Mechanism of collapse of tall steel moment frame buildings under earthquake excitation

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Abstract

The mechanism of collapse of tall steel moment frame buildings is explored through three-dimensional nonlinear analyses of two 18-story steel moment frame buildings under earthquake excitation. Both fracture-susceptible as well as perfect-connection conditions are investigated. Classical energy balance analysis shows that only long-period excitation imparts energy to tall buildings large enough to cause collapse. Under such long-period motion, the shear beam analogy alludes to the existence of a characteristic mechanism of collapse or a few preferred mechanisms of collapse for these buildings. Numerical evidence from parametric analyses of the buildings under a suite of idealized sawtooth-like ground motion time histories, with varying period ($T$), amplitude (peak ground velocity, $PGV$), and duration (number of cycles, $N$), is presented to support this hypothesis. Damage localizes to form a “quasi-shear” band over a few stories. When the band is destabilized, sidesway collapse is initiated and gravity takes over. Only one to five collapse mechanisms occur out of a possible 153 mechanisms in either principal direction of the buildings considered. Where two or more preferred mechanisms do exist, they have significant story-overlap, typically separated by just one story. It is shown that a simple work-energy relation applied to all possible quasi-shear bands, combined with plastic analysis principles can systematically identify all the preferred collapse mechanisms.

1. Introduction

Ever since the collapse of two tall steel buildings in the 1985 Mexico City earthquake, there has been sustained interest in understanding the response of tall steel buildings under strong earthquake excitation through simulations. Steel construction is not common in Mexico City. Of the few isolated steel buildings existent there at the time of the 1985 earthquake, two 14- and 21-story towers out of five tall steel frame buildings collapsed in the Conjunto Pino Suarez apartment complex, and a third 21-story tower, although remaining standing, was leaning six feet out of plumb at the roof level. These collapses have been attributed to the strong amplification of the lake bed on which parts of the city are located and the long-period nature of the ground motion [1]. The amplification was enabled by the long duration of the main shock resulting in a resonant buildup of seismic waves within the thick clay layer just beneath the surface. The three identical 21-story structures consisted of five 6m-bays in the long direction and two 6m-bays in the short direction. The lateral force-resisting system consisted of moment frames at all column lines with two bays in the short direction braced using X braces, and one bay in the long direction braced using V braces [40]. Collapse is deemed to have occurred due to weld failure in the built-up box column and subsequent local buckling of the flanges. In addition to demonstrating that even steel highrise

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structures can collapse, this earthquake brought to the fore the possibility of upper-story collapses in midrise and highrise structures, due in part to the pounding of adjacent buildings, drastic tapering of columns in the upper stories, and more generally to the dynamics of the structural response.

The $M_w = 6.7$ 1994 Northridge earthquake caused fractures to occur in the connections of many steel moment frame buildings [11] including many that were tall, but the shaking was not intense enough to cause collapse (most of the energy was directed away from the LA basin into the Santa Susanna mountains [52]). The effort by the Structural Engineers Association of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering [42, 43, 44] to understand the causes of damage to moment-frame buildings during this earthquake provided a huge thrust to computationally modeling damage in steel buildings. Challa and Hall [5] conducted a series of 2-D nonlinear analyses on a 20-story building using six idealized pulse-like ground motions that are similar to pulses in near-fault ground motion records. MacRae [35] and Gupta and Krawinkler [12, 13] analyzed 20-story steel moment frame models under a suite of 20 ground motion records compiled by Somerville et al. [45] and determined the structural response as a function of the spectral acceleration at the fundamental period of the building. The buildings were designed to be hypothetically sited in Los Angeles, Seattle, and Boston. To study the effect of frequency content on the structural response, MacRae modified the time step-size of the record to modulate the period of peak spectral response relative to the fundamental period of the structure. He found that large drifts occurred in the first story when the period of peak spectral acceleration was greater than the fundamental period of the structure, and in the upper stories when there was a large spectral response at a shorter period than the fundamental period of the structure. Gupta and Krawinkler reported that the distributions of story drifts over height depends strongly on ground motion and structure characteristics, and that no simple rules could be developed to predict the location of greatest drifts. Medina and Krawinkler [36, 37] evaluated the drift demands and their associated uncertainties in non-deteriorating regular moment resisting frames subjected to ordinary ground motions, i.e., without near-fault effects. They reported a migration of the location of the peak interstory drift ratio (IDR) from the top story to the bottom with increased levels of inelasticity in the frame. Krishnan [24] studied the response of four 19-story steel moment frame buildings with irregular configurations under near-source ground motion from the 1994 Northridge, the 1995 Kobe, and the 1978 Iran earthquakes. He alluded to the possible connection between the period of the near-source pulse and the location of the region of greatest inelasticity following the uniform shear beam analogy to steel moment frame buildings [17]. More recently, incremental dynamic analysis [51] has been used to assess the collapse capacity as a function of the spectral acceleration at the fundamental period of the building (e.g., [54, 55, 56]). In this approach, a structure is analyzed under progressively scaled-up ground motion records and its response is computed. The spectral acceleration level at which structural response becomes unbounded is the collapse capacity of the building. Finally, the emergence of rupture-to-rafters simulations [31] have led to extensive investigations of the performance of tall steel moment frame buildings under large simulated earthquakes in Los Angeles, San Francisco, and Seattle (e.g., [28, 29, 39, 53, 38]). Conducting experiments on full-scale models of tall buildings is beyond the existing capacities of existing
large-scale testing programs such as the Network for Earthquake Engineering Simulation (NEES) in the US and the E-Defense program in Japan. The closest that researchers have come to emulating tall building response on a shake-table has been a recent series of large-scale tests in Japan [7, 32]. The test specimen consisted of a four-story steel moment frame topped by three layers of concrete slabs sitting on rubber bearings that represent the upper stories of the 21-story prototype structure. This assemblage when subjected to long-period motion exhibited cumulative ductilities more than four times that considered in the design. The fracturing of beam bottom flange welds continues to be of great concern in multi-cycle long period excitation.

All these studies provide insights into the nature of tall steel moment frame building to earthquake excitation. But they are not comprehensive and systematic in exploring the evolution of damage and the formation of the mechanism of collapse. Questions such as “where does damage localize over the building height?”, “does this depend on the frequency content and amplitude of ground shaking?”, “can a relationship be developed between these features of the ground motion and the region of damage localization?”, “could there be more than one damage localization region?”; “if so, when and which of these regions evolves into a collapse mechanism?”, “are there any preferred mechanisms of collapse in these buildings?”, “if so, can these be determined independent of the ground motion?”, have not been answered in a comprehensive and conclusive manner. Here, we make an attempt to address these questions through computational case-history studies of two 18-story steel moment frame buildings and their variants.

The first building is an existing 18-story office building, located within five miles of the epicenter of the 1994 Northridge earthquake. An isometric view of its FRAME3D model is shown in Figure 1[A]. It was designed according to the 1982 UBC [18] and completed in 1986-87. The height of the building above ground is 75.7 m (248’ 4”) with a typical story height of 3.96 m (13’ 0”) and taller first, seventeenth, and penthouse stories. The lateral force-resisting system consists of two-bay welded steel moment-frames, two apiece in either principal direction of the structure as shown in Figure 1[C]. The location of the north frame one bay inside of the perimeter gives rise to some torsional eccentricity. Many moment-frame beam-column connections in the building fractured during the Northridge earthquake, and the building has been extensively investigated since then by engineering research groups [42, 6, 3]. Fundamental periods, computed assuming 100% dead load and 30% live load contribution to the mass, are 4.52s (X-translation), 4.26s (Y-translation) and 2.69s (torsion). We consider two models of the existing building, one with connections susceptible to fracture, and the other with perfect connections. Two orthogonal orientations (with respect to the strong component of the ground motion) are considered for the model with perfect connections. Greater details of the structural modeling are provided in Appendix A.

The second building, a FRAME3D model of which is shown in Figure 1[B], is similar to the existing building, but the lateral force-resisting system has been redesigned according to the 1997 UBC [19]. It has been designed for larger earthquake forces and greater redundancy in the lateral force-resisting system, with 8 bays of moment-frames in either direction (although lateral resistance will likely be dominated by the three-bay moment frames shown in Figure 1[D] as opposed to the single-bay moment frames). The frame located in the interior
of the existing building 1 has been relocated to the exterior, eliminating the torsional eccentricity. Of course, torsion may still occur in the building as a result of differential yielding in the moment frames. Fundamental periods, computed assuming 100% dead load and 30% live load contribution to the mass, are 4.06s ([X+Y-] translation), 3.85s ([X+Y+] translation) and 2.60s (torsion). Note that the fundamental translational modes are oriented approximately along diagonals on the building plan view. Detailed floor plans, beam and column sizes,
and the gravity, wind and seismic loading criteria for the two buildings can be found in [27] and [28]. Only one variant of the redesigned building is modeled here, that with perfect connections.

We start with a simplified (idealized) representation scheme that allows for the characterization of seismic ground motion frequency content, intensity, and duration using three parameters. The tall building models are analyzed for damage under idealized ground motion time histories with varying parameters. Key structural response metrics that track collapse (transient and residual peak interstory drift ratios) are mapped against the idealized ground motion time history parameters. The evolution of the collapse mechanism is also tracked as a function of these parameters by imaging the damage localization region (i.e., extent and distribution of plastic rotations and/or fractures in beams, columns, and panel zones).

