Near Source Fault Effects on the Performance of Base-Isolated Hospital Building vs. a BRBF Hospital Building

Badri K. Prasad, S.E., Principal
DASSE Design Inc.
Oakland, California

Troy A. Morgan, Ph.D., P.E., Postdoctoral Researcher
University of California
Berkeley, California

Tomasz Wienskowski, S.E.
DASSE Design Inc.
Oakland, California

ABSTRACT

Analysis of the data from Northridge earthquake clearly demonstrated, that for sites close to fault rapture (i.e. sites located less than 15km from fault), the ground motion spectra was significantly higher than design spectrum used in the earlier code. The new four story Replacement Hospital – Phase 1 building for the Washington Hospital, Fremont, California, is located at a site which is less than 0.3 km from the Hayward fault. Due to the proximity of this critical care facility to the Hayward fault, this building can be subjected to significant acceleration from a major seismic event on this fault. The building's primary lateral load resisting system consists of Moment Frames in both directions. Base isolation was selected to meet the enhanced seismic performance objective required by the California Building Code for critical care facilities. Viscous dampers were added at the isolation level to control isolator displacement and to limit the moat size. Currently this building is under design and is expected to go into construction in 2010.

This paper presents the results of the non-linear response history analysis on the above building and in particular the performance of the building's non-structural systems with out the near source effects. Next, for the same building a Buckling Restrained Braced Frame (BRBF) system is used as the primary lateral load resisting system and a non-linear response history analysis is carried out. Finally, the results of the building performance using the two different lateral systems are presented. Actual floor acceleration values obtained from the non-linear response history analysis is used to study the effect of ground motions on the performance of non-structural elements of the building and is then compared with current code requirements for Immediate Occupancy and Life Safety performance criteria.

INTRODUCTION

The seismic performance of hospital and acute care facilities is of critical importance to communities located is regions vulnerable to earthquake hazard. Such facilities provide primary treatment of injuries following a major seismic event, and therefore are expected to satisfy *immediate occupancy* following such an event. Recent model building codes [1] have defined this level of performance as the observation of negligible structural damage, and functionality of all mechanical, electrical, plumbing, medical equipment and other secondary systems. Immediate occupancy of hospital facilities can be interpreted as that which allows acute care staff to effectively perform the duties that constitute post-disaster healthcare delivery. Since seismic performance assessment of hospital facilities is a function of damage to structural systems, non-structural systems, and building contents, the structural design must consider multiple demands parameters. These demand parameters include:

a. Peak inter-story drift ratio (PIDR): an indicator of damage to deformation-sensitive components
b. Residual inter-story drift ratio (RIDR): An indicator of post-earthquake functionality
c. Peak floor acceleration (PFA): An indicator of damage to essentially rigid building nonstructural

components

   d. Peak floor spectral acceleration (PFSA): An indicator of damage to non-rigid building nonstructural components (and is dependent on natural frequency of components under consideration).

It should be noted here that this paper may be considered as Part II of the earlier paper by Morgan, et. al. (2007), which dealt with the effect of near source effects on Base-Isolated Washington Hospital Building, located in Fremont, CA.

DESCRIPTION OF FACILITY

The Washington Hospital replacement building is located in Fremont, California (USA), and consists of a basement, first, second and third floors for a total of approximately 300,000 ft^2 (28,400 m^2.) The typical bay size is 32 ft (9.8 m) x 32 ft (9.8 m) with a floor-to-floor height of 16 ft (4.9 m). Lightweight stone panels are to be used as exterior cladding. Floors and roofs are to be constructed with lightweight concrete fill over the metal deck. The building will house food services, material management, central processing, mechanical and electrical rooms, an emergency room, catheterization labs and ICU units. The building ties into a utility tunnel that comes out of the new central plant and enters the building on the north side. The building is located at less than 1 kilometer from the Hayward fault and hence may be subjected to significant ground shaking from a major seismic event. However, for this study the Hospital is assumed to be located not near a major fault and hence will not have the near source effects.

