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DISPLACEMENT-BASED DESIGN OF SEISMICALLY-ISOLATED WOODFRAMED STRUCTURES

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ABSTRACT

Seismic isolation systems have been implemented in numerous structures worldwide but with relatively few applications to woodframed buildings, in spite of the fact that recent shaking table tests demonstrate significant improvement in their performance when they are isolated. To facilitate future applications to woodframed buildings, this paper discusses the development of two displacement-based methods for seismic design of light-framed wood structures with base isolation systems. One method involves a conventional direct displacement-based design procedure. In addition, a simple and quick design procedure based on inter-story drift spectra has been developed. The validity of both displacement-based procedures is confirmed using results from nonlinear dynamic response-history analyses and experimental results from shaking table tests of a half-scale, two-story, woodframed building supported on a sliding isolation system.

Introduction

Although woodframed construction in the U.S. has generally been considered to perform well during earthquakes, the 1994 Northridge Earthquake (Moment Magnitude = 6.8) clearly demonstrated the seismic vulnerability of such construction. As such, the NEESWood project focused on the development of a performance-based seismic design (PBSD) approach for mid-rise wood-frame contruction as well as damage mitigation in low-rise woodframed structures (van de Lindt et al. 2006). One approach to preventing or minimizing damage in woodframed structures is to introduce a horizontally flexible layer (an isolation system) between the foundation and superstructure, thereby increasing the fundamental natural period and thus filtering out damaging frequencies in the ground motion. A comprehensive literature review on the application of advanced seismic protection systems (both base isolation and supplemental damping systems) to woodframed structures is presented by Symans et al. (2002).

There have been continuing research efforts to develop or implement base isolation systems in woodframed buildings, particularly in Japan where different base isolation systems have been applied to residential structures. The design of seismic isolation systems within the context of PBSD procedures has been discussed within the literature. PBSD with isolation systems have been proposed by Priestly et al. (2007) in which direct displacement-based design (DDD) of seismically isolated bridges and isolated buildings (assuming both rigid and flexible behavior of superstructure) were covered. Cardone et al. (2008) proposed a new approach for DDD of seismically isolated bridges using the guidelines of Priestly et al. (2007).

To facilitate future applications to woodframed buildings, this paper discusses the

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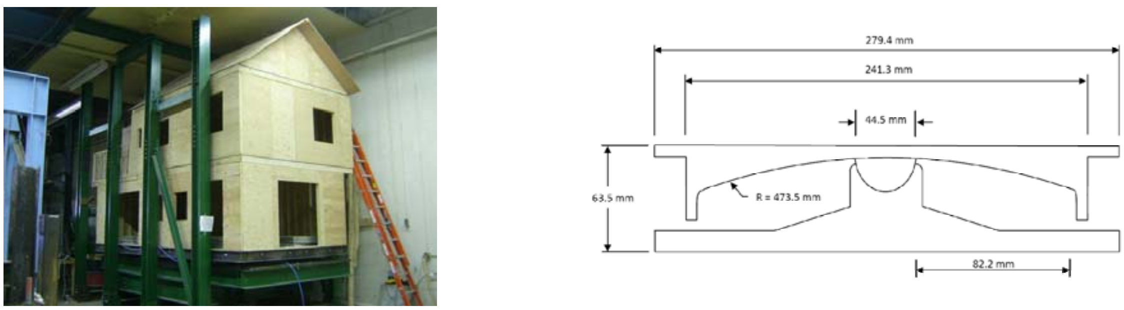
development of two displacement-based methods for seismic design of light-framed wood structures with base isolation systems. The validity of both displacement-based procedures is confirmed using results from nonlinear dynamic response-history analyses and experimental results from shaking table tests of a half-scale woodframed building supported on a sliding isolation system. The results from the tests clearly demonstrated the effectiveness of sliding isolation bearings for seismic protection of light wood-framed buildings.

Half-Scale Seismic Isolation Test

Seismic shaking table tests were performed at Colorado State University on a half-scale woodframed building supported on Friction Pendulum System (FPS) bearings. The tests were conducted as part of the NEESWood project (see Fig. 1). The building is assumed to have been located in either Northern or Southern California and may be regarded as a single family home with three bedrooms and an attached one-car garage. The height of the first and second story, H_{SS} , was equal to 1.23 m, thus making the total height of the superstructure (foundation to roof eave) equal to 2.46 m. Note that the shaking table tests were conducted with motion imposed only along the longitudinal direction (i.e., stronger direction) of the half-scale test structure.



