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Reliability of steel frame buildings under seismic load

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ABSTRACT: The purpose of this investigation is to evaluate the reliability of steel frame buildings designed for seismic loads in accordance with the Uniform Building Code. Six frame types and two sites in California are being considered. Fifty artificial accelerograms are derived for each site and a Monte Carlo simulation of the building responses is performed. Preliminary results indicate that damage to connections and maximum interstory drifts will not be at levels that produce threats to human life. The buildings, however, are very flexible which may lead to excessive nonstructural and structural damage.

1 INTRODUCTION

The development of modern building code provisions for seismic loads in the United States is based on the understanding that a major earthquake is an infrequent event that has only a very small probability of occurrence during the useful life of a building. Therefore, the philosophy imbedded in the seismic design codes is not based on the desire to prevent damage to a building during a major earthquake since this would lead to excessive construction costs and provide a level of safety that is inordinately high for the low risk involved. Rather, it is based on the desire to limit damage during a moderate earthquake to nonstructural elements with, perhaps, some minor damage to structural members and to prevent collapse during a major earthquake even though the building may be unusable after this large event.

The purpose of this investigation is to evaluate the reliability of steel buildings designed in accordance with the design provisions in the 1988 Uniform Building Code. This document is used for the design of buildings in most of the seismically active areas of the United States. In addition, the study will be restricted to low-rise construction since these buildings constitute the largest percentage of the building stock.

The total design base shear is given by

$$V = \frac{ZICW}{R_w} \quad [1]$$

where Z is the zone factor and represents the effective peak ground acceleration at bedrock; I is the importance factor and is usually equal to 1.0; C represents the normalized elastic response spectrum ordinate and is a function of the building period and the soil condition at the site; W is the total weight of

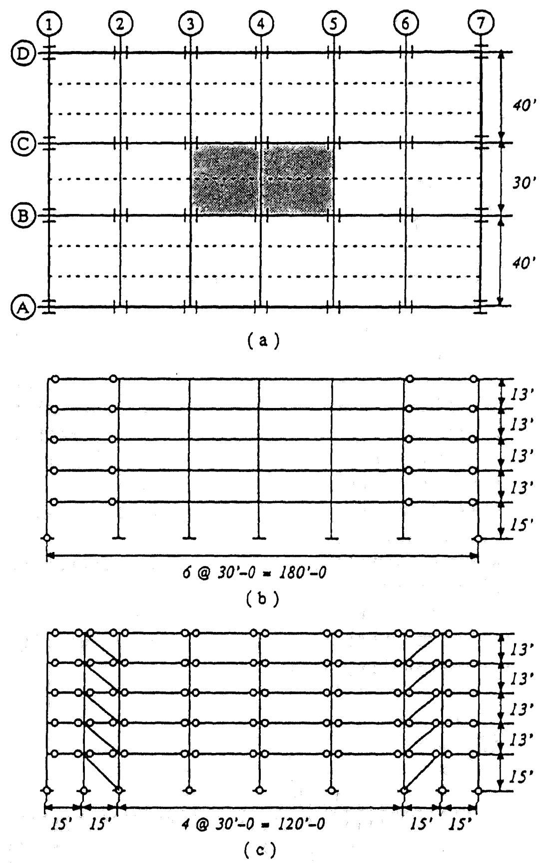


Figure 1 (a) Plan view of building. (b) Elevation of SMRSF and OMRSF. (c) Elevation view of CBF.

the building; and R_w is the response modification factor and is used to reduce the elastic design forces to account for inelastic behavior. Thus, the product ZCW represents the base shear that the building would experience if it remained elastic during the earthquake. The design forces are distributed along the height of the building by an inverse triangular relationship.

In addition to a strength requirement controlled by Eq. 1, a drift limitation is also imposed by the code. The story drift limitation is given by

$$d=\frac{0.03}{R_w} \qquad [2]$$

but not greater than 0.004. This turns out to be a crucial provision since even low-rise moment frame buildings are usually governed by drift rather than strength.

