ASCE 41 Assessment of the Imperial County Services Building and Comparison with Recorded Response and Damage

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Abstract

ASCE 41 is the U.S. standard to for seismic evaluation and rehabilitation; it also contains widely-used modeling guidelines. As part of the Applied Technology Council project 134, this paper documents the comparison of the historic response of the Imperial County Services Building with an ASCE 41 nonlinear dynamic assessment. To benchmark the ASCE 41 assessment of this building, a nonlinear response history analysis is performed using a 3D OpenSees model of the building following ASCE 41 concentrated plasticity modeling criteria and subjected to the recorded ground motion. Model response assuming both a fixed base connection and including soil-structure-interaction are investigated. The ASCE 41 assessment correctly identifies a deficiency in the first story columns that cause the building to fail to meet Collapse Prevention performance objectives. The response of the nonlinear dynamic model in terms of drifts and accelerations are similar to the recorded response of the building on the first story and show improved peak roof response when foundation flexibility is considered. Even though the ASCE 41 assessment correctly identified this structure as failing to meet Collapse Prevention, the damage assessment over predicted damage to most of the first story columns where shear cracking and some minor spalling was observed.

Keywords: ASCE 41; Performance Based Earthquake Engineering; OpenSees, Instrumented Buildings

1. Introduction

In the U.S., ASCE 41 is viewed as the "first generation" of performance-based seismic design. It is widely used as a standard and guideline for the seismic evaluation and retrofit of existing buildings. To do so, ASCE 41 provides numerical modeling guidelines for linear and nonlinear analyses, along with criteria for assessing the performance of existing buildings in terms of Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). For nonlinear analyses, component modeling is defined based on a provided nonlinear backbone relationship along with defined component-based acceptance criteria. The overall building performance is controlled by the poorest performing primary component. However, several recent studies have indicated that assessment methods of ASCE 41 may provide conservative outcomes. For example, Maison et al. [1] showed that, while the nonlinear ASCE 41 assessment correctly predicted drifts and the location of failure, the assessment identified CP at a significantly lower shaking level compared to the collapse of a shake table test structure. More recently, the U.S. National Institute of Standards and Technology (NIST) performed a series of studies that quantified the performance of modern code-conforming buildings using ASCE 41 [2,3,4,5]. Those studies showed that many modern building designs for various kinds of buildings failed to meet the ASCE 41 CP acceptance criteria. The authors interpreted this result to indicate either a deficiency in modern design, or a conservatism in the component-based acceptance criteria of ASCE 41. Harrington [6] showed that design of retrofits according to the ASCE 41 CP criteria retain significant post-earthquake strength and stiffness, indicating that buildings designed to component-based CP criteria performed better than the ASCE 41 document's qualitative damage descriptions would suggest.

To quantify the performance of the ASCE 41 assessment procedure, this study develops an ASCE 41 numerical model of a real, earthquake-damaged instrumented building. The simulations and evaluation from the ASCE 41 assessment are compared to recorded results. In particular, this paper summarizes the ASCE 41 numerical model and predicted response of the Imperial County Services Building, which experienced a partial collapse of several columns during the 1979 Imperial Valley Earthquake. The response of the Imperial County Services Building was recorded by 13 accelerometers as part of the California Strong Motion Instrumentation Program. Comparisons between predicted and recorded response are made and recommendations for updates to ASCE 41 are provided. This study is part of the Applied Technology Council (ATC) project 134 and funded by NIST. The project is developing ASCE 41 numerical models of several instrumented buildings to benchmark the performance of the ASCE 41 procedures and acceptance criteria for concrete buildings.

1.1 The ASCE 41 Standard

ASCE 41 grew out of FEMA 273 [7] and FEMA 356 [8], and was first published in 2006 by the American Society of Civil Engineers as ASCE/SEI 41-06 and is now in its third generation as ASCE/SEI 41-17 [9]. ASCE 41 systemizes performance-based seismic design and assessment methods for use by practicing engineers. In ASCE 41, numerical modeling guidelines and performance assessment criteria are provided for linear static, linear dynamic, nonlinear static, and nonlinear dynamic procedures to identify poor-performing components and evaluate overall building performance. Modeling parameters and acceptance criteria are derived based on experimental evidence [10]. Acceptance criteria define either force-based or deformation-based limits states for each component and classify the performance of the component as either IO, LS, or CP. If the performance objective acceptance criteria of any component is exceeded, the building is flagged as being in need of further evaluation or retrofit. Each performance objective is defined by a qualitative description of overall building performance. IO defines "the postearthquake damage state in which a structure remains safe to occupy and essentially retains its pre-earthquake strength and stiffness" (ASCE 41-17 2.3.1.1) [9]. CP defines "the postearthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin against collapse" (ASCE 41-17 2.3.1.5) [9]. While the descriptions of the performance objectives are based on global metrics, performance is evaluated using component-based metrics.

1.2 Imperial County Services Building and Earthquake Damage

The Imperial County Services Building was a six-story reinforced concrete frame and shear wall structure, constructed in 1971, damaged in the 1979 Imperial County Earthquake, and subsequently demolished. The response of the structure during the earthquake was recorded by 13 accelerometers attached to the building as part of the California Strong Motion Instrumentation Program [11].

