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Relative performance of fixed and isolated structures

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ABSTRACT: Base isolation has yet to receive widespread acceptance and implementation in the design of non-critical facilities. One reason for this is the lack of equivalent performance/cost comparison data for base-isolated and fixed-base structures. Presented are the results of an investigation in which the performance of several fixed-base and base-isolated structures were compared. Four-story, steel and concrete moment resisting frames were designed in accordance with the 1990 Structural Engineers Association of California recommended provisions. The base-isolated structures were designed to varying fractions of the recommended lateral force. Non-linear time history analyses were conducted for an ensemble of recorded earthquakes. Statistical analysis of peak response quantities demonstrates the superior performance of the isolated structures. Results illustrate that comparable performance is generally achieved when the isolated frames are designed to between 25% and 50% of the recommended lateral force.

1 INTRODUCTION

This paper presents the results of an analytical investigation in which the performance of fixed-base and base-isolated structures are compared. The purpose of the investigation is to determine, approximately, the design force level for the base-isolated structure which will yield comparable performance to that of the fixed-base structure. The structure considered is intended to be typical of a lateral load resisting frame from a commercial office building located in the United States. Two types of construction have been considered for the frame, a steel special moment-resisting frame (SMF) and a concrete moment-resisting frame (CMF). Each frame has been designed as conventional fixed-base and base-isolated in accordance with the 1990 Recommended Lateral Force Requirements and Commentary of the Structural Engineers Association of California (SEAOC) (Recommended, 1990), using the static lateral load procedure. To establish comparable performance, the base-isolated frames have been designed to varying fractions of the SEAOC recommended base shear: 100%, 50% and 25%. The behavior of the fixed-base design when situated on isolators has also been considered.

The following nomenclature has been adopted to reference the various design cases: "frame type-design case", where "frame type" is SMF or CMF, and design case is FB (fixed-base), BI.0 (fixed-base design on isolators), BI.1 (base-isolated, 100% of code), BI.2 (base-isolated, 50% of code) and BI.3 (base-isolated, 25% of code). Thus, SMF-BI.2 refers to the steel moment frame designed to 50% of the code recommended lateral force.

Nonlinear time history analyses have been conducted using the computer program DRAIN-2D (Kanaan and Powell, 1973), for an ensemble of 18 recorded earthquakes. Analyses have been conducted for three scalings of the ensemble: as recorded, scaled to a peak ground acceleration of 0.4g, and scaled to a peak ground velocity of 500 mm/s. As a basis for comparison, a number of response quantities have been tabulated. These include peak roof displacement, peak first story drift, number of frames that exhibit yielding or collapse and number of yielded superstructure elements. Statistical analyses of these results have been conducted to determine the approximate design force level required to achieve comparable performance.

The results presented herein are part of a larger effort to examine the relative performance of fixed-base and base-isolated structures. In addition to the steel and concrete moment frame, relative performance is being investigated for a steel braced frame and concrete shear wall. Results of the investigation for the steel frames have previously been reported (Lin and Shenton 1992). The results for the steel moment frame are summarized herein for comparison with the concrete moment frame.