Although elastic analysis and design concepts are used to proportion the members of the building, it is implicitly understood that a building will undergo several large excursions of inelastic response if it is struck by the design earthquake. The amount of inelastic behavior that is expected in a building type is crudely controlled by the design factor, R_w. The larger the value of R_w used in the design the greater the amount of inelastic behavior that should be expected. The maximum interstory drift that is expected during the earthquake is estimated by

$$d_{max}=\frac{3R_w d}{8} \qquad [3]$$

The value of R_w used for the design of a building depends on the type of framing system that is used and on the expected residual strength and ductility of that framing system.

The actual performance of a building during an earthquake depends on how well the simple elastic design rules discussed above capture the essence of the inelastic response of a multi-degree-of-freedom structure. Implicit in the design assumptions is the expectation that inelastic behavior and energy dissipation will be distributed evenly throughout the height of the building. It is assumed that connections will not fail through low-cycle fatigue. Finally, it is assumed that a considerable amount of strength is possessed by buildings that is not quantifiable using ordinary design procedures.

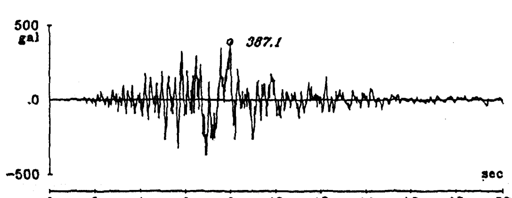


Figure 2. Accelerogram no. 49 for Imperial Valley site.

The purpose of this study is to estimate the reliability of steel buildings designed by current procedures. We hope to answer the following questions:

1. If a building is hit by a major earthquake, will the maximum story drifts be within acceptable limits?
2. Will the expected maximum story drifts be roughly equal throughout the height of the building?
3. Is the reliability consistent among different frame types?
4. Are connection failures likely to occur?

A summary of the investigations and of the preliminary results are given below.

2 PROCEDURES

2.1 Site Selection and Modelling of Ground Motion

Two sites in Southern California were chosen, both in Zone 4. One site is in the Imperial Valley 5 km from the Imperial Fault which is characterized by characteristic earthquakes. These are major events which occur on a known fault with relatively better understood magnitude and recurrence time behavior. The second site is located in downtown Los Angeles which is most affected by smaller, local earthquakes whose occurrences, when taken collectively, can be treated as a Poisson process. The major parameters of the characteristic earthquake are recurrence time, epicentral distance to the site and attenuation. The major parameters describing the non-characteristic earthquake are occurrence rate, intensity and duration. Only results for the Imperial Valley site will be given in this paper.

The ground motion model is described as a nonstationary random process whose intensity and frequency content vary with time. The model allows for identification of parameters from actual ground motions where these are available. The model also uses seismological data such as magnitude, intensity, duration, etc. Using this model and any available data, artificial accelerograms are generated for the site in question. These artificial accelerograms are then used in Monte Carlo simulations of building response. Complete details of the ground motion model are given by Eliopoulos and Wen (1991) and a summary is given in this proceedings (Wen, et. al., 1992).

2.2 Building Design and Analysis

Six types of steel building frames are the focus of this study. Their designations along with their R_w value

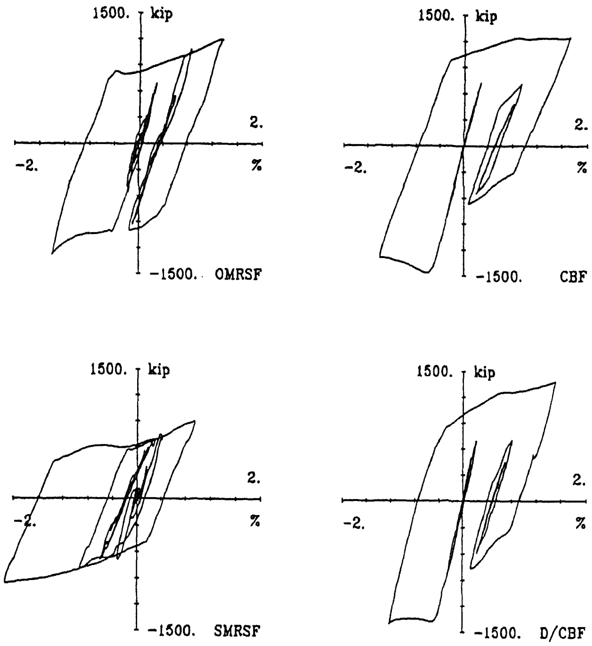


Figure 3. Strong shear vs. drift for the first story of each building subjected to accelerogram no. 49 for the Imperial Valley site.