The building consists of four moment frame lines in the east-west (E-W) direction and discontinuous shear wall in the north-south (N-S) direction, as shown in Figure 1. Shear walls are present on the exterior of the building at stories two and above, and in the interior of the building at the first story. The exterior upper level shear walls rest on beams such that walls are cantilevered approximately 70 inches from the column line. Columns generally have moderately ductile detailing, with transverse ties spaced at 2-3 inches in the hinge regions. However, at the base of the first-story columns, a ground slab connects to and disrupts the designed plastic hinge region; above this region, reinforcement spacing is 12 inches. Beams are present only in the E-W direction. Typical wall reinforcement consists of two curtains of #4s spaced at 16 in. vertically and 12 in. horizontally on the first-story walls, and a single curtain of #4s at 16 in. vertically and #5s at 12 in horizontally at upper levels. The foundations are pile caps on 45 ft. tapered piles connected by grade beams. Design material properties are 3, 4, and 5 ksi concrete for foundations, columns, and beams, respectively, and 40 ksi steel. Based on the instrumentation, the ambient period of the building before the earthquake was estimated to be 0.67 seconds in the E-W Frame direction, and 0.44 seconds in the N-S wall-frame direction [12].

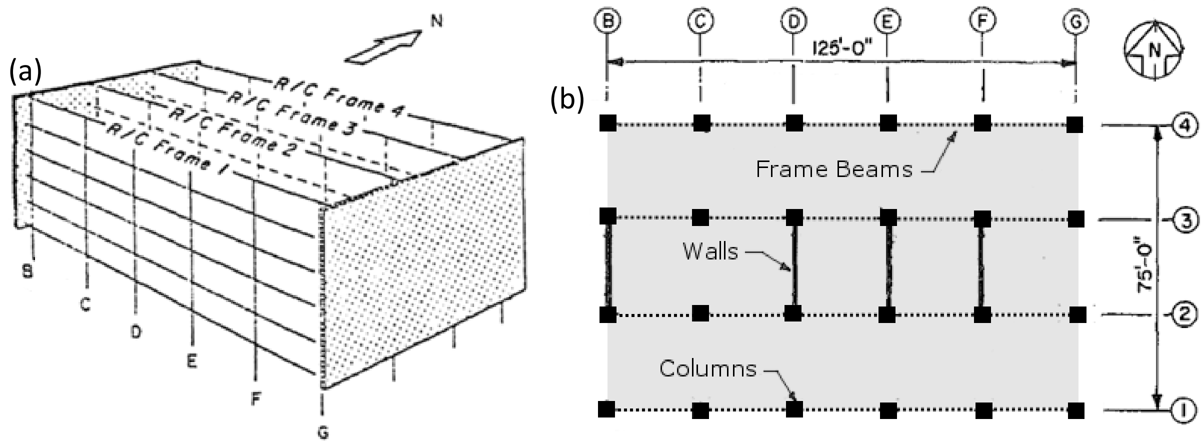

Figure 1 – (a) Lateral force resisting system of the Imperial County Services Building, with 4 moment resisting frame lines in the E-W direction and discontinuous shear walls in the N-S direction; (b) Plan view showing the structural components in the first-story (figures are modified from original diagrams [13,14]).

The ground motion recorded at the foundation during the 1979 earthquake had a peak ground acceleration (PGA) of 0.33 g in the E-W direction and 0.29 g in the N-S Direction. The response spectra in Figure 2 show significant energy in the range of 0.3 to 1.5 seconds. The vertical component of the ground motion has substantially smaller accelerations, except at periods < 0.3s. In the free field, the response spectra have PGA values of 0.24 g in the E-W direction and 0.21 g in the N-S direction. The motion recorded at the foundation is used for all analyses, except the case that considers foundation flexibility, for which the free field recordings are used.

A number of reports documented damage to this building [15,16,17]. The building sustained significant damage to the first-story columns, especially on the eastern-most frame line (column line G), as shown in Figure 3. This eastern column line exhibited axial column failure above the ground slab where spacing transverse reinforcement was wider, likely following shear failure. Interior columns' damage in the first story consisted of shear cracks primarily associated with response in the E-W direction. First-story walls showed some cracking. After the earthquake, the building was demolished due to the failure of the east-side columns.

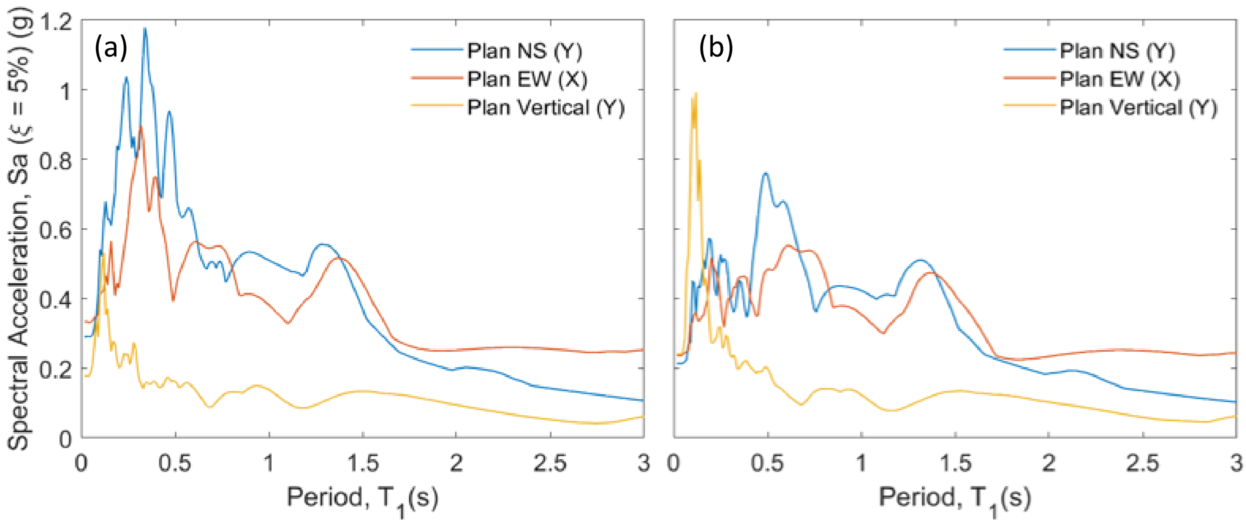

Figure 2 - Recorded response spectra, calculated with 5% damping: (a) at the foundation and (b) in the free field.