2. Ground Motion Idealization Scheme

The sensitivity of tall building response to peak ground velocity can be established using the classical, but approximate, analysis of energy budget in multi-story buildings subjected to earthquake excitation [49, 50]. Starting from the governing differential equation of motion and integrating all terms with respect to the structural relative displacement vector, \( \mathbf{v} \), the equation for energy balance can be written as:

\[
E_k(t) + E_\xi(t) + E_s(t) = E_I(t)
\]  

(1)

where \( E_k(t) \) is the instantaneous kinetic energy of the system, \( E_\xi(t) \) is the energy dissipated by viscous forces until time \( t \), \( E_s(t) \) is the recoverable strain energy stored in the system plus the dissipated hysteretic energy until time \( t \), and \( E_I(t) \) is the energy imparted to the system by the input excitation until time \( t \), given by

\[
E_k(t) = \frac{1}{2} \sum_{i=1}^{N_f} m_i (\ddot{v}_i + \dot{v}_g)^2; \quad E_\xi(t) = \int f_\xi^T \mathbf{d}v; \quad E_s(t) = \int f_s^T \mathbf{d}v = \sum_{i=1}^{N_f} \int f_{si} \mathbf{dv}_i
\]

(2)

\[
E_I(t) = \sum_{i=1}^{N_f} m_i (\ddot{v}_i + \dot{v}_g) \mathbf{dv}_g = \sum_{i=1}^{N_f} m_i \ddot{v}_i \mathbf{dv}_g + \frac{1}{2} \sum_{i=1}^{N_f} m_i \dot{v}_g^2
\]

(3)

In the above equations, \( f_\xi \) is the damping force vector, \( f_s \) is the restoring force vector, \( m_i \) is the mass of floor \( i \) and \( N_f \) is the number of floors in the building. \( v_i, \dot{v}_i, \ddot{v}_i \) are the displacement, velocity and acceleration, respectively, of floor \( i \) relative to the ground. \( v_g, \dot{v}_g, \ddot{v}_g \) are the ground displacement, velocity, and acceleration respectively. \( E_I \) is the total work done by all the inertial forces (base shear) on the foundation (displacing through a displacement equal to the ground displacement). In other words, it is the energy imparted to the structure during seismic shaking (e.g., [48]). If the input excitation period is much shorter than that of the structure, we have \( v_i \approx -v_g \); as a result, \( E_I \approx 0 \). For the long-period buildings of this study, the energy imparted from short-period excitation is small and the peak transient IDR must consequently be quite small. If the input excitation period is much longer than that of the structure, \( v_i \approx 0 \), and \( E_I \approx \frac{1}{2} \sum_{i=1}^{N_f} m_i \dot{v}_g^2 \), i.e.,
the input excitation energy is proportional to the square of the ground velocity. Two facts become clear from this analysis: (a) for the long-period structures of this study, only long-period ground motion can induce a strong response; (b) this response is extremely sensitive to the peak ground velocity (PGV). These observations suggest that the best candidate for idealization in as far as ground motion time histories is concerned is the ground velocity history. The three most important parameters in the idealization scheme must be the frequency content of the time history (period of predominant shaking), the peak ground velocity, and the duration represented by the number of cycles. It should be noted that the energy balance analysis is not appropriate for excitation velocities that are extreme where conservation of momentum may be more applicable. However, peak ground velocity from earthquakes seldom exceeds 2.5m/s [17] and energy balance would generally be applicable. In addition, rotational kinetic energy associated with the rotation of floor slabs due to differential axial deformation of columns and/or differential foundation settlement is not explicitly considered in this formulation. This energy, however, is usually only a small fraction of the translational kinetic energy associated with the lateral motion of the floor slabs.

![Figure 2](image)

**Figure 2:** Time histories for the [A] displacement, [B] velocity, and [C] acceleration of the idealized pulses used as input ground motions.

Here, ground velocity time histories are idealized as triangular (sawtooth-like) wave-trains as shown in Figure 2[B]. This ground motion representation scheme was first used by Hall et al. [17] to study the effects of near-source ground motion on tall building response. The displacement history in this representation closely mimics the displacement pulse that would result from the rupture of a penny-shaped crack on a fault surface (point-source) in the vicinity of the crack [8]. Although a single cycle is shown in the figure, multi-cycle extensions with identical period and amplitude are also used to represent long-duration ground motion time histories. The acceleration time history corresponding to this velocity history is a rectangular wave-train (Figure 2[C]), while the displacement is a one-sided parabolic wave-train (Figure 2[A]). The one-sided nature of the displacement should not be of concern. For multi-cycle excitation, displacement is cyclic but always has a positive sign. This is an artefact of the idealization scheme, mathematically equivalent to shifting the origin of the frame of reference, and should have little or no effect on the dynamics of the structure.
Figure 3: Near-source ground motion records from the Cape Mendocino, Chi-Chi, Landers, and Imperial Valley earthquakes with the best-fitting the idealized saw-tooth ground motion time histories.

Three parameters are used to characterize the ground velocity time history: period $T$, amplitude $PGV$ (peak ground velocity), and number of cycles $N$. The ability of this ground motion representation to accurately emulate the true seismic ground motion time histories in as far as impacts on the buildings of interest are concerned must be ensured. Toward this end, the best-fitting single-cycle idealized time history from a suite of idealized time histories to the strong component of 18 near-source records (velocity histories) is determined using the Least Absolute Deviation method ($L_1$ norm). The idealized time history suite comprises of time histories with period varying between 0.5s and 6.0s at 0.25s intervals, $PGV$ varying between 0.125m/s and 2.5m/s at 0.125m/s intervals, and the number of cycles $N$ taking the values of 1 to 5 as well as 10 to emulate long duration records. The near-source records are from the 1971 San Fernando, the 1978 Iran, the 1979 Imperial Valley, the 1987 Superstition Hills, the 1989 Loma Prieta, the 1992 Cape Mendocino, the 1992 Landers, the 1994 Northridge, the 1995 Kobe, and the 1999 Chi-Chi earthquakes. The idealized time history fits for 4 cases are shown in Figure 3. Best-fitting 2-, 3-, 4-, 5-, and 10-cycle time histories are shown as well. These are not utilized
in the forthcoming analysis since all the records have a prominent near-source pulse that is likely to dominate the structural response. They are, however, used in characterizing ordinary multi-cycle ground motion. The fits for the remaining 14 cases can be found in [30].

The FRAME3D models of the existing building (perfect and susceptible connections), and the redesigned building (perfect connections) are analyzed under the 18 three-component near-source records. For the existing building model with susceptible connections and the redesigned building model with perfect connections, the strong component of ground motion is oriented in the building X direction. For the existing building model with perfect connections, two cases are considered: strong component oriented in the building X and Y directions. The peak transient interstory drift ratio (IDR), which is the peak value for all stories of the relative displacement between the top and bottom of a story normalized by its height, is used as a measure of structural performance. It is a good indicator of damage to both structural elements (plasticity and fracture) as well as many types of nonstructural elements. The same models are also analyzed under the one-component best-fitting single-cycle idealized time histories. Shown in Figures 4 and 5 is the comparison of the profiles of peak transient IDR over the height under the actual and idealized motions for the existing building (susceptible connections) and

**Figure 4:** Comparison of peak transient interstory drift ratio (IDR) profile over building height computed using real record (labelled “real”) against that computed using the best-fit idealized 1-cycle saw-tooth time history (labelled “ideal”): Existing building (susceptible connections).

**Figure 5:** Comparison of peak transient interstory drift ratio (IDR) profile over building height computed using real record (labelled “real”) against that computed using the best-fit idealized 1-cycle saw-tooth time history (labelled “ideal”): Redesigned building (perfect connections).
the redesigned building (perfect connections) models. The consistently good match of the profiles indicates that the particular idealization adopted here to characterize the ground motion can be very effectively used in studying the response of the target buildings. The peak values of IDR in all four building models from the two sets of analysis are compared against each other and the errors are quantified in Figure 6. The mean and standard deviation of the best-fitting Gaussian distribution function to the IDR errors are 0.00056 and 0.0069, respectively. All 18 ground motion cases are included in this calculation.

Figure 6: [A] Peak transient interstory drift ratio (IDR) in all the building models computed using the near-source records plotted against that estimated using best-fit idealized 1-cycle saw-tooth time histories. [B] Histogram of and the best-fitting Gaussian distribution to the estimation error.

3. Structural Response to Idealized Ground Motion

To explore the nature of damage localization and the evolution of the collapse mechanism as a function of ground motion features, a series of 3-D nonlinear response history analyses are conducted on the four building models: (a) Existing building (susceptible connections) under X direction excitation, (b) Existing building (perfect connections) under X direction excitation, (c) Existing building (perfect connections) under Y direction excitation, and (d) Redesigned building (perfect connections) under X direction excitation. The models are subjected to the idealized 1-component ground motion time histories introduced in the last section with ground motion period $T$ varying between 0.5s and 6.0s at 0.25s intervals, $PGV$ varying between 0.125 m/s and 2.5 m/s at 0.125 m/s intervals, and the number of cycles $N$ taking the values of 1 to 5 and 10. Key response metrics are computed and stored in a database. These include the peak transient inter-story drift ratio (IDR) and its location over the
building height, the peak residual IDR and its location, permanent roof drift (or tilt) following seismic shaking, plastic rotations in beams, columns, and joints (panel zones), and locations of fractures in the model with fracture-susceptible connections. For the peak transient IDR, the larger of the peak values at two diagonally opposite corners of each story is taken in order to accurately include the effects of torsion in the performance assessment. The peak residual IDR is computed by lowpass-filtering the interstory drift ratio histories and averaging the points within a 5s time-window that has the lowest variance of all such time-windows in the record. A two-pass Butterworth filter with a corner at 10s is employed. The frequency response of the Butterworth filter is maximally flat (has no ripples) in the pass-band and rolls off towards zero in the stop-band [2]. A similar approach is adopted for computing the permanent roof drift which is the roof residual displacement normalized by building height. The penthouse is excluded from the peak transient IDR calculations. It has a much smaller floor-plate than the typical floor of the two buildings. Moreover, the primary moment frames are terminated at the 17th story. The results for each building model, in the form of maps and/or figures, are catalogued in a comprehensive report [30].