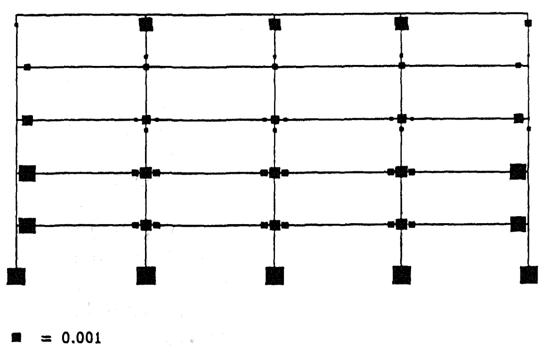


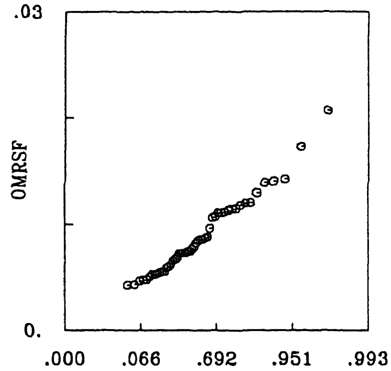
Figure 4. Damage index for the SMRSF frame subjected to accelerogram no. 49 for the Imperial Valley site.

as given in the UBC are as follows:

1. Ordinary moment-resisting space frame (OMRSF), $R_w = 6$;
2. Special moment-resisting space frame (SMRSF), $R_w = 12$;
3. Concentric braced frame (CBF), $R_w = 8$;
4. Eccentric braced frame (EBF), $R_w = 10$;
5. Dual system with CBF, $R_w = 10$;
6. Dual system with EBF, $R_w = 12$.

One five-story building using each of the above framing systems was designed for Zone 4 in accordance with the 1988 UBC. Structural engineers from a Los Angeles engineering firm were consulted to ensure that the floor plan and the design loads and procedures would be consistent with those used in practice. A plan view of the building is shown in Figure 1 (a). Lateral loads are carried by the perimeter frames; Frames 1 and 7 in the transverse direction and Frames A and D in the longitudinal direction. All beam-to-column connections at interior joints are assumed to be pinned. An elevation view for the SMRSF and OMRSF frames is shown in Figure 1 (b) and for the CBF frame in Figure 1 (c). The EBF frames are still under investigation and will not be discussed in this paper.

Fifty accelerograms were simulated for each site. An example of one of these (No. 49) is shown in Figure 2. The computer program Drain 2DX was used to calculate the inelastic dynamic response for each earthquake. The statistics of the response and the level of risk for each of the following response characteristics were determined: maximum story drifts; energy dissipation; and damage index. Maximum story drifts are related to the amount of nonstructural and structural damage that will occur during an earthquake. Energy dissipation and the damage index are related to the amount of low-cycle fatigue damage that will occur in the connection regions. A damage model based on work by Krawinkler (1983) was used in this evaluation.



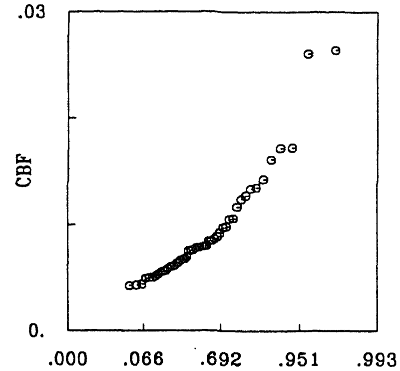


Figure 5. Maximum first-story drift plotted on type 1 extreme value probability paper for 50 Imperial Valley accelerograms for OMRSF and CBF frames.

Table 1. Maximum story drifts at the Imperial Valley site for given reliability levels for each frame type.