The lateral force-resisting system of the building is steel moment resisting frames and a base-isolation system between the basement slab and the foundation. Moment frames were selected to provide flexibility for planning and future modifications, and base isolation was selected to meet the enhanced seismic performance objectives required for critical care facilities. Viscous dampers were added at the isolation interface to control isolator displacements, and thus limit the necessary moat size. See Figure 1 for a rendered elevation of the building, as currently envisioned.

Fig. 1: Rendering of south elevation of Washington Hospital (courtesy of Fong & Chan Architects)

SEISMIC PERFORMANCE OBJECTIVES

Design Basis Earthquake
The structure is intended to remain *elastic* for the code-prescribed Design Basis Earthquake (DBE) with minimal damage to the lateral load resisting system and with light-to-moderate damage to non-structural systems. This performance level corresponds to an inter-story drift ratio of no more than 1.0% at any floor, in either principle direction of the building. This drift limit is selected to achieve essentially elastic behavior in the lateral system and limit damage to displacement-sensitive nonstructural components, indicating that a building is ready for immediate occupancy following an earthquake. The DBE is defined as a seismic event with a 10% probability of exceedance in a 50 year interval (equivalent to an earthquake with a 475 year return period.)

Maximum Considered Earthquake
The structure should resist the code-prescribed Maximum Considered Earthquake (MCE), also known as Upper Bound Earthquake, *without collapse*. Moderate damage is expected in the lateral system as well as non-structural systems. For a base-isolated structure, the isolators should remain stable in the MCE. In addition, the isolator displacement should not exceed the seismic gap (moat) in the MCE. The MCE is defined as a seismic event with a 10% probability of exceedance in a 100 year interval (equivalent to an earthquake with a 975 year return period.) The basis for the design of the isolation bearings and supplemental dampers is to limit the MCE isolator displacement to not exceed 36 inches (0.92 m), in combination with the above-described DBE drift limit.

SEISMIC HAZARD CHARACTERIZATION

The seismic environment for the subject site is characterized through site-specific response spectra developed for multiple levels of seismic hazard, developed by Fugro West [Geotechnical and Geologic Hazards Evaluation, 2005]. These spectra form the basis for development of a suite of seven earthquake acceleration records, each of which incorporate near-field effects through specific record selection. In order to be consistent with our assumption for this study i.e. the Hospital is not subjected to near source effects, Fugro West modified the above ground motions to eliminate near source as well as directivity effects. This was accomplished by using the fault parallel component for both horizontal directions since the response spectra for the fault parallel component is very similar to the response spectra with out the near source and directivity effects. These acceleration records are incorporated in a nonlinear dynamic analysis of the structure, and proportioning of structural elements is based on the mean response. See Figure 2 below for a plot of the response spectra for the acceleration histories.
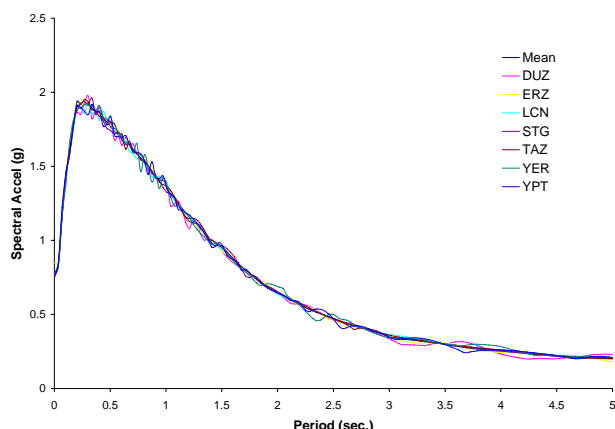
Fig. 2: Summary of DBE ground motion response spectra (5% damped)

SEISMIC ISOLATION SYSTEM DESIGN

Double concave friction pendulum (DC-FP) bearings with a radius of 88 inches provide isolation for the building. When including the effect of the height of the slider, the effective radius is 168 inches which results in a nominal isolated period (excluding the effect of friction) of 4.14 seconds. The behavior of DC-FP bearings are described by Fenz et al. [3]. An expected friction coefficient of 0.053 has been assumed for design, with a variation of between 0.043 and 0.072 providing bounds for the analysis. Linear viscous dampers are used at the isolation interface, with 16 devices in each principal direction of the building. Each device has a stroke capacity of +/- 36 inches, the maximum allowable displacement of the isolation system. An expected damping coefficient of C = 5.4 k-sec^2/in has been specified for design, with a variation of +/- 15% assumed for bounding analysis.

