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NON-LINEAR TIME HISTORY ANALYSIS OF REFINED MESH STEEL STRUCTURES

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ABSTRACT

Non-linear time history analysis of steel structures has traditionally utilized models consisting of one dimensional elements that have discrete hinge regions or non-linear springs to simulate inelastic behavior. The non-linear behavior of these hinges or springs must be known prior to the analysis to incorporate them into the model. Very different models are typically used in investigating steel connection performance. Such models typically utilize shell or brick elements subjected to static monotonic or static cyclic loading.

This study investigates non-linear time history analysis of models where connection regions are modeled explicitly with shell or brick elements. These analyses are computationally expensive, but provide important insight to seismic performance. Low cycle fatigue failure criteria based on local stress and strain data can be used in conjunction with such models. These models capture strength degradation associated with yielding and buckling (local and global). The predictive power of these models exceeds that of one dimensional element models where non-linear regions must be pre-supposed. This paper compares results obtained using these advanced modeling techniques, with those obtained from more traditional time history analysis.

Introduction

The usual goal in the seismic design of steel buildings is to provide frames that remain elastic under mild earthquakes and will not collapse under strong ones. This is achieved economically by designing structures with an appropriate level of stiffness and strength to resist small earthquakes elastically and with sufficient ductility to survive the inelastic deformations produced by strong events. For steel buildings, ductility is provided through a desirable inelastic mechanism with yielding confined to carefully selected locations of the frame. In most steel systems yielding is expected to occur in or near connection regions. Connections are usually designed so that the ultimate system failure mode is low cycle fatigue material failure of the intended yielding elements.

Traditional Approaches to Connection Verification

A traditional approach to verifying connection performance under seismic loading is indicated in Fig. 1 (next page). The work done by the SAC project to develop new steel moment frame connections

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following the 1994 Northridge California earthquake typifies this approach (SAC 2000). The traditional approach of Fig. 1 will be discussed briefly with corresponding references to the SAC work as examples.

The first two steps of the traditional approach deal with connection demands (Fig. 1). Connection deformation demands under earthquake loads are investigated using building models subjected to a suite of earthquake acceleration time histories (Gupta and Krawinkler 1999). The models used for this type of analysis consist of one dimensional (1D) elements with discrete hinge regions or non-linear springs to simulate inelastic behavior [Fig. 2(a)]. Using results from the time-history analyses a simplified loading protocol, usually consisting of symmetric cycles of increasing displacement magnitudes, is developed to reflect appropriate cumulative and maximum deformation demands (Clark et al. 1997, Krawinkler et al. 2000).