(a) **(b)**

Figure 1. (a) Half-Scale Isolated Test Structure (b) Details of FPS bearing used for Testing.

The seismic weight at the first and second floor level and the roof level was 9.79 kN, 16.46 kN and 11.12 kN, respectively. The total weight of the superstructure, W_{SS} , was 27.58 kN and the total weight of the isolated structure, W_{IS} , was 37.38 kN. The radius of curvature, R , of the FPS bearings used for testing was 437.5 mm with a displacement capacity , Δ_{IS} , equal to +/- 82.2 mm (see Fig. 1 for details). Thus, the natural period of a rigid mass supported on the FPS bearings, T_{IS} , is 1.38 sec as given by the natural period of a pendulum:

$$T_{IS} = 2\pi\sqrt{\frac{R}{g}} \tag{1}$$

Displacement-Based Design using Inter-story Drift Spectra

The simple displacement-based design (DBD) procedure presented herein is based on the generation of inter-story drift spectra and has the potential to be used as a simple and expeditious PBSD approach for design of isolated structures. This method was conceptualized based on an important observation that inter-story drifts are more sensitive than mode shape values to isolation system parameter values. The general method for generating inter-story drift spectra for use within a DBD procedure was adapted from Pang and Rosowsky (2007). This approach provides a reasonably accurate solution and thus serves as a suitable tool for preliminary analysis.

The DBD procedure involves the following steps:

1. For given seismic hazard levels, define desired performance level in terms of limiting inter-story drifts.
2. Select isolation system properties and model the isolation system as a separate story and assume a suitable height of the isolation story and inter-story drift at the isolation level. Estimate the mass and stiffness ratios for each story (relative to the first floor level (i.e., floor above isolation story)).
3. Perform normalized modal analysis to obtain inter-story drift factors and natural frequency parameters.
4. Assume a suitable value of equivalent damping (based on selected isolation system parameters) and construct design inter-story drift spectra for the given hazard level. From design inter-story spectra obtain estimate of design inter-story drift in each story of superstructure for the defined value of isolation inter-story drift ratio.
5. Compare results with design inter-story drift. Revise isolation design if necessary and repeat steps 2-5.

Design Process

Target Performance Levels and Seismic Hazard Levels

The test structure is designed for a location in Southern California with stiff soil conditions (Site Class D). The design 5%-damped spectral acceleration values for MCE hazard Level were determined in accordance with ASCE/SEI-41 (2006). The MCE spectral response acceleration for short-period, S_{MS}, value was taken equal to 1.5g and its value for one-second period, S_{M1}, was taken as 0.9g. The design performance level was chosen as an operational level (0.5% design inter-story drift limit θ_D).

Modeling of Isolation System and Estimation of Mass and Stiffness Ratios

Since the isolation system is modeled as an individual story, the height of the isolation story, H_{IS}, must be determined. For the design of the isolated test structure, it is taken equal to ten times the height of the curved surface of the FPS bearings used in the experimental shaking table tests. The height of the curved surface (used in the denominator of Eq. 2) is also used in the determination of the inter-story drift limit for the isolation story, θ_{IS}, as defined by the following empirical relation:

$$\theta_{IS} = \frac{\eta H_{SS}}{R\left(1-\cos\sigma\right)} \tag{2}$$

where H_{SS} is the typical height of the stories within the superstructure , σ is one-half of the central angle made by the FPS arc and η is an empirical constant. Note that Eq. (2) gives drift expressed as a percentage. σ was equal to 10 degrees for the FPS bearing used in testing. For the design of the isolated test structure, a value of 1.30 was used for η. The above empirical equation is based on the fact that inter-story drift (actually, inter-story drift ratio) is inversely proportional to the height of the story. As indicated previously, for a constant value of σ, the height of the isolation story, H_{IS}, is directly proportional to the radius of curvature of the FPS bearing (*R*). Thus, the inter-story drift limit for the isolation story, θ_{IS}, is inversely proportional to the radius, R ,and, therefore, to the square of the natural period of the isolated structure, T_{IS} . Introducing the height of the stories of the superstructure, H_{SS} , in Eq. 2 allows for a connection to be established between the stiffness of the superstructure and that of the isolation system, that relation being important as it controls the degree of isolation effect. For other types of isolation system and different number of stories in superstructure, a parametric study would be needed to determine a suitable empirical relation.