The Imperial County Services Building was selected for this study as it provides a unique opportunity to compare a full-scale instrumented response of a real U.S. structure with an ASCE 41 assessment. The building has also been extensively studied, documented, and modeled over the past four decades. Kroger and Cozen [17,14] compared the recorded response of the structure with the observed damage to identify the causes of failure. Their study found that the discontinuity of the shear walls on the east side of the building was the key factor in the failure of the columns. This asymmetric N-S wall layout and discontinuity amplified the torsional response and axial loads on the east side of the building, overloading the columns. Other studies, e.g., [18,19], came to similar conclusions. Kojic et al. [19] also noted that soil-structure interaction had a significant effect on N-S response.

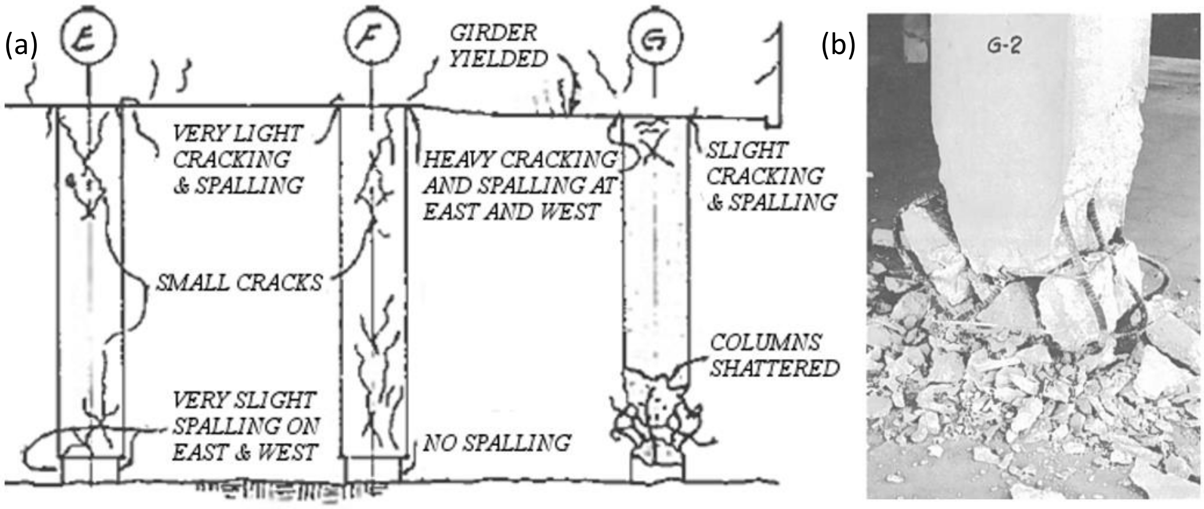

Figure 3 – Illustrations of damage showing: (a) detailed damage description of frame line 1 at the first story; heavy shear cracking and minor spalling is apparent at the base of the first-story columns; (b) damage to columns on grid line G resulting in 9 in. of shortening of eastern-most bay (diagram and picture from [16]).

2. Methods

To benchmark the outcomes from an ASCE 41 evaluation of this building, a 3D nonlinear numerical model of the Imperial County Services Building is developed according the modeling guidelines in ASCE 41-17. Predictions of drifts, accelerations, component damage, and performance acceptance are quantified according to the ASCE 41-17 Nonlinear Dynamic Analysis. Nonlinear dynamic analysis is performed using the ground motion recorded at the foundation of the building during the 1979 Imperial Valley Earthquake. The results from the analysis are compared with recordings of floor accelerations during the event, calculated displacements based on double integration of the acceleration time histories, as well as documented observations of damage to the building.

2.1 Nonlinear Simulation Model of Imperial County Services Building

A 3D nonlinear model of the Imperial County Services Building was created in the open-source simulation platform, OpenSees [20]. Material properties are based on expected material strengths and element stiffness is reduced to represent the effective cracked stiffness. These modifications are based on Table 10-5 of ASCE 41-17.