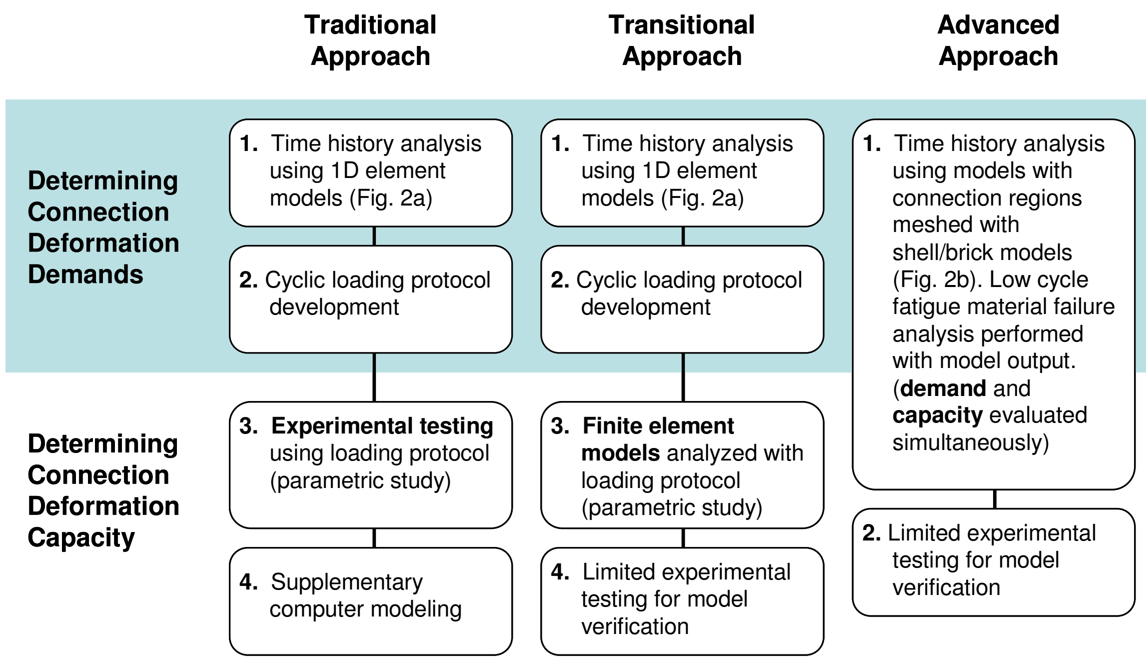
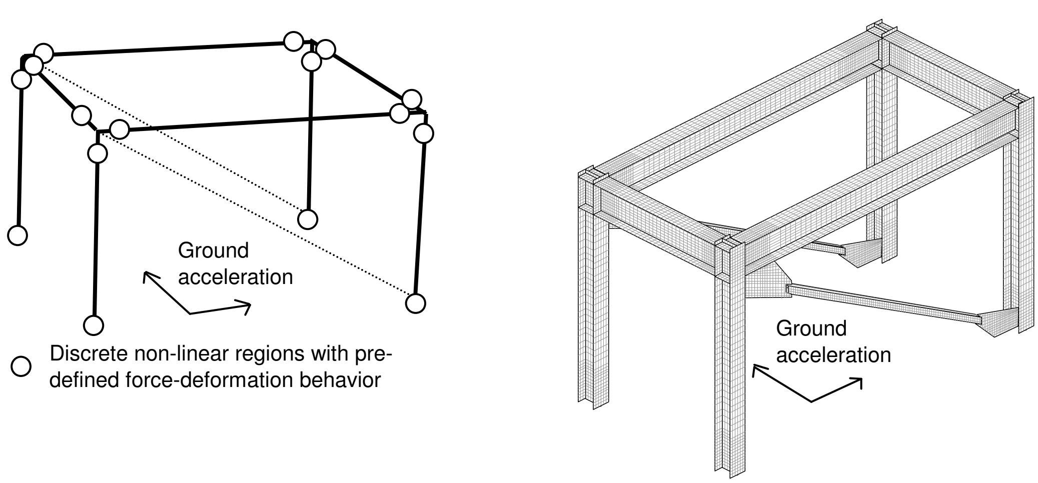


Figure 1. Procedures for verifying connection performance under seismic loading.



(a) 1D elements with lumped plasticity (b) Shell elements with inelastic material properties

Figure 2. Models used for dynamic analysis.

In step 3 of the traditional approach (Fig. 1), connection deformation capacity is investigated through experimental testing. Pseudo-static experimental testing of connections, using entire frames or subassemblies, is conducted following the developed loading protocol (Chi and Uang 2002; and Jones et al. 2002 for example). If the maximum connection deformation capacity is greater than the demands indicated by the time history analyses, then the connection is acceptable. In the final step (Fig. 1), analytical models are used to extrapolate experimental results to untested cases or to investigate specific failure modes observed in testing (Kim et al. 2002; and Ricles et al. 2002 for example).

While this traditional approach seems to provide satisfactory results, it has several drawbacks. First, the time history analysis and loading protocol development require significant effort. As a result generic or arbitrary protocols are often used in experimental testing. Second, large-scale experimental testing is expensive and time consuming so a limited number of cases can be considered. Finally, since experiments usually consider in-plane loading with simplified protocols; connections are usually not subjected to real multi-directional demands they would see under a real earthquake.

Other traditional methods to validate connection performance include pseudo-dynamic testing, hybrid testing, or shake table testing. These methods have the advantage of applying more realistic demands on connections. Disadvantages of these tests are the cost and time to complete testing. High costs permit only a small number of configurations to be investigated under a few earthquakes. This results in a small data set to justify a general connection design procedure.

Transitional Approach for Connection Validation

Finite element modeling has been used increasingly in recent years to address some of the disadvantages of the traditional approach. Researchers have used shell and brick element models subjected to experimental loading protocols for parametric studies (Richards and Uang 2005; Berman and Bruneau 2006; and Zhang and Ricles 2006; for example). This type of modeling is the thrust of the “transitional” approach indicated in Fig. 1.

In the transitional approach, shell or brick finite element models essentially replace many of the experimental test specimens of the traditional approach (Fig. 1). These models can simulate strength degradation associated with local buckling and provide detailed stress and strain information through-out the model. In the transitional approach, models are used for parametric study thereby increasing the amount of data and decreasing the number of experiments required. Experiments are used to validate the models and explore issues that can not be addressed in the models. Many configurations can be analyzed, with verified computer models, providing more thorough verification than experimental testing alone.