Shown in Figure 7 are maps of peak transient IDR on the $PGV - T$ plane for the existing and redesigned building models with perfect connections under idealized 1- and 2-cycle excitation applied in the X direction. The data from the parametric analysis computed at discrete values of $T$ and $PGV$ is first interpolated on a fine parameter grid using a triangle-based linear interpolation technique. It is then filtered using a disk-shaped correlation filter to smoothen sharp transitions in the contours. Also plotted on the maps are contours corresponding to the upper limits on IDR of the Federal Emergency Management Agency (FEMA [9]) performance levels of Immediate Occupancy (IO; IDR=0.007), Life Safety (LS; IDR=0.025), and Collapse Prevention (CP; IDR=0.05). Contours corresponding to peak transient IDRs of 0.075 (Red-Tagged, RT) and 0.100 (Collapsed, CO) are shown as well. Gravity-driven progressive collapse invariably takes hold of our numerical models beyond peak transient IDRs of 0.100. However, since our models do not include degradation due to local flange buckling, we believe the probability of collapse in real-world buildings to be significant beyond peak transient IDR values of 0.075. That said, for the purposes of this paper, definitive inferences on collapse can be made from the analyses of this study only at peak transient IDR of 0.100. Figure 7 shows that for collapse to occur in the two building models with perfect connections under 1- or 2-cycle excitation (e.g., near-source records), the excitation period must exceed roughly the building fundamental period. For the existing building, the excitation period must exceed 4.5s, whereas, for the redesigned building, it must exceed 4.0s. The story location of peak transient IDR closely tracks the ground excitation period $T$, steadily dropping from the top of the building with increasing $T$. The downward migration halts not at the bottom story, but slightly higher.

Shown in Figure 8 are the peak transient IDR curves in the existing building with susceptible connections and the redesigned building with perfect connections under X direction excitation as a function of $PGV$. Each curve corresponds to a particular idealized time history period $T$. The two subfigures for each building correspond to excitation consisting of $N = 1$ and 3 cycles. The corresponding curves for the existing building model with perfect connections under X and Y direction excitation are given in [30]. It can be seen that under short
Figure 7: Peak transient IDR maps on the $T - PGV$ plane for the existing building (perfect connections) and the redesigned building (perfect connections) under 1- and 2-cycle idealized X-direction ground motion. The story location where the peak occurs is labeled at each of the 460 $[T, PGV]$ combinations for which analyses were performed. Contours corresponding to the empirical immediate occupancy (IO), life-safety (LS), collapse prevention (CP), red-tagged (RT), and collapsed (CO) performance levels are shown in bold font. The principal direction building fundamental periods are indicated for reference.

period excitation ($T < 1.5s$), the peak transient IDR saturates to a value below the CP limit with increasing $PGV$. Under longer period ground motion, the peak transient IDR grows with $PGV$ (linearly for moderate period excitation, more rapidly for long period excitation with $T > T_x$ or $T_y$). Collapse risk is negligible/minor if ground motion $PGV < 0.5m/s$ or $T < 1.5s$ for $N \leq 5$ for all four building models. Collapse is probable (peak transient IDR $> 0.075$) in all four models when ground excitation has more than one cycle with period exceeding 5s and velocity exceeding 1m/s.

The simulations agree well with the theoretical energy balance analysis presented earlier. Collapse-level response is induced only by long-period ground motions. Under such motions, the structural response degrades rapidly as a result of a quadratic growth in input energy with increasing $PGV$. If the intensity of shaking ($PGV$)
Figure 8: Peak transient interstory drift ratio (IDR) is shown plotted against the idealized ground motion peak ground velocity (PGV) for the existing building (susceptible connections) and the redesigned building (perfect connections) models at various levels of wave-train periods (T). The two subfigures for each building correspond to 1- and 3-cycle excitations. The one-component ground motion is applied in the building X direction.

is very strong, then the period does not have to be as long to induce collapse-level response. Alternately, within certain limits, a longer period motion relative to the building fundamental period requires a smaller PGV to cause collapse-level response. These findings apply to the PGV range of 0-2.5m/s and an excitation period range of 0-6s. If ground motion period is much longer (greater than, say, twice or thrice the fundamental period of the building), then loading is almost static and does not induce strong enough dynamic response.

4. Localization of Damage: Quasi-Shear Band Formation

To first illustrate the anatomy of a collapse mechanism, one typical instance of simulated collapse is dissected and presented here. The existing building model collapses when subjected to synthetic 3-component motion at Northridge from an 1857-like magnitude 7.9 earthquake on the San Andreas fault. The deformed shape of the structure as it is collapsing along with the plastic hinges on one of the frames are shown in Figure 9. The figure
shows the formation of plastic hinges at the top of all columns in an upper story, at the bottom of all columns in a lower story, and at both ends of all beams in the intermediate stories. Such a pattern of hinging results in shear-like deformation in these stories, resembling plastic shear bands in ductile solids that are severely (shear) strained (e.g., [41]). We coin the term “quasi-shear band (QSB)” to refer to this yield localization region in moment frame buildings. Most of the lateral deformation due to seismic shaking is concentrated in this band. The severe plastic hinging causes it to be far more compliant than the overriding block of stories above and the supporting basal block of stories below. When the overturning 1st-order and 2nd-order ($P - \Delta$) moments from the inertia of the overriding block of stories exceeds the moment-carrying capacity of the fully plasticized quasi-shear band (and if the subsequent seismic waves impart a velocity to the base that is opposite in direction to that of the block of stories over-riding the QSB), it loses stability and collapses. This initiates gravity-driven progressive collapse of the overriding block of stories. Thus, the collapse mechanism initiates as a sidesway mechanism that is taken over by gravity once the quasi-shear band is destabilized. This mode of collapse was predicted by Lignos et al. through analysis and subsequently realized in earthquake simulator tests of two 1:8 scale models of a 4-story code-compliant steel moment frame building [34, 33]. A similar sidesway collapse mode was observed in a full-scale 4-story steel moment frame building that was shaken by the Takatori record from the 1995 Kobe earthquake on the E-Defense Shake Table in Japan [46]. An idealized representation of the collapse mechanism is shown in Figure 10. In structures where the moment frame is proportioned such that the panel zones are weaker than the beams, the quasi-shear band may start with yielding in the panel zones of the intermediate stories rather than at the ends of the beams. However, shear yielding of panel zones is quite a stable mode of deformation (note that the material model for panel shear stress-strain behavior in FRAME3D has no upper limit on the shear strain). Well before some form of instability sets in the panel zone region, there will typically be sufficient moment build-up in the beams to cause yielding. Since deterioration can be much faster in beams, the eventual collapse mechanism will be due to excessive plastic rotations in the beams of the intermediate stories of the quasi-shear band. In an $N_s$-story building, $N_s$ 1-story quasi-shear bands are theoretically possible, $(N_s - 1)$ 2-story QSBs are possible, and so on. Thus, there are a total of $N_s(N_s + 1)/2$ possible QSBs in either principal direction of the building as shown in Figure 11. During strong shaking one or more of these quasi-shear bands can form. It is reasonable to postulate that the most prominent of these bands, the “primary” quasi-shear band, will evolve into a sidesway collapse mechanism. Then, in order to help establish the relationship between the collapse mechanism of the buildings considered in this study and the ground motion parameters, $T$, $PGV$, and $N$, the first step would be to identify the primary quasi-shear band in each of the analysis cases archived in the database of Section 3. This is accomplished by attributing and computing a damage index for each of the $N_s(N_s + 1)/2$ possible quasi-shear bands. The band with the largest damage index is the “primary” quasi-shear band. This damage index is an aggregate of the damage indices (extent of plasticity) of all the components comprising the band. The steps involved in determining the quasi-shear band damage index are outlined in Appendix B.
Figure 9: Anatomy of a collapse mechanism: [A] Typical mechanism of collapse from the simulation of the existing building subjected to strong ground motion (synthetic 3-component motion at Northridge from an 1857-like $M_w = 7.9$ earthquake on the San Andreas fault). Deformations are scaled by a factor of 5 for visual clarity. [B] Plastic rotations at the ends of beams and columns (squares) in one of the frames oriented in the direction of sidesway collapse.

Figure 10: Simplified schematic of a sidesway-collapse mechanism.

Shown in Figure 12[A] is the variation of the three floor damage indices (average of column top damage, average of column bottom damage, and average of joint damage) over the height of the existing building (perfect connections) when subjected to strong 3-component ground motion simulated at a southern California site in the ShakeOut scenario earthquake [38]. Also shown is the identified primary quasi-shear band (red dashed line).
Figure 11: All possible quasi-shear bands (and hence sidesway-collapse mechanisms) for a building with \( N_s \) stories in one principal direction.

It is clear from the actual distribution of frame plastic hinging, shown in Figure 12[B], that there is uniformly heavy yielding throughout the identified primary QSB. While there is some beam yielding above and below this band, collapse is not likely to extend beyond the stories within the selected band because significant column yielding has occurred only at the upper and lower limits of this band and not outside. This can be seen from Figure 12[A] which shows that the top and bottom of the primary QSB correspond to local maxima of the floor damage indices computed using the column top and bottom damage, respectively.