Steel intermediate moment resisting frames (IMRF) constitute the lateral system for the base isolated building. IMRF were chosen instead of Special Moment Resisting Frames (SMRF) to take advantage of the fact that the moment frames will remain essentially elastic during the DBE ground motion. Moment frames provide the greatest architectural flexibility due to open bays. While conventional moment frame buildings may be subject to large inelastic rotations in severe earthquakes, the seismic isolation system limits beam rotations to be nearly elastic even in the MCE level of seismic hazard. To achieve the necessary stiffness such that the isolation system is effective, substantial beam and column sections are required. However, the enhanced detailing requirements of a special MRF are not required because of the negligible expected ductility demands. The final design results in an isolated building with a fixed-base fundamental period of 1.0 sec in each direction.

BUCKLING-RESTRAINED BRACE FRAME DESIGN

The buckling-restrained brace frame (BRBF) is a lateral force-resisting system that had gained wide popularity and seen many application among steel structures in California and around the world. BRBFs consist of concentric steel braced frames, but instead of standard steel sections (i.e. HSS, wide flange, etc) as diagonal braces, specially-manufactured BRBs are used. These BRBs typically consist of a low-strength steel coreplate, a mortar encasement to prevent buckling of the steel coreplate, and some means of decoupling axial strain in the coreplate with stress in the surrounding mortar. A compete description of the mechanics and behavior of BRBs is given in Black, et al. [2004], and a summary of important BRB design parameters is given by Kalyanaraman, et al. [2003].

In the present study, single diagonal braces were assumed in a sufficient number of bays to control displacement (drift) without excessively large braces sizes. The stiffness of each brace element as defined by the Young's modulus of steel, the coreplate area (selected as part of the design) and the effective brace length. Based on previous experience with BRBF design, the effective length of the brace is assumed to be 0.75 times the workpoint-to-workpoint length of the brace. The coreplate areas and number of braces were selected such that the drift criteria is met. The final BRBF design results in a building with a fundamental period of about 0.78 sec in each direction, markedly stiffer than the equivalent isolated building design described above.

PERFORMANCE COMPARISON

Inter-Story Drift Demands
An important demand parameter for the evaluation of likely performance of structural elements and other deformation-sensitive components is peak inter-story drift ratio. For this hospital facility, a drift ratio limit of 1.0% is imposed for the DBE for the immediate occupancy performance level. That is, regardless of the structural system, the median inter-story drift demands computed from the suite of nonlinear analyses may not exceed this limit. As a result, structural elements for both BRBF and base isolation designs are proportioned to achieve this drift limit, and are thus equivalent between the two designs. This is reflected in Table 1 below, summarizing both peak and residual inter-story drift ratios for both BRBF and base isolated structures. Results shown in Table 1 are based on a previous paper by Morgan, et. al., (2007) for the Washington Hospital project, considering near source effects. Table 2 shows the results of inter-story drift ratios for the same building for a Life Safety Performance goal and with out near source effects.

Table 1: Summary of DBE peak and residual inter-story drift ratios for BRBF and base isolated hospital buildings with Near Source Effects

Floor	BRBF				Base Isolated			
	$PIDR_x$	$PIDR_y$	$RIDR_x$	$RIDR_y$	$PIDR_x$	$PIDR_y$	$RIDR_x$	$RIDR_y$
Roof	0.93%	0.85%	0.08%	0.08%	0.75%	0.70%	--	--
3rd	0.88%	0.88%	0.08%	0.11%	0.83%	0.88%	--	--
2nd	0.81%	0.88%	0.09%	0.14%	0.83%	0.92%	--	--
1st	0.80%	0.80%	0.14%	0.17%	0.76%	0.88%	--	--

PIDR: Peak inter-story drift ratio
RIDR: Residual inter-story drift ratio

Table 2: Summary of DBE peak and residual inter-story drift ratios for BRBF and base isolated buildings without Near Source Effects