While more cost effective than the traditional approach, the transitional approach retains the loading protocol related weaknesses of the traditional approach.

Advanced Approach for Connection Verification

A new approach, designated the “advanced” approach in Fig. 1, takes advantage of current computation power and a recently validated low cycle fatigue model. In the advanced approach, a detailed model of the system is developed using shell or brick elements in the connection areas [Fig. 1 and Figure 2(b)]. This model is subjected to multi-directional earthquake ground motions. Under each ground motion, connection performance is evaluated and low cycle fatigue failure is investigated.

The advanced approach addresses the loading protocol weaknesses inherent in the traditional and transitional approaches. It provides comparable results to full-scale multi-directional shake table testing with the possibility to consider numerous geometries and suites of earthquakes at a reasonable cost.

Recent work by Chi et al. (2006) has produced a micromechanics-based model that captures the fundamental processes of void growth, collapse, and damage responsible for low cycle fatigue failure. This fatigue failure criterion has been verified using experimental data for braces that buckle and eventually fail due to low cycle fatigue tearing (Fell et al. 2006). Ductile steel systems are designed such that low-cycle material failure is the desired and expected ultimate limit state. The low cyclic fatigue failure criterion can be used with refined shell or solid element models where stress and strain states are known throughout.

The advanced approach offers several advantages over the others. The need for simplified loading protocols is eliminated; connections are subjected to real earthquake loads. This includes multi-directional loading which is almost always absent from studies using the traditional or transitional approaches. Numerous geometries and earthquake histories can be evaluated with the advanced approach since the primary costs are computational. The advanced approach makes valuable information available that cannot be obtained from current methods.

The power of the advanced analyses (Fig. 1) is that, with no other input beyond frame geometry and material properties, locations where low cycle fatigue will occur under actual earthquake loading can be predicted. The predictive power of advanced models significantly exceeds that of 1D element models because the location and performance of the inelastic regions do not need to be presupposed and actual material failure can be evaluated.

Objective

The purpose of the study discussed herein is to demonstrate the feasibility and usefulness of the advanced method. A moment frame bay is analyzed under earthquake loading using a traditional 1D element model with lumped plasticity and again using a refined mesh model as described above. By comparing results, advantages of each system can be identified and discussed.

Computer Modeling

Prototype Frame

A one-story moment-frame building (Fig. 3) was designed for Loma Prieta, California per seismic guidelines in ASCE 7-05 (ASCE 2006)(see Fig. 4 for design spectra). Only two bays on each side were considered moment frames to simplify modeling (Fig. 3). Member sizes ended up being governed by drift criteria rather than strength considerations (Fig. 5, next page). Reduced beam section (RBS) moment connections, which are typical in the western U.S., were included in the design (Fig. 6, next page).

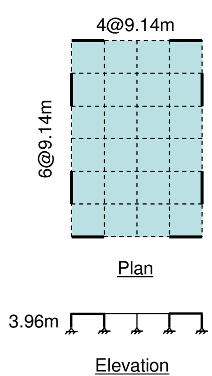


Figure 3. Prototype building.

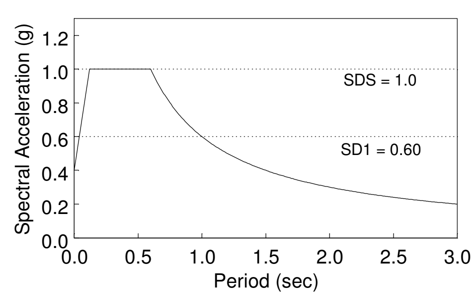


Figure 4. Design spectra.

Figure 5. Frame members.

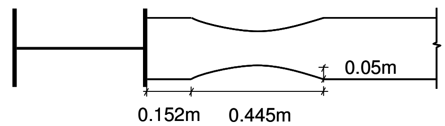


Figure 6. RBS detail.

1D Element Models

A 1D element model of the moment frame was developed and analyzed using Ruaumoko (Carr 2006). RBS hinge regions were modeled utilizing the built-in hinges in the beam element (Fig. 7). The moment-axial yield surface used for the RBS hinges is shown in Fig. 8. The area, A, and plastic section modulus, Z, varied through the RBS region so average values over the region were used in defining the plastic hinge properties. A bi-linear hysteretic model was used with post yield stiffness equal to 0.01 times the initial.