The identified QSBs in the existing building (perfect connections) under the idealized 1-cycle X-direction ground excitation are shown in Figure 13. Shown there are bars indicating the location and extent of the primary quasi-shear band over the height of the building as a function of the excitation period \( T \), for each \( PGV \) intensity.
The fill color represents the peak transient IDR. There is no column damage when the pulse amplitude is less than or equal to 0.5 m/s for any pulse period in the range 0.5-6.0s. Hence, no quasi-shear band has formed and none could be identified in these cases. For a given pulse in the relatively low amplitude regime (0.125-1.375 m/s), say with amplitude 1.25 m/s, the primary QSB is located at the top of the building for small pulse period and migrates down with increasing pulse period. This downward migration stops at the third or fourth floor for pulse periods longer than about the fundamental period of the building, with the location of the band becoming invariant with ground excitation period. For a given pulse in the relatively large amplitude regime (1.875-2.5 m/s), say with amplitude 2.00 m/s, the primary quasi-shear band is located at the bottom of the building for small pulse period and migrates up with increasing pulse period until it coalesces nominally to the same invariant QSB location as for pulses in the low-amplitude, long-period excitation regime discussed above. For a pulse in the intermediate amplitude regime (1.50-1.75 m/s), say with amplitude 1.50 m/s, the primary QSB is located at the bottom stories for short periods, at the top stories for intermediate periods (< building fundamental period) and migrating down with increasing period. The migration once again stops (when the pulse period roughly equals the fundamental period of the building) nominally at the same invariant QSB location as was observed for the low-amplitude and large-amplitude, long period excitation regimes. These observations more or less hold true for all four building models (existing building with susceptible connections under X excitation, existing building with perfect connections under X and Y excitation, and redesigned building with perfect connections under X excitation) subjected to 1-cycle excitation [30]. There are subtle differences from one case to the next. However, under long period excitation in both the low amplitude and high amplitude regimes, the convergence of the primary quasi-shear band to one or two building-specific sets of stories holds true in majority of the cases.
Figure 13: Primary quasi-shear band (QSB) in the existing building (perfect connections) subjected to idealized single-cycle X direction excitation. Pulse period $T$ varies from 0.5 seconds to 6 seconds; peak ground velocity $PGV$ varies from 0.125 m/s to 2.5 m/s.

The results for multi-cycle excitation are quite similar to those under single-cycle excitation, except that the PGV thresholds demarcating low-, moderate-, and high-intensity ground motions are progressively lower with increasing number of cycles. The excitation period thresholds for collapse are lower as well. But, collapse continues to occur nominally in the same set of stories under multi-cycle excitation as for the single-cycle excitation. It should be pointed out, however, that degradation due to local flange buckling, which is not included in the analysis, may have a bigger influence under multi-cycle excitation, rendering the analysis results less accurate. Figure 14 shows the primary quasi-shear band in the existing building (susceptible connections) and the redesigned building model (perfect connections) under 1-, 3-, and 5-cycle X excitation, re-arranged in the order of increasing peak interstory drift ratio (IDR). It is clear that with few exceptions collapse (peak transient IDR $> 0.100$) occurs only in cases where the primary quasi-shear band has formed in the set of stories corresponding
to the nominally invariant QSB that forms under 1-cycle excitation. This suggests a characteristic mechanism or one or two preferred mechanisms of collapse for each building under all forms of earthquake excitation. For instance, there is a strong preference for collapse to occur between floors 3 and 9, floors 3 and 10, and floors 4 and 10 in the existing building (susceptible connections) under X direction excitation. There is a weaker preference for collapse to occur between floors 5 and 10, and 5 and 11. Thus, there are just five preferred mechanisms out of a total of 153 possible mechanisms. Similarly, there is a strong preference for collapse to occur between floors 3 and 8 in the redesigned building (perfect connections) under X excitation. There is a weak preference for collapse to occur between floors 4 and 9, floors 3 and 9, floors 3 and 6, and floors 3 and 7 (five preferred mechanisms out of \( \left[ N_s(N_s + 1)/2 \right] = 153 \) possible mechanisms). Note that the preferred collapse mechanisms are spatially clustered together with significant story-overlap. Collapse is restricted to occur in this narrow band of stories. Similar results are observed for the existing building model with perfect connections under X direction and Y direction excitations [30]. The occurrence of just 1–5 preferred collapse mechanisms that are clustered together in the simulations of all four models can be explained through the shear beam analogy.
Figure 14: The location and extent of the primary quasi-shear band (QSB) plotted against peak interstory drift ratio (IDR) in the existing building (susceptible connections) and the redesigned building (perfect connections) models when subjected to idealized 1-, 3-, and 5-cycle X direction excitation. Pulse periods vary from 0.5s to 6s and amplitudes vary from 0.125 m/s to 2.5 m/s. Ground excitation $T$ and $PGV$ can be identified by the pen color of the bar and the circle, respectively.
5. Understanding quasi-shear band and collapse mechanism formation through the uniform shear-beam analogy

The distribution of moments in a steel moment frame subjected to lateral loads is such that it produces double curvature in all the columns and beams resulting in shear-racking of the frame [47]. Thus, in an overall sense, shear-beam-like behavior and not cantilever-like behavior dominates moment-frame response. This allows for the drawing of an analogy between steel moment frames excited by earthquake ground motion and a shear wave traveling through a uniform shear-beam [20, 17]. However, three significant differences exist between steel moment frame buildings and uniform shear-beams. First, the buildings are not uniform, there is typically stiffness and strength gradation as well as some mass variation over the height of the structure. Second, gravity is not usually considered in the uniform shear-beam, whereas it plays an important role in the collapse behavior of the building structure by causing second order \((P - \Delta)\) effects associated with the self-weight of the structure acting through its deformed configuration under lateral loading. \(P - \Delta\) effects, however, simply amplify the 1\(^{st}\) order overturning moments caused by the floor plate inertial forces. This amplification becomes perceptible only beyond peak transient IDRs of 0.025 (Figure 23) and becomes significant beyond peak transient IDRs of 0.05 (evidence is presented in Appendix C). At the peak transient IDR level of 0.05, the primary quasi-shear band has typically coalesced and become invariant (e.g., Figures 13 and 14), suggesting that \(P - \Delta\) effect has only a minor role to play in the formation of the quasi-shear band. Lastly, steel-frame buildings do exhibit low levels of damping which are not typically present in the uniform shear-beam. Damping has the effect of attenuating the response (which impacts the response to multi-pulse excitation more than single-pulse excitation) and lengthening the apparent period. But the low level of damping inherent in steel structures means that it plays a relatively minor role in the damage localization phenomenon. Thus, the differences in the building response and the analogous shear-beam response, if they exist, are attributable primarily to non-uniformity.

When the fixed end of a uniform elastic shear-beam is subjected to a two-sided pulse excitation at its fixed end, the disturbance propagates as a shear wave through the beam, causing a shear strain that is proportional to the velocity of the support motion [17]. In order to maintain the traction-free boundary condition at the free end and the zero-displacement boundary condition at the fixed end, it can be shown that strain doubling occurs at the fixed end and at a distance of \(HT/T_1\) from the free end of the beam, where \(H\) is the length of the beam, \(T\) is the period of the incident pulse, and \(T_1\) is the fundamental period of vibration of the shear beam. The strain-doubling within the beam occurs due to constructive superposition of the upward-propagating reverse phase of the incident pulse and the downward-propagating reflection of its forward phase. The longer the period of the incident pulse, the lower is the location of strain-doubling. The strain-doubling at the fixed end occurs later as the reflected pulse traverses down and reflects a second time off the fixed boundary. This phenomenon is numerically demonstrated by subjecting the base of a simple discretized uniform shear-beam model to the saw-tooth pulse of Figure 2 with a period of 1s. The model consists of 400 shear spring-mass elements in series. The mass \(m\) and the spring elastic shear stiffness coefficient \(k_1\) are chosen to result in a fundamental period \(T_1\) of 4s for the shear-beam model. First, the springs are modeled to remain elastic. The snapshots of strain
distribution in the model at time intervals of 0.1s are shown in Figure 15[A]. For comparison, the redesigned building model with perfect connections is subjected to the same pulse in the X direction. The period of the pulse is roughly one-quarter of the building’s X-translation fundamental period of 4.06s, while its amplitude is small enough to keep the building elastic. The traveling wave is imaged through snapshots of the inter-story drift ratio at various times (Figure 15[B]). The IDR, being the difference in the displacements of the top and bottom of a given story normalized by its height, is approximately analogous to the shear strain observed in the uniform shear-beam. The IDR distribution tracks the strain in the shear-beam model very closely. The subtle differences in the locations of the peaks can be attributed to the non-uniformity of the steel building. From this example it is clear that the uniform shear-beam analogy to tall steel moment frame buildings holds quite well in the elastic regime of structural response.