The mass ratios, β_m, (relative to the isolation story) were computed as 1.68 and 1.14 for the first and second stories, respectively. For initial design purposes, the stiffness of all stories of the superstructure can be assumed equal and thus a uniform distribution of design inter-story drift can be assumed. Thus, a preliminary estimate of the stiffness ratio for each story within the superstructure is given by:

$$\beta_{kj} = \frac{F_t / \Delta_{SS}}{W_{IS} / R + \mu W_{IS} / \Delta_{IS}} \tag{3}$$

where the numerator represents the stiffness of the j-th story of the superstructure, k_{SSj} , and the denominator represents the secant stiffness of the isolation system, k_{IS} .

Note that the substitute structure approach used in the classical DDD method developed by Priestley et al. (2007) can also be used herein to define the effective stiffness of the superstructure. The method described herein is meant to simplify the calculations. In Eq. 3, F_t is the effective lateral force as given by:

$$F_t = \frac{S_{MS} W_{SS}}{B_\xi} \tag{4}$$

and Δ_{SS} is the displacement at the top of the superstructure. Also note that, due to the relatively low pressure on the bearings for light weight woodframed structures, the value of the coefficient of friction, μ , was taken as 0.10. In Eq. 4, B_ξ is the damping reduction factor determined in accordance with (ASCE/SEI-41, 2006):

$$B_\xi = \frac{4}{5.6 - ln\left(100\xi_{SS}\right)} \tag{5}$$

where ξ_{SS} is the equivalent damping of the superstructure and is assumed equal in all modes of vibration. The value of B_ξ is equal to unity as an equivalent viscous damping of 5% is assumed for the superstructure (considering that elastic response of the superstructure is expected).

Normalized Modal Analysis & Construction of Inter-story Drift Spectra

After the mass and stiffness ratios have been determined, a normalized modal analysis is performed to compute frequency parameters, α_n , and mode shapes, ϕ_n , as given by:

$$\alpha_n^2 M \phi_n = K \phi_n \tag{6}$$

Knowing these parameters, the inter-story drift factor for the j-th story in the n-th mode, γ_{jn} , is obtained as follows:

$$\gamma_{jn} = \Gamma_n \left(\phi_{jn} - \phi_{j-1,n}\right) \tag{7}$$

where Γ_n is the modal participation factor corresponding to the n-th mode. The design inter-story drift spectra are then generated from the design acceleration response spectrum (Pang and Rosowsky, 2007) using:

$$\Delta_j\left(\bar{T}\right) = \frac{1}{H_j}\sqrt{\sum_n \left[\gamma_{jn}\left(\frac{\bar{T}}{2\pi\alpha_n}\right)^2 S_a\left(\frac{\bar{T}}{\alpha_n}\right)\right]^2} \tag{8}$$

where $\Delta_j\left(\bar{T}\right)$ is the inter-story drift for the j-th story (with contributions from all modes), H_j is the height of the j-th story, $S_a\left(\bar{T}/\alpha_n\right)$ is the interpolated design acceleration response spectra value, and $\bar{T}$ is the normalized first-story period. Note that the inter-story drift spectra should be further adjusted for the equivalent damping of the isolated structure, ξ_{eq}, as given by (Priestley et al. 2007):

$$\xi_{eq} = \frac{\Delta_{SS}\ \xi_{SS} + \Delta_{IS}\ \xi_{IS}}{\Delta_D} \tag{9}$$

where Δ_D is the design displacement of the isolated structure which is equal to the sum of the top structural displacement, Δ_{SS}, and the displacement capacity of the isolation bearings, Δ_{IS}. Equation 9 is derived assuming that the ratio of equivalent damping of the structure and isolation system is equal to the ratio of the corresponding hysteretic areas used to compute them. Note that the damping reduction factor B_ξ defined in Eq. 5 is used to scale down the inter-story drift spectra by additional equivalent damping of the isolated structure, ξ_{eq}, by replacing the ξ_{SS} term in Eq. 5. The damping ratio associated with the FPS isolation system, ξ_{IS}, is calculated based on Jacobsen's Equation (Cardone et al. 2008) which uses the ratio of the total energy dissipated by the isolation system to the strain energy stored at the maximum amplitude of force and displacement as given by:

$$\xi_{IS} = \frac{2}{\pi}\frac{\mu}{\mu + \Delta_{IS}/R} \tag{10}$$

After generating the design inter-story drift spectra, the estimate of the peak inter-story drift in each story of the superstructure was obtained at the defined value of isolation inter-story drift ratio (228%) calculated using Eq. 2 (see Fig. 2). The first and second story will experience only 0.37% and 0.15% drift, respectively. These values are less than the design inter-story drift limit, θ_D, of 0.5% and thus there is no need to redesign the isolation system.