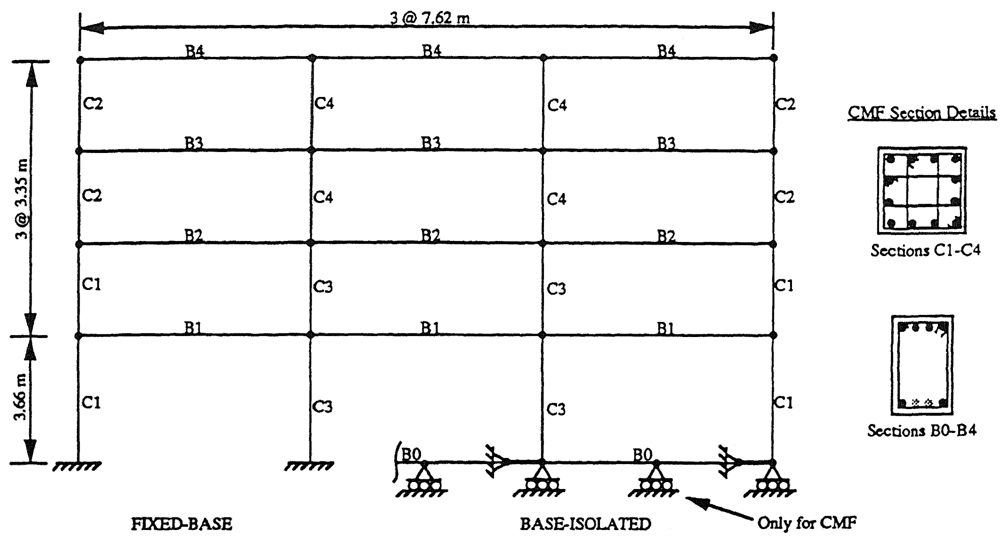

Figure 1. Schematic of frames (refer to Tables 1 and 2 for member sizes)

2 FRAME DESIGNS

The frames under consideration are assumed to be part of a 4 story office building which is located in seismic zone 4 (peak ground acceleration, PGA=0.4g), founded on dense or stiff soil (S_2) and rated for ordinary occupancy. The building is rectangular in plan, measuring 45.7m by 22.9m with 7.62m bays in both directions. Story heights are 3.66m for the first story and 3.35m for the second through fourth stories. The lateral load resisting frame under investigation is located along the transverse centerline of the building. Alternating frames are designed to carry the lateral load, hence, the tributary width for lateral load is 15.3m, and for gravity load is 7.62m. The roof and floors are assumed to act as rigid diaphragms. The frame is illustrated in Figure 1.

The isolated frames are assumed to be supported on laminated elastomeric isolation bearings. Other parameters relevant to their design include: isolated period equal to 2.5 sec, effective isolator damping = 15%, distance to the nearest fault > 15 kilometers. This results in a design isolator displacement of 280 mm (Recommended, 1990).

2.1 Steel Frame Designs

The steel frames are designed for a roof dead load of 2.75 kPa and a floor dead load of 3.92 kPa. The corresponding live loads are 0.90 kPa and 2.40 kPa, respectively. The total seismic dead load (W) is 5190 kN for the fixed-base frame and 6600 kN for the base-isolated frame. The design lateral force for the fixed-base frame, based on the 1990 SEAOC Recommended Provisions is 0.070W, for the base-isolated frame designed to 100% of the code recommended lateral force, the base shear is 0.060W (based on an effective isolator stiffness at the design displacement = 4.22 kN/mm). Member sizes for the SMF designs are presented in Table 1.

2.2 Concrete Frame Designs

The concrete frames are designed for a roof dead load of 7.18 kPa and a floor dead load of 7.89 kPa. The corresponding live loads are 0.90 kPa and 2.40 kPa, respectively. The total seismic dead load is 10,930 kN for the fixed-base frame and 13,720 kN for the base-isolated frame. The design lateral force for the fixed-base frame is 0.078W, for the base-isolated frame designed to 100% of the code recommended lateral force, the base shear is 0.060W (based on an effective isolator stiffness at the design displacement = 8.82 kN/mm). Member sizes for the CMF designs are presented in Table 2.

3 EARTHQUAKE TIME HISTORIES

For the dynamic analysis 18 recorded earthquakes have been selected from the original 30 used by Seed et al. (1974) to develop the S_2 design spectrum of the 1990 SEAOC Recommended Provisions. The 18 have peak ground accelerations ranging from 0.11g to 0.28 g, and peak ground velocities ranging from

Table 1. Member sizes (per American Institute of Steel Construction) for SMF. Refer to figure 1 for member location.

Member	FB and BI.0	BI.1	BI.2	BI.3
C1	W12 x 65	W12 x 58	W10 x 49	W10 x 45
C2	W12 x 50	W12 x 40	W10 x 39	W10 x 39
C3	W12 x 96	W12 x 87	W10 x 68	W10 x 54
C4	W12 x 58	W12 x 50	W10 x 39	W10 x 39
B0 & B1	W21 x 57	W21 x 50	W18 x 40	W18 x 40
B2	W21 x 50	W18 x 46	W18 x 40	W18 x 40
B3	W21 x 44	W18 x 40	W18 x 40	W18 x 40
B4	W18 x 35	W16 x 31	W16 x 31	W16 x 31

120 mm/s to 360 mm/s. The original 18 form the "As-recorded" ensemble. To conform to the zone 4 design spectrum PGA of 0.4g, a second ensemble has been created by scaling each record to a peak ground acceleration of 0.4g. This is referred to as the "PGA" ensemble. A third, the "PGV" ensemble, has been generated by scaling each record to a peak ground velocity of 500 mm/sec.