If the shear-beam were to yield, the greatest amount of yielding can be expected to occur at locations where strain doubling occurs in the elastic case. To confirm this, the base of the discretized uniform shear-beam model is excited by pulses of the type shown in Figure 2 with periods 1 to 5s. Here, the spring restoring force is modeled in a nonlinear fashion and is hysteretic. The backbone curve is taken to be bilinear. The hysteretic rules are governed by an extended Masing hypothesis [21]. The post-yield stiffness coefficient $k_2$ is chosen to be 10% of the elastic stiffness coefficient $k_1$. If the intensity of the incident pulse is moderate, it will not yield the structure on its way up. But upon reflection off the roof, it causes yielding due to constructive interference of the reflected forward phase of the pulse and the upward traveling reverse phase. Figure 16 shows the accumulated plastic strain in the discretized shear-beam resulting from 1-cycle pulses with periods of 1 to 5 seconds. The yield stress of the springs is chosen such that the pulse with PGV intensity of 1.5m/s does not yield the beam on its way up. Peak plastic strain occurs at distances of $H/4$, $H/2$, $3H/4$, and $H$ from the free end of the shear-beam when excited by the 1s, 2s, 3s, and 4s pulses, respectively. In each case, this corresponds to the location of the strain-doubling in the elastic case ($HT/T_1$ from the free end). For the 5s pulse with wavelength $5H$, strain doubling does not occur in the elastic case, but peak strain still occurs at the base when the incident part of the pulse entering the shear-beam interacts positively with the reflected part of the pulse arriving from the free-end of the model. Greatest yielding can thus be expected to occur at the base in the nonlinear case, and this is indeed the case as (Figure 16[E]). Analogously, Figure 17 shows the distribution of plastic rotations in the beams, columns, and joints of the existing building (perfect connections, $T_1 = 4.52s$), when subjected to 0.75m/s pulses with periods 1s-4s in the X direction. The pulse intensity is chosen such that it does not yield the building on its way up. Overlaid on the plastic rotation maps are the theoretical locations of strain doubling under the elastic uniform shear-beam assumption (dashed red lines). The yield localization region does migrate down from the top of the building with increasing pulse period, but, unlike the inelastic uniform shear-beam, the migration slows down and gets arrested nominally between floors 3 and 9; not coinciding with the strain-doubling location in the corresponding elastic case. Going from the top of the building to the bottom, there is a gradual increase in the strength and stiffness of the structure. The stiffness of the structure affects only the speed of the traveling wave and does not affect the location of greatest yielding. However, the increased strength at
Figure 15: Comparison of [A] the strain response of an elastic discretized uniform shear-beam model to an incident pulse with a period equal to one-quarter of the fundamental period of the beam; and [B] the transient interstory drift ratio (IDR) response of the redesigned building to a low-intensity incident pulse with a period approximately equal to one-quarter of the fundamental period of the building.
the bottom of the structure does not allow yielding to permeate into those stories. Greatest yielding occurs a few stories above the base where the strength of the structure is low enough, yet there is sufficient, if not perfect, constructive interference of the incident and reflected waves. The long wavelengths at these long periods ensure that the superposition is fairly positive and peak strain is not that far off from the maximum possible strain from the uniform shear-beam analogy. Since moderate $PGV$ motions with only long periods can impart energy enough to induce collapse-level responses in a tall building (from Eq. 3), it follows that a collapse mechanism can form only in the few stories where the migration of yielding is arrested. This points to the existence of a small set of preferred mechanisms of collapse, that are clustered together, under moderate intensity excitations.

Figure 16: Accumulated plastic strain over the height of the discretized uniform shear-beam when subjected to 1-cycle velocity pulses with peak velocity of 1.5 m/s and periods varying from 1s-5s.

If the incident pulse has a large enough amplitude relative to the yield strength of some portion of the structure it may cause yielding to occur in the building on its way up. To demonstrate this, the responses of the existing building model (perfect connections) under two single-cycle time histories with identical period ($T = 2.25s$), but different $PGVs$ (0.5m/s and 1.5m/s) are compared. Shown in Figure 18 is the strain of a fiber in a beam at each floor of the model for the two cases. If fiber has not yielded, its response is shown using a dashed line-style. Once the fiber yields, the line-style is changed to solid. Under the moderate level excitation
Figure 17: Plastic hinge rotations in the existing building (perfect connections) resulting from single-cycle \( (PGV = 0.75 \text{m/s}) \) X-direction excitation with periods of [A] 1s, [B] 2s, [C] 3s, and [D] 4s. The dashed lines indicate the predicted location for the peak shearing force in an elastic shear-beam and based on Figure 16, the peak strain in an inelastic shear-beam with the same fundamental period as the building model.

of 0.5m/s the fiber yields only after the wave reflects off the roof. On the other hand, the 1.5m/s pulse is strong enough to yield the fibers on its way up. Due to the gradation of strength over the building height, plastic strain is not greatest at the base, but between floors 3 and 9 for the existing building. The strength of the building drops as the pulse travels up the building. However, inertial forces drop as well, as a result of fewer stories above contributing to the mass. There exists a narrow band of stories with an optimal combination of low-enough strength and high-enough inertia where peak yielding occurs and the primary quasi-shear band forms. This optimal region is nominally between floors 3 and 9 in the existing building model, identical to the location of the preferred collapse mechanism under moderate intensity excitation.

After yielding the structure on its way up, the pulse gets reflected off the roof and the reflected phase of the pulse constructively interferes with the incident phase to produce yielding in the upper stories. The post-reflection yielding in this case occurs in the upper stories because the pulse-period (2.25s) is not long enough relative to the fundamental period of the building in the X direction (4.52s). Thus, under intense, but short-period motions, two distinct quasi-shear bands form in the structure, one in the lower stories and the other in
Figure 18: Strain of a fiber in a beam at each floor of the existing building (perfect connections) for an incident pulse with a period of 2.25s and amplitudes of [A] 0.5 m/s and [B] 1.5 m/s. The dashed lines indicate pre-yield response, the heavy solid lines indicate post-yield response (if any).

the upper stories. But yielding is not great enough at either locations to cause collapse since the input excitation energy is not large enough (Eq. 3). Even though the period of high-intensity excitation does not need to be as long as for moderate-intensity motions to cause collapse, it still needs to be sizable enough relative to the fundamental period of structure. As the period of the incident pulse is increased, the post-reflection quasi-shear band converges toward and, for long-period excitation, merges with the pre-reflection quasi-shear band. Thus, under both large-amplitude motion with moderately long periods as well as moderate $PGV$ motion with long periods, there is a propensity for the primary quasi-shear band to form in an optimal set of stories governed by the mass and strength distribution of the building over its height. These are characteristics solely of the structure and not of the ground motion. When $T$ and $PGV$ are large enough, it is this band that evolves into a collapse mechanism. This points to the existence of a "characteristic" collapse mechanism or only a few preferred collapse mechanisms (out of the $\frac{N_s(N_s+1)}{2}$ possible mechanisms) in either principal direction of the building. If multiple preferred collapse mechanisms exist, they would be clustered together with significant
6. Identifying the Characteristic Collapse Mechanism Using the Principle of Virtual Work

If the non-uniformity in strength is the primary reason for the existence of a characteristic collapse mechanism for tall steel moment frame buildings, it must be possible to identify this mechanism using a work-energy principle such as the Principle of Virtual Work (PVW). The PVW can be applied to each of the \( N_s(N_s + 1)/2 \) possible quasi-shear bands in either principal direction of building (Figure 11), at the instant that the QSB has been fully plasticized. Using the notation in Figure 10, the PVW for the fully plasticized quasi-shear band that extends from floor \( i \) to floor \( i + m \), with height \( h = h_{i+m} - h_i \), can be written as:

\[
\ddot{v}_{cr}^t \Delta \sum_{k=i+m}^{N_f} m_k + (h - h \cos \gamma) \sum_{k=i+m}^{N_f} P_k = \sum_{k=i,j=1}^{N_f} M_p^{(k,j)} \gamma
\]

where \( \gamma \) is the plastic rotation at each joint under sidesway or the shear strain in the quasi-shear band, \( \Delta = h \sin \gamma \), \( N_f = N_s + 1 \) is the number of floors in the building, \( N_j \) is the number of joints in each floor, \( P_k \) and \( m_k \) are the weight and mass of the \( k^{th} \) floor, respectively, and \( \ddot{v}_{cr}^t \) is the critical absolute acceleration of the over-riding block above the quasi-shear band that fully plasticizes the band. For \( k = i \) (the bottom floor of the quasi-shear band) and \( k = i + m \) (the top floor of the QSB), \( M_p^{k,j} \) refers to the column plastic moment capacity (suitably reduced by the factor \( \left[ 1 - \frac{P_{col}}{P_{col,y}} \right] \) to account for the column axial compressive load \( P_{col}; \) \( P_{col,y} \) is the column axial yield force). For the joints on the intermediate stories, \( M_p^{k,j} \) refers to the minimum of (a) the column plastic moment capacity in the excitation direction, (b) the sum of the plastic moment capacities of the beams framing into the joint in the excitation direction, and (c) the panel zone plastic moment capacity.

For small \( \gamma \) (a reasonable approximation for collapse initiation), Eq. 4 reduces to:

\[
\ddot{v}_{cr}^t h \sum_{k=i+m}^{N_f} m_k = \sum_{k=i,j=1}^{N_f} M_p^{(k,j)}
\]

and the critical acceleration of the overriding block at which the quasi-shear band fully plasticizes is given by:

\[
\ddot{v}_{cr}^t = \frac{\sum_{k=i,j=1}^{N_f} M_p^{(k,j)} / h \sum_{k=i+m}^{N_f} m_k}{\sum_{k=i,j=1}^{N_f} M_p^{(k,j)} / h \sum_{k=i+m}^{N_f} P_k}
\]

where \( g \) is the acceleration due to gravity. From plastic analysis principles, the quasi-shear band that becomes unstable (completely plasticizes) at the lowest acceleration of the over-riding block is the characteristic
plastic mechanism of collapse. Note that since this approach does not account for fracture, it will predict the same collapse mechanism for buildings with susceptible connections and perfect connections alike. Also, note that the inertial forces from the floors within the quasi-shear band are neglected in the above formulation. If these forces are included, the relative and absolute accelerations of the over-riding block must be treated separately, breaking down the simplicity of the solution. The following additional term will appear on the left hand side of Eq. 4:

\[ \sum_{k=i}^{k=i+m} m_k \left[ \frac{h_i - h_k}{h} \ddot{v}_{cr} + \ddot{v}_g \right] \left( \frac{h_i - h_k}{h} \right) \Delta, \]

where \( \ddot{v}_{cr} \) is the critical relative acceleration of the over-riding block above the quasi-shear band, \( h_i \) is the elevation of floor \( i \), and \( \ddot{v}_g \) is the ground acceleration. It should be mentioned that the formation of a plastic mechanism as dictated by this approach is not a sufficient condition for collapse. Sufficient lateral drift is needed before the mechanism results in complete collapse. It may not even be a necessary condition for modes of collapse other than sidesway collapse. So collapse mechanisms of structures, say with vertical discontinuities, that may have a propensity to collapse in other ways cannot be assessed using this approach.

