13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3276

EARTHQUAKE RESISTANT DESIGN USING HYSTERETIC ENERGY DEMANDS FOR LOW RISE BUILDINGS

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SUMMARY

An energy based procedure for the earthquake resistant design of new buildings and the seismic upgrading of existing buildings is presented. The procedure uses an energy balance equation and plastic design concepts to either design or modify the members of moment resistant frames. Hysteretic energy demand due to inelastic deformation is the key parameter for the design. Hysteretic energy demand over the height of prototypical buildings is determined by conducting nonlinear analyses for ensembles of recorded earthquake ground motions that have been adjusted to have a common probability of occurrence. The proposed design method is demonstrated for a simple two story, single bay moment frame. The design method uses energy demands from actual earthquake records, simulated SAC 10/50 and SAC Near Fault records. Following the design or upgrade, nonlinear time history analyses are used to check that the design parameters are not exceeded.

INTRODUCTION

In order to efficiently design new buildings and retrofit existing buildings for strong earthquake ground motions, it is necessary to accurately predict the seismic demands on the building system including the effects of duration. It has been recognized that structural damage is the result of a combination of maximum deformation (drift) and cumulative inelastic deformation cycles. It has also been shown that multiple inelastic excursions below the maximum response can still cause significant damage to the structure. An early paper, Housner [6], suggested an energy based design procedure in which the maximum strain energy in an elastic system was related to the spectral pseudo-velocity and indicated a promising approach for accounting for nonlinear dynamic response through the use of an energy balance equation of the form:

$$E_i = E_s + E_k + E_h + E_d \tag{1}$$

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Where E_i is the energy input to the structure by the earthquake, E_s is the elastic stain energy, E_k is the kinetic energy, E_h is the cumulative plastic (hysteretic) strain energy and E_d is the energy dissipated by damping. The energy terms that contribute to structural damage are the cumulative plastic strain energy (hysteretic energy) and the elastic strain energy. A small amount of system damping (2% - 5% of critical) is routinely used in dynamic analyses of building structures and can dissipate a substantial amount of the input energy over time. The kinetic energy term is generally small and can be neglected for purposes of design.

In order to be successful, the capacity of the structural design to absorb energy must be greater than the energy input by the earthquake minus the energy dissipated by viscous damping. This requirement can be expressed as

$$E_a = E_s + E_h \geq E_i - E_d \tag{2}$$

In order to increase the energy absorbed by the building frame, E_a, it is necessary to increase the yield resistance of the system, to increase the yield displacement with the corresponding reduction in stiffness or to increase both the yield resistance and the yield displacement. An increase in the yield resistance requires more material resulting in increased weight and cost. The permissible yield displacement is dictated by serviceability considerations, which are often expressed in terms of drift limitations. In order to increase E_h, it is necessary to increase the inelastic deformation capacity (ductility) of the frame. The beam to column connections or the interaction of the axial load acting on the increased displacements often limits this capacity. For an existing building, increasing the energy absorption capacity may be difficult and uneconomical. An alternative that could satisfy Equation (2) may be the dissipation of excess input energy through the addition of supplemental damping, particularly for extreme earthquake environments and older buildings.

Of these energy components, the hysteretic energy due to inelastic deformation is the more important for purposes of design since it is directly related to the cumulative ductility that defines the inelastic deformation demands on the structural members. Once the hysteretic energy demand is defined, the moment resistant frame is designed on a story-by-story basis subject to the hysteretic energy demand and required drift constraints. A plastic design procedure based on the concept of minimum weight is used for member selection at each story level. Conducting a nonlinear time history analysis then checks the final design.

GROUND MOTIONS AND HYSTERETIC ENERGY DEMAND

For multistory buildings, the energy input by the earthquake at the base will be distributed to each story. If the structure is not well proportioned, this energy will be concentrated at a particular story, Kato and Akiyama [7]. A concentration of energy can also occur due to the characteristics of the input ground motion. For ground motions that are harmonic in character, the concentration may occur in the upper third of the building, whereas if the motions are pulse-type in character, the concentration will occur in the lower third.

In an earlier study Kato and Akiyama [8] determined a shear distribution profile over the building height by conducting trial-and-error dynamic response analyses for sample structures under the El Centro, 1940, ground motion. A recent study, Estes and Anderson [4], considered the hysteretic energy distributions in buildings three, nine and twenty stories in height under an ensemble of 20 earthquake records that had been adjusted to have a 10% probability of exceedance in 50 years, Somerville [12]. Hysteretic energy demands at each story level were evaluated using a modified version of the DRAIN2D computer program, DRAIN2D+, Tsai [13]. This version of the program allows the grouping of elements into energy groups thereby permitting the study of the energy distribution, story by story. The ability of the DRAIN2D+ program to group elements and calculate their cumulative hysteretic energy is an important step in quantifying the energy demands that earlier researchers have described as difficult to establish. The hysteretic energy demand over the height of a nine story moment frame under the SAC 10/50 ensemble of records is shown in Figure 1.