Floor	BRBF				Base Isolated			
	$PIDR_x$	$PIDR_y$	$RIDR_x$	$RIDR_y$	$PIDR_x$	$PIDR_y$	$RIDR_x$	$RIDR_y$
Roof	1.88%	1.69%	0.09%	0.08%	0.55%	0.56%	-	-
3rd	1.55%	1.57%	0.09%	0.09%	0.62%	0.63%	-	-
2nd	1.61%	1.53%	0.09%	0.09%	0.62%	0.61%	-	-
1st	1.82%	1.56%	0.09%	0.10%	0.55%	0.53%	-	-

PIDR: Peak Interstory Drift Ratio
RIDR: Residual Interstory Drift Ratio

For reference, the drift limit for structures classified as Seismic Use Group IV (defined in 2006 IBC as essential facilities) is 1.5%, a slightly less stringent limit than what has been specified here. The reported values represent an ensemble average of the seven DBE ground motions. Since the steel moment frame forms a yield mechanism at about 1.0% rotation, the base isolated structure exhibits elastic behavior in the DBE, and therefore shows negligible residual deformation. The drift limit for structures for Life Safety (L/S) performance level (Seismic Use Group 11) is limited to 2.5% in 2006 IBC. The maximum drift obtained for the BRBF building for a L/S performance level is equal to 1.88% per Table 2. Residual drift for the building were not observed to exceed 0.10%% at any level per Table 2. While no universally-accepted criteria for residual drift limits exists, FEMA 356 [6] states that steel buildings shall have a negligible permanent deformation for Immediate Occupancy performance. A residual drift of 0.10% is likely negligible, but further examination of possible consequences of such a permanent deformation should be conducted, particularly for windows and means of egress.

Floor Acceleration Demands
Another important aspect of seismic performance assessment is potential damage to acceleration-sensitive non-structural systems. Such systems include medical equipment, ceilings, parapets, and unanchored or lightly anchored architectural systems. In evaluating damage to nonstructural components, it is important to consider not only the peak floor acceleration (PFA) at each floor, but also the peak floor spectral acceleration (PFSA) over a frequency range consistent with expected natural frequencies of nonstructural components. For this building, the range of important frequencies considered were 3 – 16.67 Hz. This is consistent with the definition of rigid and flexible components as defined in FEMA 356. A component is considered rigid if the fundamental period is less than or equal to 0.06 seconds and is considered as flexible if the fundamental period is greater than 0.06 seconds.

To evaluate peak floor acceleration demands for a given design, an ensemble of spectra was developed at each floor level, in each principal direction of the building. Each ensemble contained 7 spectra, one for each ground motion, and these spectra are averaged to form an expected floor spectrum. This spectrum would be used to design anchorage for equipment, or evaluate damage to nonstructural components (given some estimated nonstructural frequency.) As an example, the expected *roof* spectra are indicated in below in Figure 3 for both the BRBF and base isolation design.
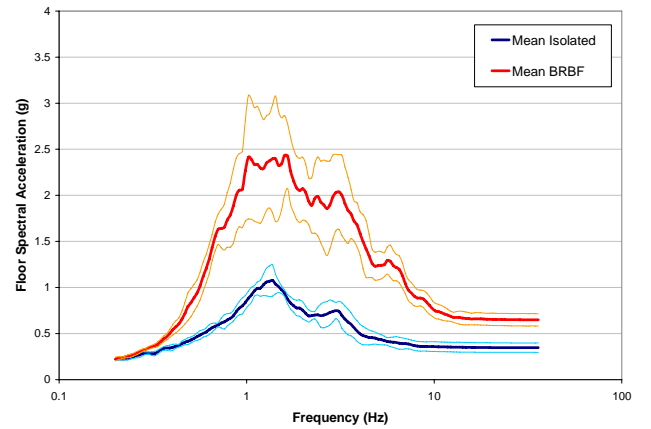

Fig. 3: Comparison of longitudinal roof-level floor spectra for BRBF and Base Isolated, mean shown bold with 16th and 84th percentile