	Prob (%)	Story 1	2	3	4	5
OMRSF	50	.64	.90	.96	1.03	1.20
	25	.89	1.15	1.22	1.28	1.42
	15	1.04	1.31	1.39	1.43	1.55
	10	1.16	1.43	1.51	1.54	1.65
	5	1.36	1.63	1.71	1.72	1.81
SMRSF	50	.76	.97	1.07	1.10	1.40
	25	1.04	1.25	1.36	1.38	1.70
	15	1.21	1.43	1.55	1.56	1.89
	10	1.34	1.56	1.69	1.70	2.03
	5	1.55	1.78	1.93	1.92	2.26
CBF	50	.59	.64	.77	.98	1.00
	25	.87	.90	1.01	1.26	1.23
	15	1.09	1.08	1.16	1.44	1.37
	10	1.26	1.21	1.27	1.57	1.47
	5	1.54	1.44	1.45	1.78	1.63
D/CBF	50	.47	.75	.82	.88	.82
	25	.70	1.02	1.09	1.09	.98
	15	.88	1.21	1.26	1.23	1.08
	10	1.01	1.36	1.39	1.33	1.16
	5	1.23	1.59	1.60	1.49	1.28

3 RESULTS

Typical examples of the story shear vs. story drift for the first story for each of the four frame types are shown in Figure 3. These all represent the response to accelerogram No. 49 at the Imperial Valley site which was one of the most intense of the group. The two braced frames (CBF and dual with CBF) were the strongest, followed by the OMRSF and then the SMRSF frames. One should note, however, that the OMRSF frame is not two times stronger as might be expected based on the R_w values used for their design. This is because drift governed each of their designs rather than strength.

An example of the damage index calculations is shown in Figure 4 for the SMRSF frame. A damage index of 1.0 at any location indicates failure of the connection by low-cycle fatigue. These results indicate that failures of well-constructed connections are not likely to be a problem during earthquakes.

Only uncertainties in the ground motion parameters were considered for computing the statistics of the responses. It was assumed that the variations in material and member properties are small compared to variations in the properties of the earthquakes. The level of risk for each response quantity was calculated by assuming that it was governed by a type 1 extreme value distribution. A plot of the maximum first-story drift on type 1 extreme value distribution paper is shown in Figure 5 for each frame type and for all fifty accelerograms.

The levels of risk for interstory drift are shown in Table 1 for the Imperial Valley site. For this table the dependent variable is the probability of exceedance and the independent variable is the story drift given as a percentage of the story height. Some drift limits to keep in mind are the threshold of nonstructural damage which is about 0.5 percent and the maximum tolerable drift due to life threatening considerations which is on the order of 1.5 to 2.0 percent.

The variation in maximum drift among the different frame types is surprisingly small at all risk levels considering the range in R_w values used in their design. This is particularly true, and perhaps troublesome, for the two moment frames. The reason for using the smaller R_w value for OMRSF frames is that the detailing requirements are less stringent than for SMRSF frames and therefore less ductility capacity is provided.

The results indicate that there is a probability of greater than 50 percent that nonstructural damage will occur during the 50-year life of the building regardless of which type of framing system is used. Drifts exceeding 1.0 percent will likely result in loss of most or all of the nonstructural elements in the building. The probability is between 15 and 25 percent that this will happen for all of the frame types. This seems to be too high of a risk for an event that could lead to the complete economic loss of the building. Since none of the expected maximum drifts is greater than 2.0 percent life safety does not appear to be a problem.

The maximum expected drift levels are reasonably uniform with height for the two braced frames at the larger response levels. This is not the case, however, for the two moment frames. Both of these frames show a trend towards higher drifts at the upper stories. This is probably not desirable since it leads to higher maximum drifts than would occur if the damage was uniformly distributed over the height. This could be remedied by using a different distribution of seismic forces for design.

Two important components of actual buildings that are not included in the models used in this study are nonstructural walls and cladding. These may add a considerable amount of stiffness and strength to a steel building, particularly at low to moderate drift levels. This is a topic that is currently under study. The results may show that the damage potential for steel frame buildings is not as great as the current results indicate.

4 CONCLUDING REMARKS

The preliminary results of this study indicate that the current design rules will result in steel buildings that will provide life safety against major earthquakes. These buildings, however, are quite flexible which may lead to excessive damage to nonstructural and structural elements even during moderate earthquakes. The moment frame buildings show a tendency for greater interstory drifts in the upper stories. This may or may not be detrimental and is a topic for further research.

This is a continuing project. Results for eccentric braced frames and dual systems with EBF are currently being generated. The effect of cladding and other nonstructural elements is also being investigated.

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