ECONOMICS, EXPECTED DAMAGE, AND COSTS OF SEISMIC STRENGTHENING

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SUMMARY

This paper presents a detailed discussion of costs of seismic strengthening for various strengthening concepts for a typical low-rise structure. Damage is discussed in terms of drift and acceleration, and a damage matrix is formulated in terms of design levels and applied loading. The economics of seismic strengthening is discussed.

INTRODUCTION

The United States Navy has numerous bases located in active seismic regions, and each of these bases resembles a small city containing work areas and residential areas. With any seismic plan establishing appropriate design levels which are safe, consistent with established knowledge, and economically effective must be considered. Because of the limited amount of available construction funds, an investigation of the economics of seismic strengthening is appropriate. What level of seismic design should be utilized considering costs of strengthening, the expected damage, and loss of life? This complex problem is the topic of this paper.

SEISMIC DESIGN AND ANALYSIS

A typical structure was selected for detailed study. The structure chosen was representative of a class of structures utilized by the Navy for administration, light industrial work, or living quarters. The structure selected was an actually constructed three-story building for which detailed cost data and drawings were available. The building was recently constructed at an eastern Navy base in a nonseismic area. Thus, the nonseismic baseline cost condition was established. The building was a frame structure, 185 by 185 feet in plan. The structure utilized a structural frame system.

The selected structure was redesigned considering the structure to be new construction and being located in seismically active areas. Seismic design concepts were typical of conventional West Coast standard engineering design practice.

The structure was designed for six levels of peak ground acceleration: 0.10g to 0.35g with .05g increments. Elastic design spectra utilizing Newmark standard spectral shapes were utilized. Five concepts of seismic strengthening were utilized: (1) steel moment frame, (2) steel braced frame, (3) steel frame and concrete shear wall, (4) concrete moment frame, and (5) concrete frame and shear wall. The performance level of the structure under the specified spectra was required to be a ductility equal to 1.0 design, such that members were to be at yield. This performance level was specified for several reasons. First, specifying a ductility of 1.0 is the same as specifying a higher acceleration

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and some ductility greater than 1.0. Second, use of a ductility equal to 1.0 criteria allows the structural design engineer the use of all elastic computer codes without need for a nonlinear analysis; further, nonlinear spectral techniques need not be used.

Cost of Seismic Strengthening

Detailed structural costs were estimated based on the results of the six design cases for the five concepts of strengthening. The cost of the existing exterior frame construction was deducted from the total building cost, and then each new seismic framing system was added to obtain a new total building cost. Concrete or masonry seismic shear wall configurations, when utilized, were assumed to replace the existing 6-inch concrete block. Foundation redesign was included. Costs were adjusted to 1981 costs in the Los Angeles area. Figure 1 shows the increase in cost for seismic strengthening. Figure 2 gives the first mode periods of the structure for the strengthening concepts as a function of design level. The moment frames period show greatest variation with design acceleration.

Damage Evaluations

Damage to structural frame members, shear walls, and other elements associated with displacement are influenced by the interstory drift. Other elements tied to the floors, such as equipment or contents, are influenced by floor acceleration. Reference 1 is a detailed study of previous work in damage evaluation and will not be repeated here.

To evaluate the damage expected to the structure, each of the six design levels for each of the five design concepts of strengthening was analyzed for a series of applied seismic load levels. Nonlinear finite element techniques were employed. The program DRAIN-TABS was utilized to perform the analysis. Damping increased with the ratio of applied load to design level. Drift and floor acceleration time history responses were computed in the analysis. Effective response levels were selected at 65% of peak values and used in the damage prediction. The value of 65% has been used in past studies to approximate effective peak ground acceleration. This value, based on engineering practice, is used to reduce the peak values to a level of repeated sustained loading.

The detailed cost estimate was utilized to identify key elements of the structure to which dollar values could be associated. Repair factors for damage were estimated. The key elements were divided into drift- or acceleration-sensitive components, and values of drift and acceleration were then related to damage for each element.

Tables 1 and 2 give the damage ratios for each key element for the steel framing concepts. A similar table was developed for the concrete framing concepts. It should be noted that a value is included for contents and that utilization of repair multipliers can result in costs exceeding the total cost of the structure. This is reasonable since demolition and removal costs would be required for major repairs.