Direct Displacement-Based Design

This method involves a conventional direct displacement-based design procedure (Priestley et al. 2007) in which a new approach has been taken to define an equivalent single-degree-of-freedom (SDOF) model for an isolated building structure. This DDD procedure involves the following steps:

1. For given seismic hazard levels, define desired performance level in terms of limiting inter-story drifts.
2. Define equivalent SDOF model for a fixed-base superstructure using substitute structure

approach.

3. Select isolation system properties and model isolation system as a separate story. Define an equivalent SDOF model for the isolated superstructure (note that the superstructure is converted to an equivalent SDOF model in the previous step) and determine its actual effective stiffness.
4. Enter displacement response spectra (modified with equivalent viscous damping of equivalent SDOF model of isolated structure) with the design displacement of isolated structure to determine the required effective stiffness.
5. Estimate approximate peak inter-story drift by dividing the design inter-story drift ratio by the ratio of actual to required effective stiffness and compare with design inter-story drift. Revise isolation system design if necessary and repeat steps 2-6.

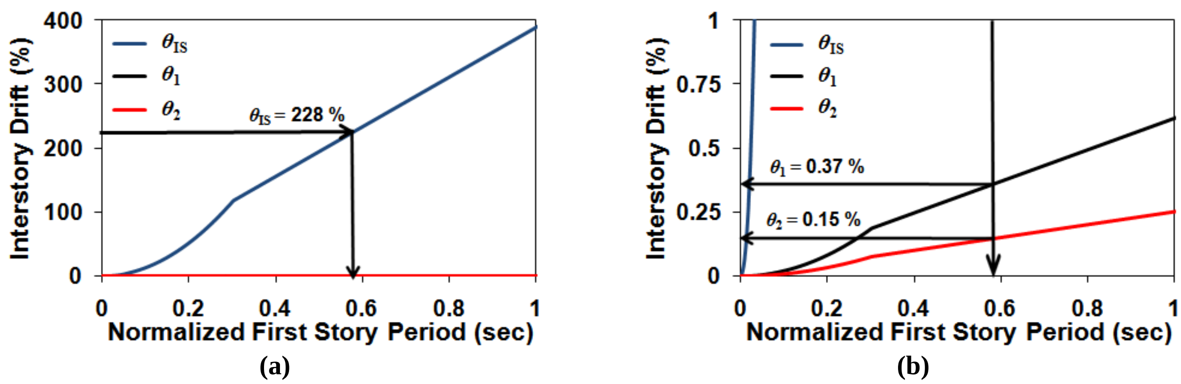


Figure 2. Inter-story Drifts for Superstructure obtained from Isolation Story Drift Demand.

Design Process

DDD of the half-scale test structure presented previously is also used herein to explain the steps of the DDD procedure. Note that target performance levels and hazard levels are the same as defined previously in displacement-based design using inter-story drift spectra.

Equivalent SDOF Model of Superstructure

An equivalent SDOF model for a fixed-base superstructure based on DDD guidelines (Pang et al. 2009) is developed herein. Note that the procedure used to define the effective weight and height of the structure is the same as the Substitute Structure Approach discussed by Priestley et al. (2007). Table 1 shows the design parameters for the equivalent SDOF model of the superstructure which are defined in Fig. 3. Note that the design inter-story drift limit for each story, θ_j, is the same as the design inter-story drift limit, θ_D ,defined previously. A uniform distribution of inter-story drift is assumed.

Table 1 Design Parameters for Equivalent SDOF Model of Superstructure.

Story	h_j (m)	h_j^t (m)	W_j (kN)	θ_j (%)	Δ_j (mm)	Δ_j^t (mm)	C_{vj}
1	1.23	1.23	16.46	0.50	6.10	6.10	0.43
2	1.23	2.46	11.12	0.50	6.10	12.20	0.57

Superstructure

An equivalent SDOF model for the isolated structure can also be defined as shown in Fig. 3 wherein the isolation system is combined with the equivalent SDOF model of the superstructure.