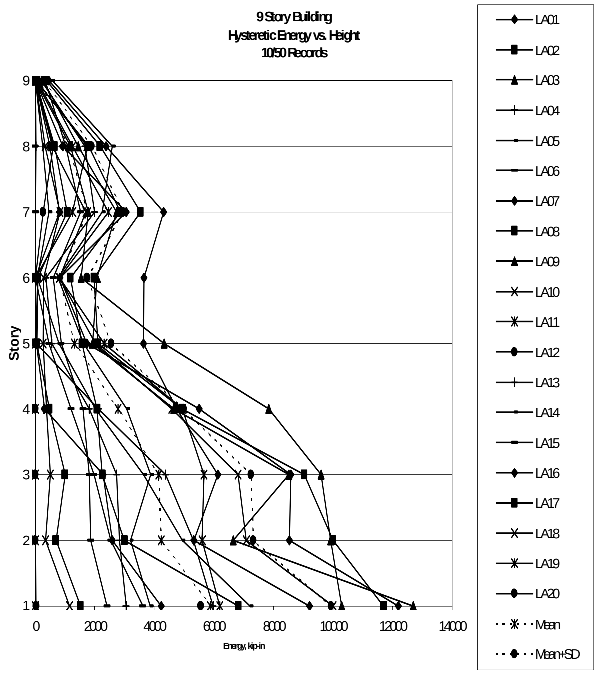

Figure 1. Hysteretic Energy vs. Height, Nine Story Building

DESIGN PROCEDURE

Using plastic design techniques, Neal [9], and the minimum weight theorems, Foulkes [5], the weight of a single story of the frame can be minimized subject to the system constraints. For multistory buildings, a method of piecewise optimum design suggested by Ridha and Wright [10] is applied by optimizing one story at a time, starting at the upper story and proceeding down the frame. The geometry and loading considered for each story are shown in Figure 2. The forces considered in the design of each story include the hysteretic energy demand for the story, the column axial loads from the columns above the story and moments equal to the required plastic moment capacity of the columns directly above the story.

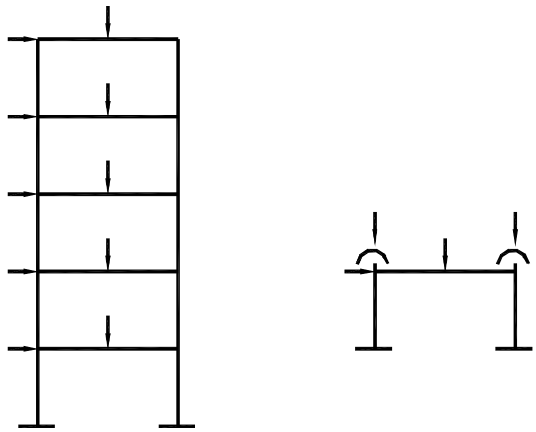

Figure 2. Multistory Frame and Individual Story

Using this model, a work equation can be written for the individual story level, equating internal work to external work. In this study, the internal work is equated to the hysteretic energy demand for the story. The energy demand as determined from nonlinear dynamic analyses may include many load reversals that contribute to the cumulative hysteretic energy. On the other hand, a static, plastic analysis will only include monotonic hysteretic energy. Therefore, an adjustment is necessary to correlate the static monotonic hysteretic energy with the dynamic hysteretic energy as shown in Figure 3.

In an earlier paper, Akiyama and Kato [8], it has been argued that the ratio of the dynamic hysteretic energy to the monotonic should be two. In a more recent report considering the evaluation of existing reinforced concrete buildings, ATC [2], the use of a factor of four is suggested along with reduction factors to account for reduction of the hysteretic energy due to pinching of the hysteresis curve. Since the present work is concerned with ductile steel moment frames, it was considered that the factor of four is appropriate for frames with well-designed connections and stable hysteresis curves. In the course of this study the use of a factor for four produced reasonable and consistent results that were confirmed by nonlinear dynamic analyses. Other values were evaluated and found to give inconsistent results. It is also possible that in the analysis of existing buildings or buildings with degrading hysteretic characteristics, the use of reduction factors may be necessary and could be integrated into the proposed procedure.

For the purposes of design, it was determined that a distribution of energy to each story in equal amounts produced the most uniform results. Consequently, the total energy demand was divided by the number of stories and applied equally to each story. Unequal amounts of energy, especially in the lower stories of a building, produced wide swings in resulting plastic rotations.