Seismic mass associated with the frame was lumped at the top of the columns (Fig. 6). Total mass associated with one frame was 259.3 kN-s^2/m. The frame was analyzed using an earthquake record from the 1994 Loma Prieta, California earthquake (Fig. 9). Two versions of the record were used for analysis: the original record (designated 1x); and the record scaled such that the response spectra would match the spectra used for design at the fundamental period of the building (designated 3.366x). Spectra for the 1x and 3.36x records are shown in Fig. 10. The 3.366x record represents a “design” level earthquake for the frame being studied (T_1 for the frame = 1.3 sec).

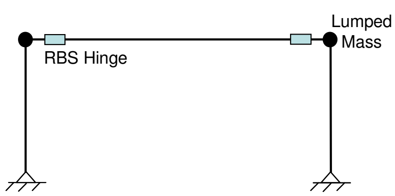


Figure 7. 1D Model with Plastic Hinges.

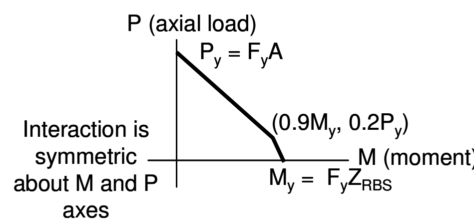


Figure 8. Yield Surface for Plastic Hinges.

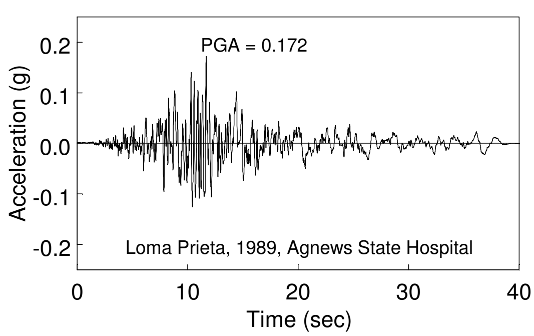


Figure 9. Earthquake record used for analyses.

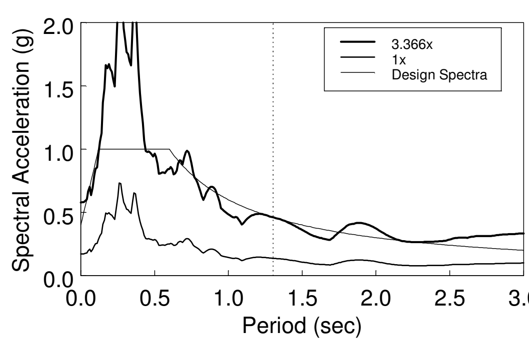


Figure 10. Spectra for 1x and 3.366x events.

Refined Mesh Model

Another model of the moment frame was developed and analyzed using ABAQUS (HKS 2006). This model consisted of a combination of 1D beam elements and shell elements (Fig. 11). At locations where 1D beam elements connected to the shell mesh, rigid body constraints force the shell section to match the rotations and translations at the end of the 1D beam element. The mesh was refined in the region with the greatest potential for low cycle fatigue failure (Fig. 12).

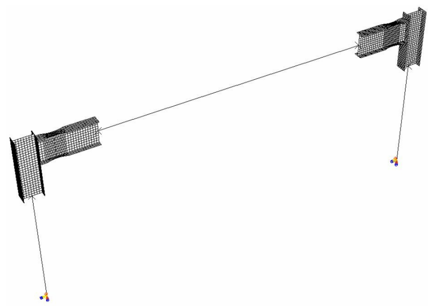


Figure 11. ABAQUS model with refined mesh in connection regions.

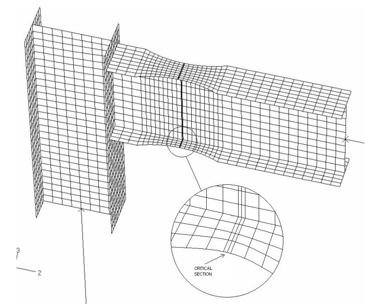


Figure 12. Refined mesh at region most susceptible to low cycle fatigue.

For this model material properties and geometry were the only input. Material properties were based on cyclic coupon testing of A575 steel (Kaufmann et al. 2001). The material plasticity is based on a Mises yield surface and non-linear kinematic hardening. This model was subjected to the same ground accelerations (in plane) as the 1D element model (previous section).