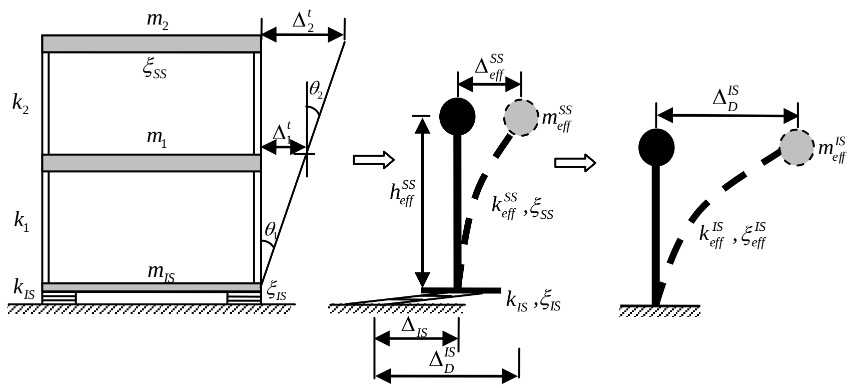


Figure 3. Equivalent SDOF Model of Isolated Structure.

The design displacement of the isolated structure, Δ_D^{IS}, is determined using:

$$\Delta_D^{IS} = \Delta_{eff}^{SS} + \Delta_{IS} \tag{11}$$

The effective mass of the equivalent isolated structure, m_{eff}^{IS}, is obtained as follows:

$$m_{eff}^{IS} = \frac{m_{eff}^{SS}\ \Delta_{eff}^{SS} + m_{IS}\ \Delta_{IS}}{\Delta_D^{IS}} \tag{12}$$

The displacement response spectra in step 4 should be further adjusted for the equivalent damping of the isolated structure. The effective damping of the equivalent isolated structure, ξ_{eff}^{IS}, is determined using (Priestley et al. 2007):

$$\xi_{eff}^{IS} = \frac{\xi_{SS}\Delta_{eff}^{SS} + \xi_{IS}\Delta_{IS}}{\Delta_D^{IS}} \tag{13}$$

Note that the above equation is the same as Eq. 9 except with some different notation as used in the DDD procedure described herein. Finally, the actual effective stiffness of the equivalent isolated structure, k_{eff}^{IS}, is determined using:

$$k_{eff}^{IS} = \frac{k_{eff}^{SS} k_{IS}}{k_{eff}^{SS} + k_{IS}} \tag{14}$$

Required Effective Stiffness using Displacement Response Spectra

Displacement response spectra (adjusted with damping reduction factor B_ξ) are used to determine the required effective period, T_{reff}^{IS}, corresponding to the design displacement of the isolated structure, Δ_D^{IS}. Next, the required effective stiffness, k_{reff}^{IS}, of the equivalent isolated structure is obtained:

$$k_{reff}^{IS} = \left(\frac{2\pi}{T_{reff}^{IS}}\right)^2 m_{eff}^{IS} \tag{15}$$

Note that, for the given design example, the displacement response spectrum for the full-scale test structure (i.e., prototype) is determined as follows:

$$S_d\left(T_p\right) = \left(\frac{T_p}{2\pi}\right)^2 S_a\left(T_p\right) \tag{16}$$

where $S_d\left(T_p\right)$ is the spectral displacement of the prototype, $S_a\left(T_p\right)$ is the spectral acceleration of the prototype and T_p is the natural period of the prototype. The spectral displacement for the half-scale test structure is also given by Eq. 16 except that the values at model scale are used instead of at prototype scale. In that case, the natural period at model scale, T_m, is obtained by invoking similitude principles and results in $T_m = T_p / \sqrt{2}$.

Estimation of Peak Inter-story Drift

An approximation of the expected peak inter-story drift, θ_E, can be obtained from:

$$\theta_E = \frac{\theta_j k_{reff}^{IS}}{k_{eff}^{IS}} \tag{17}$$

The expected peak inter-story drift was estimated as 0.35% for the half-scale test structure using the DDD procedure described herein. This value is less than the 0.5 % design inter-story drift limit and thus there is no need to redesign the isolation system. Note that this estimated value is almost identical to the peak inter-story drift prediction (i.e., 0.37% in first story; see Fig. 2) from the displacement-based design procedure using the inter-story drift spectra.

Design Appraisal

Design Appraisal via Non-linear Response History Analysis

Numerical Modeling

The numerical model that was developed during testing of the half-scale isolated woodframed building was used herein for the design appraisal. In this model, the FPS bearing element was modeled as a three-dimensional element with coupled horizontal stiffnesses and restoring forces. The model also included a time-varying vertical force component and a pressure- and velocity-dependent dynamic coefficient of friction. The FPS bearing model was integrated into the SAPWood software package (Pei and van de Lindt, 2007) wherein it was combined with a non-linear flexible model of the woodframed superstructure. Complete details regarding the numerical model are available in Liu et al. (2008).