Beams and columns are modeled as elastic beam-column line elements with lumped-plasticity rotational springs. This type of model is one of the allowable ASCE 41 approaches for reinforced concrete beam-column elements. The rotational springs follow the Ibarra-Medina-Krawinkler deterioration model with peak-oriented response [21]. No cyclic deterioration is modeled in the plastic hinges. Column nonlinear deformations are determined from Table 10-8 of ASCE 41-17 similar to the backbone shown in Figure 4a. This table assigns modeling parameters based on the column axial loads, transverse reinforcement ratio, and the ratio of flexure to shear strength. Column flexural strengths are calculated from a moment-curvature analysis using the maximum axial loads from a limit state analysis which pushed a nonlinear model of the building out to loss of lateral load carrying capacity (1.5% roof drift in the E-W direction). Due to the effect of the ground slab, the transverse reinforcement used in these calculates for the base of the first-story columns are based on the reinforcement above the ground slab (spaced at 12 inches). This creates first-story columns that are shear critical at their base, and flexure controlled at the top. All columns above the first story are flexure controlled. To account for post yield strain hardening, the peak strength (point C on Figure 4a) is calculated using material properties equal to 1.15 times the expected strength properties for flexure controlled elements. For shear controlled elements, no post-yield strain hardening is modeled. Once the point of failure is reached (point E on Figure 4), columns and beams retain 5% residual lateral strength to help maintain convergence out to large drifts. All beams are controlled by flexure and included stiffness and strength contributions of the floor slabs. The cantilever sections of the beams that support the upper story exterior walls are modeled as elastic.

Joints elements connect the beams and columns using elastic beam-column elements to represent finite joint size that are connected by a rotational spring at the center with nonlinear properties determined from Table 10-11 of ASCE 41-17. To account for joint stiffness, some of the elastic elements in each joint are modeled as rigid and some use the elastic properties of the beam and column elements, depending on column-to-beam strength ratio. Column-to-beam strength ratios at the top of the first story range from 1.0 to 1.2. For interior joints at all stories, rigid elements go from the center of the joint to halfway between the joint center and joint edge for both the vertical and horizontal directions. For exterior joints at all stories, the vertical elements are modeled as rigid for the full height of the joint and the horizontal elements are modeled as elastic for the full length of the joint.

In their in-plane (N-S) direction, walls were determined to be shear controlled and are modeled as line elements with shear springs. Wall nonlinear deformations are determined from Table 10-20 of ASCE 41-17 similar to the backbone shown in Figure 4b. These properties depend on the longitudinal reinforcement, the cross-sectional area, and axial loads of the wall. First-story walls are also modeled nonlinearly in the out-of-plane direction to account for the added stiffness and strength they provide to the E-W frame lateral resistance.

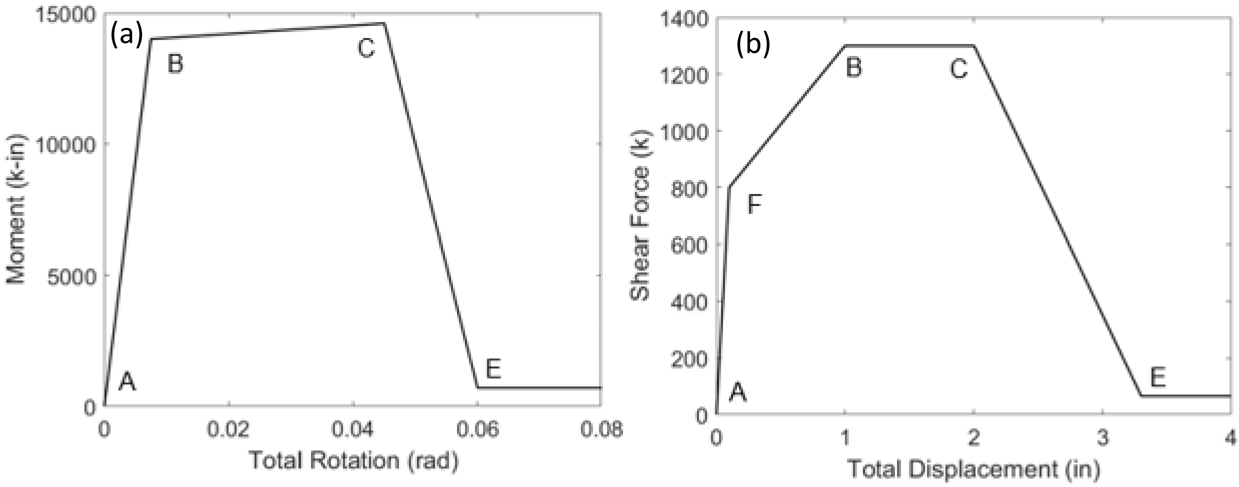

Figure 4 - Modeled nonlinear deformation behavior of (a) the upper first-story hinge of column B1, and (b) the western-most wall on the first story. Total rotation of each hinge is defined as the sum of the elastic rotation of half of the column and the plastic rotation of the hinge.

Figure 5 shows the total deformation capacity (point E on Figure 4) for the first-story columns and walls determined for the Imperial County Services Building from ASCE 41-17 modeling guidelines. There is a significant difference in deformation capacity between the base of the first-story columns and the top of the first-story columns, due to the wider spacing of transverse reinforcement resulting in shear critical behavior of the column bases. For columns, the CP limit state is taken as 70% of the plastic deformation capacity (point E) of the element. For shear controlled walls, the CP limit state is taken as the total deformation capacity of the element (point E on Figure 4b). During the assessment, if the deformation demand of any element exceeds the CP deformation limit state, the building is flagged as failing the CP performance objective.