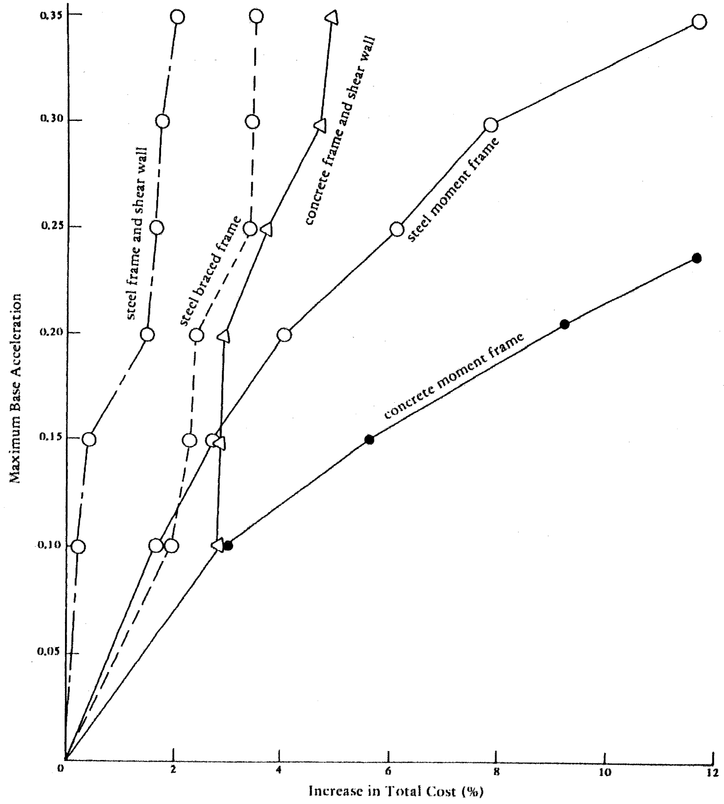

Figure 1. Increase in cost for each type of strengthening.

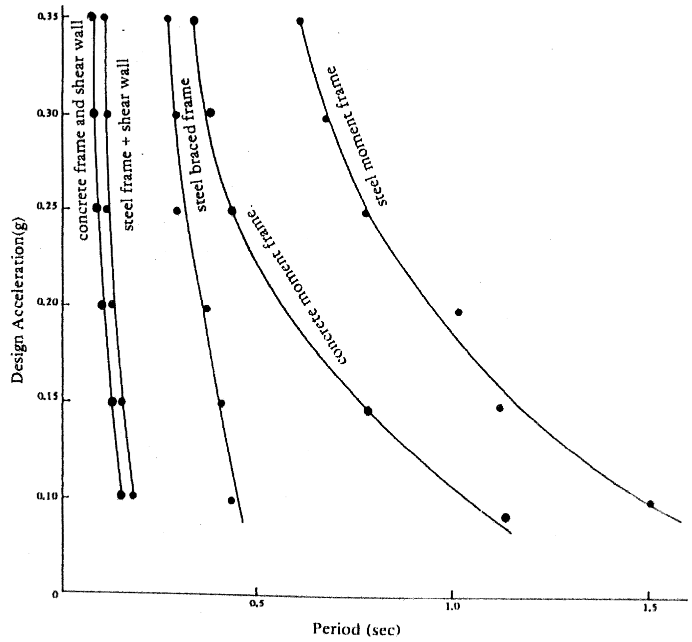

Figure 2. First mode period.

Table 1. Damage Ratios – Drift Steel Structure

Element	Cost	Repair Multiplier	Interstory Drift .001	.005	.010	.020	.030	.040	.070	.100	.140
1a. Rigid Frames	117,500*	2.0	0	.01	.02	.05	.10	.25	.35	.50	1.00
b. Braced Frames	*Varies	2.0	0	.03	.14	.22	.40	.85	1.00	1.0	1.0
c. Shear Walls	w/Design	2.0	0	.05	.30	.30	.60	.85	1.00	1.0	1.0
2. Non-Seismic Struc. Frame	625,500	1.5	0	.005	.01	.02	.10	.30	1.00	1.0	1.0
3. Masonry	417,600	2.0	0	.10	.20	.50	1.00	1.0			
4. Windows & Frames	120,600	1.5	0	.30	.80	1.00					
5. Partitions, Architect. Elements	276,200	1.25	0	.10	.30	1.00					
6. Floor	301,200	1.5	0	.01	.04	.12	.20	.35	.80	1.00	1.0
7. Foundation	412,100	1.5	0	.01	.04	.10	.25	.30	.50	1.0	1.0
8. Bldg. EQ & Plumb.	731,600	1.25	0	.02	.07	.15	.35	.45	.80	1.0	1.0
9. Contents	500,000	1.00	0	.02	.07	.15	.35	.45	.80	1.0	1.0