Non-linear Response History Analysis

The optimal design of the isolation system was determined based on the displacement-based design procedure and, as a verification step in the PBSD procedure, earthquake simulations using a suite of twenty earthquake ground motions (Krawinkler et al. 2000) were performed using SAPWood. Similar to the work by Krawinkler et al. (2000), these earthquake ground motions were scaled such that their mean 5%-damped spectral values over a period range from 0.1 to 0.6 seconds is equal to 1.5g corresponding to the flat region of the response spectrum for the Southern California region for a 2%/50 yr hazard level.

In Fig. 4, the simulation results (with and without isolation) for the suite of 20 motions are plotted in terms of cumulative probability. Note that the simulations are carried out only in the longitudinal direction (i.e., stronger direction) to be consistent with the experimental tests. As shown in Fig. 4, the first story drift ratio (which corresponds to the peak drift ratio of the structure) for the no isolation case exceeds the 2% drift limit corresponding to the Life Safety (LS) performance level for four out of twenty earthquakes scaled for a 2%/50 yr hazard level. Since the test structure is stronger in the longitudinal direction, the LS performance level is achieved for most of the earthquakes (median value = 1.72% and probability of nonexceedance (P_{NE}) = 81%) in spite of the strong hazard level (2%/50 year). Although the code criterion for design is well satisfied, the results indicate the possibility of damage to the structure that could render it nonrepairable or uninhabitable. For the isolated structure, it is apparent that the specified performance objective (i.e, operational performance level with a design inter-story drift of 0.5%) can be achieved for a 2%/50 yr hazard level (median value = 0.46% and probability of nonexceedance (P_{NE}) = 57.5%). Also note that the degree of isolation effect would have been even more pronounced for earthquake loading along the weaker direction of the test structure.

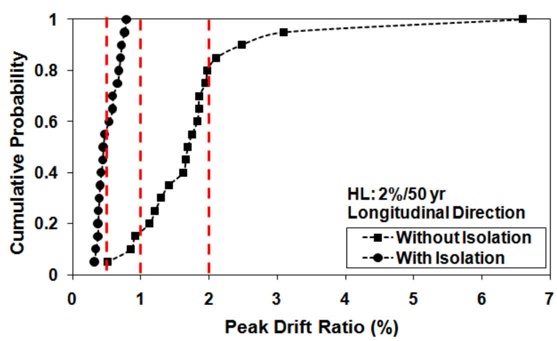


Figure 4. Peak Drift Distributions from Nonlinear Dynamic Analysis of Half-Scale Test Structure With and Without Isolation.

The reasonably good agreement between the design inter-story drifts (based on displacement-based design procedures) and the median peak drifts obtained from response-history analyses serves to validate both displacement-based design procedures and thus suggests that they are suitable for design of isolated structures.

Design Appraisal via Experimental Results

Historical ground motions from three different earthquakes were used for the seismic shaking table tests: 1989 Loma Prieta, 1994 Northridge, and 1987 Superstition Hills. The isolated test structure experienced a peak inter-story drift of 0.44% for the Northridge ground motion scaled to a 2%/50 yr hazard level, indicating that, for a hazard level associated with a collapse prevention performance level for conventional structures, an operational performance level was achieved without any significant damage. For the fixed-base test structure, the peak inter-story drift was more than 5% for the same motion, indicating severe damage and a near-collapse condition for the test structure. Note that, for this particular experimental test

(Northridge ground motion applied to isolated test structure), the measured peak drift of 0.44% is reasonably close to the design inter-story drifts (i.e., 0.37% and 0.35% based on DBD and DDD, respectively) and thus the proposed displacement-based design procedures are further validated.

Summary and Concluding Remarks

Displacement-based PBSD procedures for design of multi-story buildings supported on sliding isolation systems have been presented. These methods were applied to the design of a half-scale woodframed test structure. The final design was evaluated using results from experimental testing and nonlinear response-history analyses. The results demonstrated that the predicted design inter-story drifts based on the selected operational performance level were in reasonable agreement with the median peak inter-story drifts obtained from nonlinear response-history analyses and the peak inter-story drifts from experimental testing, thus validating the proposed displacement-based design procedures.

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