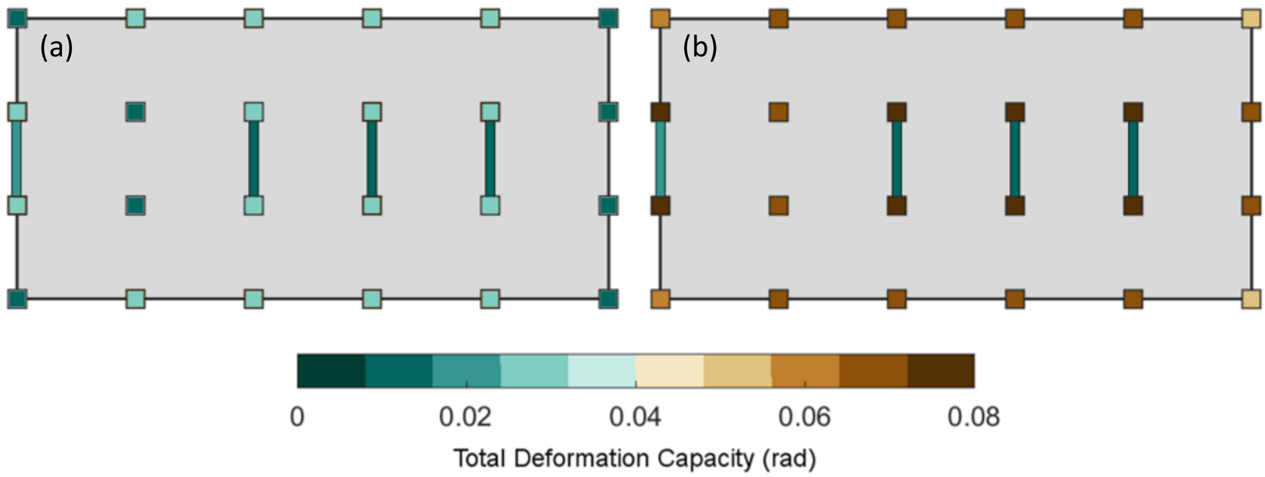

Figure 5 - Total deformation capacities in the backbone curves for the first-story walls and (a) the base of the first-story columns and (b) the top of the first-story columns.

The analysis employed 3% Rayleigh damping anchored at both the fundamental periods of the building in the E-W and N-S directions. It accounted for large geometry effects using the P-delta transformation. A 5% mass eccentricity is modeled towards the north-east corner of the building to account for accidental torsion with a rigid diaphragm. To avoid numerical convergence issues in the dynamic analysis, a convergence solution is implemented that iterates on alternative solution algorithms, reduces the earthquake time step, and increase solution tolerance until convergence or instability is reached.

2.2 Simulation Model Including Foundation Flexibility

ASCE 41 suggests "the base of a column [for pile foundations] can typically be modeled as fixed" (ASCE 41-17 C10.12.2) [9]. However, ASCE 41-17 guidelines also instruct engineers to consider foundation flexibility and soil-structure interaction (SSI) when "fixed or pinned boundary elements cannot be justified" (ASCE 41-17 10.12.2) [9]. As previous studies have contributed some of the N-S response of the Imperial County Services Building to soil-structure-interaction [19], both a fixed base model and a flexible foundation model are assessed to explore the effect that modeling foundation flexibility has on the response of the building.

For further guidance on the inclusion of SSI, this study turns to the NIST Report GCR 12-917-21 [22]. The structure-to-soil stiffness ratio check is used to determine if the effects of soil-foundation-structure interaction should be considered, where the structure-to-soil stiffness ratio is the ratio of the height of the structure to the product of the fundamental period and the shear wave velocity. If the structure-to-soil stiffness ratio is greater than 0.1 [22] suggests that SSI effects be considered. For the Imperial County Building, the structure-to-soil stiffness ratio is 0.45 in the N-S direction, indicating that SSI effects should be considered.

Pile foundations are represented by translational, rotational, and axial springs at the pile cap location. The properties of the axial and rotational springs were determined by Equations 8-13 and 8-14 of ASCE 41-17, respectively. The properties of the translational springs were determined from P-Y analysis of individual piles using OpenSees. The bases of the first-story walls are supported by grade beams framing into the pile caps as well as axial springs intended to represent of the soil bearing resistance. For this model, the free-field acceleration recording, rather than the foundation recording, was used to represent the earthquake excitation.

3. Results

This sections presents the results from the ASCE 41-17 nonlinear dynamic analysis and the Incremental Dynamic Analysis of the Imperial County Services Building. To benchmark the ASCE 41 modeling guidelines, peak accelerations and drifts from the nonlinear dynamic analysis are compared with recorded results. To benchmark the ASCE acceptance criteria, the hysteretic response of the components are compared with documented observations of component damage.

3.1 ASCE 41 Nonlinear Dynamic Assessment

Figure 6 compares the acceleration and displacement profiles between the instrumentation recordings and the simulation in both the E-W and N-S directions. In the E-W direction, accelerations show good agreement (difference < 12 %) over the height of the building, except at the second floor where the model produces a 33% overestimation of the accelerations. Displacement estimates for the first story show good agreement (< 9 % difference), although the roof displacement is underestimated by the model by 18%. The underestimation of upper story drifts in the E-W direction is explained by three factors. First, models of the deep beams (54 inches deep) on the upper-story exterior frame lines only account for flexural deformations, and shear deformations, which are estimated to be as high as 10% of the flexural deformations, are neglected. Secondly, the implicit stiffness model of the joints as outlined in ASCE 41-17 10.4.2.2.1 can overestimate the stiffness of joints with deep beams. Lastly, the base of first-story columns appear to fail prematurely in the model compared to the real building (see discussion below), leading to early concentration of damage in the first story of the model, effectively protecting the upper stories and reducing upper story displacements. In the N-S direction, roof displacements are substantially underestimated (by 50%). Soil-structure-interaction is more significant in that direction, the effects of which are discussed below. This interaction also contributes to the high accelerations at the second-floor level shown in Figure 6 in the N-S direction.