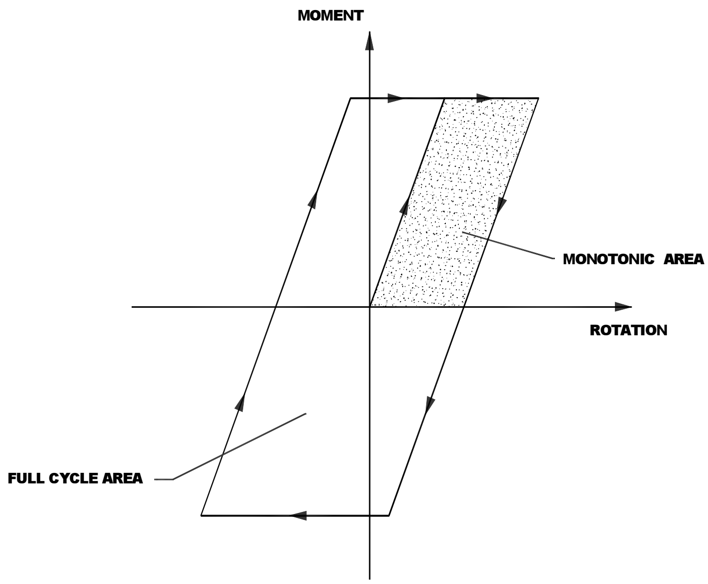

Figure 3. Idealized Hysteresis Loop

The minimum weight formulation requires minimizing a function subject to constraints and is a well-known mathematical problem that can readily be programmed for computation. A convenient procedure for solving this problem in a systematic way is referred to as the Simplex Method. This procedure begins with a "feasible solution" that satisfies all the constraints and then proceeds from one feasible solution to another in a prescribed manner. Rubenstein and Karagozian [11] have described the application of the procedure to structural design in an earlier paper. For the purposes of this study, the procedure was programmed to run on a commercial spreadsheet program.

CASE STUDY

As a simple example of the procedure, a two story, one bay moment frame is considered. This building frame was part of an actual building in the Santa Clarita Valley, CA that was severely damaged by the Northridge Earthquake in 1994 and later demolished. An isometric view of the lateral framing system of the building is shown in Figure 4. Frame #1 used in this study is shown with original sizes in Figure 5.

Each story of the building is assumed to have a constant plastic rotation (drift angle). The work equation for a plastic sway mechanism for the top story of the frame with hinges in the columns, similar to Figure 6, can be written as

$$HE = (2M_{Col} + 2M_{Rf\ Bm})*\theta_p*4.0 \qquad (3)$$

HE is the hysteretic energy demand at the story level, M_{Col} is the effective plastic moment for the columns considering the effect of axial load, $M_{Rf\ Bm}$ is the plastic moment for the roof beam, θ_p is the plastic rotation angle and 4.0 is the adjustment factor for dynamic hysteresis. For a lower story sway mechanism with hinges in the beams and columns, shown in Figure 6, the work equation becomes