The critical acceleration (in units of “g”) for plasticizing each of the \( N_s(N_s+1)/2 \) possible quasi-shear bands of the existing and redesigned buildings in the X and Y directions, determined using this approach, are given in matrix form in Tables 1–4, respectively. The row number represents the bottom floor whereas the column number represents the top floor of the QSB. Thus, the \( ij^{th} \) element of the matrix, \( \ddot{v}_{cr}^{ij} \), corresponds to the quasi-shear band that extends from floor \( i \) up to floor \( j \). The penthouse is excluded from this calculation in both buildings.

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Table 1: Critical acceleration (in units of “g”) for plasticizing each of the \( N_s(N_s+1)/2 \) possible quasi-shear bands of the existing building in the X direction.

The characteristic plastic mechanism of collapse for the existing building in the X direction, predicted using this approach, extends from floor 3 to floor 10, with an \( \ddot{v}_{cr} \) of 0.130g. Since the \( \ddot{v}_{cr} \) for QSBs that run from floors 3 to 9 and floors 3 to 8 are not too different (they are within a couple of percentage points–0.131g
and 0.132g, respectively), these QSBs could evolve into collapse mechanisms as well. Thus, three preferred mechanisms may be predicted using this approach. Shown underlined and in bold are the strongly preferred collapse mechanisms in the perfect- and susceptible-connection models of the existing building under X direction excitation (discussed previously; see e.g., Figure 14). It is clear that the strongly preferred mechanisms from the simulations are very well predicted by this minimization approach. Shown in bold, but not underlined, are the weakly preferred mechanisms. The $\ddot{v}_{cr}$ for the weakly preferred mechanisms occurring between floors 4 and 9, and floors 4 and 10 is within 10% of the minimum value. However, they are higher than the $\ddot{v}_{cr}$ values for mechanisms between floors 1 and 7, and floors 1 and 8. These mechanisms are subsets of the two most critical

### Table 2: Critical acceleration (in units of “g”) for plasticizing each of the $N_s(N_s+1)/2$ possible quasi-shear bands of the existing building in the Y direction.

|   | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |
| Top floor of quasi-shear band |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| 1 | 0 | 0.283 | 0.221 | 0.170 | 0.155 | 0.144 | 0.141 | 0.140 | 0.143 | 0.147 | 0.155 | 0.165 | 0.182 | 0.204 | 0.240 | 0.293 | 0.392 | 0.610 |
| 2 | 0 | 0 | 0.471 | 0.252 | 0.200 | 0.172 | 0.161 | 0.154 | 0.154 | 0.155 | 0.162 | 0.171 | 0.187 | 0.208 | 0.243 | 0.294 | 0.393 | 0.609 |
| 3 | 0 | 0 | 0 | 0.277 | 0.177 | 0.141 | 0.129 | 0.122 | 0.122 | 0.123 | 0.129 | 0.135 | 0.149 | 0.166 | 0.194 | 0.236 | 0.316 | 0.491 |
| 4 | 0 | 0 | 0 | 0 | 0.294 | 0.179 | 0.149 | 0.133 | 0.130 | 0.128 | 0.133 | 0.138 | 0.151 | 0.167 | 0.195 | 0.236 | 0.315 | 0.488 |
| 5 | 0 | 0 | 0 | 0 | 0 | 0.277 | 0.180 | 0.146 | 0.136 | 0.130 | 0.133 | 0.137 | 0.146 | 0.163 | 0.190 | 0.229 | 0.305 | 0.471 |

### Table 3: Critical acceleration (in units of “g”) for plasticizing each of the $N_s(N_s+1)/2$ possible quasi-shear bands of the redesigned building in the X direction.

|   | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |
| Top floor of quasi-shear band |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| 1 | 0 | 0 | 0.305 | 0.226 | 0.158 | 0.152 | 0.149 | 0.154 | 0.154 | 0.162 | 0.173 | 0.187 | 0.204 | 0.227 | 0.259 | 0.306 | 0.374 | 0.498 | 0.760 |
| 2 | 0 | 0 | 0.505 | 0.243 | 0.203 | 0.185 | 0.181 | 0.177 | 0.182 | 0.191 | 0.204 | 0.220 | 0.243 | 0.275 | 0.323 | 0.392 | 0.520 | 0.790 |
| 3 | 0 | 0 | 0 | 0.215 | 0.160 | 0.143 | 0.142 | 0.139 | 0.145 | 0.154 | 0.167 | 0.181 | 0.202 | 0.230 | 0.272 | 0.332 | 0.441 | 0.672 |
| 4 | 0 | 0 | 0 | 0 | 0.232 | 0.168 | 0.156 | 0.146 | 0.150 | 0.158 | 0.170 | 0.183 | 0.203 | 0.230 | 0.272 | 0.330 | 0.438 | 0.666 |
| 5 | 0 | 0 | 0 | 0 | 0.238 | 0.181 | 0.156 | 0.156 | 0.162 | 0.172 | 0.184 | 0.204 | 0.230 | 0.271 | 0.327 | 0.433 | 0.657 |

and 0.132g, respectively), these QSBs could evolve into collapse mechanisms as well. Thus, three preferred mechanisms may be predicted using this approach. Shown underlined and in bold are the strongly preferred collapse mechanisms in the perfect- and susceptible-connection models of the existing building under X direction excitation (discussed previously; see e.g., Figure 14). It is clear that the strongly preferred mechanisms from the simulations are very well predicted by this minimization approach. Shown in bold, but not underlined, are the weakly preferred mechanisms. The $\ddot{v}_{cr}$ for the weakly preferred mechanisms occurring between floors 4 and 9, and floors 4 and 10 is within 10% of the minimum value. However, they are higher than the $\ddot{v}_{cr}$ values for mechanisms between floors 1 and 7, and floors 1 and 8. These mechanisms are subsets of the two most critical
mechanisms between floors 3 and 9, and 3 and 10. Collapse after initiating between floors 3 and 9, and 3 and 10, could localize further during the collapse process. Thus, the observation of collapse between floors 4 and 9, and 4 and 10 despite other mechanisms having smaller $\ddot{v}_t^{cr}$ does not debunk this minimization approach. The collapse mechanisms which are subsets of the preferred collapse mechanisms predicted by this approach should also be considered possible.

In the Y direction, the predicted characteristic collapse mechanism for the existing building extends from floor 3 to floor 8 or 9. Both QSBs have a $\ddot{v}_t^{cr}$ of 0.122g. The simulations of the existing building model with perfect connections show a strong preference for collapse to occur exactly between these floors as well as within two subsets of stories (floors 4 and 8, and floors 4 and 9). The ability of the PVW-plastic analysis approach to robustly predict the preferred collapse mechanisms is clear.

The best results are obtained for the redesigned building under X direction loading. Collapse is predicted to occur between floors 3 and 8 ($\ddot{v}_t^{cr} = 0.139g$). Collapse occurs between these floors in majority of the cases (refer to earlier discussion and Figure 14). The weakly preferred mechanisms between floors 3 and 6, 3 and 7, and 3 and 9 in the simulations have the next smallest $\ddot{v}_t^{cr}$ (within 5% of the minimum $\ddot{v}_t^{cr}$ of 0.139g for the critical mechanism between floors 3 and 8). The other weakly preferred mechanism between floors 4 and 9 is a subset of the mechanism between floors 3 and 9. It is clear from this discussion that if one or more mechanisms have a $\ddot{v}_t^{cr}$ very close (say within 5%) to the minimum $\ddot{v}_t^{cr}$, the characteristic collapse mechanism may be non-unique. Collapse may occur in one of these preferred mechanisms.

To confirm that these findings are not limited to idealized single-component motions, but applicable equally to 3-component earthquake excitation, three of the models (existing building with perfect and susceptible connections, and redesigned building with perfect connections) are analyzed under a suite of 636 3-component

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Table 4: Critical acceleration (in units of “g”) for plasticizing each of the $N_s(N_s + 1)/2$ possible quasi-shear bands of the redesigned building in the Y direction.
earthquake excitations. These are synthetic time histories computed at 636 sites in southern California from the simulation of an 1857-like earthquake \( (M_w = 7.9) \) on the San Andreas fault \([28, 29]\). The East component of horizontal motion is oriented in the building X direction. While the East component is the stronger component in majority of the 636 sites, the North component is stronger at some of the locations. The primary quasi-shear band in each building at each of the 636 analysis is shown plotted against the peak transient IDR (taken to be the maximum IDR in either direction) on Figure 19. For the existing building with perfect or susceptible connections, the primary QSB coalesces between floors 3 and 10, with mechanisms forming between floors 3 and 8, 3 and 9, 3 and 10, 4 and 9, as well as 4 and 10 in equal measure. This is consistent with the predictions using the Principle of Virtual Work, where these five mechanisms had the lowest critical over-riding block accelerations for collapse in either direction (within a few percentage points of each other). Given the complexity associated with 3-component ground motion and the torsional eccentricity in this building, this is more than satisfactory. Likewise, the collapse mechanism in the redesigned occurs between floors 3 and 8 in majority of the cases. In a few cases, it forms between floors 3 and 6, 3 and 7, 3 and 9, and 4 and 8. This is consistent with the \( \dot{w}_{cr}^l \) values for these mechanisms being close to (within a few percentage points of) the minimum \( \dot{w}_{cr}^l \) value corresponding to the mechanism between floors 3 and 8 for collapse in either direction. Thus, the findings of the possible existence of preferred (characteristic) collapse mechanism(s) in these buildings hold true for earthquake excitation as well. It should be mentioned, however, that the synthetic time histories from the San Andreas earthquake are not broadband in character. They are lowpass-filtered through a Butterworth filter with a corner at 2s. This is because the velocity model characterizing the earth structure is not well-resolved to reliably propagate high-frequency waves. Having said this, the soft lower cutoff of 2s for the seismic wave period is well below the fundamental periods of both buildings. Hence, the band-limited nature of the ground motions does not diminish the value of using these synthetic time histories in lieu of recorded time histories for the validation of this study.