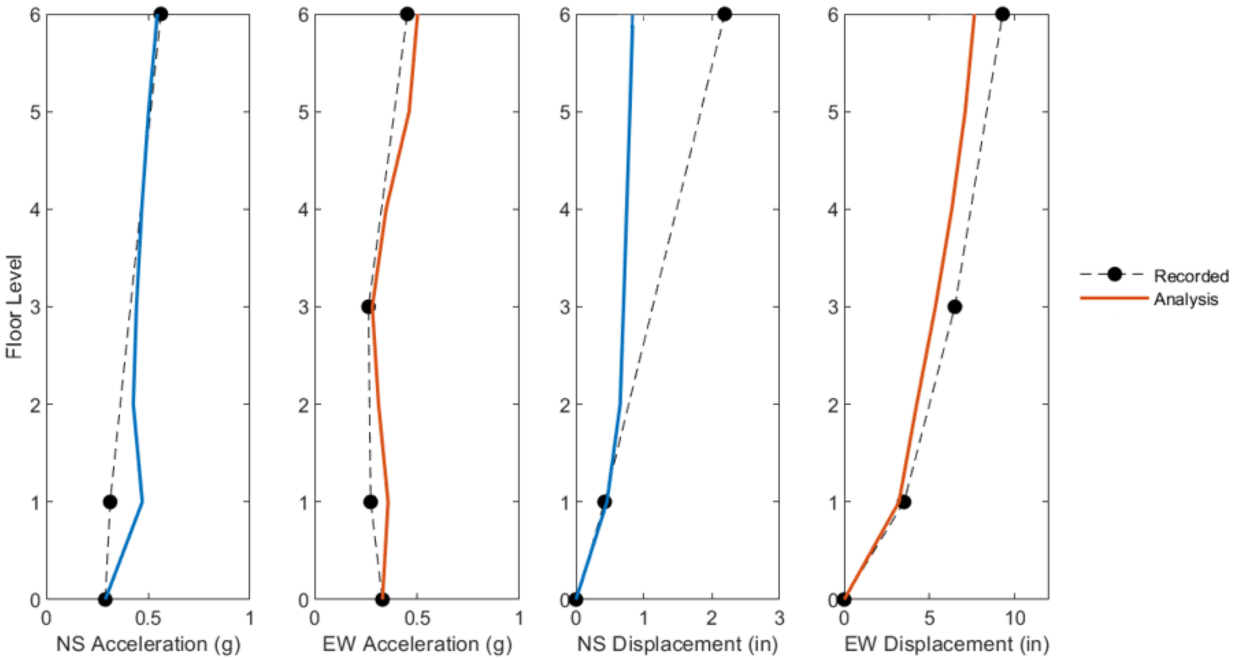

Figure 6 - Comparison of recorded and simulated peak acceleration and displacement profiles. These displacements are measured at the center of building in plan in each direction.

The distribution of damage over the height of the building is illustrated by comparison to ASCE 41-17 acceptance criteria in Figure 7. The model predicts that the columns fail CP at the base of the first-story columns, and IO at the top of some of the first-story columns, effectively creating a pinned-base response. Post-earthquake reconnaissance reported little to no damage in the upper stories, and the model response is consistent with these reports. The model predicts that most of the beams did not yield, although the second-floor beams failed the IO acceptance criteria. Damage surveys found only minor cracking in beams throughout the building. No damage was reported to joints in any of the available damage surveys, and no damage to joints was indicated by the analysis.

The building underwent significant torsion due to an asymmetric wall layout, which was evident in recordings and in the model. However, the peak twist (corresponding to about 1.5 inches of additional displacement at the first-story eastern most column line) was underestimated by the model by about 50%. Previous authors [15] estimated a torsion period of 0.35 s before the earthquake and 0.43 s after the earthquake. The torsional period of the model of 0.23 s is less than the ambient recordings, at least partly due to the fixed-based of the model. The model predicts that the first-story walls in the N-S direction fail IO acceptance criteria, consistent with observations of wall cracking from damage surveys.

The response of the exterior first-story column G-2 is examined in more detail in Figure 8. These results show that, at the column base, the modeled column has lost practically all of its strength and stiffness, especially in the E-W direction. At the top of the column, however, where transverse reinforcement spacing is much narrower (2 – 3 in.), the columns have just reached their flexural capacity in the model in the E-W direction. These results seem qualitatively consistent with the damage photos, which showed limited flexural cracking at the tops of the columns. The model predicts the least ductile behavior and response most beyond the total rotation capacity (point E on Figure 4a) at the base of the columns on the eastern-most column line, due to the higher axial loads and consistent with observations of damage. The model response also indicates that all columns have exceeded their total rotation capacity and lost gravity-load bearing capacity, according to the definition of the ASCE 41 backbones. However, the observed damage would indicate that only the eastern most columns have lost gravity-load bearing. This would imply that the ASCE 41 damage assessment of the first story columns is conservative for most columns other than the eastern-most columns.