$$HE = (2M_{Col} + 2M_{Flr\ bm} - 2M_{Col(abv)})*\theta_p*4.0 \qquad (4)$$

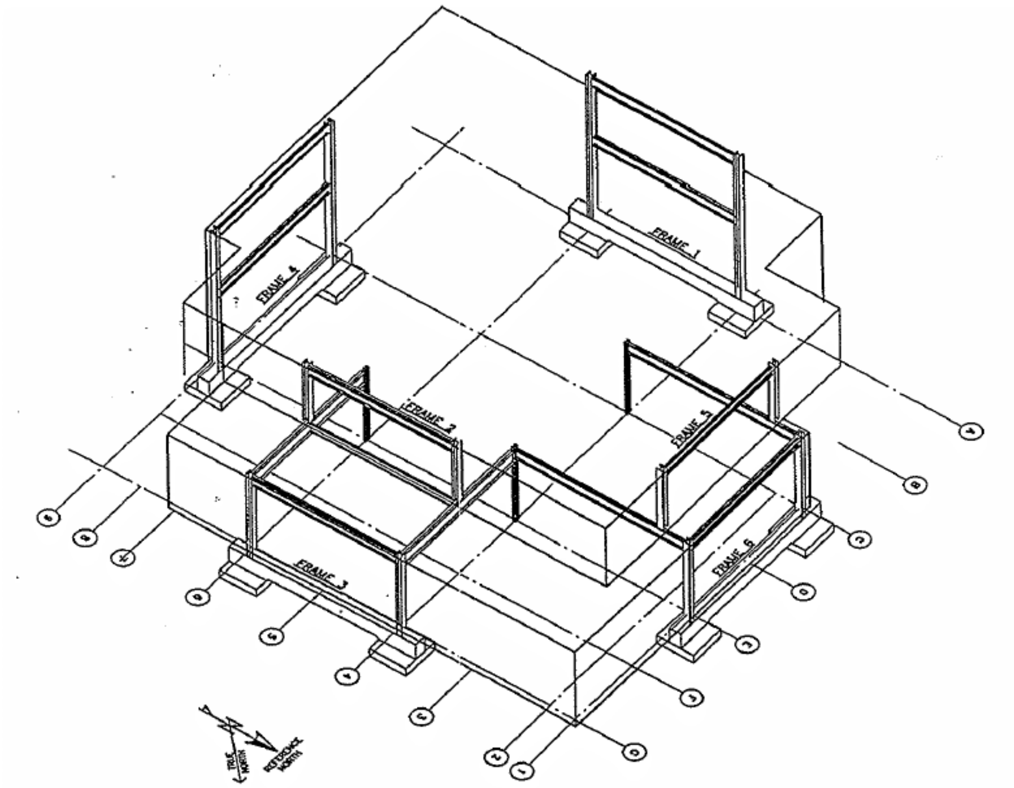

Figure 4. Isometric View of Two Story Moment Frame Building

where $M_{Flr\ Bm}$ is the floor beam plastic moment capacity. For rolled sections, it can be shown that the weight per unit length is proportional to the plastic moment capacity. Hence, the weight function for the story is the sum over all members of the story of the member length multiplied by the member plastic moment capacity. The weight is then minimized by minimizing the weight function. For this building, the weight function for the first story can be expressed as

$$F = 35.2M_{Col} + 32M_{Flr\ Bm} \qquad (5)$$

The plastic moments are determined from equations (3), and (4) subject to the weight function (5). The effect of axial load on the plastic moment capacity of the columns was considered by reducing the column capacity based on the equations in the AISC Load and Resistance Factor Manual [1].

For the purposes of this study, a design plastic rotation of 0.03 radians is used. This is a value that was originally recommended following initial testing of steel moment connections after the Northridge earthquake. The FEMA 350 recommendation suggests an allowable plastic rotation (when converted from drift angle) of 0.038. Use of this value assumes the collapse prevention level of performance, a special moment frame, a 95% confidence level and use of a nonlinear dynamic analysis. Each of the listed criteria are variable and must be evaluated in conjunction with the details of the specific building under consideration.