A summary of the ensemble average peak floor accelerations and peak floor spectral accelerations (over the frequency range f = 3Hz – 16.67 Hz) is provided below in Table 3. It can be seen that the maximum spectral acceleration at roof level for a Base-Isolated building is 0.39g at 16.67 Hz (rigid component condition) vs. 0.66g for BRBF – an increase of 88%. For the flexible component condition range of 16.67Hz to 3Hz, spectral acceleration in BRBF building is 2.76 times that of the Base-Isolated building (2.04g Vs. 0.74g). The corresponding value obtained from FEMA 450 is 2.88g – this is quite high and is about 1.4 times the value for BRBF and 3.89 times the value for Base-Isolated building with I = 1.0.

Table 3: Summary of DBE peak floor and peak floor spectral accelerations for BRBF and base-isolated buildings for frequency range of interest and without near source effects

		Base Isolated Building			
	Floor Level	NLRHA > 16.66 Hz	FEMA > 16.66 Hz	NLRHA 16.66 Hz -3 Hz	FEMA 16.66 Hz -3 Hz
X Dir.	Roof	0.35	2.16	0.74	2.88
	3	0.27	1.80	0.61	2.88
	2	0.27	1.44	0.91	2.88
	1	0.26	1.08	0.51	2.70
Y Dir.	Roof	0.39	2.16	0.80	2.88
	3	0.31	1.80	0.72	2.88
	2	0.31	1.44	0.99	2.88
	1	0.29	1.08	0.59	2.70

		BRBF Building			
	Floor Level	NLRHA > 16.66 Hz	FEMA > 16.66 Hz	NLRHA 16.66 Hz -3 Hz	FEMA 16.66 Hz -3 Hz
X Dir.	Roof	0.66	2.16	2.04	2.88
	3	0.81	1.80	2.14	2.88
	2	0.72	1.44	2.18	2.88
	1	0.80	1.08	1.89	2.70
Y Dir.	Roof	0.66	2.16	1.92	2.88
	3	0.80	1.80	2.22	2.88
	2	0.72	1.44	2.25	2.88
	1	0.78	1.08	1.83	2.70

It is worthwhile to compare the floor acceleration results from nonlinear analysis with the provisions for non-structural anchorage present in current building codes. FEMA 450 contains the following equation for computation of anchorage force:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \leq 1.6 S_{DS} I_p W_p \qquad (1)$$

Here, a_p is an amplification factor to account for component flexibility, S_{DS} is the mapped short-period spectral acceleration, W_p is the component weight, R_p is the ductility-based reduction factor, and I_p is the importance factor. For consistent comparison, the factors W_p, R_p, and I_p must be removed from the left hand side of equation (1), in which case the elastic component pseudo-acceleration is given as:

$$F_p = 0.4 a_p S_{DS}\left(1 + 2\frac{z}{h}\right) \leq 1.6 S_{DS} I_p \qquad (2)$$

Assuming flexible components, $a_p = 2.5$ (i.e. the peak of the floor spectrum)*,* and $0.4 S_{DS}$ is an approximation of the peak ground acceleration. For the subject site, the USGS defined PGA for the 475-year event is 0.73 g. A comparison of the results of Eq. (2) to the data presented in Table 2 above is given in Figure 4 below.
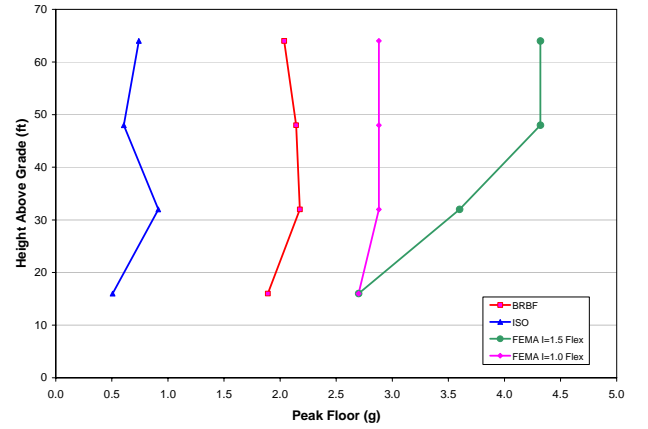