4 METHOD OF ANALYSIS

Models were developed for the SMF and CMF for analysis using DRAIN-2D. The SMF has 24 nodes, 35 elements and 20 point masses. The CMF has 27 nodes, 38 elements and 20 point masses. In each case, nodes are located at the geometric beam-column centerline intersection, seismic mass is lumped at the beam-column joint, beam-column joints are idealized as rigid and single elements are used to represent each story column and girder.

Particular to modeling of the SMF, girders and columns are modeled using beam/column elements with typical A36 steel properties (E=200 GPa, F_y=248 MPa) and nominal section properties. Members are assumed to exhibit an elastic-plastic moment-curvature relation, assuming 2% hardening. The effect of axial force on yielding is considered.

Particular to modeling of the CMF, girders are modeled using beam elements with degrading stiffness properties. Hysteresis for the element is governed by a modified Takeda model (Takeda 1970), which accounts for degrading stiffness and reduced unloading stiffness (in particular, α=0.3, β=0.2 [Kanaan and Powell, 1973]). Columns are modeled using beam/column elements with the concrete yield surface option (stiffness degradation is neglected) and assuming 2% hardening. Nominal concrete properties are: E=24.8 GPa, f'_c=27,600 kPa and ultimate strain ϵ_u=0.003. Nominal reinforcement properties are: E=200 GPa and f_y=415 MPa.

Isolators are modeled using bilinear, elastic-plastic springs located at the base of each first story column. The isolation system for the CMF has an initial stiffness of 66.4 kN/mm, a post-yield stiffness of 6.64 kN/mm and a yield displacement equal to 10.4 mm. The isolation system for the SMF has an initial stiffness of 31.5 kN/mm, a post-yield stiffness of 3.15 kN/mm and a yield displacement equal to 10.4 mm.

In the analyses, P-Δ effects are considered approximately, the dynamic analysis is performed from the deformed static equilibrium position and original elastic stiffness proportional damping, corresponding to 2% in the fundamental mode, is assumed. Analyses have been conducted for the first 30 seconds of each strong motion record in the 3 ensembles, for the 10 design cases. A total of 540 time history analyses have been conducted.

5 RESULTS

5.1 STEEL MOMENT FRAME

A statistical summary of response quantities for the SMF is presented in Table 3. Results include, for each ensemble scaling: number of frames that "collapse" (defined as a roof displacement greater than 2.5 m), number of frames which exhibit superstructure yielding (i.e., the incidence of yielding, the maximum of which is 18 for each design case), the average number of superstructure plastic hinges, the average peak displacement of the roof relative to the isolators (or ground, for the fixed-base design) and the average first floor ductility ratio (defined as the ratio of average peak

Table 2. Member details for CMF ("t" refers to top and "b" refers to bottom). Refer to figure 1 for member location.

Member	FB and BI.0	BI.1	BI.2	BI.3
C1 & C2	28"x28", 12 #9, #4 ties	28"x28", 12 #8, #4 ties	25"x25", 12 #8, #4 ties	25"x25", 12 #8, #4 ties
C3 & C4	28"x28", 12 #11, #4 ties	28"x28", 12 #10, #4 ties	25"x25", 12 #10, #4 ties	25"x25", 12 #9, #4 ties
B0	-	20"x24", 4 #10 t, 4 #8 b, #5 ties	20"x22", 4 #9 t, 4 #7 b, #5 ties	20"x20", 4 #8 t, 4 #6 b, #5 ties
B1	20"x30", 4 #11 t, 2 #10 b, #5 ties	20"x30", 4 #11 t, 2 #10 b, #5 ties	20"x30", 4 #10 t, 2 #9 b, #5 ties	20"x26", 4 #10 t, 2 #9 b, #5 ties
B2	"	20"x30", 4 #10 t, 2 #10 b, #5 ties	20"x30", 4 #9 t, 2 #9 b, #5 ties	"
B3	"	"	"	20"x26", 4 #9 t, 2 #9 b, #5 ties
B4	20"x30", 4 #9 t, 2 #9 b, #4 ties	20"x24", 4 #9 t, 2 #9 b, #4 ties	20"x22", 4 #9 t, 2 #9 b, #4 ties	20"x20", 4 #9 t, 2 #9 b, #4 ties

first story drift to the first story drift required to cause yielding).