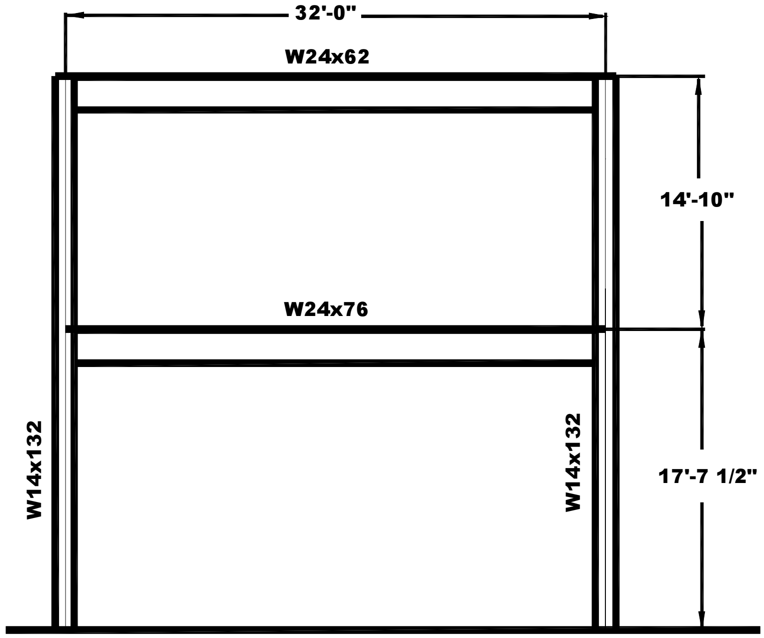

Figure 5. Two Story Moment Frame Building, Frame #1 with Original Sizes

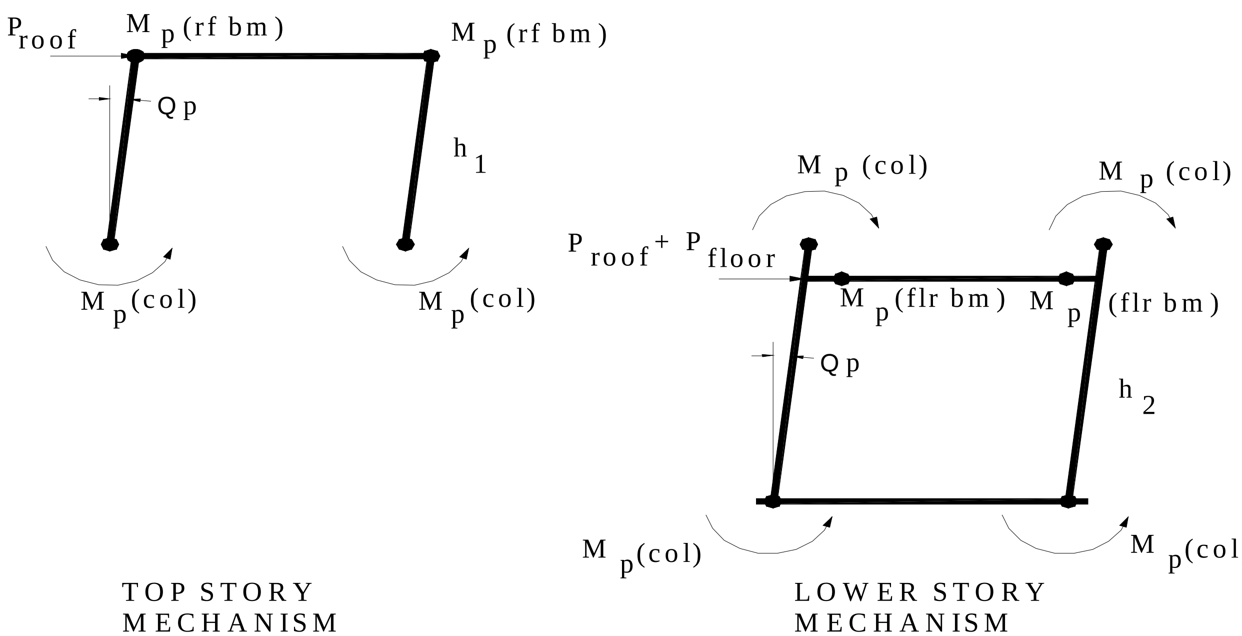

Figure 6. Separated Story Mechanisms

The outlined design procedure is followed for each story with the resulting required plastic moments determining the member sizes. The member sizes are not reduced from the initial code-designed sizes. Following the determination of the new sizes, the building frame is reanalyzed by nonlinear time history analysis. First, the building frame was analyzed with the N-S component of the record obtained at the nearby Newhall Fire Station during the Northridge earthquake. Also, several records from the SAC 10/50 ensemble were used that were close to the mean plus one standard deviation energy demand for this building. Figure 7 shows the energy distribution of the subject building analyzed for all twenty of the SAC 10/50 records. The left dotted line is the mean value of the energy, the right hand dotted line is the mean plus one standard deviation of the energy.

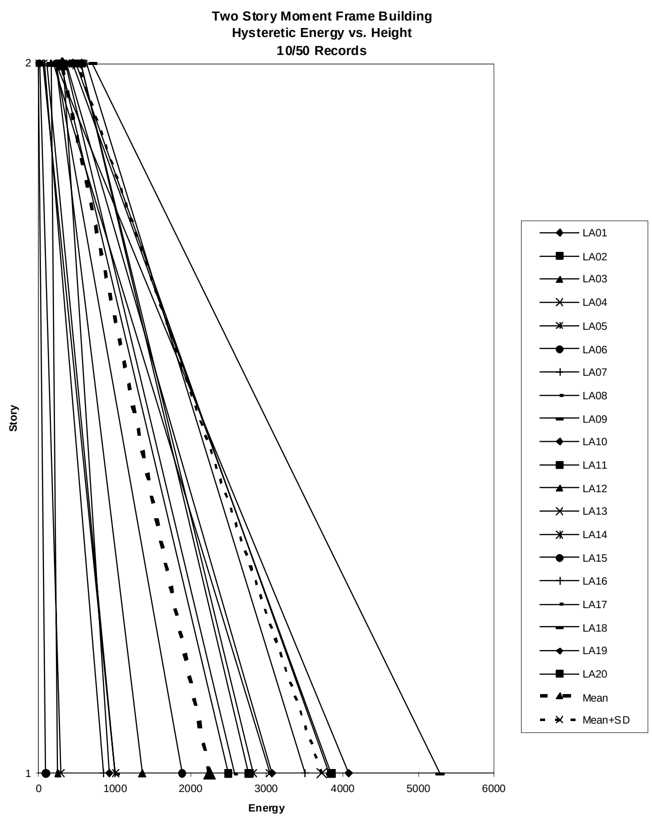

Figure 7. Hysteretic Energy vs. Height for Two Story Moment Frame Building for SAC 10/50 Records

The plastic rotations for the Newhall Fire Station N-S record for the original member sizes are shown in Figure 8. The results for the revised energy-based sizes are shown in Figure 9 for the energy demands from the Newhall Fire Station N-S record. Note that the application of the energy considerations fully reflects the damage that the building experienced. The energy procedure required no increase in the top story sizes, but required the bottom story of the frame to be strengthened. This reflects the actual building behavior as the top story seems to translate as a rigid body, where the bottom story experienced the vast majority of damage. It can also be seen that the maximum plastic rotations are less than or right at the

0.03 criteria. The results of the procedure applied for the SAC 10/50 records are shown in Figure 10 with similar rotations.

It was also desirable to study the effects of so-called Near Fault (NF) records on this design procedure. A chart of the resulting energies for the two story frame building subjected to the SAC NF records is shown in Figure 11. Figure 12 shows the resulting very large plastic rotations for the original frame sizes when subjected to one of the NF records closest to the mean of the NF energies. Proceeding through the design procedure using the mean NF energies produces the member sizes shown in Figure 13. The plastic rotations resulting from the same NF record indicates they are within the specified limit.

