use of judgment.

Case Study 4: A Three-Story Concrete Shear Wall Structure

A small three-story concrete shear wall building was selected for use as an emergency communications center. The structure had perimeter, mostly solid, lightly reinforced concrete shear walls. Aside from the ground floor, which had three garage door openings along one side, the structure was basically a rigid box. A code-check of the structure was performed, comparing code-level demands with capacities allowed by the current code -- the 1997 Uniform Building Code. The code-check indicated that while the existing detailing wasn't quite up to current detailing requirements, the structure had substantially greater strength than would be required for a new building (even accounting for the garage door openings), thus greatly offsetting if not completely compensating for the limited detailing. However, ASCE-31 and ASCE-41 had been selected as the criteria by which the structure should be analyzed and upgraded. The ASCE-31 and ASCE-41 analyses were significantly flawed, primarily because of the rigid rules incorporated into these two standards.

According to ASCE-31, if the structure had a foundation less than 3-feet below grade, the maximum acceleration for consideration would be limited to 0.75g, presumably because this shallow foundation structure would tend to slide or rock as a rigid body, limiting the amount of energy that can be transmitted to the structure. However, because the ground at the back of the structure is slightly higher than three feet, ASCE does not allow a similarly rational approach, resulting in spectral accelerations more than double for that of a building of similar construction but with just slightly less soil at the back.

Overturning, as evaluated by ASCE-31, was also difficult to resolve. Given the very high levels of spectral acceleration required by these standards, the building was subjected to lateral loads approaching 2.0g. Since the structure only weighs 1g and since the dead load tributary to the walls was much smaller, there was insufficient dead load to prevent uplift, necessitating the installation of soil anchors, which have the potential to preclude beneficial sliding. Another problem involved the extreme conservatism in ASCE-41 (even when considering Supplement No. 1); as a concrete shear/bearing wall building with concrete coupling beams, the m-factors were limited to 2.0 for the walls and 1.5 for the coupling beams. Combined with the knowledge factor, which mandates a 0.75 factor unless large numbers of disruptive and unnecessary material tests are performed, the code equivalent R-factor was approximately 1.5 -- an extremely conservative value, even for an essential facility. In the end, the use of ASCE-31 and ASCE-41 resulted in strengthening beyond that deemed necessary by the experienced engineers evaluating the structure, and beyond that which would have been required if the structure had been evaluated according to current code (subject to slight modifications according to the judgment of the evaluating engineer).

Case Study 5: A Multi-Story Concrete Shear Wall Building

In another project, a multi-story concrete shear wall structure with a back-up steel moment frame was evaluated by two different teams of engineers using ASCE-31. Both teams of engineers were experienced earthquake engineers, with decades of experience. One team of engineers concluded that the structure represented a significant life-safety hazard under the design level earthquake. Using ASCE-31, they predicted the presence of multiple soft stories, multiple weak stories, weak diaphragms, buckling columns under discontinuous walls, and failure of the rooftop steel-framed cupola. This team concluded that the existing structure had a fairly low capacity, so low that the structure would have had to have experienced significant structural damage -- almost to the point of collapse -- in a prior earthquake.

Since the building actually did not experience any structural damage during that prior earthquake, the first team’s analysis and conclusions were obviously suspect. Conversely, evaluating the same building under the same design level earthquake and using the same standards, the second team concluded that the building was robust and clearly met the life-safety goal under the design level earthquake. The building owner, faced with diametrically opposed results based on the same documents, was left with the dilemma of determining what to do.

DISCUSSION AND CONCLUSION

Our point is this: if these standards are intended to provide engineers who do not know what they are doing with a procedure for evaluating and designing strengthening measures for existing buildings, then the first two examples demonstrate that these documents don’t work. Engineers who do not know what they are doing simply do not know what they are doing and, standards or not, that is not going to change. If the standards are intended to allow proper consideration of the actual capacities of existing elements, even if they may not be detailed in full accordance with current code, then the second two examples belie that goal. If the standards are intended to provide some consensus as to what constitutes a good design and what constitutes a bad design, then they do not do that either, as all the examples show. If the standards are intended to remove most or all judgment from earthquake engineering, then this is simply the wrong goal for these standards, since the practice of earthquake engineering is often more of an art than a science and removal of judgment in such a complicated field where every structure is different and where every structure has different strengths and weaknesses is a catastrophic setback to our profession. In reality, these standards are very similar to codes; instead of an “R-factor” there is an equally arbitrary "m-factor". Instead of dividing the demand by R, the capacities are multiplied by an "m-factor"; with this sleight-of-hand, it has been claimed that a major improvement in earthquake engineering has occurred. Unfortunately, ASCE-31 and ASCE-41 actually discourage or even prohibit the use of engineering judgment in determining whether a building or design complies with the standards. Since engineering judgment is critical to proper seismic assessment and strengthening of a structure, these standards have a great potential to force engineers who have significant performance-based experience in earthquake engineering to condemn or upgrade elements of structures that do not deserve to be condemned or upgraded, and conversely, since they differ from the codes that inexperienced seismic engineers use on a daily basis, these standards are much more likely to result in nonsensical answers when used by the inexperienced engineer.

REFERENCES

[1] American Society of Civil Engineers (ASCE), *Seismic Evaluation of Existing Buildings* (ASCE/SEI 31-03), 2003, Reston, Virginia.
[2] American Society of Civil Engineers (ASCE), *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06), 2007, Reston, Virginia.
[3] American Society of Civil Engineers (ASCE), *Supplement No. 1, Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06), 2007, Reston, Virginia.
[4] Federal Emergency Management Agency (FEMA), *Handbook for the Seismic Evaluation of Buildings: A Prestandard* (FEMA 310), 1998, Washington D.C.
[5] Federal Emergency Management Agency (FEMA), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 356), 2000, Washington D.C.