The SMF generally performed well, regardless of the base fixity or ensemble scaling. In only a few cases did the analyses result in collapse, with the majority occurring in the frame designed to 25% of the code recommended base shear (SMF-BI.3). Yielding of one or more superstructure elements, however, occurred in 85% of all the analyses conducted for the SMF. A majority of the analyses conducted on the code designed base-isolated frame (SMF-BI.1) produced some yielding in the superstructure: this is contradictory to the usual assumption of elastic superstructure response for isolated structures. Comparing the number of frames which exhibit yielding, the performance of the code designed fixed-base (SMF-FB) and base-isolated (SMF-BI.1) frames are similar, regardless of the ensemble scaling.

Although the incidence of superstructure yielding is comparable for the SMF-FB and SMF-BI.1, the extent of yielding is quite different, as illustrated by the average number of superstructure plastic hinges. Regardless of the ensemble scaling the number of yielded elements in the SMF-BI.1 is less than the corresponding number for the SMF-FB: the average number of plastic hinges in the SMF-BI.1 is approximately 65% of that of the SMF-FB for the PGA and PGV ensembles, and 33% for the As-recorded ensemble. Based on this criterion, comparable performance to that of the SMF-FB is achieved in the base-isolated frame designed to 50% of the code recommended base shear (SMF-BI.2). Note, however, that the performance of the isolated frame designed to 25% of the code recommended base shear (SMF-BI.3) is also comparable to, or marginally inferior to that of the fixed-base design.

The SMF-BI.1 exhibits superior performance over the SMF-FB when comparing the average peak relative roof displacement: averages for the SMF-BI.1 range from 64% to 73% of that of the SMF-FB. The base-isolated frame designed to 50% of the code recommended base shear (SMF-BI.2) exhibits performance comparable to that of the SMF-FB. The performance of the SMF-BI.3, however, is definitely inferior to that of the SMF-FB.

Considering first floor ductility, the SMF-BI.1 is again superior to the SMF-FB. The ductility ratio for the SMF-BI.1 is on the order of one half that of the SMF-FB for all ensembles. Note, however, that the first floor ductility of the base-isolated frame designed to 50% of the code recommended base shear is greater than that of the fixed-base frame, regardless of the ensemble scaling.

Finally, note that the fixed-base design on isolators (SMF-BI.0) out performed the SMF-FB in every regard, as would be expected. It also out performed the SMF-BI.1.

5.2 CONCRETE MOMENT FRAME

The concrete moment frames also performed well, regardless of the ensemble scaling or base fixity, and slightly better than the steel moment frames. A statistical summary of response quantities for the CMF is presented in Table 4. Results presented include: number of frames which exhibit

Table 3. Response statistics for SMF

Scaling	As-recorded					PGA=0.4g					PGV=50cm/s				
Design	FB	BI.0	BI.1	BI.2	BI.3	FB	BI.0	BI.1	BI.2	BI.3	FB	BI.0	BI.1	BI.2	BI.3
# Frames that collapse	0	0	0	0	0	0	0	0	2	5	0	0	0	0	1
# Frames that yield	11	4	12	14	15	17	14	17	18	18	18	17	18	18	18
# Superstructure plastic hinges	16.8	0.7	5.7	18.4	19.3	34.1	13.1	21.7	33.2	33.5	40.6	15.4	26.2	38.5	39.9
Roof disp. relative to isolators* (mm)	100	61	73	107	136	149	78	96	137	163	210	107	139	212	316
First floor ductility	1.6	0.8	0.9	1.9	3.6	2.1	1.0	1.2	2.6	4.3	3.8	1.6	2.0	4.9	10.2

* Relative to ground for the fixed-base design.

superstructure yielding, average number of plastic hinges, average peak displacement of the roof relative to the isolators and average first story ductility (defined as for the SMF). All 270 time history analyses for the CMF were carried to completion, no collapses were predicted.