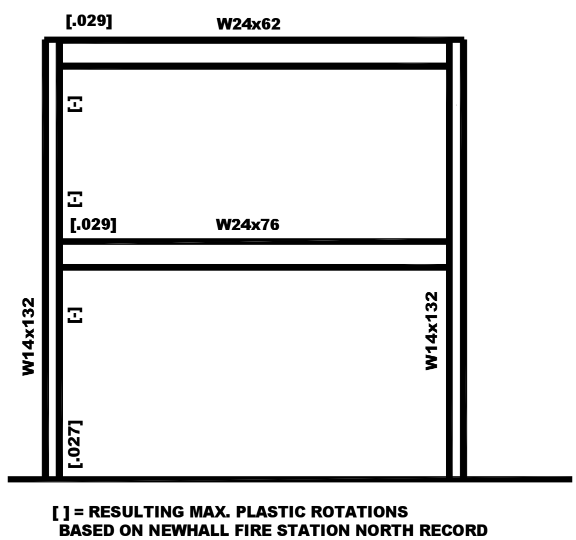

Figure 8. Original Member Sizes and Plastic Rotations based on Newhall Fire Station N-S Record

Plots of demand and capacity for the various design conditions are summarized in Figure 14. It can be seen that the demand for the ground accelerations recorded at the Newhall Fire Station are very similar to the demand for the Mean + Standard Deviation obtained for the 10/50 ensemble of ground accelerations. It can also be seen that the capacity of the revised sizes based on energy demand is only slightly higher than the capacity of the original design (1994 UBC). Comparing these curves to the two demand curves indicates that both of these designs should have withstood the specified ground motions. However, this was not the case.

Measurements and observations at the building following the Northridge earthquake indicated that the first story had a residual displacement of 3 inches. This amount of displacement represents a drift of 0.014 (1.4%). It was also noted that all of the inelastic deformation was concentrated in the first floor (soft story). Hence the beam-to-column moment connections in the first story were only able to develop a plastic rotation of approximately 1.4% rather than the 3% assumed in the energy design. The hysteretic energy capacities based on this reduced plastic rotation are also shown in Figure 14. It can be seen that the upper story still has adequate capacity to resist the imposed ground motions, however, the lower floor has only 50% of the required demand.

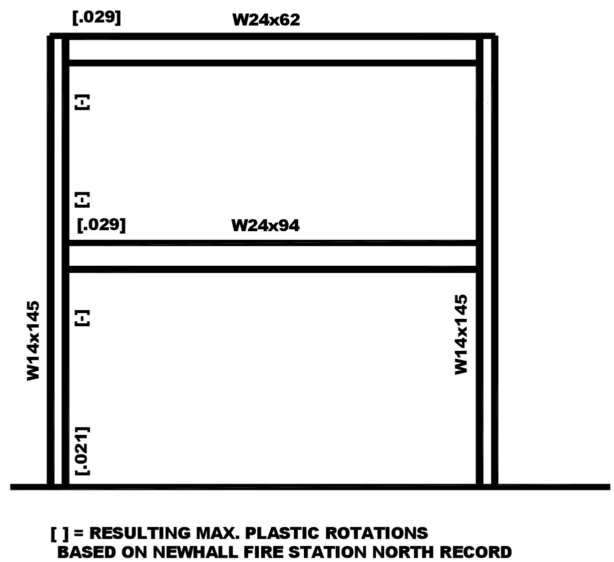

Figure 9. Revised Member Sizes and Plastic Rotations from Energy Design Procedure based on Newhall Fire Station N-S Record

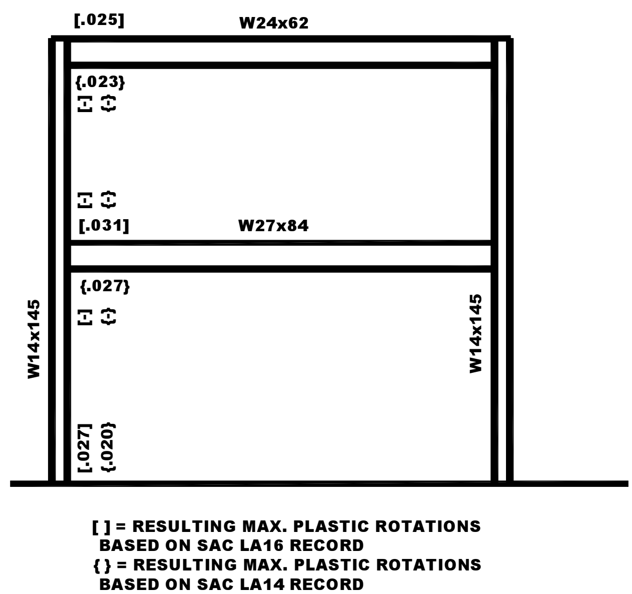

Figure 10. Revised Member Sizes and Plastic Rotations from Energy Design Procedure based on SAC 10/50 Records