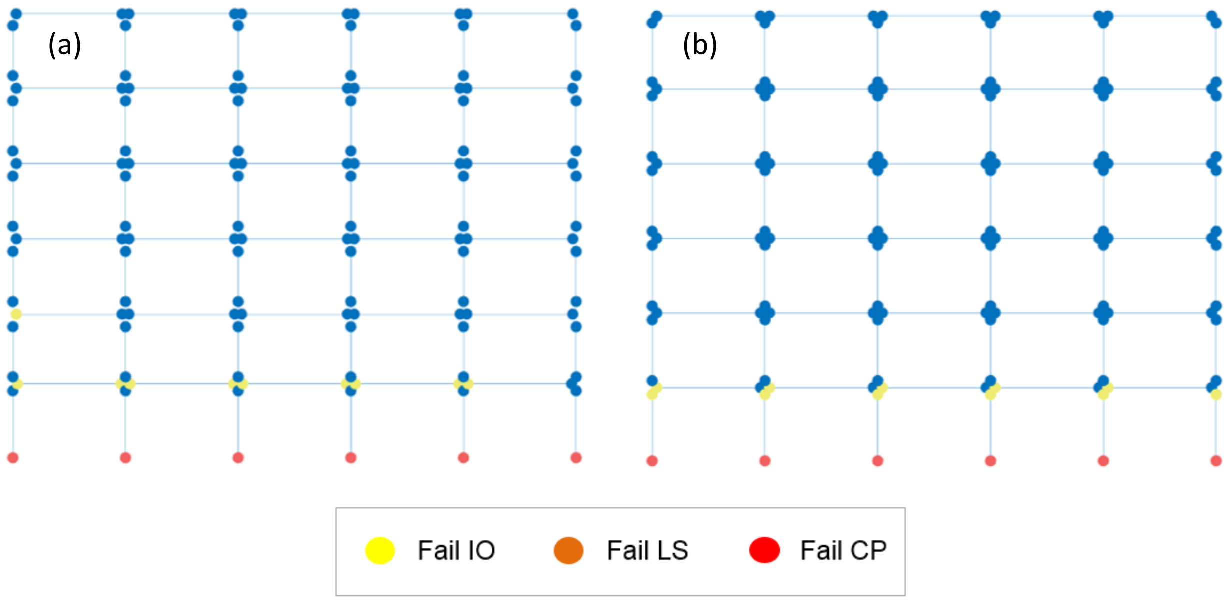

Figure 7 - Assessment of column and beam response relative to ASCE 41-17 nonlinear acceptance criteria for (a) exterior frame line 1 and (b) interior frame line 2.

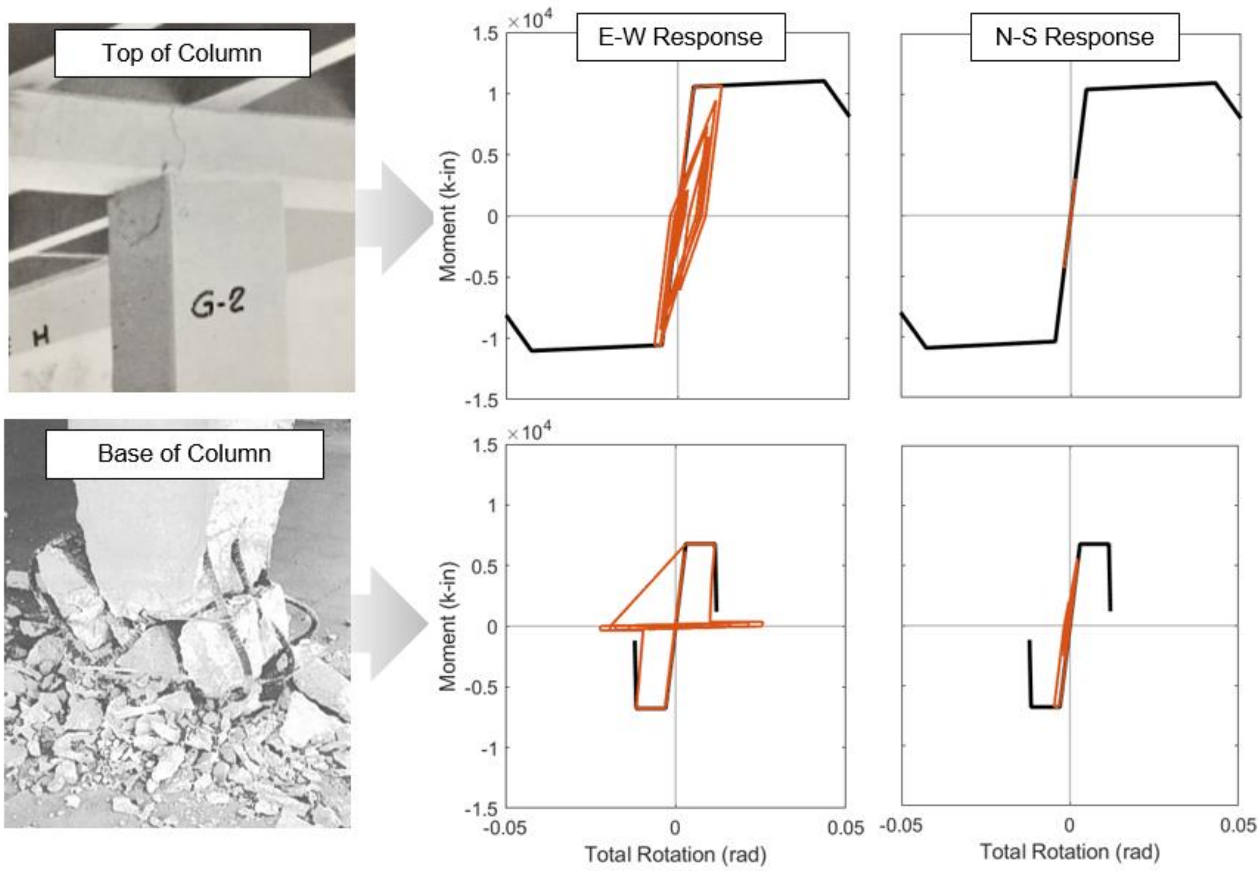

Figure 8 - Damage photos from [16] of column G-2, with model backbones and hysteretic response in the recorded ground motion. Total rotation of each hinge is defined as the sum of the elastic rotation of half of the column and the plastic rotation of the hinge.