The above plastic analysis approach which quantifies the relative propensity for all the possible collapse mechanisms to occur could possibly be used in the optimal design of moment frame systems. The objective could be to size the members in such a way as to maximize the number of preferred mechanisms \( (\dot{w}_{cr}^l \) for all these mechanisms would have to be similar). It may not be feasible to make all mechanisms equally plausible; getting collapses in upper stories is hard in general, as sufficient mass will not be present in the stories overriding the quasi-shear band. However, one could aim to maximize the number of preferred mechanisms by boosting the \( \dot{w}_{cr}^l \) for the lower story mechanisms.

7. Limitations of the Study

(i) The findings of the study are based on analyses of just two moment frame buildings in the 20-story class. These buildings cannot be realistically considered to cover the entire class of steel moment frame buildings. Having said this, the periods and strengths of these two buildings are significantly different, the existing building has a prominent torsional eccentricity whereas this is eliminated in the redesigned building, and different levels of vulnerability to fracture have been considered. Thus, the models analyzed
**Figure 19:** The location and extent of the primary quasi-shear band (QSB) plotted against peak transient interstory drift ratio (IDR) in three of the building models when subjected to synthetic 3-component excitation at 636 sites in southern California from an 1857-like magnitude 7.9 earthquake on the San Andreas fault: [A] existing building (susceptible connections); [B] existing building (perfect connections); and [C] redesigned building (perfect connections).

Do feature some distinct qualities found within this class of structures. Furthermore, the sensitivity of structural response of these models to ground motion features is consistent with theoretical energy balance and shear beam predictions. The strong theoretical basis for the findings offers an interesting prospect that they may be more robust and perhaps be expandable to a broader set of structures within the class of tall steel moment frame buildings. However, further studies are needed to explore this promising possibility. In addition, sensitivity of the collapse mechanism predictions to the modeling assumptions and limitations needs to be systematically investigated.

(ii) The findings of the study related to the mechanism of collapse are limited to regular buildings that do not have discontinuities in the vertical load-carrying system (in the moment frame or the gravity columns). For instance, say a column carrying gravity load of many upper floors is terminated at a lower floor and its load is transferred to two adjacent columns through a transfer beam. In this case, if the transfer beam fails during earthquake excitation, then there could be a rapid vertical progressive collapse similar to an
implosion, not the progressive collapse initiated by sidesway destabilization of the QSB.

(iii) As with any modeling techniques, the FRAME3D models of the buildings have limitations too. These, in the order of importance in the collapse mechanism context, are:

(a) Structural degradation due to local flange buckling of I-sections is not included in the structural modeling. Thus, while collapse mechanism results have been derived for single-cycle as well as multi-cycle excitation, they are most credible for one or two-cycle near-source excitation. The degradation due to local buckling will play a far greater role in dictating the structural response under multi-cycle excitation. For the results to be extended reliably to multi-cycle excitation, models that can accurately capture local buckling must be analyzed.

(b) Column splices have not been modeled. Column splices are typically located three feet above the floor slab with the intention of locating them away from the high-moment (high flexural stress) regions near beam-column joints. In the absence of axial load, the theoretical point of contraflexure (zero moment) is at mid-height of the column. In the case of columns, axial load does exist and buckling failure could occur at mid-height (first mode buckling). So the splice location of three feet above the floor slab is chosen to avoid the most vulnerable locations of the column. These splices are weak points and could fail especially if the column goes into tension during the earthquake.

(c) Composite action of moment-frame beams has not been included. Moment-frame beams are connected to the concrete slab on metal deck through shear connectors (studs). This leads to some part of the slab in the vicinity of the beam to act as being part of the beam, leading to increased stiffness and strength. The effect of this is two-fold. Firstly, it could make the moment-frames stiffer attracting greater seismic forces, but this could be partly offset by the increased strength from composite action. In addition, since this would make the beams stronger in relation to the columns, it could have the effect of pushing the location of plastic yielding into the columns.

(d) Floor framing beams that support the dead weight of the floors are not modeled. While they are typically assumed pin-connected, in reality they do offer partial restraint. Of course, the sections are much shallower and smaller than the moment frame beams. This factor, in conjunction with the fact that only partial restraint is offered by the connections, implies that their contribution may be quite small relative to the moment frame beams.

(e) Damage to floor slabs is not modeled.

(f) Stiffness of partitions, and stair & elevator enclosures is not included.

(g) Foundations have not been modeled. Soil-structure interaction is not included in the analyses.

8. Conclusions

Based on case studies of two tall steel moment frame buildings subjected to idealized 1-component ground motion time histories (11040 simulations) as well as synthetic 3-component seismic time histories (1908 simu-
lations), we predicate the existence of a characteristic mechanism of collapse or a few preferred mechanisms of collapse. The evidence presented is most credible for 1- or 2-cycle near-source excitation since the numerical models do not include local flange buckling which would become more important under multi-cycle excitation. Damage (yielding and/or fracture) localizes in a few stories to form a “quasi-shear” band. When the band is destabilized, sidesway collapse is initiated and gravity takes over. Using classical energy balance analysis and the shear beam analogy it has been shown that under collapse-inducing long-period, moderate- to large-amplitude motions, the critical quasi-shear band in the typically non-uniform buildings must occur in an optimal set of stories where the strength of the structural elements is low enough, but the driving mass of the over-riding floors is sufficiently large. The location of this critical band does not depend upon the finer details of the ground motion. A convenient plastic analysis approach has been developed to ascertain the potential of each of the possible quasi-shear bands in a building to collapse. The characteristic and/or preferred collapse mechanisms identified by this method agree well with the simulations of all the tall building models. The method can alternately be used to develop an improved design of the building that would be close to optimal, where yielding occurs all over, by proportioning the system to make all collapse mechanisms equally likely.

9. Acknowledgments

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Appendix A: Specifics of Structural Modeling

Nonlinear damage analyses of the building models subjected to the simulated ground motion in this study are carried out using the program FRAME3D (http://www.virtualshaker.caltech.edu) that is based on the finite-element method and is capable of performing three-dimensional nonlinear time-history analysis under 3-component ground motion. A three-dimensional structural model of a framed building using this program consists of grids of beams and columns. The setup of the model is comprised of three element classes: panel zone elements for joints, beam elements for beams and columns, and diaphragm elements for floor and roof slabs. The elastofiber beam element is divided into three segments – two end nonlinear segments and an interior elastic segment, as shown in Figure 20. The cross-sections of the end segments are subdivided into fibers. Associated with each fiber is a nonlinear hysteretic stress-strain law, proposed by [16], for axial stress, $\sigma_n$, and axial strain, $\epsilon_n$, where $n$ denotes the $n^{th}$ fiber. This hysteresis model defines a backbone curve consisting of a linear portion, a yield plateau, a strain-hardening region which is described by a cubic ellipse, and a strain softening region described by a continuation of the same cubic ellipse. The backbone curve is characterized by seven parameters: yield stress $\sigma_y$, ultimate stress $\sigma_u$, Young’s modulus $E$, strain at initiation of strain hardening $\epsilon_{sh}$, strain at ultimate stress $\epsilon_u$, rupture strain $\epsilon_r$, and the tangent modulus at initiation of strain hardening $E_{sh}$. Hysteresis loops (Figure 21) consist of linear segments and cubic ellipse segments, and the hysteretic rules to define the cyclic response of each panel are given by [4]. A fiber fracture capability, in the form of a general probabilistic description of the fracture strain, has been added to approximately represent fracture of welded beam-to-column connections [14, 15]. When the fiber strain reaches the fracture strain, it fractures and can no longer take tension, but upon reversal of loading the fractured and separated parts can come in contact, and the fiber is able to resist compression again. The fiber segment is based on the finite element method, wherein the beam translations and rotations are interpolated linearly and independently from their nodal values, requiring a one-point integration on the shear terms to prevent locking.

![Figure 20](image)

**Figure 20:** Schematic representation of the elastofiber beam element used to model columns and beams. Each element is divided into a linear elastic middle segment and two non-linear fiber segments.

The panel zone element models nonlinear shear deformation in the region of the joint where the beams and columns intersect. The joint region consists of a length of column within the depth of the connecting beams.
Figure 21: An example of the fiber response in an elastofiber element.