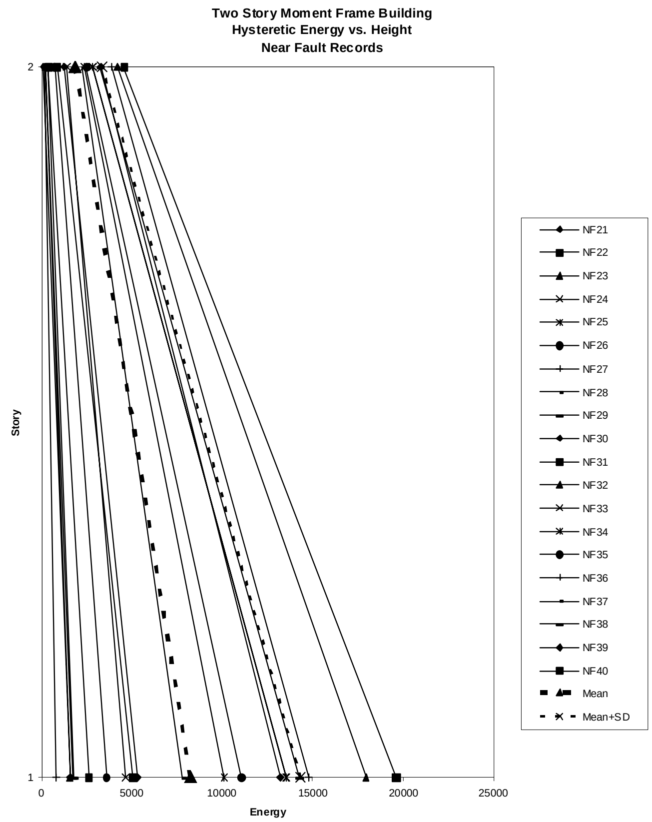

Figure 11. Hysteretic Energy vs. Height for Two Story Moment Frame Building for SAC Near Fault Records

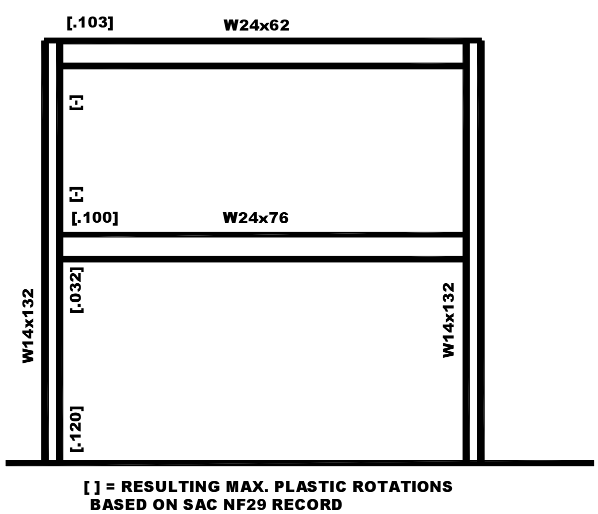

Figure 12. Original Member Sizes and Plastic Rotations for Two Story Moment Frame Building for SAC Near Fault Record

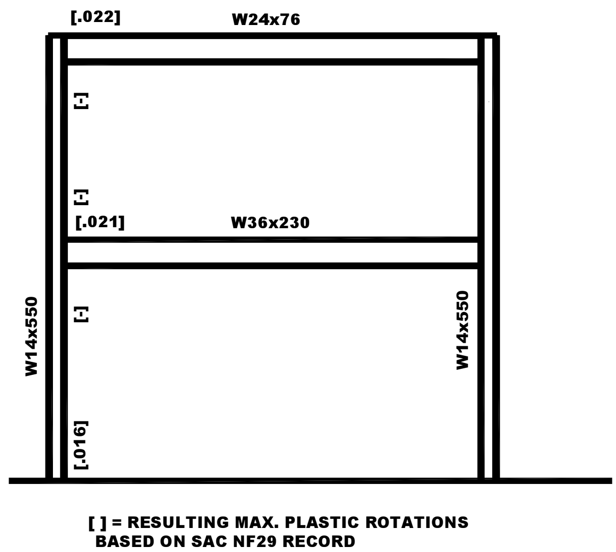

Figure 13. Revised Member Sizes and Plastic Rotations from Energy Design Procedure for SAC Near Fault Record