Fig. 4: Comparison of peak floor spectral accelerations for frequency range of 3 Hz to 16.67 Hz and computed from nonlinear dynamic analysis for BRBF and base isolated hospital buildings

The nonlinear analysis data incorporates the elastic properties of the building as well as the strength and ductility demands present. None of these things as captured in Eq. (2), and as a result is a cruder estimation of non-structural demands than those of Table 3. However, it can be seen that Eq. (2) overestimates the unreduced anchorage requirements at all floors. This is true for Immediate Occupancy and Life Safety Performance levels. It should be noted here that the value of $S_{DS} = 1.83$g used in Eq. (2) is based on the average peak spectral acceleration established according to the 2001 CBC, since the Hospital is designed using the 2001 CBC. If the current 2007 CBC code is used, the value of S_{DS} will be 1.35g, a reduction of about 35% from the value of S_{DS} calculated using the 2001 CBC. This implies that there will be further reduction in the anchorage forces calculated by using Eq. (2). This is shown graphically in Fig. 4. Thus it can bee seen that for the frequency range from 3Hz to 16.67Hz (i.e. from rigid to very flexible range), anchorage forces computed using FEMA 450 may be overly conservative. Note that code-based anchorage forces are approximately between 2.5 and 4 times the required forces determined from nonlinear dynamic analysis for the base isolated building.

CONCLUSIONS

This paper is a comparison of the expected seismic performance of a hospital building considering two alternative seismic systems: seismic isolation and buckling-restrained braced frames. These two systems are increasingly being used for seismic protection of essential facilities, and several key features of their performance has been highlighted. The BRBF frames experience residual deformations following a large seismic event, as well as significantly increased floor spectral accelerations relative to the isolated building. It is noteworthy that code-based anchorage design forces may be overly conservative for BRBF frames designed for immediate occupancy as well as

Life Safety performance goals for the subject building that has no near source effects. However, anchorage forces are significantly overestimated by the code for isolated buildings, designed either as an essential facility or for Life Safety performance goal. In the design of critical care facilities, seismic isolation should be evaluated relative to conventional systems accounting for the important demand parameters of expected residual drift and peak floor spectral accelerations in a major seismic event.

ACKNOWLEDGEMENTS

AUTHOR AFFILIATIONS

Principal, DASSE Design, Inc., Oakland, CA, USA, p*rasad@dasse.com*
Postdoctoral Researcher, University of California, Berkeley, USA, *troy@berkeley.edu*
Project Engineer, DASSE Design Inc., Oakland, CA, USA, *wienskowskwi@dasse.com*

REFERENCES

1. FEMA 450 (2004) *"NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – 2003 Edition,"* Federal Emergency Management Agency, Washington D.C.

2. Fugro West, Inc. (2005) *"Geotechnical Study and Geologic Hazards Evaluation, CC/ED Building, Washington Hospital, Fremont California."*

3. Fenz, D.M., Constantinou, M.C. (2006) *"Behaviour of the double concave Friction Pendulum bearing."* Earthquake Engineering and Structural Dynamics, **35**(11), pp. 1403-1422

4. Black, C.J., N. Makris, and I.D. Aiken (2004) *"Component testing, seismic evaluation, and characterization of buckling-restrained braces,"* ASCE Journal of Structural Engineering, **130**(6), pp. 880-894

5. Kalyanamaran, V., B. Ramachandran, B.K. Prasad, B.N. Sridhara (2003) *"Analytical Study of 'Sleeved Column' Buckling Restrained Braced System",* Proceedings, SEAOC 72nd Annual Convention, Squaw Creek, California, USA

6. Morgan,T.A. and Prasad, B.K., "Comparision of the Seismic Performance of a Base-Isolated Hospital Building versus BRBF Hospital Building Considering Near Source Effects", Structural Engineers World Congress" Bangalore, India, 2007

7. FEMA 356 (2000) *"Prestandard and Commentary for the Seismic Rehabilitation of Buildings,"* Federal Emergency Management Agency, Washington D.C., 2000

8. International Conference of Building Officials, *California Building Code, 2001 Edition*, Whittier, CA