Yielding of one or more superstructure elements occurred in 79% of all the analyses conducted for the CMF. The incidence of yielding in this case is dependent on ensemble scaling: all 4 isolated designs performed better than the CMF-FB for the As-recorded ensemble; comparable performance to that of the CMF-FB is achieved with the CMF-BI.2 for the PGA ensemble; every design, fixed-base and base-isolated, performed comparably for the PGV ensemble.

Considering the average number of superstructure plastic hinges, the code design base-isolated frame (CMF-BI.1) performed better than the fixed-base design, regardless of ensemble scaling: the average number of plastic hinges ranges from 3% to 43% of the number of hinges in the CMF-FB. The performance of the isolated frame designed to 25% of the code recommended base shear is also superior to that of the CMF-FB, regardless of ensemble scaling. The extent of superstructure yielding is slightly dependent on ensemble scaling, although not to the same degree as with the incidence of yielding.

Based on average peak relative roof displacement the CMF-BI.1 performed better than the CMF-FB: the peak displacement for the CMF-BI.1 ranges from 51% to 72% of the peak displacement for the CMF-FB. The performance of the base-isolated frame designed to between 25% and 50% of the code recommended base shear is comparable to that of the fixed-base design, for all ensemble scalings: the peak displacement for the CMF-BI.3 ranges from 93% to 114% of the displacement of the CMF-FB.

Consider next the relative performance based on first story ductility. The isolated frames do exhibit lower ductility demands in all but a few instances (the exception is the CMF-BI.3 for the As-recorded and PGA ensembles), the relative difference in demand, however, is not significant. The isolated frame designed to 100% of the code recommended base shear performed marginally better than the fixed-base design: the ductility computed for the CMF-BI.1 ranges from 55% to 87% of the ductility for the CMF-FB. The most significant difference is noted between the CMF-FB and CMF-BI.0 for the PGV ensemble: the first story ductility for the CMF-BI.0 is approximately 1/3 that of the CMF-FB. On this basis, comparable performance to that of the CMF-FB is achieved with the base-isolated frame designed to between 25% and 50% of the code recommended base shear.

6 CONCLUSIONS

In this investigation the relative performance of fixed-base and base-isolated steel and concrete moment frames have been compared. The results presented indicate that, based on a number of performance criteria, isolated structures perform better than conventional fixed-base structures. Comparable performance is achieved with a base-isolated frame designed to a lateral force level less then that required by the 1990 SEAOC Recommended Provisions. This is generally true

Table 4. Response statistics for CMF

Scaling	As-recorded					PGA=0.4g					PGV=50cm/s				
Design	FB	BI.0	BI.1	BI.2	BI.3	FB	BI.0	BI.1	BI.2	BI.3	FB	BI.0	BI.1	BI.2	BI.3
# Frames that yield	16	3	3	12	12	18	13	13	17	17	18	18	18	18	18
# Superstructure plastic hinges	14.8	0.6	0.4	6.8	11.8	33.1	13.2	14.2	23.1	29.4	40.2	12.1	15.2	27.2	32.8
Roof disp. relative to isolators* (mm)	74	40	46	58	73	159	98	114	135	182	163	68	83	118	151
First floor ductility	3.7	2.0	2.8	2.8	3.7	11.3	6.7	9.8	9.1	11.7	10.0	3.7	5.5	6.4	8.6

* Relative to ground for the fixed-base design.

regardless of the scaling of the strong motion records.

The base-isolated steel moment frame designed to 50% of the code recommended base shear performed comparably to the fixed-base design, when based on the average number of superstructure plastic hinges and the average peak roof displacement. Performance comparable to that of the fixed-base frame is achieved with the isolated frame designed to between 50% and 100% of the code recommended base shear, when based on the incidence of superstructure yielding and the average first story ductility ratio. The results for the steel moment frame are relatively insensitive to the scaling of ground acceleration.

The base-isolated concrete moment frame designed to between 25% and 50% of the code recommended base shear performed comparably to the fixed-base design, when based on the average number of superstructure plastic hinges, the average peak roof displacement and the average first story ductility ratio. The design force level which results in comparable performance, based on the incidence of superstructure yielding, varies depending on ensemble scaling. Overall, the results for the concrete moment frame are somewhat sensitive to ensemble scaling; the effect of ensemble scaling varies depending on performance criterion.

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