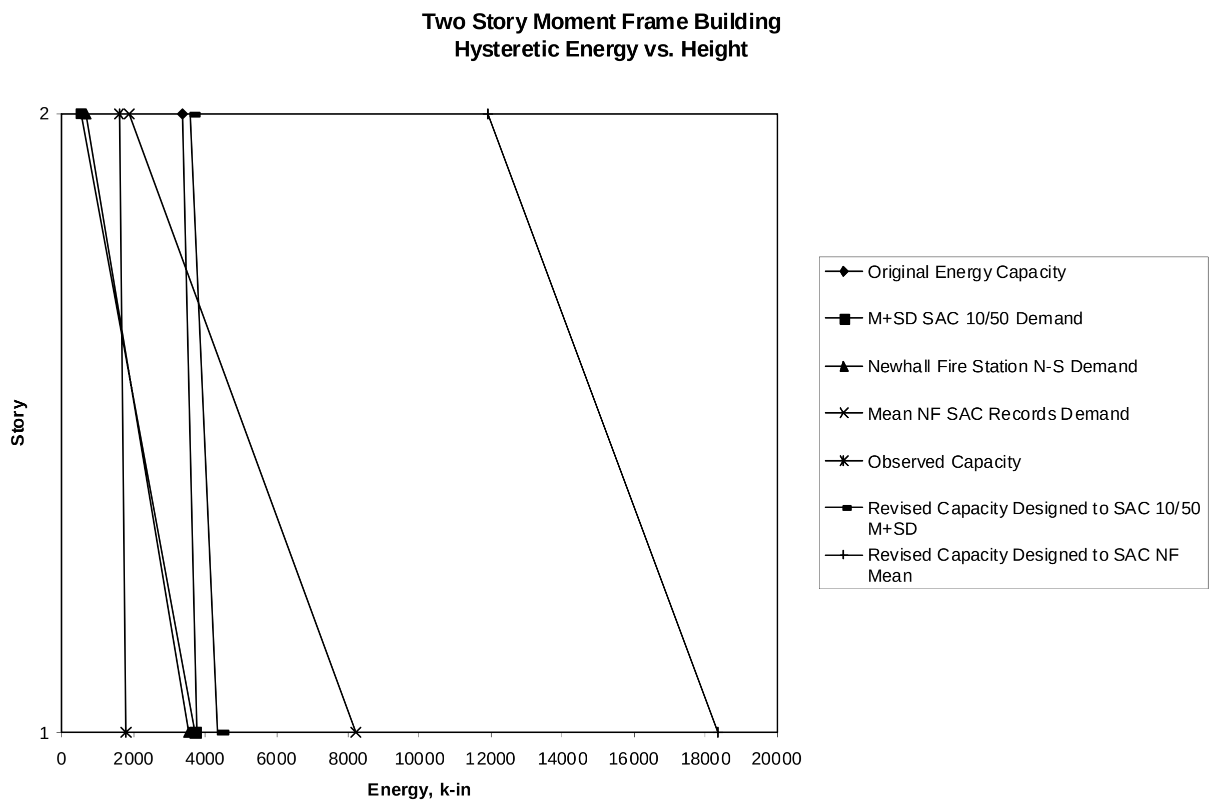

Figure 14. Plot of Energy Capacity vs. Demand for Two Story Moment Frame Building

Also shown on Figure 14 is the demand for the Mean of the more extreme, Near Fault motions. It can be seen that these demands are much larger than those of the 10/50 records. In order to meet these hysteretic energy demands by limiting the plastic rotation to 3% and using available member sizes results in the use of very large members as shown in Figure 13. As shown in Figure 14, the extreme energy demand can be satisfied within the plastic rotation constraint of 3%. However, there is considerable over-design due to the limited number of large sizes that are available and the design constraint to hold the column size constant over the height of the building. A more economical design for this condition may be achieved through the use of supplemental damping in the first story.

CONCLUSIONS

Based on the results of this study, the following conclusions could be put forth:

1. Conducting inelastic time history analyses with DRAIN 2D+ for a representative steel building frame and recorded earthquake ground accelerations, it is possible to study the distribution of derived hysteretic energy over the building height. For low to mid-rise buildings this distribution is approximately linear, being least at the top and maximum at the base.
2. Once the energy distribution over the height is determined, a story by story optimization procedure can be used to optimize the size of members of the frame, subject to the constraints.
3. The current design procedure was formulated for computation on a commercial spreadsheet program (Excel).
4. Using plastic design concepts and incorporating the hysteretic energy demand for a story level, the necessary plastic moment capacities of the members can be determined if the plastic rotation for the story is set at a recommended limiting value.
5. Application of the energy based design procedure is demonstrated on a low rise, two-story building for a nearby actual record and 10/50 plus Near Fault SAC records using an energy distribution over the building height. A distribution of energy for design that considers an equal amount of energy per story was found to produce the most consistent results.
6. The energy demands for SAC Near Fault records are several times the other records and produce correspondingly large member sizes. Near Fault energies for larger building frames have been found to produce many times the corresponding 10/50 energies which may merit a reexamination of the code Near Fault factors.
7. Graphing energy demand versus energy capacity helps indicate areas of weakness and likely damage in a building frame. The plot of energy capacity using actual plastic rotations for a two story frame clearly indicates deficiencies in the moment connections of the original member sizes.

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