Table 2. Damage Ratios -- Acceleration Steel Structure

Element	Cost	Repair Multiplier	Floor Acceleration (g's) .08	.18	.50	1.2	1.4
1. Floor & Roof Sys.	301,200	1.5	.01	.02	.10	.50	1.0
2. Ceilings & Lights	288,500	1.25	.01	.10	.60	.95	1.0
3. Building Equipment & Plumbing	731,600	1.25	.01	.10	.45	.60	1.0
4. Elevators	57,000	1.5	.01	.10	.50	.70	1.0
5. Foundations (Slab on Grade, Sitework)	412,100	1.5	.01	.02	.10	.50	1.0
6. Contents	500,000	1.05	.05	.20	.60	.90	1.0

Table 3. Damage Ratio – Steel Moment Frame

Applied Load (g's)	Design Acceleration (g's) 0.00	.10	.15	.20	.25	.30	.35
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
.10	.10	.08	.08	.09	.11	.11	.11
.20	.17	.11	.13	.15	.17	.19	.21
.30	.24	.16	.20	.23	.22	.22	.24
.40	.37	.24	.26	.27	.27	.26	.29
.50	.56	.37	.29	.32	.29	.29	.31
.60	.82	.55	.35	.36	.33	.33	.32
.70	1.03	.68	.40	.39	.35	.34	.35
.80	1.13	.75	.43	.43	.38	.38	.38
.90	1.25	.83	.46	.45	.42	.40	.39
1.00	1.50	.89	.50	.48	.46	.42	.41

Use of Tables 1 and 2 in conjunction with the drift and acceleration values from the nonlinear analysis resulted in Table 3, which presents a typical damage matrix giving damage as a function of design level and applied loading. Included in the damage matrix is the damage to the structure and the contents using the noted repair factors.

Steel Moment Frame. The response of the moment frame structure is in the constant velocity region of the spectra for all six design ranges. It is significant to note that as the structure is stiffened, displacement is reduced; however, acceleration is increased. Damage is dependent on both displacement and acceleration. Note also that for a given applied load level, each of the six design cases is at a different damping level, with the weakest structure being most heavily damped. In the low applied loading level the strong structures are lightly damped, responding elastically with higher floor accelerations. The weaker structures are more heavily damped, responding inelastically with lower floor accelerations. In this range, stiffer structures receive greater damage; this condition exists to about 0.5g for the range of structures studied. Over 0.5g the stiffer structures exhibit lower damage, as might be expected. The use of a single time history event with its unique frequency content results in minor response variations. Any single time history has unique frequency gaps and high points. Since the period of the structure changes with strengthening, secondary interactions occur between the frequency high points and structure periods such that the responses at a particular design level might be slightly reduced or amplified over the response of an ideal time history without gaps and high points. Further, the six design cases are not exact multiples but rather depend on human selection of available structural shapes. These factors induce very minor dispersion in the results. A clear conclusion, however, is that stiffening in the low applied acceleration region does not reduce total damage. The damage ratio is a complex function of period, damping, range of non-linear behavior, and the mix of total damage caused by drift and acceleration.

Steel Braced Frame. The response of the braced frame structure is in the constant acceleration region of the spectra for all six design ranges. The structure in its basic configuration with bracing is a much stiffer structure than the corresponding moment frame, pushing the response from the constant velocity region to the acceleration region. The resulting floor accelerations produced by the applied loading are higher than those of the moment frame while story drifts are reduced. In the medium and low level applied loading range, damage decreases with stiffening; however, at high load levels the acceleration dominates, resulting in higher damage with stiffening. Again, note that damping varies with the ratio of applied to design load level. Note also that three of the designs utilized 2-bay bracing, and three of the designs utilized 3-bay bracing.

Steel Shear Wall. The shear wall/frame structure was the stiffest of the three steel concepts studied. Damage was generally least with this structure; however, collapse did occur for the 0.1g design at 0.9g applied load. The brittle nature and sudden shear failure are illustrated by the 0.29 damage ratio at 0.8g loading and the collapse at 0.9g loading (Table 5). In general, because of the low period of the structure, floor acceleration resulting from amplification of base motion was least; and in high applied acceleration load levels, attenuation occurred.

Concrete Moment Frame. The response of this structure is similar to the steel moment frame. In the low applied load range, stiffer structures receive greater damage; this condition exists to about 0.25g for the range of structures studied. Over 0.25g the stiffer structures exhibit lower damage, as might be expected.