Failure Index

A stress modified critical strain (SMCS) criteria was applied to analysis output from the ABAQUS model. The failure index used to predict low-cycle ductile fracture is the accumulated equivalent plastic strain (PEEQ in ABAQUS) divided by a critical plastic strain. The critical plastic strain is taken as:

$$\varepsilon_{\mathrm{p,critical}} = \alpha \ \exp\left(-1.5\frac{\sigma_{\mathrm{m}}}{\sigma_{\mathrm{e}}}\right) \tag{1}$$

(Hancock and Mackenzie 1976), where σ_m is the mean stress, σ_e is the effective stress, and α is a material constant. For the purpose of this study, α was taken as 2.6 (Chi et al. 2006). When the equivalent plastic strain exceeds the critical value (a failure index greater than one) low cycle fatigue failure occurs.

This procedure for predicting low cycle fatigue failure was verified using experimental results from RBS connection testing (Richards and Uang 2002). A finite element model of the experimental specimen was subjected to the same loading protocol as used in testing. The failure index was computed for the critical element at the middle of the RBS cut. The failure index reached a value of one at the point of the analyses that corresponded to the point in experimental testing where low cycle fatigue failure was observed.

Stress and strain data from the center of the RBS cut (Fig. 12) were used in computing the failure index for the ABAQUS model investigated dynamically in this study.

Results

Frame Drifts

Figure 13 shows the roof drift for the Ruaumoko and ABAQUS models under the 1x event. The frames remained essentially elastic. From Fig. 13, responses are similar for Ruaumoko and ABAQUS models. with peak roof drifts for the different models within ten percent. The difference in responses is due to the RBS cut not being represented in the elastic response of the Ruaumoko beam element.

Figure 14 shows the roof drift for the Ruaumoko and ABAQUS models under the 3.366x event. Recall the 3.366x event represents a design earthquake. Under the 3.366x event, the frames experiences yielding in the RBS regions. Differences in the results are due to approximations made to describe the RBS behavior using the plastic hinges in the Ruaumoko model. For the Ruaumoko beam element, hinge properties cannot vary through the plastic region, so average properties were used in describing the RBS region. In spite of these inaccuracies, agreement between the models is reasonable with peak drifts within ten percent. Similar results could be shown for velocity and acceleration demands.

The refined mesh model (ABAQUS) offers little or no advantage over the simpler 1D element model (Ruaumoko) in determining global displacement, velocity, or acceleration demands (Figs. 13 and 14). The power of the advanced model is in failure prediction capabilities.

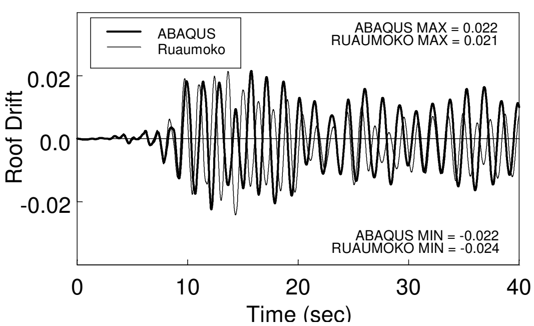


Figure 13. Roof drift under 1xrecord.

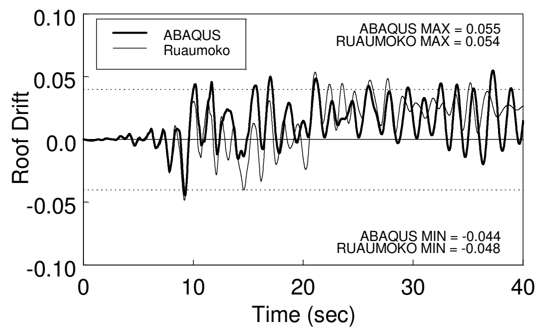


Figure 14. Roof drift under 3.366xrecord.

Failure Prediction

Given the results of the Ruaumoko model in Fig. 14, an engineer could easily conclude that the RBS connections of the frame would have failed or be close to failure during the 3.366x event. This judgment would be based on experimental observations that RBS connections typically fail due to low cycle fatigue at drifts between 0.04 and 0.05 (Richards and Uang 2002).

It should be remembered; however, than in experimental testing connections are subjected to a cyclic loading protocol and experimental results are loading protocol dependent. Estimating connection failure based on only maximum drift, neglects the cumulative deformation effects.

A better approach to estimating failure with the Ruaumoko model output, might be to compute cumulative drifts and compare that with cumulative drifts observed from experimental testing. However, it is known that large cycles are more damaging that multiple small cycles with the same cumulative drift (Krawinkler et al. 2000) and response of connections vary depending on the beam section. Connection specific experimental testing must be used with the Ruaumoko model output to predict failure with confidence.