3.2 Effects of Soil Structure Interaction

As anticipated, including the foundation flexibility effects of SSI improves the estimation of global response in the N-S direction. In both the N-S and E-W direction, peak roof drifts are closer to the recorded roof drifts for the SSI model as compared with the fixed-base model. While the peak roof drift in the N-S direction is still underestimated, the error reduces from around 50% underprediction to around 35%. The torsional period increased from 0.23 s to 0.31 s and the amplitude of the torsional response in the N-S direction provides a closer match to the response observed (maximum errors in torsional amplitude reduced from about a 67% to about 50%). Figure 9 shows that the effects of the soil-structure interaction/foundation flexibility also provide more realistic representation of the building frequency content (as interpreted from spectra computed from roof accelerations). Component response and damage is very similar between the fixed-base and the SSI model, except for slightly reduced forces in the first-story N-S walls of the SSI model due to deformations in the pile foundations.

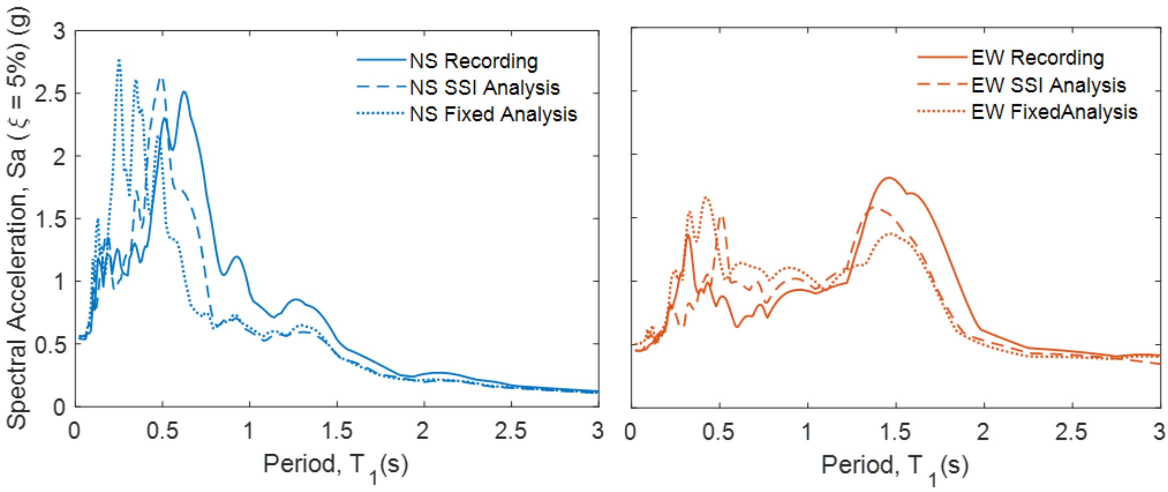

Figure 9 - Comparison of roof acceleration spectra from models with and without soil-structure-interaction. These spectra are computed from the accelerations recorded at the center of the roof in each direction.

4. Conclusions

This study presents comparisons between an ASCE 41-17 nonlinear dynamic analysis of the Imperial County Services Building and recorded and observed building response and damage from the 1979 Imperial Valley Earthquake. ASCE 41 is commonly used for seismic evaluation and retrofit of reinforced concrete buildings. This building provides an opportunity to compare observed response and damage to the outcomes of the ASCE 41-17 standard modeling and acceptance criteria, benchmarking that document.

Overall, the ASCE 41 damage assessment of the Imperial County Services Building successfully identifies the concentration of damage in the first-story columns such that a satisfactory retrofit could be implemented. The assessment also correctly identifies limited damage to components above the first story. The peak displacements of the fixed based model match the recorded displacements in the first story well, and the model response captured the high axial loads on the east corner columns and simulated, albeit underpredicted, the torsion response of the east side. Torsional response and roof displacements are improved when foundation flexibility is modeled. The ASCE 41 damage assessment also identifies more damage to some components than was observed. In particular, the results of the nonlinear ASCE 41 assessment indicate that close to all of the first-story column bases are well beyond their rotation capacity, indicating the loss of the ability to resist axial load. In the actual building, only the 4 east side columns showed loss of axial capacity.

These observations, based on an instrumented building, show that ASCE 41 models can provide good response predictions. In addition, the results confirm findings from previous studies, which indicate the ASCE 41 damage assessment provision (i.e., the CP acceptance criteria) may provide conservative predictions. When ASCE 41 is used as a design tool, a conservative estimate of damage may be desirable and can likely result in an acceptable retrofit design. However, the relationship between component-based acceptance criteria and global performance depends on building-specific characteristics, such that the the conservatism provided by the assessment provisions may vary from building-to-building. Therefore, it is recommended that more work be done to quantify how structural response and building characteristics affect the conservatism in component-based response metrics.

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