Concrete Shear Wall. The shear wall/frame structure was stiffer than the concrete moment frame. Damage was generally less with this structure. In general, because of the low period of the structure, floor acceleration resulting from amplification of base motion was less and in high applied acceleration load levels, attenuation occurred.

DISCUSSION AND CONCLUSION

Five sites* were examined in light of the cost and damage data presented earlier and the probability of site acceleration distributions. Based on the probability distribution data from the five sites, Figure 3 indicates the least-cost design acceleration in terms of the 225-year return-time acceleration (80% probability of not being exceeded in 50 years).

Seismic strengthening costs are seen to be dependent on the type of strengthening system utilized; damage is correlated both to drift and acceleration. Strengthening alone limits drift damage but increases acceleration damage. Damage to a structure is a complex mechanism influenced by damping level, degree of inelastic behavior, acceleration level as well as drift level, and spectral region of response. Economic design levels appear to be somewhat greater than those indicated by building codes. The most cost-effective design acceleration is a function of construction type and site seismic exposure.

Acceleration produces a significant amount of damage, and special care should be taken to design ceilings and lights to withstand acceleration. Shaking produces overturning of equipment, which is a significant factor, accounting for most mechanical and electrical losses. Since stiffening produces increased acceleration, consideration should be given to development and utilization of isolation techniques.

A value of design acceleration with a 60 to 100 year-return-time appears to be reasonable. A 100 year return-time acceleration would have a probability of not being exceeded in 50 year exposure of 0.62. Figure 4 shows a histogram of the probability distribution of acceleration based on data from a number of sites. Use of a 100 year return time acceleration would represent a design level of about 70 percent of the 225 year return time level, and a 60 year return time acceleration would be about 50 percent of the 225 year return time level.

An examination of the computed results of the probabilistic damage analysis over the life of the structure shows that most damage comes from the exposure to low level acceleration. Structures which respond elastically in this range being designed for high acceleration exhibit high floor accelerations which cause much of the damage.

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*Bremerton, Wash.; Memphis, Tenn.; San Diego, Calif.; Port Hueneme, Calif.; and Long Beach, Calif.

Figure 4 shows a histogram of the distribution for a typical site with a 225-year return-time acceleration of 0.25g. Also shown in the figure is the damage ratio for a steel-moment-frame structure for three design levels, 0.1g, 0.2g, and 0.3g. As noted, strengthening produces little or no reduction in damage at low acceleration levels, which are most probable because floor acceleration increase from the resulting stiffening of the structure.

REFERENCES

1. J.M. Ferritto. An economic analysis of earthquake design levels, Naval Civil Engineering Laboratory, Technical Note N-1640. Port Hueneme, Calif., Jul 1982.