Shear deformation is due, primarily, to opposing moments from the beams and columns at the joint caused by the frame being subjected to lateral loads. The joint is modeled by two planar orthogonal panels forming a cruciform section. Edges of these panels contain attachment points \( a, b, c, \) and \( d \) where beams attach, and \( e \) and \( f \) on the top and bottom, respectively, where columns attach (Figure 22[A]). Each panel may yield and strain harden in shear. Material nonlinearity in each panel is included by assuming a linear-quadratic shear stress-strain backbone behavior until ultimate shear stress is reached, and perfectly plastic behavior thereafter, as first proposed by \( [16] \). Hysteresis loops, defined by linear segments and quadratic ellipse segments, and hysteresis rules, based on an extended Masing’s hypothesis, are used to model the cyclic response of each panel (Figure 22[B]).

A diaphragm element is used to model the in-plane stiffness of floor slabs. It is essentially a 4-noded plane-stress element that remains elastic at all times. Refer to \( [23] \) for the detailed theory of each of these element types. A key feature of FRAME3D is that full geometric updating is included in both static and dynamic analyses to accommodate large nodal translations and rotations. This automatically accounts for the \( P - \Delta \) effects and allows the analysis to follow a building’s response well into collapse. It involves updating the locations of the joint nodes, attachment points, and the local beam nodes, as well as the orientations of the local element coordinate systems \([22, 25, 26]\) and ensuring equilibrium in the updated configuration. The program utilizes an iteration strategy applied to an implicit time-integration scheme to solve the nonlinear equations of motion at each time-step.

The FRAME-3D models for the two buildings use panel zone elements and elastofiber elements to model
the structural frame, and plane stress elements to represent the floor diaphragms. The story masses are lumped at the column locations based on plan tributary area. Composite action due to the connection between the floor slabs and the moment-frame beams is not considered. A rigid foundation is assumed, with the base of all columns fixed. Soil-structure interaction is not included.

There is great uncertainty in the performance of the beam-to-column connections in older welded steel moment frame buildings as evidenced in the 1994 Northridge earthquake, where brittle behavior was observed in many of these connections. To take into account this vulnerability of older steel moment-frames to fracture, FRAME3D allows for a user-specified probabilistic description of the fracture strain of fibers in elastofiber elements. This feature is used in modeling the existing building. Two susceptibility models are considered, one with perfect connections representing the best-case scenario in as far as system ductility is concerned, and the other with susceptible connections representing a less optimistic and perhaps more realistic assumption in this regard. Since there were significantly greater number of fractures observed in the bottom flanges of beams during the Northridge earthquake, a more susceptible probability distribution is assumed for the fibers in the beam bottom flange when compared against that assumed for the fibers in the top flange and the web. The fracture strain for the fibers in the bottom flanges of moment frame beams, represented by fibers 8 to 14 in Figure 20, is drawn from the following distribution: probability is 20% that the fracture strain is 0.9 times the yield strain, \( \gamma_y \); 20% that it is \( 2\gamma_y \); 20% that is \( 5\gamma_y \); 20% that is \( 15\gamma_y \); and 20% that it is \( 40\gamma_y \). For the top-flanges, represented by fibers 1-7 in Figure 20, and the webs of the beams, represented by fibers 15-20 in the figure, fracture strains are drawn from the following distribution: 30% probability that the fracture strain is \( 10\gamma_y \); 30% that it is \( 20\gamma_y \); 20% that it is \( 40\gamma_y \); and 20% that it is \( 80\gamma_y \). At each connection, the fracture strain in all

Figure 22: [A] Idealization of the beam-column connection into a panel-zone element. [B] Backbone curve for the non-linear hysteretic stress-strain relationship in the panel-zone element.
the beam bottom flange fibers is taken to be a random variable. At the beginning of the time-history analysis, a single realization of this random variable is generated from the corresponding distribution and assigned to the fracture strain of all the beam bottom flange fibers at that connection. This is repeated for the bottom flange fibers as well as the web and the top flange fibers at either segment of each beam. For column flange and web fibers, it is assumed that the fracture strains are far greater than the rupture strain, thus precluding the occurrence of fractures.

The specifications [10] developed by the Federal Emergency Management Agency (FEMA) for moment-frame construction following the Northridge earthquake should result in superior connection performance. Hence, the connections in the redesigned building (designed according to UBC97) are assumed to be flawless, and the fracture-susceptible case is not considered in this study.

Appendix B: Quasi-Shear Band Damage Index Computation

The following are the steps involved in determining the quasi-shear band damage index:

(i) Component Damage Index:

(a) Fibers in the nonlinear end segments of elastofiber elements: The fiber damage index is the fiber plastic strain (ductility demand), \( \epsilon - \epsilon_y \), normalized by the plastic strain to fiber rupture or fracture, whichever is smaller \( \min[\epsilon_r, \epsilon_{frac}] \) - \( \epsilon_y \). \( \epsilon \) is the maximum fiber strain, \( \epsilon_y \) is the fiber yield strain, \( \epsilon_r \) is the fiber rupture strain, and \( \epsilon_{frac} \) is the fiber fracture strain.

(b) Nonlinear end segments of elastofiber elements: The segment damage index is the average fiber damage index for the 20 fibers comprising the segment.

(c) Panel zone damage index: The panel zone damage index is the plastic shear strain, \( \gamma - \gamma_y \), normalized by the plastic shear strain at ultimate shear stress, \( \gamma_u - \gamma_y \). \( \gamma \) is the peak shear strain in panel, \( \gamma_y \) is the panel yield strain, and \( \gamma_u \) is the panel strain at ultimate shear stress.

(ii) Joint Damage Index: The component damage indices are used as is or combined into a joint damage index defined as the root-mean-square of the damage indices of all elements framing into the joint:

\[
D_{\text{joint},j} = \left[ \frac{1}{N_{\text{beam}} + N_{\text{col}} + N_{\text{pz}}} \left( \sum_{i=1}^{N_{\text{beam}}} (D_{ef,i})^2 + \sum_{i=1}^{N_{\text{col}}} (D_{ef,i})^2 + \sum_{i=1}^{N_{\text{pz}}} (D_{pz,i})^2 \right) \right]^{\frac{1}{2}} \tag{7}
\]

where \( D_{\text{joint},j} \) is the damage index of any joint \( j \), \( D_{ef,i} \) is the damage index of the \( i^{th} \) elastofiber element, \( D_{pz,i} \) is the damage index of the \( i^{th} \) panel-zone element, \( N_{\text{beam}} \) is the number of beams framing into the joint in the direction of shear deformation of the quasi-shear band, \( N_{\text{col}} \) is the number of columns framing into the joint (typically one or two), and \( N_{\text{pz}} \) is the number of panel zones (typically one) at the joint in the direction of shear deformation of the QSB.
(iii) **Floor Damage Index (FDI):**

(a) The floor damage index of the top floor of the QSB is the average of the damage indices of the top nonlinear segments in all columns of the top story of the band. Similarly, the FDI of the bottom floor of the QSB is the average of the damage indices of the bottom nonlinear segments in all columns of the bottom story of the band.

(b) The FDI for each intermediate floor is the average of the joint damage indices of all joints on the floor.

(iv) **Quasi-Shear Band Damage Index:** The damage index for each quasi-shear band is the average of the FDIs of all floors within the band. The band with the largest damage index is the primary QSB in a given response history analysis case. If the extent of plasticity is severe enough to render this band unstable, it will evolve into a sidesway collapse mechanism driven by gravity to progressive global collapse of the structure.

**Appendix C: Regime of Significance of $P - \Delta$ Effects**

The existing building with perfect and susceptible connections and the redesigned building with perfect connections are analyzed under a suite of 13 ground motion records from the September 21, 1999, magnitude 7.7 Chi-Chi earthquake in Taiwan, and the September 25, 2003, magnitude 8.3 Tokachi-Oki earthquake in Japan. Only the north component of these records is used to excite the models. Response spectra of these records are given in [29]. The 3-component ground motion records are scaled by factors ranging from 0.125 to 3.000 at 0.125 intervals, and 4.000 to 24.000 at 1.000 intervals, for a total of 585 excitation time-histories. Nonlinear analyses of all three models subjected to these ground motions are conducted with and without consideration of $P-\Delta$ effects. FRAME3D accounts for $P - \Delta$ effects by updating the structural geometry at each step, and ensuring force equilibrium in the deformed configuration. This feature is “turned off” to simulate structural response in the absence of $P - \Delta$ effects. The FRAME3D models are the same as those used in the parametric study using idealized ground motion time histories, except that panel zones are assumed rigid here. Joint element response is intimately tied into coordinate updating, especially of the beam-to-column attachment points, with the panel zone strain being determined by changes in the coordinates of the attachment points [25, 26]. So “turning off” coordinate updating to preclude $P - \Delta$ will result in joint element response differences that cannot be traced to $P - \Delta$, and hence corrupt the results. The peak transient IDR from the analysis including $P-\Delta$ is shown plotted against that from the analysis neglecting $P-\Delta$ in Figure 23. Each point represents the response for a certain scaled ground motion record. Also shown are quadratic relations that fit the data the best. It is quite clear that $P - \Delta$ effects become perceptible only beyond peak transient IDRs of 0.025 and become significant only beyond IDRs of 0.05 in all three building models. This result is consistent with work done by Gupta and Krawinkler [13], who found that for a 20-story structure $P - \Delta$ effects result in change in roof drift from about -15% to +30%, as long as the roof drift does not exceed the value of 0.02. A roof drift of 0.02 would be consistent with a peak IDR of about 0.05-0.08 [30].
Figure 23: Peak transient interstory drift ratio (IDR) in the two 18-story steel moment frame buildings under 13 ground motion records, scaled by factors ranging from 0.125 to 24.000, with and without P-Δ effects plotted against each other.
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