The refined mesh model provides more powerful failure prediction capabilities. Stress and strain data from the ABAQUS model can be used with mechanics-based failure criteria that are independent of loading protocol and section geometry. Figure 15 illustrates the cumulative plastic strain, PEEQ, for the connection at the end of the 3.366x event. As expected, the greatest strains occur in the middle of the RBS cut. In Figure 16 the failure index is plotted for the critical element in the RBS (see Eq.1 and associated discussion). The maximum value for the failure index is 0.294. This indicates that the RBS connection was not close to experiencing low cycle fatigue, and would be able to perform adequately under two more earthquakes of similar intensity.

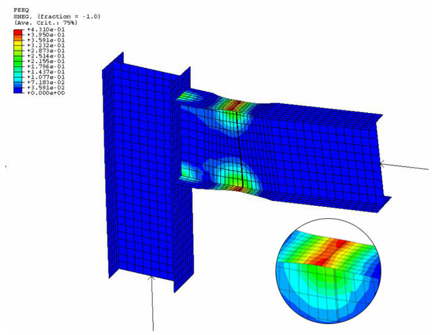


Figure 15. Cumulative plastic strain at end of earthquake.

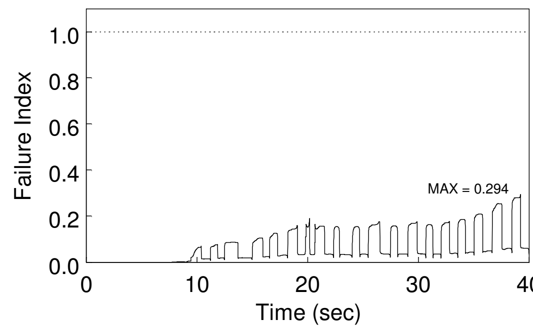


Figure 16. Failure index at RBS connection.

Computational Demands

The analysis of the ABAQUS model under the 3.366x event took 8.5 hrs to complete on a Unix HP-UX machine while the corresponding Ruaumoko analyses took less that one second to complete on a PC. For determining drifts, velocities, and accelerations the simpler models with 1D elements and lumped plasticity are efficient and reasonably accurate.

The power of the advanced models is in determining connection failure without experimental testing. The computational costs of the advanced analyses should be compared against the costs of dynamic testing of full-scale systems. Even if supercomputing capabilities are required, the time and expense to perform the advanced analyses are small compared to the time and expense associated with comparable experimental testing.

Conclusions

Traditional methods for connection verification rely on experimental testing. Experimental methods that subject connections to realistic dynamic loading include shake table testing, pseudo-dynamic testing, or hybrid testing. It is unrealistic to conduct full-scale dynamic experiments to verify current and new connections and design practices. Most experimental testing of connections is pseudo-static using simple loading protocols.

Advanced finite element analyses provide a relatively low-cost means of investigating connections under realistic dynamic loads. Current computation capabilities permit dynamic analysis of finite element models with refined shell element meshes in connection regions. When these analyses are coupled with low cycle fatigue failure criteria, it is possible to directly predict if and where low cycle fatigue failure will occur during a specific event. These advanced finite element models provide much more information than traditional models used for dynamic analyses that consist of one-dimensional elements with lumped plasticity.

In this paper, a case study was presented comparing analysis results from traditional dynamic analysis (1D elements with lumped plasticity) with the results from dynamical analysis of advanced models (shell elements in connection regions). From this case study, the following conclusions can be made:

1. Simple and advanced models give similar results for overall system drifts, velocities, and accelerations; the computational effort of advanced models is not justified if these are the only

required results.

2. Maximum drift or connection rotation is frequently used as a failure criteria with traditional models. These criteria do not account for cumulative effects. Maximum drift criteria inaccurately implied failure for the case study considered.
3. Using advanced models coupled with a low cycle fatigue failure criteria, the plastic strain demands on connections can be quantified and failure can be accurately predicted.
4. Through advanced dynamic analysis connections can be validated under real earthquake loads, rather than simple loading protocols.

The case study demonstrates the feasibility of using dynamic analysis of advanced models in researching connections. This is an ideal tool to explore issues that have been difficult to address in the laboratory or with traditional dynamic analysis. Such issues include: multi-directional loading of steel frames; gusset plate performance under dynamic loads; validation of new steel connections and systems; and damage assessment of steel buildings that have experienced some earthquake damage (determination if connections need to be repaired or if they have adequate capacity to withstand another event).

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