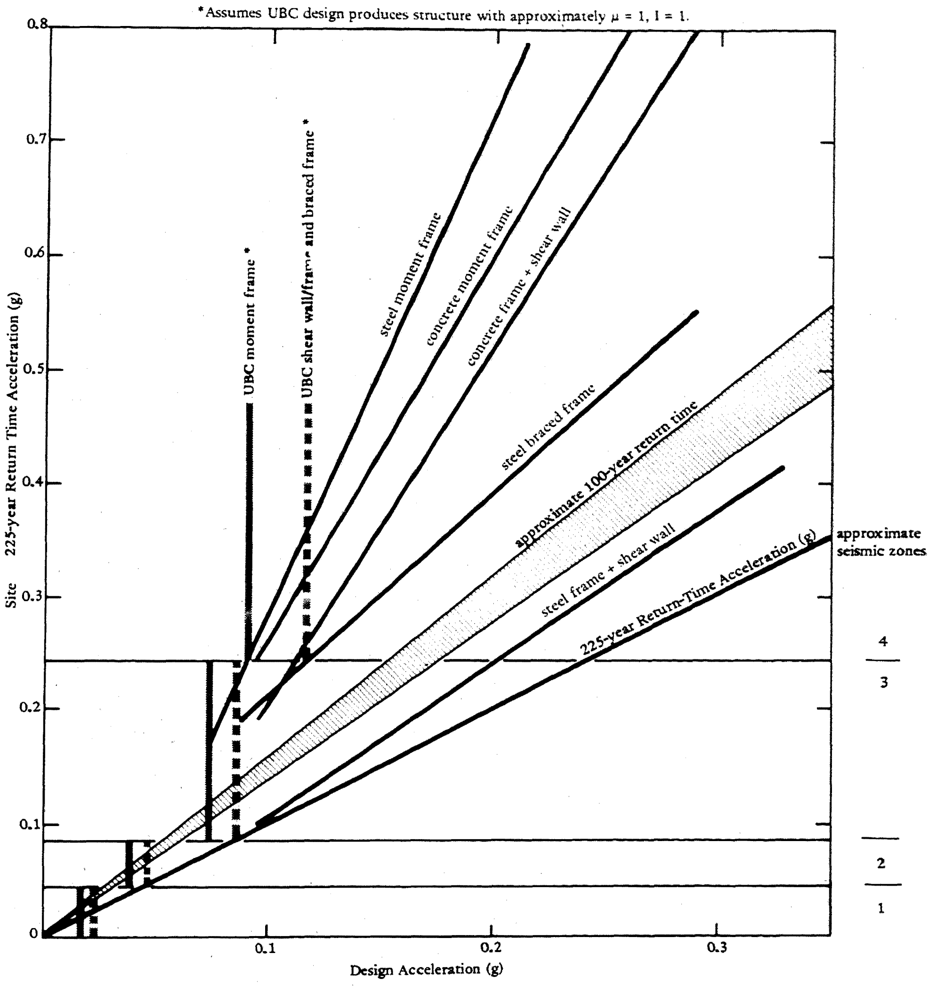

Figure 3. Least cost design acceleration for structures (ductility = 1).

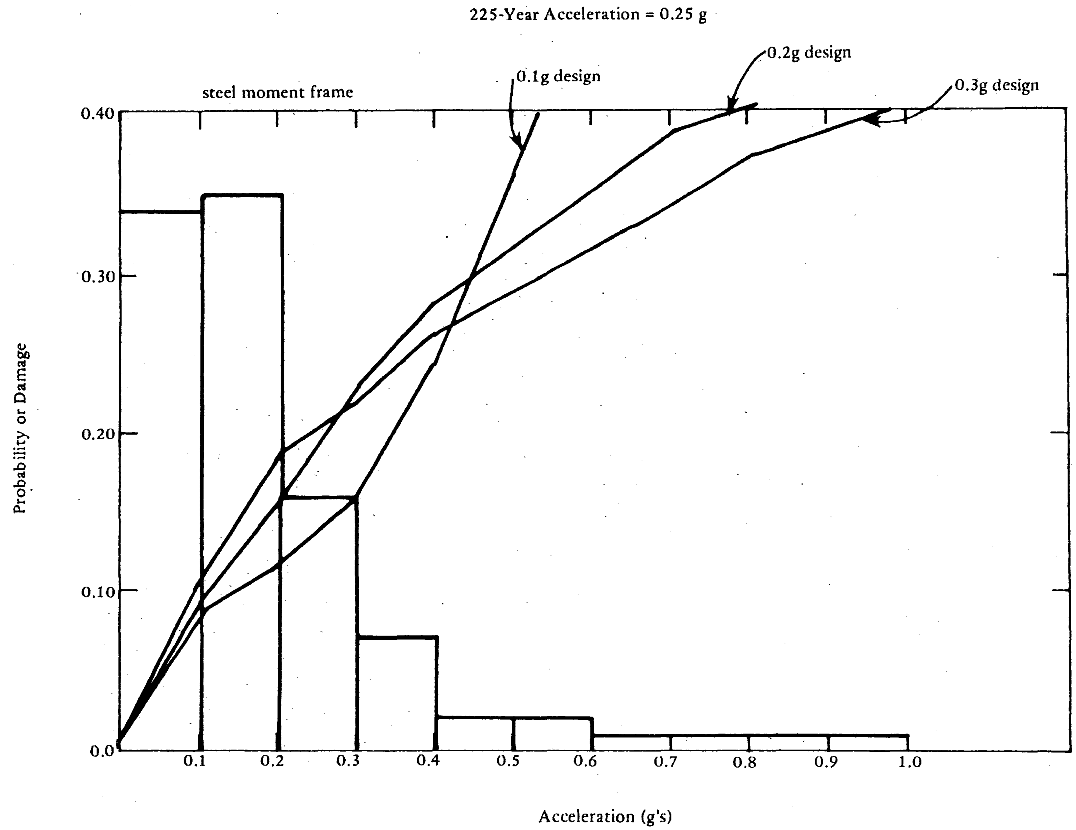

Figure 4. Histogram of probability distribution of acceleration and distribution of damage ratio with acceleration.